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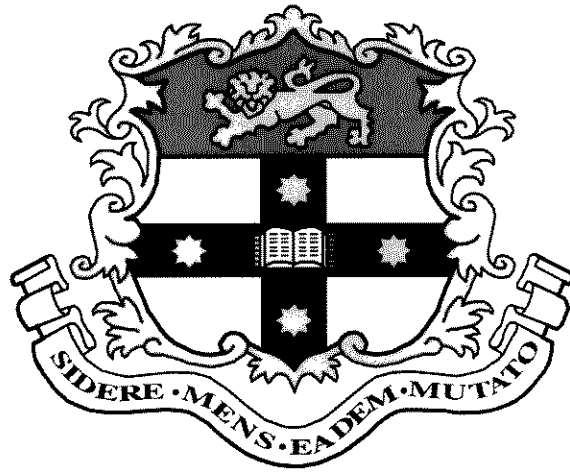
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This thesis has been
accepted for the award
of the degree in the
Faculty of Engineering

GROUND WATER PUMPING ANALYSIS AND ITS INFLUENCE ON PILE FOUNDATIONS

by

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A thesis submitted for the
Degree of Doctor of Philosophy

Department of Civil Engineering
The University of Sydney

August 2003

*Dedicated to
my parents
for their love*

SYNOPSIS

This thesis presents a study of the behaviour of soil due to excessive ground water pumping or excavation with pumping of ground water to dry excavation sites and the influence of these factors on piled foundations. Numerical analysis has been used to determine soil movements in both the vertical and horizontal directions due to pumping of ground water. The area that is most influenced by pumping of ground water is also investigated. Since pumping of ground water or excavation causes an influence on structures and utilities in the vicinity of the pumping area, a study of the behaviour of piled foundations that are subjected to both vertical and horizontal soil movements has been carried out. A combined technique that uses both finite element and boundary element methods has been used for these analyses. The main objectives of this study can be summarised as follows:

- (a) To investigate lowering of the water table due to excessive pumping of ground water for living and industrial purposes. Two case studies of ground water pumping in Hanoi, Vietnam are examined and a comparison between the measured vertical soil movements and the numerical analysis is carried out. The horizontal soil movement due to ground water pumping is also obtained from the numerical analysis.
- (b) Excavation with lowering of the ground water table is investigated. Undrained excavation and partially drained excavation (excavation with pumping of ground water) analyses have been carried out to investigate the influences of both excavation and lowering of the ground water table on soil movements.
- (c) Combined technique using both the finite element and the boundary element methods is used to study piled foundation behaviour where the piles are close to the area of ground water pumping or excavation with ground water pumping.

Numerical analysis has been applied to the analysis of a series of reported case studies. The good agreement between the measured data and the analysis results show that the finite element methods used can predict field behaviour.

Two case studies are analysed where soil movements due to ground water pumping occurred. In this analysis, lowering of the ground water table and vertical soil movements over several years have been analysed and compared to the measured data. Three analyses of excavations, which were reported and published, have been conducted. Among them, one excavation with lowering of the ground water table has been carefully investigated. As may be expected, lowering of the ground water table significantly increases the settlements of the ground surface behind the retaining wall. Numerical analysis was shown to be an effective means of predicting soil movements due to excavation with lowering of the ground water table behind retaining walls. Two case studies of the behaviour of a pile next to an excavation have been considered. One involves results from a centrifuge tests and the other is a full-scale case study.

Numerical methods have also been used to conduct parametric studies of the soil behaviour due to lowering of the ground water table or due to excavation with lowering of the ground water table behind a retaining wall. Behaviour of piled foundations in the area of influence has also been explored. The results can be used to predict the behaviour of piled structures that may be affected by lowering of the ground water table so that an effective management scheme can be implemented to avoid potential damage.

PREFACE

The work described in this thesis was carried out during the period from 1999-2003 in the Department of Civil Engineering, the University of Sydney under the supervision of Professor H. G. Poulos and Associate Professor J. C. Small.

According to the By-Laws of the University of Sydney, a candidate has to indicate which portion of the work is original and which is the work of others and the sources from which he obtained the work of others. Apart from the work that was obtained from other sources and acknowledged in the text, the Author claims originality of the following work:

- In Chapter 2, the summary of the current state of land subsidence around the world and the current state of land subsidence in Hanoi, Vietnam due to ground water pumping.
- In Chapter 5, the case study analyses of Phap Van and Ngoc Ha. Comparison between measured data and the numerical results.
- In Chapter 5, the parametric study using the finite element program (EF4). The study is based on soil properties and pumping rates that were taken from the Hanoi area, however the results can be applied to areas with similar soil conditions, especially in South East Asia.
- In Chapter 6, the pile behaviour analysis where boundary element methods (GEPAN, PALLAS and PIES) are used in conjunction with finite element analysis. The analysis is based on real soil movements from the Hanoi case studies used to investigate the influence of soil movements due to pumping of ground water.
- In Chapter 6, the parametric study of the behaviour of a pile foundation in the area close to a pumping station. Programs PALLAS and PIES have been used in the numerical analysis.
- In Chapter 7, the numerical analyses using the fully coupled finite element method (EF4) to study three reported case studies with different soil conditions with and without lowering of the ground water table.

- In Chapter 8, case studies involving analysis of pile behaviour behind a retaining wall bracing an excavation, one from centrifuge test results and one from a full-scale test.
- In Chapter 8, numerical analysis of undrained excavation and excavation with lowering of the ground water table behind a retaining wall (using programs EF4 and AFENA), and comparison of soil behaviour with two case studies. Conclusions about the influence of lowering of the ground water table during excavation.
- In Chapter 8, the parametric study of the behaviour of an existing piled foundation behind a retaining wall of an excavation with lowering of the ground water table (EF4).

A research paper has been published:

Nguyen, V. D., Small, J. C., and Poulos, H. G. "The effects of ground water pumping on piled foundations",

ACKNOWLEDGEMENTS

I would like to express my deepest gratitude to Professor Harry G. Poulos, who encouraged me to do postgraduate study in 1999, for his supervision, enthusiasm, support, and helpfulness through out the entire period of my candidature.

I am also indebted to Associate Professor John C. Small, who most influenced my study in Australia, for his supervision, patience, support, encouragement, and assistance throughout the course of this study.

I wish to thank academic staff and non-academic staff of the Department of Civil Engineering at the University of Sydney. Especially, I would like to thank Associate Professor Rob Wheen, Dr. Tim Hull, Mrs Kim Pham, and my colleagues Dr. Hong Ngoc Bui, Dr. Sii Chung Wong, Mr. Behrooz, Mr. Ezzat Williams, and Mr. Bossco Poon.

I like to express special thanks to Dr. Gerard Gabriel, Dr. Michel Chaaya and to all staff at the Computer Aided Design Group-Facilities Management Office, The University of Sydney for their support.

To my future wife to be, Miss Kim Chi Hoang, I wish to thank her for her love, sacrifices, assistance and encouragement.

Finally, I would like to say many thanks to my family, especially to my parents for their continued support, love and sacrifices throughout the last four difficult years.

August 2003

Viet Duong Nguyen

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NOTATIONS

Roman

$[A]$	Influence factor matrix relating element displacement and stress
$A(t)$	Response due to the unit pumping function
$[AD]$	Summation matrix
A_i	Area of soil element i
AIR	Apparent influence range
$[B]$	Influence factor matrix relating element displacement and pile tip displacement
b	Thickness of confining layer
B_w, B_c	Initial thickness of the aquifer and aquitard respectively
c_a	Negative skin friction
C_c	Coefficient of consolidation
c_m	Coefficient of uniaxial compression
C_p, C_s, C_p, C_s	Primary and secondary compressibility coefficients
c_u	Undrained shear strength of soil
c_v	Average coefficient of consolidation during cyclic loading
C_v	Vertical coefficient of consolidation
$[D]$	Influence factor matrix relating element displacement and pile head load
D	Depth of the soil element i
$d\varepsilon$	Total strain increment
$d\varepsilon_p$	Total strain increment due to pore pressure change
d_w, d_c	Change in total water head at any point in the aquitard and aquifer respectively
D_w	Penetration depth of the retaining wall
E'_1, E'_2	Drained Young's modulus of the soil layers number 1 and 2
E^*	$= E_s / (1-\nu)$
e, e_o	Void ratio and initial void ratio respectively
E_d, S_d	McNamee – Gibson displacement functions
EI_{d-wall}	Flexural stiffness of diaphragm pile wall
EI_{s-wall}	Flexural stiffness of sheet pile wall

E_p, E_{eq}	Pile and equivalent smeared wall Young's modulus respectively
E_u, E'_s	Undrained and drained Young's soil modulus respectively
F	Overall safety factor (Chapter 6)
F	Strength of the point sink (unit volume / unit time) (Chapter 2)
F_1	Ratio = 1.7 (Chapter 6)
F_{11} to F_{44}	Constant coefficients
F_2	Ratio = 2.5 (Chapter 6)
f_b	Base resistance of pile
f_{ci}	Limiting pile-soil interaction stress in compression, at element i
[FE]	Pile compression matrix
F_i	Components of the body force
f_s	Shaft resistance of pile
f_{ti}	Limiting pile-soil interaction stress in tension at element i
[G]	Influence factor matrix relating cap displacement and cap-tie-cap beam force (to allow for pile caps jointed by tie beam)
G	Shear modulus
g	Acceleration due to gravity
G^{FS}	Term to allow for release of fluid when the free surface (water table) falls
[H]	Influence factor matrix relating cap load and pile head load
h	Hydraulic head
h'	Hydraulic head in confining layer
H, H_i	Soil thicknesses and thickness of soil layer i
H_1, H_2	Thickness of the soil layers that are defined in Figure 2.6 (Chapter 2)
H_1, H_2	Thickness of the soil layers number 1 and 2 (Chapter 5)
H_{di}	Drained path length of soil layer i
h_j	Amount by which the water level is lowered in the j^{th} consolidation stage
H_o	Final excavation depth
h_{ps}	The depth of the point sink
H_{UB}, H_{LB}	Depths of upper and lower boundaries of the compressible layer

I_p, I_{eq}	Second moment of area of pile cross-section and equivalent smeared wall respectively
K	$= 1 - \sin \phi'_a$
k	Isotropic permeability
k_1, k_2	Isotropic permeability of soil layers number 1 and 2
$k_B^s, k_B'^s$	Maximum bending pile moment and maximum horizontal pile deflection correction factors respectively for excavation width (lateral pile behaviour analysis)
K_c	Relative flexibility of pile
k_d^s	Maximum pile bending moment correction factor for pile diameter (Chapter 5)
$k_d^s, k_d'^s$	Maximum pile bending moment and maximum horizontal pile deflection correction factors respectively for pile diameter (lateral pile behaviour analysis)
$k_E^s, k_E'^s$	Maximum pile bending moment and maximum horizontal pile deflection correction factors respectively for drained Young's modulus (lateral pile behaviour analysis)
k_H, k_V	Horizontal and vertical permeability
k_B^h, k_B^v	Maximum horizontal and vertical soil movement correction factors for excavation width (Excavation parametric study)
k_E^h, k_E^v	Maximum horizontal and vertical soil movement correction factors for drained soil Young's modulus (Excavation parametric study)
k_h^h, k_h^v	Maximum horizontal and vertical soil movement correction factors for excavation depth (Excavation parametric study)
k_H^h, k_H^v	Maximum horizontal and vertical soil movement correction factors for the thickness of the homogeneous soil layer (Excavation parametric study)
$k_H^s, k_H'^s$	Maximum bending pile moment and maximum horizontal pile deflection correction factors respectively for the thickness of the homogeneous soil layer (lateral pile behaviour analysis)
$k_h^s, k_h'^s$	Maximum pile bending moment and maximum horizontal pile deflection correction factors respectively for excavation depth (lateral pile behaviour analysis)

k_{ss}^h, k_s^v	Maximum horizontal and vertical soil movement correction factors for strut stiffness (Excavation parametric study)
K_o	Coefficient of lateral earth pressure
$[K_p]$	Pile stiffness matrix
K_p	$= (1 + \sin \phi') / (1 - \sin \phi')$
K_r, K_z	Hydraulic conductivity in two directions (r, z) of the radial coordinate systems respectively
k_r^s	Maximum pile bending moment correction factor for pile distance to the edge of the pumped region (Chapter 5)
$[K_s]$	Soil stiffness matrix
$[K_{sm}]$	Incremental stiffness matrix
K_{SPT}	Empirical correlation ratio for estimating base resistance of a pile from standard penetration test results
$k_s^s, k_s'^s$	Maximum pile bending moment and maximum horizontal pile deflection correction factors respectively for excavation strut spacing (lateral pile behaviour analysis)
k_{st}	Axial stiffness of the struts
K_{xx}, K_{yy}, K_{zz}	Hydraulic conductivity in three directions (x, y, z) of the Cartesian coordinate systems respectively
$K_z(n)$	Vertical permeability given as a function of porosity n
K_{z0}	Initial hydraulic conductivity
L^{-1}	Expressions for Laplace inverse transformation
L_c	Coupling matrix
L_p	Pile length
L_x, L_y	Half size of soil element i in x and y directions respectively
M	Modified Cam Clay parameter M
$Max \delta_h, Max \delta_v$	Maximum horizontal and vertical soil movements respectively due to excavation with drawdown of water table
$Max \delta_{hb}, Max \delta_{vb}$	Maximum horizontal and vertical soil movements of the typical analysis (the base case analysis)
$Max. d_h$	Maximum horizontal ground surface movement ($r = 0$)
$Max. d_h^r$	Maximum horizontal ground surface movement at a distance r
$Max. d_v$	Maximum vertical ground surface settlement (at distance $r = 0$)

$Max. d_v^r$	Maximum vertical ground surface settlement at a distance r
$Max. d_v^t$	Maximum vertical ground surface settlement (at distance $r = 0$) at time t
$Max. d_w$	Maximum water table drawdown
$Max. d_w^r$	Maximum water table drawdown at a distance r from the axis of symmetry
$Max. d_w^t$	Maximum water table drawdown at time t
$Max. M_b^s$,	Maximum pile bending moments and maximum horizontal pile
$Max. \rho_b^s$	deflection respectively for the base case
$Max. M^s, Max. \rho^s$	Maximum bending moment and maximum pile deflection respectively for a single pile
M_p	Friction factor for the soil acting on the pile surface $0 \leq M_p \leq 1$
m_v	Coefficient of volume compressibility
N	$= (n\alpha_2)k E / (H^2\gamma_w)$
n	Porosity
n_0	Initial porosity of soil
N_p, N_m	Transformed normal stresses
N_q	Empirical correlation factor for the ultimate pile base resistance
N_{SPT}	Standard penetration test result
$[O]$	Influence factor matrix relating pile head displacement and element stress
OCR	Over-consolidation ratio
$[P]$	Interaction factor matrix relating pile head displacement and pile tip displacement
p	Excess pore pressure
$p(t)$	Pore water pressure at time t
p_0	Initial pore water pressure
P_a	Applied axial load to pile
P_b	Maximum resisting force due to end bearing in the stable soil layer
p_g	Pre-consolidation pressure
p_i	Existing pile-soil interaction stress on element i prior to a load increment

P_{limit}	Limiting negative pore pressure
P_p, P_m	Transformed pore pressure coefficients
P_{s1}	Maximum downdrag force due to negative friction in the settling layers
P_{s2}	Maximum resisting force due to positive friction in the stable soil layer
P_u	Ultimate axial load capacity of a pile
$[Q]$	Interaction factor matrix of pile head displacement and pile cap displacement
$Q_{(a-c)i+1}$	Ground water flow quantity from the i^{th} aquifer to the $(i+1)^{th}$ confining layer
$Q_{(c-a)l}$	Ground water flow quantity from the i^{th} confining layer to the i^{th} aquifer
Q, q	Ground water pumping rate
Q_{ai}	Yield quantity from the i^{th} aquifer
q_c	Static cone penetration resistance
$[R]$	Interaction factor matrix relating pile head displacement and pile head load
R	Distance where ground water is kept constant ($R = 1000m$ for the ground water pumping parametric study in Chapter 5)
r	Radial coordinate
RFS, RFB	Hyperbolic ratios
r_p	Radius over which ground water is pumped
$[S]$	Interaction factor matrix relating pile head load and element stress
S	Storage capacity
S_c, S_u	Storage coefficients of the aquitard and aquifer respectively
$S_b, S_i(t)$	Total settlement of soil layer i and settlement of soil layer i at any time t respectively
$[SIF]$	Influence factor matrix relating element soil displacement and stress
S_s	Specific storage
$S_s(n, \sigma')$	Specific storage related to n and to effective stress σ' .
$S_T, S_T(t)$	Total ground surface settlement and total ground surface

	settlement at any time t respectively
s_u	Undrained shear strength
s_y	Specific yield ratio
T_p, T_m	Transformed shear stresses
T_x, T_y	Transmissivity in x and y directions respectively
$u(t-t_j)$	Heaviside step function
u_p	Lateral pile deflection
U_r	Radial displacement
u_s, u_{sm}	Lateral soil deflections
u_t	Ground surface settlement at any time t
$U_v, \hat{U}_h$	Vertical and horizontal ground surface movements respectively (Figures 2.4 and 2.5)
U_{xp}, U_{xm}	Displacements in x-direction due to transformed shear stresses T_p, T_m respectively
U_z, u_z	Vertical ground surface settlements
U_{zp}, U_{zm}	Displacements in z-direction due to transformed normal stresses N_p, N_m respectively
$[V]$	Interaction factor matrix relating pile head load and pile head load
W	Volumetric flux per unit volume of source or sinks of water
W_p	Distributed load associated with pile
W_r	Recharge capacity
W_s	Distributed load associated with soil
W_u	Distributed load due to interaction between pile and soil
W_y	Ultimate horizontal lateral resistance of pile-soil interaction
$\{X_e\}, \{X_b\},$ $\{X_c\}, \{X_t\}$	Vectors of pile soil stress, pile tip displacement, pile cap displacement, and capped pile head load respectively
$\{Y_q\}, \{Y_c\}, \{Y_p\}$	Vectors of element stress, cap load, and pile head load respectively.
$\{Y_{q,i}\}, \{Y_{q,e}\}$	Vector of soil displacements imposed on element 'i' due to pile head load on individual piles and soil displacement vector due to external sources respectively

Greek

Δb_{kj}	Thickness of the k^{th} finite element of the j^{th} aquitard
$\Delta \rho_p$	Vector of incremental pile displacements and incremental displacement of the pile tip respectively
Δh	Ground water head change
Δm	Compression of each unit cell
Δp	Vector of incremental pile-soil interaction stresses
Δp_i	Incremental pile-soil stress on element I
Δp^i	Change in pore pressure in soil element I
Δu	Non-linear pile-soil interface behaviour
ΔZ	Vertical length of each unit cell
$\Delta \sigma', \Delta \sigma'_l$	Increase in effective stress
α	Slope of the variation of water level with time curve (Equation 2-13)
α	Vertical compressibility of soil (Equation 2-27)
α^{pc}	Correlation ratio for Young's soil modulus estimation from the static cone penetration test results
δ_v	Vertical compression of a soil layer assuming the condition of no lateral strain
δh	Thickness of soil element
δ_{ik}	Kronecker delta
δ_v^i	Vertical compression of soil element i assuming the condition of no lateral strain
$\delta u_x(X, Y)$	Soil movement in the x direction of a soil element that has dimensions of X, Y
$\delta u_z(X, Y)$	Soil movement in the z direction of a soil element that has dimensions of X, Y
ϕ	Internal friction angle
β	Negative skin friction factor
β_0	Pore water compressibility
μ	Ratio = 0.7 (Chapter 6)
ν	Poisson's ratio

ν^*	$= \nu / (1 - \nu)$
π	Hydrostatic stress in the pore fluid;
γ_w	Ground water unit weight
κ	Modified Cam Clay parameter Kappa
λ	Modified Cam Clay parameter Lamda
λ	Lamé modulus
ν	Apparent velocity of the pore water
η	Auxiliary elastic constant $= (1 - \nu)/(1 - 2 \nu)$
ψ	$= \sqrt{k_H / k_V}$
ϕ'	Effective friction angle of soil
ϕ'_a	Adhesive effective friction angle of soil
Φ_f	Flow matrix
ρ	Total mass density of the saturated material;
ρ_2	Mass density of the fluid, assumed to be incompressible
ρ_f	Fluid density
ρ_s	Soil grain density
σ'_v	Effective vertical stress
σ^*	Stress caused by loading or boundary water level change
σ'_o	Initial vertical effective stress
σ_{ki}	Components of the symmetric stress tensor for the solid phase
σ_{soil}	Vector of induced soil stresses due to external soil movements
∇^T	$= [\partial / \partial x \quad \partial / \partial y \quad \partial / \partial z]$

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

Pumping of ground water for human consumption has been carried out for hundreds of years. As soon as industrialization began and cities were formed, different land usage patterns appeared. Forests were devastated and houses and factories built on the cleared land. Ground water began to be pumped from drilled wells to top up the storage tank in each house and building for living purposes. Ground water pumping was also used for industrial and agricultural purposes. Extraction of ground water is normally accompanied by large soil movements, and severe damage to structures has been reported. Figure 1.1 illustrates the damage caused by excessive pumping of ground water in America. Settlements over 52 years can be up to 9m as has been reported in San Joaquin Valley, the agricultural area of California. In this picture the original ground level in 1925 at the top and the ground level after 52 years of pumping ground water, in 1976, is shown compared with the man standing (Figure 1.1a). Horizontal soil movements caused by pumping of ground water has also been recorded. Figures 1.2 to 1.4 show fissures formed by pumping ground water in Picacho, Arizona. As shown by the warning sign in Figure 1.2, fissures caused by ground water pumping are a real hazard for traffic systems. In Vietnam ground water has been exploited since the beginning of the 20th century, but only during the last 20 years has land subsidence due to ground water pumping occurred in Hanoi city. This is the result of the development of the Vietnamese economy in the last 20 years. Damage to residences and offices have been reported. As the country is still developing and ground water pumping is increased more and more, land subsidence due to ground water pumping will become more severe unless there is an adequate control of ground water pumping to reduce the soil movements.

Ground water pumping is also used in excavation as a method to stabilise and allow access to the excavation. When excavating, ground water inflow can be one of the main difficulties that is encountered. The problems can arise due to erosion or collapse of the sides of the excavation, instability of the base of the excavation, or due to collapse of temporary support for the excavation. To prevent these problems, ground water is pumped out from excavation sites by several methods, such as pumping from sumps, wells or deep wells. Each method has its own merit and sets of

conditions under which it can be applied. However all methods to some degree cause the ground water table behind the retaining wall to be lowered from the original position.

Excavation itself causes both vertical and horizontal soil movements. Excavation with drawdown of the ground water table exaggerates the soil movements especially in the vertical direction. Records from excavations with drawdown of the water table that cause adjacent buildings to tilt are common and can be easily found in the geotechnical literature.

This thesis presents analysis of soil behaviour due to ground water extraction in two cases: (1) excessive pumping of ground water for domestic and industrial purposes and (2) pumping of ground water in the excavation process. The influences of the soil movements due to ground water extraction on adjacent piled foundations are investigated. The comparisons between measured data and the numerical analysis results can be used to indicate the possibility of using numerical methods in forecasting the influences of these activities on surface and underground structures.

1.2 OBJECTIVES

The objectives of this thesis are to use numerical methods to analyse soil movements (that include horizontal and vertical movements) due to heavy pumping of ground water in the Hanoi area, Vietnam. The effects of the soil movements on single piles and pile groups will then be studied theoretically.

In this thesis the main objectives are:

- Collecting information concerning the geological and hydrological conditions of the Hanoi soil, the current state of ground water extraction, as well as subsidence problems in the Hanoi area.

- Using a numerical analysis program that is available to analyse the problem of soil movements due to excessive pumping of ground water in the Hanoi area.
- Comparing the numerical results with the field data collected in the area.
- Theoretically looking at the effects of the soil movements due to pumping of ground water on a single pile.
- Collecting information about excavation and excavation with ground water pumping.
- Using finite element methods to analyse and compare the results with collected data from excavations.
- Conducting parametric studies of piled foundation behaviour due to excavation induced soil movements with ground water pumping during the excavation period.

1.3 OUTLINE OF THE THESIS

Chapter 1 is the introduction to the thesis.

Chapter 2 presents a review of the current state of land subsidence due to ground water extraction around the world. This is followed by a literature review of methods that have been developed and used to analyse soil behaviour that take into account ground water pumping and the drawdown of the ground water level. The available methods are classified into three groups: (1) analytical methods, (2) transform methods, (3) numerical methods.

Chapter 3 gives an introduction to Hanoi, Vietnam, the problem of land subsidence due to ground water pumping and the problems that have arisen in Hanoi, Vietnam. The geological and hydrological conditions of the Hanoi soils are presented in Chapter 3. The geological history of the Quaternary sediments in the Red River Delta, the geological properties of the Hanoi soil, and the hydrological conditions of the water-bearing layers are introduced.

Chapter 4 presents measurement equipment used for recording land subsidence and the devices used to measure the lowering of the ground water table level due to

pumping of ground water in Hanoi, Vietnam. The measuring instruments that have been used in Hanoi are also introduced in this chapter.

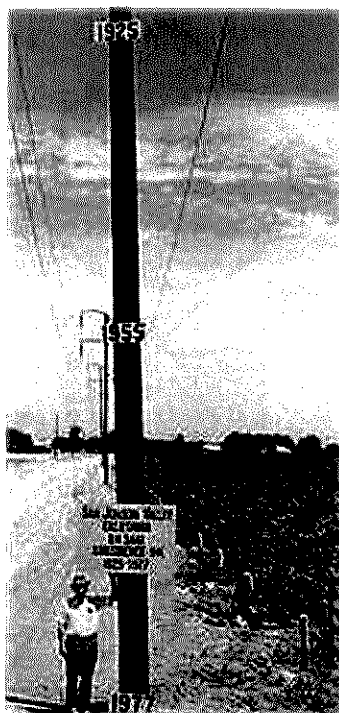
In Chapter 5 the details of the numerical analysis used is explained and the numerical analysis results are compared with the field data collected.

Chapter 6 presents a theoretical analysis of a piled foundation that exists close to a ground water pumping station.

Chapter 7 gives a clear view of effects of excavation and excavation with ground water pumping on soil movements.

Chapter 8 presents case studies of piled foundation behaviour for piles near an excavation. Both full-scale and laboratory scale case studies have been investigated. A parametric study of piled foundation behaviour due to excavation induced soil movements is presented.

(a)



Photograph from U.S. Geological Survey Internet site
http://water.wr.usgs.gov/subsidence/ls_3.html

(b)



Early in the 20th Century land near the Alviso Marina, top, sank nearly 13 feet due to over-pumping of local ground water (Picture from the website of Santa Clara valley water districts <http://www.valleywater.org>).

Figure 1.1: Land subsidence in San Joaquin Valley, California.



Figure 1.2: Earth fissure warning in Arizona, 1981 (Picture from website of United States Geological Survey <http://geochange.er.usgs.gov>).



Figure 1.3: Earth fissure near Picacho, Arizona, 1981 (Picture from website of United States Geological Survey <http://geochange.er.usgs.gov>).

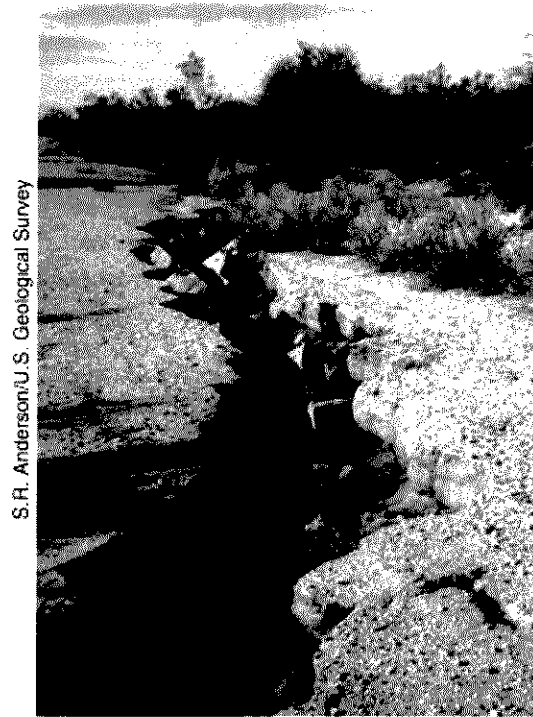


Figure 1.4: Earth fissure near Picacho, Arizona, 1981 (Picture from website of United States Geological Survey <http://geochange.er.usgs.gov>).

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

This chapter will present a brief history of the occurrence of surface subsidence due to ground water extraction in the world. The latter part will introduce a review of some of the previous theories that have been developed by several researchers involving land subsidence due to ground water withdrawal.

Excessive pumping of ground water from wells causes a decline of artesian head in a confined aquifer or a lowering of the water table in an unconfined aquifer. Both a decline in artesian head and decline in the water table cause a decrease of pore pressure, and consequently an increase in the effective stress, which in turn causes settlement of the soil and subsidence is seen at the ground surface. The subsidence often happens in areas where water is pumped from recent alluvial soil deposits (or unconsolidated deposits). The extent of pore pressure reduction and the geometry and properties of the region in which pore pressure is reduced, affects the magnitude of the ground surface movement. Both lowering of ground water and consolidation of the soil are time dependent, depending on the permeability and compressibility of the soil in the reservoir being pumped as well as the surrounding zones of material.

As a result, analysis of land subsidence due to ground water extraction involves:

- The theory of consolidation or time dependent deformation of the ground.
- The theory of fluid flow.

Many land subsidence models have been based on Terzaghi's consolidation theory. This is based on the assumption that the water flow occurs in the vertical direction only. Other land subsidence models employ the consolidation equation, which was first developed by Biot (1941). It is a three-dimensional consolidation equation and this equation is valid for any arbitrary time dependent load while Terzaghi's equation is restricted to the one-dimensional problem. Biot's equations are based on the assumption that the soil is a saturated, poro-elastic medium, and that the change in volume of an element of the soil is equal to the volume of water that flows out of that element.

Flow of fluid in saturated soils is often based on Darcy's law that states that the superficial velocity of the pore fluid is proportional to the hydraulic gradient in the fluid.

A number of different analysis models that have been applied to solving the problems of ground movement due to excessive pumping of ground water can be categorized into the following groups: (1) analytical methods; (2) transform methods; (3) numerical methods.

2.2 OCCURRENCE OF SURFACE SUBSIDENCE DUE TO GROUND WATER EXTRACTION IN THE WORLD

2.2.1 INTRODUCTION

Ground water has been seen as a very important resource through the ages. This is because ground water is very clean, bacterially pure, locally available, and of constant quality and temperature. Today ground water is a major source of water for many municipalities and industries and for irrigation, suburban homes and farms. Ground water has been exploited and used for quite a long time (800BC in Greece and then in the 18th century in Europe and in the United States from the 19th century).

After the Second World War the world population rose rapidly, urbanization extended over great areas, and industrialization and irrigated cropland increased sharply. In order to meet the needs of the burgeoning world economy and population, ground water pumping intensified significantly and widened over large areas in many large cities in the world. The increase in pumping has resulted in intensification and spread of land subsidence and some appearance of horizontal movement of the ground surface.

The mechanism involved in the subsidence phenomenon is the reduction of pore pressure due to pumping ground water. Usually, in a saturated geological layer, the total overburden stress is the sum of the effective stress (or intergranular pressure) and pore pressure (or hydrostatic pressure). Through the process of pumping or lateral drainage, the pressure of fluid is reduced while the total overburden stress is unchanged. The pore pressure reduction must be taken by the soil skeleton thereby causing an increase in the grain-to-grain or intergranular stress. Consequently under the appropriate combination of conditions, a considerable amount of subsidence may occur.

Subsidence of the ground surface will occur if the following conditions prevail:

- (a) The reservoir fluid pressures are lowered causing a load transfer from the water to the soil skeleton.
 - (b) The reservoir soils are compressible (usually uncemented) and/or are unable effectively to resist deformation on the transfer of load from the fluid phase to grain-to-grain contacts.
 - (c) The overburden lacks internal self-support and can easily deform downward.
- (Allen, 1969)

The magnitude of subsidence depends on many factors such as: the degree of over-consolidation of the soil due to previous depth of burial during geologic time, types of clays and sands, shape and size of sand grains and relative proportions of interbedded clays and sands (Allen, 1969).

There are examples where subsidence is so small as to be negligible even though a head decline of 30m or more has been experienced. For example, in Memphis, Tennessee, the artesian-head decline in confined aquifers of Eocene age has been 30m or more since 1886, yet subsidence as shown by repeated levelling has been negligible (Davis, 1987).

2.2.2 SUMMARY OF CASE HISTORIES

- *Areas of Honshu, Japan*

Land subsidence due to ground water extraction in Japan first began at the beginning of the 19th century. However, the first land subsidence was noticed in 1923 in Tokyo after the great Kanto earthquake that hit Tokyo in the same year. Land subsidence also occurred in Osaka around 1928. According to a study of the Ministry of Construction's Geographical Research Institute and the Environmental Protection Agency of Japan, at the second symposium on land subsidence in 1974, besides Tokyo and Osaka there were 46 land subsidence areas totalling 7380 km² in area. At present, although new policies and solutions have been adopted, many areas in Japan are still affected by land subsidence due to ground water withdrawal. There are 62 subsiding areas and 11 of them have a subsidence rate that exceeds 20mm/year. The total area of subsidence amounts to 276 km² (Environment Protection Agency of Japan, 1995).

Tokyo, Osaka, and Niigata are three areas that have experienced appreciable subsidence. In Tokyo the ground water level has dropped about 60m due to surplus withdrawal of ground water from various aquifers. The cumulative subsidence since the early 1900's was 4.57m in 1975. Recently, due to the restriction of ground water extraction, the water level has recovered to a drawdown of 10-30 m since 1971.

Osaka is the second largest city in Japan and also one of the typical examples of land subsidence in Japan. Osaka City experienced a maximum subsidence of about 2.5m during the period from 1928 to 1965. The area of subsidence that was centred on Osaka City was about 280 km². According to the Disaster Science Laboratory study of land subsidence in Osaka from 1938-1942, about 50% of the subsidence was due to consolidation in the top 30m and about 90% was caused by the consolidation in the top 180m. The maximum subsidence in the Niigata basin reached about 2m in 1968.

- ***Mexico City, Mexico***

Mexico City is situated in a closed basin at an elevation of about 2240 m above mean sea level. The basin contains lacustrine deposits that consist of a clayey layer of variable thickness overlying a sand and gravel aquifer.

The surface subsidence in Mexico City probably began in the 19th century but was only first noticed early in the 20 century. In 1925, the bottom of the Mexico valley was sinking based on the evidence of two precision levellings (performed in 1877 and 1924) of a monument located near the Cathedral. Later measurements showed that by 1959 the subsidence was more than 4m all over the old Mexico City and as much as 7.5m in the northeast part of the City (Poland and Davis, 1969).

The maximum observed subsidence recorded up to the year 1973 reached 9 (m), which makes it one of the most remarkable examples of subsidence in the world. The damage to buildings, pavements and other structures was inevitable especially to the sewage systems of the City.

- ***Areas in California***

San Joaquin Valley

The San Joaquin Valley is a broad alluviated intermontage structural trough, it is about 400 km long and from 40 km to 90 km wide. Pumping of ground water for irrigation purposes started in about 1900 but since the Second World War, the ground water pumping rate has been intensive and this has broadened the area of ground water level decline. During the period between the early 1940's and the mid 1960's the ground water pumping rate rose by as much as three times or more. The pumping extraction rate increased from $3.7 \times 10^9 \text{ m}^3$ in 1942 up to $12.3 \times 10^9 \text{ m}^3$ in 1964. The ground water overdraft has caused subsidence of nearly half of the farmland area (13500 km^2), and maximum subsidence has reached 8.8 m. The large land surface subsidence resulted in severe problems such as design and maintenance of canals and waterways, the expenditure of millions of dollars in repair and replacement cost of deep irrigation wells, and changes in irrigation and farming practices.

Santa Clara Valley

Santa Clara Valley is located to the south of San Francisco, California. It is the first area in the United States where land subsidence was recognized in 1933 due to ground water extraction. Pumping of ground water from the confined aquifer system that is about 240m thick has caused a drawdown of the artesian head by as much as 75 m since 1912. The total land subsidence during the period 1915-1967 was 3.9 m.

- ***Savannah, Georgia***

During the time between 1933-1955, land subsidence of as much as 200 mm has occurred over an area of 50 square miles in Savannah, Georgia. The increase in pumping rate of the ground water from 23 million gallons in 1933 to 57 millions gallons in 1963 caused a marked decline in artesian head. Over 40 years from 1933, the artesian head reduced 40 feet and it continued declining another 100 feet in the 22 years from 1933 to 1955.

- ***Houston-Galveston area, Texas***

The Houston-Galveston area is underlain by a thick section of unconsolidated lenticular deposits of sand and clay. Removal of water and oil from surface strata caused surface subsidence of 5 feet to occur corresponding to a 200-foot decline of water level in the period from 1943 to 1964.

- ***South-central Arizona, U.S.A***

The land subsidence in south central Arizona due to intensive ground water extraction was first detected in 1948. From 1948 to 1967 the ground water was lowered 61m in the 700m thick permeable sediments. The ground surface subsided 2.3m during that time and 65% of the subsidence happened in the upper 253m from the surface. The differential subsidence of the ground caused earth fissures and damaged irrigation systems, an interstate highway, and railroads.

- ***Taipei basin, Taiwan***

The 280m thick sediments of the Taipei basin comprise unconsolidated sediments of alternating layers of sand, gravel, clay and silt. Uncontrolled pumping of ground water started in 1957 at a rate of 9million m³/year. The pumping rate gradually increased and reached 249 millions gallons in 1972 and 212 millions gallons in 1974. The maximum subsidence of the land surface in the Taipei basin was 1.78 m between 1960 and 1975.

- ***Other known subsidence***

Apart from well-known areas that suffered large subsidence as mentioned above there are many other areas where there have been land surface subsidence due to ground water extraction, for example Bangkok (Thailand), Shanghai (China) and Venice (Italy).

Damage from land surface subsidence due to ground water extraction has cost hundreds of millions of dollars throughout the world. It causes flooding in subsided areas, changing gradients that affect the capacity of canals, drains, and sewer systems,

and structural failure of buildings, bridges, pipe lines, railroads and other engineering structures. The consolidation and flexure of sediments at depth have caused damage by compression or shear failure of oil and water well casings.

At present, in many areas in the developed countries, in order to support the needs of industrialization, ground water is still being pumped at a high rate that causes a continuing land subsidence. As developing countries expand their industry, ground water is still in great demand and overdraft pumping of ground water is likely to happen that will result in the subsidence of the land surface. Some examples of new areas where subsidence due to ground water extraction have occurred are Keletan (Malaysia), Hanoi (Vietnam), Kerman (Iran), and some areas in Japan. In some well-known areas, though the land subsidence phenomenon has already stopped, the potential for further subsidence still exists. It is doubtless that land subsidence will occur if ground water is still unwisely pumped at a high rate and there is no proper ground water resource management.

2.3 ANALYTICAL METHODS

Analytical methods are those methods that employ closed-form mathematical solutions for calculating land surface subsidence. Changing of the water table level causes changes in pore pressure in the soil. As mentioned above, two consolidation theories that have been used to calculate the changes of pore pressure in the soil due to the fluctuation of water table are Terzaghi's consolidation theory and the consolidation theory developed by Biot (1941). In general, calculation of land subsidence due to ground water extraction using analytical methods is simplified by solving the one-dimensional problem (with Terzaghi's consolidation theory) or three-dimensional problems using Biot's theory. Boundary conditions need to be applied before a solution can be obtained.

An analytical method developed by Viergever (1991) was applied to predict land subsidence in a reclaimed area in the central part of the Netherlands. In the reclaimed area, where the soil consisted of five to six meters of compressible clay and peat layers on top of sandy subsoil, the water table was lowered about five meters. The result of changes in pore water pressure due to the hydraulic filling was up to one meter of land subsidence. The author employed the one-dimensional consolidation

theory of Terzaghi to calculate the process of consolidation and subsidence. Because this simplified method only gives the pore water pressure for a uniform soil while in reality the soil is often layered, a method presented by Hansbo (1987) was used to transform the actual layered soil conditions to a uniform soil layer with an adapted thickness and the equivalent soil properties. The thickness of the transformed uniform soil is given in the following equation:

$$h_{uniform} = h_{original} \sqrt{\frac{k_{uniform} m_{v original}}{m_{v uniform} k_{original}}} \quad (2.1)$$

Any fluctuation of the water level causes a triangular change in pore pressure in the clay or peat layers. Pore pressure at a time t and depth z is given by using the consolidation equation of Terzaghi (1951):

$$p(t) = \sum_{m=0}^{\infty} \frac{4p_0}{(2m+1)\pi} \sin\left[\frac{(2m+1)\pi}{h} z\right] e^{\left\{\frac{-k}{m_v \gamma_w} \left[\frac{(2m+1)\pi}{h}\right]^2 t\right\}} \quad (2.2)$$

where $p(t)$ = pore water pressure at time t ; p_0 = pore water pressure at time $t = 0$; h = thickness of the compressible layer; z = depth at which $p(t)$ is calculated; k = permeability; m_v = compressibility coefficient; γ_w = volume weight of water; t = time of calculation.

Figure 2.1 shows pore pressure generated due to change in water level or change in pore water pressure at the bottom of the clay and peat layer. A change in pore pressure means an equal change in effective stress and then subsidence may occur. Surface subsidence is calculated according to the Terzaghi-Buisman-Koppejan equation:

$$s = h \left\{ \frac{1}{C_p} + \frac{1}{C_s} \log(t) \right\} \ln \frac{\sigma'_o + \Delta\sigma'_1}{\sigma'_o} + h \left\{ \frac{1}{C_p'} + \frac{1}{C_s'} \log(t) \right\} \ln \frac{\sigma'_o + \Delta\sigma'}{p_g} \quad (2.3)$$

where s = subsidence; h = thickness of the layer; C_p , C_s , C_p' , C_s' = primary and secondary compressibility coefficients; t = time; σ'_o = original vertical effective stress;

$\Delta\sigma', \Delta\sigma'_f$ = increase in effective stress; p_g = pre-consolidation pressure occurring in the soil.

The superposition rule is employed in this method hence the deformations caused by the fluctuation of water level or the change in pore water pressure at the bottom of the clay and peat layer are calculated separately and added afterward to the total subsidence. The soil swelling is also taken into account by using the same coefficients as for the consolidation calculation multiplied by a factor of 0.1. Comparisons have been carried out between the calculated results and the in-situ measurement results in the central part of the Netherlands with the result that five out of ten comparisons were within 20% of the measured values.

Yaw and Jyh (1995) presented the so-called step-loading method to model the stress-strain behaviour of soil due to the complicated process of a changing ground water level. The lowering and rising of the ground water level leads to a complicated ground water level versus time relationship. In order to have a constant water level over a period of time, which then drops to a new level for a new stage, Yaw and Jyh (1995) simplified the measured complex processes of the lowering and rising the ground water level to form a stepped diagram approximating the ground water level versus time. In each period of time, as the soil deformation due to the rising and lowering of the ground water level in that period is not taken into account, then the magnitude of the ground water level is taken as the lowest value in that period. Figure 2.2 illustrates an example of the measured fluctuation in ground water level against time. Figure 2.3 shows the simplified ground water level versus time plot in which the ground water level is constant within a period of time. This simplified water level is the input water level value for the step-loading model used to calculate soil settlement. Unlike Viergever (1991) who used Terzaghi's one-dimensional consolidation method to compute the settlement of the soil due to the lowering of the ground water over a period of time, Yaw and Jyh (1995) employed a one-dimensional Biot's coupled consolidation theory. If there are k periods of time to be considered, then the one-dimensional Biot's coupled consolidation equation that can be used to calculate the consolidation of a homogeneous layer of soil is given below:

$$S_i(t) = \frac{4H_i}{\Pi^2 \eta_i G_i} \sum_{j=1}^k \left\{ \gamma_w h_j \sum_{n=0}^{\infty} \frac{1}{(2n+1)^2} \left\{ 1 - \exp \left[- \frac{(2n+1)\Pi}{2H_{di}} \right]^2 C_{vi} (t - t_j) \right\} u(t - t_j) \right\} \quad (2.4)$$

where k is the number of consolidation stages; t_j , h_j represent respectively the initial time and the amount by which the water level is lowered in the j^{th} consolidation stage, and $u(t-t_j)$ is a Heaviside step function.

The settlement of the soil calculated by equation (2.4) above is only applicable to a homogeneous layer of soil. The superposition theory is again utilized for calculating the total settlement of the ground surface in the whole soil deposit, which includes m soil layers. Hence the total settlement $S_T(t)$ is equal to the sum of the settlements of each homogeneous layer.

$$S_T(t) = \sum_{i=1}^m S_i(t) \quad (2.5)$$

The step-loading model has been used to analyze the soil settlement at Wang-Kon, on the Chang-Hwa coast of Taiwan. The results of soil settlement due to the lowering of the ground water table obtained from this model compared well with the observed field data.

Viergever's (1991) method considers soil deformation caused by both the lowering and rising of the water table in a homogeneous soil layer. In the case of a layered soil, a formula is given to simplify the layered soil conditions to an equivalent homogeneous soil layer. On the other hand, Yaw and Jyh (1995) solve the problem without consideration of the fluctuation of the water table but do consider the layered properties of the soil. Their method can take into account the difference in soil deformation characteristics of different soil layers. Gu *et al.* (1991) presents a method that considers both the comprehensive lowering and rising of the ground water table and the layering of the soil. The method involves both a mathematical model and laboratory testing. The laboratory tests were utilized to study the soil deformation characteristics and provide soil parameters for the mathematical model. Then the computed results obtained were compared with field data measured in Shanghai, China. Two types of tests were conducted to determine the consolidation process of the soil under different loading conditions; namely consolidation tests under pumping

and cyclic consolidation tests. The former included consolidation tests where water was pumped out of the lower boundary of the soil test specimens and conventional consolidation tests with double drainage. The purpose of this test was only to validate the theory of superposition. The latter is called a cyclic consolidation test due to the fact that it simulates the process of loading and unloading with low frequencies to simulate the yearly water-level fluctuation. In this test, the one-dimensional consolidation equation is implied as shown below:

$$\frac{\partial p}{\partial t} = c_v \frac{\partial^2 p}{\partial z^2} + \frac{\partial \sigma^*}{\partial t} \quad (2.6)$$

where p is excess pore water pressure; t is the calculated period of time; z is soil thickness; σ^* is stress caused by loading or boundary water level change; c_v is the average coefficient of consolidation during cyclic loading.

Depending on whether the test loading boundary condition involves lowering or rising of the boundary water level, c_v in equation (2.6) will become the coefficient of consolidation c_{vc} or the coefficient of rebound c_{vs} . Excess pore water pressure can be computed by applying the appropriate boundary condition for the water level into equation (2.6). There are three boundary conditions for water levels that can be considered, namely: (1) fixed drawdown p_1 ; (2) linear rising water table p_2 ; (3) cyclic change in the water table p_3 . Solving the one-dimensional equation above for each boundary condition gives the excess pore pressure for each different case. Details of the mathematical equations for the three excess pore pressure conditions can be found in Gu *et al.* (1991). The total excess pore pressure is then equal to the summation of the three corresponding components of excess pore pressure:

$$p = p_1 + p_2 + p_3 \quad (2.7)$$

The pore pressure changes are then substituted into the equation for the relationship between the pore pressure changes and the soil deformation that is shown below:

$$S_i = m_v \int_{H_{LB}}^{H_{UB}} \Delta p \, dz \quad (2.8)$$

where S_i is deformation of the soil layer; H_{UB} and H_{LB} are the depths of the upper and lower boundaries of the compressible soil layer considered; m_v is coefficient of

volume change. The value of m_v can be coefficient of volume compression m_{vc} or coefficient of volume rebound depending on whether the lowering or the rising of the water table is considered. Corresponding to the three water level boundary components above there are also three subsidence components: (1) a fixed drawdown component S_1 ; (2) a linear rising component S_2 ; (3) a cyclic change component S_3 . These three equations for S_1 , S_2 , and S_3 are clearly explained and given in the paper by Gu *et al.* (1991).

Finally subsidence due to the fluctuation of the water table is computed by the summation of all three settlement components:

$$S_T = S_1 + S_2 + S_3 \quad (2.9)$$

Rothenburg *et al.* (1995) presented a method of computing the ground movements in both the vertical and horizontal directions in order to explain the formation of fissures as a result of a large-scale water extraction in the Sarir agriculture area in the Libyan Desert. According to Rothenburg *et al.* (1985) any reduction in pore pressure at a point in the ground will cause both vertical and horizontal components of soil displacement. Often analyses neglect horizontal ground movements and strains altogether in order to make the calculation simple. In this method, the drawdown of the water table is supposed to be known already. Strains resulting from water table decline can be solved by the theory of a nucleus of volumetric strain for a poro-elastic half space. In fact only specific patterns of water level decline corresponding to regional decline or drawdown around pumped wells have been considered in computing the horizontal ground displacements. The specific patterns of water level decline can be a uniform pressure decline of magnitude Δp within a rectangular or a circular area. It can also be a "Bell" shaped pattern simulating a regional drawdown or a transient cone of depression around a well. The method is capable of calculating the soil movements for both a rectangular and a circular area. However, an analytical method can only be used to evaluate the ground movement due to a uniform pattern of pore pressure decline over a rectangular area. The ground movements resulting from an axi-symmetric pattern can only be calculated numerically. Hence, in this part, only analytical equations that are employed to evaluate the ground movements due to a uniform pattern of pore pressure decline over a rectangular area are presented. The

details of the numerical equations can be found in Rothenburg *et al.* (1995). The process of calculation is as follows.

Firstly, the area of integration is subdivided into thin horizontal slices of thickness ∂h and located at depth D . Further, the spatial extent of each slice must be subdivided into elementary rectangles of size chosen in such a way that the variation of pore pressure decline can be neglected on the scale of an elementary area. Ground movements can be calculated as sums of contributions of strains from each area using the following equations:

$$\begin{aligned}\delta u_z(X, Y) &= \sum_i \delta_v^i \frac{1-\nu}{\pi} \int_{A_i} \frac{D dx dy}{[(X-x)^2 + (Y-y)^2 + D^2]^{3/2}} \\ \delta u_x(X, Y) &= \sum_i \delta_v^i \frac{1-\nu}{\pi} \int_{A_i} \frac{(X-x) dx dy}{[(X-x)^2 + (Y-y)^2 + D^2]^{3/2}}\end{aligned}\quad (2.10)$$

where $\delta_v^i = c_m \Delta p^i \partial h$ is the vertical compression assuming the condition of no lateral strain; c_m is the coefficient of uniaxial compression and ν is Poisson's ratio; x, y, z are the coordinates of a point where the deflections are calculated; X, Y are the two dimensions in the x and y directions respectively of the pore pressure decline area; D is the depth of the area where the pore pressure decline occurs.

Supposing that there is a uniform pore pressure decline over a rectangular area, the ground displacements of a slice of a small thickness ∂h , dimensions $2L_x, 2L_y$ and centred at $x = 0, y = 0$ at depth D can be acquired by integrating equation (2.10) over the area as follows:

$$\begin{aligned}\delta u_z(X, Y) &= \bar{\delta}_v \frac{1-\nu}{\pi} \alpha \tan \left[\frac{xy}{D \sqrt{x^2 + y^2 + D^2}} \right] \Bigg|_{X-L_x, Y-L_y}^{X+L_x, Y+L_y} \\ \delta u_x(X, Y) &= \bar{\delta}_v \frac{1-\nu}{\pi} \log \left[\frac{y + \sqrt{x^2 + y^2 + D^2}}{D} \right] \Bigg|_{X-L_x, Y-L_y}^{X+L_x, Y+L_y}\end{aligned}\quad (2.11)$$

where $\bar{\delta}_v$ is equal to $c_m \Delta p \partial h$ and the above operation of double substitution means the following.

$$U(x, y) \Bigg|_{X-L_x, Y-L_y}^{X+L_x, Y+L_y} = U(X+L_x, Y+L_y) - U(X-L_x, Y+L_y) - U(X-L_x, Y-L_y) + U(X+L_x, Y-L_y) \quad (2.12)$$

Figure 2.4 shows the vertical and horizontal displacements due to a uniform pore pressure decline within the rectangular and the circular area. Figure 2.5 illustrates the vertical and horizontal ground movements due to a bell-shape pore pressure decline and pore pressure decline near a well (using numerical calculation instead of analytical analysis). The horizontal displacements can be differentiated numerically to obtain horizontal ground strains to identify locations of tension zones and peak ground strains. From the location between tension and compression zones the fissure location can be figured out. A comparison between theory and the measured data at the Sarir agriculture area in the Libyan Desert was also conducted. The estimation of fissure locations by using this method gave a reasonable result compared with the field data.

Su *et al.* (1991) presented a method for the prediction of land subsidence in Shanghai, China. The method was concerned with creep of the clay of the Shanghai shallow layer. A solution is obtained to the one-dimensional problem of the settlement of a clay layer due to a limit variation of the water table. This solution is based on the assumption that the material is elastic. Based on Terzaghi's one-dimensional consolidation theory, the deformation of an elastic soil layer caused by linear variation of the central water level line of the layer derived by Qian and Gu (1981) is given by the following equation:

$$S(t) = \alpha m_v \left\{ \frac{1}{2H} (H_2^2 - H_1^2)t - \frac{1}{24C_v H} (2H^2 H_2^2 - 2H^2 H_1^2 - H_2^4 + H_1^4) + 2 \sum_{n=1}^{\infty} (-1)^n \frac{H^3 [\cos(n\alpha)(H_2/H) - \cos(n\alpha)(H_1/H)]}{(n\alpha)^4 C_v} e^{-(n\alpha)^2 C_v t / H^2} \right\} \quad (2.13)$$

where α is the slope of the variation of water level with time curve; C_v is the coefficient of consolidation; m_v is the coefficient of volume compressibility; k is the coefficient of permeability; and H , H_1 , H_2 are defined in Figure 2.6.

An elastic-viscoelastic analysis is then used to solve the viscoelastic problem. This involves applying a Laplace transform to the stress-strain behaviour of the viscoelastic material. On doing this, the transformed modulus may be obtained and put in to the solution for the elastic material, equation (2.13). On inverting the Laplace transform, a solution in real time is obtained.

$$S(t) = \alpha \left\{ \frac{1}{2H} (H_2^2 - H_1^2) L^{-1} \frac{1}{E_s s^2} - \frac{1}{24k E_s H} (2H^2 H_2^2 - 2H^2 H_1^2 - H_2^4 + H_1^4) L^{-1} \frac{1}{E_s^2 s} \right. \\ \left. + 2 \sum_{n=1}^{\infty} (-1)^n \frac{H^3 \gamma_w [\cos(nx)(H_2/H) - \cos(nx)(H_1/H)]}{(nx)^4 k} L^{-1} \frac{1}{E_s^2 (s+N)} \right\} \quad (2.14)$$

where

$$L^{-1} \frac{1}{E_s s^2}, \quad L^{-1} \frac{1}{E_s^2 s}, \quad L^{-1} \frac{1}{E_s^2 (s+N)} \quad (2.15)$$

are expressions for Laplace inverse transformation; E_s is elastic modulus; γ_w is the unit weight of water; $N = (n\alpha_2)k E_s / (H^2 \gamma_w)$.

This method was applied by Shu *et al.* (1991) to the Shanghai clay for a rising and falling water table with good results.

Lee *et al.* (1991) suggested a transform method that may be used to calculate land subsidence due to pumping ground water from an isolated well in a layered soil. The layered soil system consists of three horizontal layers with an aquitard sandwiched between two aquifers. All layers are infinite in radial extent. The governing equation of one-dimensional consolidation can be dealt with by a Lagrangian approach introduced by Cooper (1966) with revisions by Gambolati (1973 a, b) and Gibson *et al.* (1967). Fluid flows are considered vertical in the aquitard and horizontal in the aquifer. The drawdown of the water table and the soil deformation are non-linearly coupled. In addition to ground subsidence, two types of transient pumping schedules (intermittent pumping or cyclic pumping and recharging) are discussed and compared.

Generally, analytical methods have some limitations:

- Analytical methods often only allow computation of the soil deformation caused by the fluctuation of the water table that is supposed to have been measured or is known. They do not allow calculation of the position of the water table due to pumping.
- In many cases, only the one-dimensional consolidation equation has been used. Hence the problem has been simplified as only the vertical soil deformation due to the change in pore pressure has been taken into account with the horizontal soil

movements being neglected. Rothenburg *et al.* (1995) included into their model the horizontal soil deformation due to the depression of pore water pressure in order to explain the fissures at the Sarir agriculture area in the Libyan Desert. However, for the reason that the problems are too complicated to solve without the support of numerical methods, only the simplest form of the uniform pore water pressure depression over a rectangular area was analysed using analytical methods to work out the soil displacements in both directions.

- Though the analytical methods given here are coupled methods, the process of the fluctuations of the water table and soil deformations are still treated separately. So the superposition theorem has been used to obtain the final results.

2.4 TRANSFORM METHODS

Transform methods are those methods that employ transform techniques to simplify problems of land subsidence due to ground water extraction. It is clear that with the aid of the transform techniques more complicated problems can be solved. Transform methods allow three-dimensional behaviour, time dependent behaviour, and some other factors to be considered in land subsidence analysis.

Booker *et al.* (1985) developed a transform method called a point sink analysis. Homogeneous, anisotropic permeability and saturated poro-elastic materials are considered in the point sink method. The rate and the magnitude of the surface settlement are of interest. Pumping is supposed not to lower the water table but to just reduce the pressure in the ground water around the vicinity of the sink point. The three-dimensional consolidation equations have been used as the governing equations. The governing equations for volume strain and pore pressure are as shown below:

$$-\nabla^T \mathbf{v} + q = 0 \quad (2.16)$$

$$\frac{k_H}{\gamma_w} \left(\frac{\partial^2 p}{\partial x^2} + \frac{\partial^2 p}{\partial y^2} \right) + \frac{k_V}{\gamma_w} \frac{\partial^2 p}{\partial z^2} - q = 0 \quad (2.17)$$

As long as the steady state has been reached, the solution for the excess pore pressure distribution is as follows:

$$p = -\frac{F\gamma_w}{4\pi k} \left[\frac{1}{\sqrt{(r^2 + \psi^2 + (z + h_{ps})^2)}} - \frac{1}{\sqrt{(r^2 + \psi^2 + (z - h_{ps})^2)}} \right] \quad (2.18)$$

where, $\psi^2 = k_H/k_V$

The solution for surface settlement in this case is calculated from the formula below:

$$u_z = \frac{F\gamma_w}{2\pi k_V (\lambda + G)(\psi^2 - 1)} \ln \left[\frac{\psi h_{ps} + \sqrt{(r^2 + \psi^2 h_{ps}^2)}}{h_{ps} + \sqrt{(r^2 + h_{ps}^2)}} \right] \quad (2.19)$$

Nukulchai and Chau (1990) developed similar methods for a point sink based on the same assumption as the solution of Booker *et al.* (1985). The difference is that the governing equation of this method is based on the three-dimensional Biot's theory in cylindrical coordinates. By adopting the Mc-Namee Gibson displacement functions E_d and S_d , the governing equations for deformation and fluid flow may be written as two uncoupled differential equations:

$$U_r = -\frac{\partial E_d}{\partial r} + z \frac{\partial S_d}{\partial r} \quad U_z = -\frac{\partial E_d}{\partial z} + z \frac{\partial S_d}{\partial z} - S_d \quad p = -2G \left[\frac{\partial S_d}{\partial z} - \eta \nabla^2 E_d \right] \quad (2.20)$$

where η = auxiliary of elastic constant; E_d and S_d are the McNamee-Gibson displacement functions.

These equations are simplified by applying a Hankel transform (to simplify the equations spatially) and a Laplace transform with respect to time. The transform displacements and excess pore pressures may then be found but need to be inverted by applying numerical inversion techniques to the Hankel and Laplace transforms. The solution is obtained for a point source applied as a step function, and if the solution for the surface displacement is $A(t)$, then the solution for the settlement of the ground surface, $U_z(t)$, due to the fluid extraction at a point sink with variable strength, $q(t)$, is obtained through the use of a convolution integral:

$$U_z(t) = A(t)q(0) + \int_0^t A(t-\tau) \frac{dq(\tau)}{d\tau} d\tau \quad (2.21)$$

where $A(t)$ is response due to the unit pumping function.

The comparisons of this method with the method of Booker and Carter (1986, 1987) are illustrated in Figure 2.7 and Figures 2.8a, and b. Figure 2.7 shows the surface settlement profile along a radial direction due to a point sink at depth h . the settlement profile and the distribution of excess pore pressure along the vertical axis passing through the applied point sink at various time laps are given in Figures 2.8a, b. Both methods gave good agreement.

Tarn and Lu (1991) again have used the point sink method. In this method, the soil mass is modelled as an anisotropic, saturated half space. However not only was the permeability of the soil assumed to be anisotropic as in Booker *et al.* (1985) but also the elastic properties were assumed to be anisotropic. Both cases of a pervious and an impervious half space boundary were investigated.

In practice water is usually pumped from an aquifer of thickness t at depth h below the surface. The solution to this problem involves pumping the ground water from a layered soil profile. Small and Booker (1984) developed a fully coupled method based on finite layer theory and applied it to an elastic soil that has reached a steady state (i.e. the water pressures have reached final steady state values). In this method most assumptions are similar to the assumptions in Booker *et al.* (1985) except the soil is regarded as layered. Two cases can be considered in this method, namely, plain strain and axial-symmetry.

In the case of plain strain, Fourier transforms are used to simplify the governing equations. The relationships between the transformed displacements U_r , U_z , pore pressure P , and normal N and shear T stress at the interface of each layer are related as shown below:

$$\begin{bmatrix} U_{zp} \\ U_{xp} \\ -U_{zm} \\ -U_{xm} \end{bmatrix} = \frac{(1-\nu^*)}{E^*} \begin{bmatrix} P_p \\ \frac{1}{\alpha} \left(\frac{\partial P}{\partial z} \right)_p \\ -P_m \\ -\frac{1}{\alpha} \left(\frac{\partial P}{\partial z} \right)_m \end{bmatrix} + \frac{1}{\alpha} \begin{bmatrix} F_{11} & F_{12} & F_{13} & F_{14} \\ & F_{22} & F_{23} & F_{24} \\ \text{symm} & & F_{33} & F_{34} \\ & & & F_{44} \end{bmatrix} \begin{bmatrix} N_p \\ T_p \\ N_m \\ T_m \end{bmatrix} \quad (2.22)$$

By applying equations of continuity at the interface between each layer, the equations for each layer (2.22) can be assembled for all layers and then solved for the entire layered system. Hence the transformed displacements and excess pore pressures can

be found at each layer interface. An inverse Fourier transform can be numerically carried out to obtain the actual displacements and pore pressures.

In the case of axially symmetric problems, a Hankel transform is used to simplify the governing equations. The process of assembling the equations for each layer is the same as for two-dimensional problems.

Sepulveda (1991) suggested a pseudospectral method that was based on the known drawdown of the water table $s(t)$. Because the changes of hydraulic parameters (such as hydraulic conductivity, specific storage) due to reduction in void ratio are taken into account, the method can be considered as a one-dimensional non-linear method. The governing equation that is based upon the void ratio and effective stress relationship is the one-dimensional consolidation equation (Gibson *et al.* 1967):

$$\frac{\partial e}{\partial t} = \left(\frac{\rho_s}{\rho_f} - 1 \right) \frac{\partial}{\partial z} \left(\frac{k_z(e)}{(1+e)} \right) + \frac{\ln 10}{g C_c \rho_f} \frac{\partial}{\partial z} \left(\frac{k_z(e) \sigma_i}{(1+e)} \frac{\partial e}{\partial z} \right) \quad (2.23)$$

where e is void ratio; ρ_f and ρ_s are fluid density and soil grain density; k_z is hydraulic conductivity in the z direction that is a function of void ratio; g is acceleration due to gravity; and C_c is the compression index of the soil; and σ_i is effective stress.

As the hydraulic conductivity is a function of void ratio, any change in the void ratio can result in changes of permeability. This method utilizes the assumption of linearity between the changes in void ratio and the changes in the logarithm of effective stress that is observed in laboratory results (Terzaghi and Peck, 1967). An expression for effective stress σ_i as a function of depth z and time t is derived in Sepulveda and Zack (1990).

The equation above can be solved by applying a Fourier transform to spatial derivatives and using a fourth order Runge-Kutta scheme for time derivatives to obtain the final result for the change in void ratio which in turn results in the compression and ultimate compaction of a granular medium. At the present time, this method can only simulate compression or expansion of an individual layer and a constant value of compression index C_c . However, this method is capable of expansion to the problem of a soil layer with a variable soil compression index C_c .

Miao and Wu (1991) presented a method that included the secondary consolidation due to the viscous properties of aquifers and aquitards in the calculation of land

subsidence. They solved the problem of an aquitard lying between an upper non-confined aquifer and a lower confined aquifer that is pumped. Flow was assumed to be radial in the aquifers and vertical only in the aquitard.

Again, Hankel and Laplace transforms were applied to the governing equations to allow solution of the problem, and exact solutions are found for the settlement at any position and time in terms of infinite integrals. This method was applied to computing the land subsidence resulting from heavy pumping of ground water in Shanghai, China.

Transform techniques offer some advantages in the calculation of land subsidence due to ground water extraction.

- Non linear deformations may be considered as described in Sepulveda (1991); Miao and Wu (1991)
- The three-dimensional consolidation equation may be considered as in Booker *et al.* (1985); and Nukulchai and Chau (1990);
- Soil models such as the multi-layered systems considered by Small and Booker (1984) or the three layer systems (including an aquitard sandwiched between two aquifers-unconfined aquifer or confined aquifer) in Miao and Wu (1991) may be considered
- The fluctuation of the ground water level (drawdown, linear rising, cyclic fluctuation) due to pumping of the ground water can be considered as in the work of Miao and Wu (1991), and Sepulveda (1991).

Though transform methods offer advantages over analytical methods they still have some restrictions:

- Although the transform methods can deal with three-dimensional consolidation equations (Booker *et al.* 1985; and Nukulchai and Chau, 1990), the method has not been applied to problems where there is drawdown of the water table.
- Apart from the multiple-layered soil system analysed in Small and Booker (1984), most soil systems analysed are fairly simple such as the homogeneous soil model in Booker *et al.* (1985); and the three layer system (an aquitard sandwiched between two confined or unconfined aquifers).

- Transform methods require the soil to be horizontally layered or uniform, so that Fourier or Hankel transforms can be used to simplify the governing equations. Fortunately most sedimentary soils are horizontally layered, but in some cases they may not be. Using Laplace transforms for the time component also makes it more difficult to solve problems where pumping rates change.

2.5 NUMERICAL METHODS

Significant development of computer power over the last two decades has made numerical methods a powerful tool in solving subsidence problems. Though numerical methods can be time consuming and expensive they are rapidly growing in popularity because they can be applied to more general problems. They overcome the limitations of other methods such as analytical and transform methods.

The problems of land subsidence due to ground water extraction have been solved in two ways.

2.5.1 UNCOUPLED METHODS

In this case the fluid flow and the soil deformation problems are solved separately. Usually the fluid flow problem is solved in three or two-dimensions. Then the water pressure head changes are used to compute the soil deformation.

Kashef and Chang (1976) developed a numerical solution using a finite difference method to determine the land subsidence due to ground water pumping from an unconfined aquifer underlying an aquitard. Figure 2.9 shows a leaky unconfined homogeneous and isotropic aquifer of infinite lateral extent underlain by a homogeneous and isotropic aquitard with constant storage coefficient. The pumped well completely penetrates through the aquifer and discharges at a constant rate Q . The total settlement at any time t due to ground water pumping at a certain point is expressed in Chang (1976). The subsidence of the land surface due to ground water pumping from a system of multiple wells can be obtained by applying a modified procedure of superposition for gravity wells that was developed by Kashef (1970). The equation of total settlement at a point (x, y) at any time t due to ground water pumping from a multiple well system is as follows:

$$u_{(x,y,t)} = \left\{ \left(\frac{S_u}{B_u \gamma_w} \right) - S_y \beta_o \right\} \sum_{i=1}^n \int_0^{B_u} (d_{u,i} - s_y d_{f,i}) \gamma_w dz + \left\{ \left(\frac{S_c}{B_c \gamma_w} \right) - m_2 \beta_o \right\} \sum_{i=1}^n \int_0^{B_c} (d_{c,i} - s_y d_{f,i}) \gamma_w dz \quad (2.24)$$

where u_t is settlement at time t ; B_u is the initial thickness of the saturated zone (aquifer); B_c is thickness of the aquitard; d_c and d_u are respectively change in total water head at any point in the aquitard and aquifer; d_f is drawdown of the water table; S_c and S_u are respectively storage coefficients of the aquitard and aquifer; s_y is specific yield of the aquifer; β_o is water compressibility; and γ_w is unit weight of water.

The formulated mathematical model with moving free surface given above (2.24) is solved numerically by the “Alternating Direction Implicit (ADI)” method in dimensionless form. The total land subsidence resulting from the compressibility of both the aquifer and the aquitard is determined in the vertical direction by a time-dependent summation procedure based on the pressure head diagram along a vertical section.

One-dimensional settlement of an aquifer system near Pixley, California was simulated by Helm (1975, 1976). He used the finite different method to solve the non-linear one-dimensional governing equations that were derived from the consolidation theory of Terzaghi.

Kamata *et al.* (1976) used the finite difference technique in solving the two-dimensional flow field in the vertical plane (X-Z plane). In this method the hydrogeological profile is divided into square unit areas neglecting water flow in the y direction. The solution of the water balance equation at the unit cell is obtained by the ADI finite difference method that was devised by Peaceman and Rachford (1955). Then the compression of a unit cell (one-dimensional) was calculated by the following equation:

$$\Delta m_i = m_v \gamma_w \Delta Z_i \Delta h_i \quad (2.25)$$

where Δm is the compression of each unit cell; ΔZ is the vertical length of each unit cell; γ_w is unit weight of water; Δh is change of ground water head at each cell; m_v is coefficient of volume compressibility; i denotes the cell number in the vertical direction (column).

Figure 2.10 illustrates a unit cubic cell where the ground water is assumed to be in balance. The cumulative compression in each column, which corresponds to the settlement of land subsidence S_T , is expressed as:

$$S_T = \sum (m_v \cdot \gamma_w \cdot \Delta h_i \cdot \Delta z_i) \quad (2.26)$$

Gambolati and Freeze (1973) suggested a method of analyzing land subsidence in a complex aquifer-aquitard system. Firstly the regions of hydraulic head drawdown are calculated in a two-dimensional vertical cross section using radial coordinates. The governing water flow equation is presented as follows:

$$\frac{\partial}{\partial r} \left(K_{ri} \frac{\partial h}{\partial r} \right) + \frac{K_{ri}}{r} \frac{\partial h}{\partial r} + \frac{\partial}{\partial z} \left(K_{zi} \frac{\partial h}{\partial z} \right) = \gamma_w (\alpha_i + n_i \beta_0) \frac{\partial h}{\partial t} \quad (2.27)$$

where K_{ri} , K_{zi} , α_i , and n_i are the parameters of the i^{th} formation, whether it be aquifer or aquitard.

The computations are carried out with a model based on the diffusion equation and solved with a numerical finite element technique. The calculated head values in the aquifers are then used as time dependent boundary conditions in a set of one-dimensional vertical consolidation models solved with a finite difference technique and applied to a more refined representation of each aquitard. The one-dimensional consolidation equation is as follows:

$$\frac{\partial}{\partial z} \left(K_{zj} \frac{\partial h}{\partial z} \right) = \gamma_w (\alpha_j + n_j \beta_0) \frac{\partial h}{\partial t} \quad (2.28)$$

where K_{zj} , α_j , and n_j are the formation parameters for the j^{th} layer.

Leake (1991) presented a computer program that incorporated calculation based on Terzaghi's one-dimensional theory of vertical consolidation into the modular finite difference ground water flow model (MODFLOW) by McDonald and Harbaugh (1988). MODFLOW is a finite difference program solving ground water flow in three-dimensions and the equation governing flow is given below:

$$\frac{\partial}{\partial x} \left(K_{xx} \frac{\partial h}{\partial x} \right) + \frac{\partial}{\partial y} \left(K_{yy} \frac{\partial h}{\partial y} \right) + \frac{\partial}{\partial z} \left(K_{zz} \frac{\partial h}{\partial z} \right) - W = S_u \frac{\partial h}{\partial t} \quad (2.29)$$

where x , y , z are Cartesian coordinates, K_{xx} , K_{yy} , and K_{zz} are principal components of hydraulic conductivity tensor; h is hydraulic head; W is volumetric flux per unit volume of source or sinks of water; S_u is specific storage of the aquifer; and t is time.

The program accounts for ground water changes and consolidation in discontinuous interbeds or in extensive confining beds. An extension has been put into the MODFLOW program to simulate the relationship between the soil compression and its storage. Figure 2.11 shows a system of discontinuous interbeds and extensive confining beds.

Bravo *et al.* (1991) used the same method as Leake (1991). However the one-dimensional consolidation equation is different.

Sato *et al.* (1986) used three-dimensional and one-dimensional finite element models to predict land subsidence due to the withdrawal of ground water in the Nobi Plain, Japan. The three-dimensional model was used to estimate the relation between the withdrawal of ground water and the piezometric heads of confined aquifers. The one-dimensional model was used to calculate the land subsidence in relation to changes of piezometric heads in confined aquifers.

Saxena and Mohan (1978) developed a finite element program to estimate the potential for subsidence at an industrial site, which is located in the Southern part of the State of Florida. The program can calculate the flow of ground water in three dimensions. For calculating settlement due to the change in the ground water head a two-dimensional finite element computer program capable of handling linear elastic material was used. The program can consider axi-symmetric and plane strain problems.

A quasi 3-D model was adopted for calculating the flow of ground water. This model solves the problem of ground water flow in an aquifer under the condition of horizontal two-dimensional flow and vertical one-dimensional flow in the confining layer.

Harada and Yamanouchi (1983) used a quasi three-dimensional model to solve the ground water flow problem. The flow of ground water in an aquifer can be treated horizontally in the x - y Cartesian coordinate plan:

$$\frac{\partial}{\partial x} \left(T_{xi} \frac{\partial h_i}{\partial x} \right) + \frac{\partial}{\partial y} \left(T_{yi} \frac{\partial h_i}{\partial y} \right) + W_{ri} = S_i \frac{\partial h_i}{\partial t} \quad (2.30)$$

where h = ground water head; T = transmissivity; S = storage capacity and storativity (no dimension); x, y = orthogonal coordinates in the horizontal plain; t = time; i indicates the i^{th} aquifer from the top surface; and W_r = recharge capacity or yield capacity. W_r can be divided into two components as presented below:

$$W_{ri} = Q_{(c-a)_i} - Q_{(a-c)_{i+1}} - Q_{ai} \quad (2.31)$$

where $Q_{(c-a)_i}$ is ground water flow quantity from the i^{th} confining layer to the i^{th} aquifer; $Q_{(a-c)_{i+1}}$ is the ground water flow quantity from the i^{th} aquifer to the $(i+1)^{th}$ confining layer; Q_{ai} is yield quality from the i^{th} aquifer.

The vertical ground water flow in the i^{th} confining layer is expressed as:

$$\frac{\partial}{\partial z} \left(k_i \frac{\partial h'_i}{\partial z} \right) = S_{si} \frac{\partial h'_i}{\partial t} \quad k_i \frac{\partial h'_i}{\partial z} = Q_{(a-c)_{i-1}} (z = b_i) \quad k'_i \frac{\partial h'_i}{\partial z} = Q_{(c-a)_i} (z = 0) \quad (2.32)$$

where h' = head in confining layer; k = coefficient of permeability of confining layer; S_s = specific storage; b = thickness of confining layer.

Equation (2-32) has been solved by employing an analytical method (Bredehoeft and Pinder, 1970), by finite difference methods (Kamata *et al.*, 1973), or by means of finite element methods (Fujinawa, 1977a,b, Fujisaki *et al.*, 1979). To simplify the analysis that was used to calculate subsidence in the Saga plain, Japan, Harada and Yamanochi (1983) solved equation (2-32) by combining the model developed by Fujisaki *et al.* (1979) with a deformation submodel of the ground layer. Figure 2.12 shows a schematic drawing of the quasi-three-dimensional model with a deformation submodel incorporated.

Using a linear subsidence model in which the coefficient of volume compressibility does not change with head (pressure) at the time of depression or rise, the subsidence can be calculated by the following equation, adopting the ground water head determined from the equations above:

At the time of head depression: $u_z = H m_v \Delta h$

At the time of head rise: $u_z = H m_v \Delta h$ aquifer

$u_z = 0$ confining layer

where: u_z = amount of ground subsidence and H = thickness of deforming layer.

Teatini *et al.* (1995) adopted the same Quasi-3-D technique as Harada and Yamanochi (1983) did. The governing flow equation in the aquifer between aquitards is written as an equation for flow in the two horizontal dimensions and another equation for the vertical flow in the aquitard. Instead of using linear subsidence equations as Harada and Yamanochi (1983), the governing flow equations are non-linear in that the vertical permeability $K_z(n)$ is given as a function of porosity n . The specific storage $S_s(n, \sigma')$ is also related to n and to effective intergranular stress σ' . The empirical non-linear relationships between the vertical permeability and porosity n and effective stress σ' are given below:

$$K_z(n) = K_{z0} \left[\frac{n(1-n_0)}{n_0(1-n)} \right]^m \quad (2.33)$$

$$S_s(n, \sigma') = \gamma_w \left[0.434 C_c \frac{1-n}{\sigma'} + n \beta_0 \right] \quad (2.34)$$

$$n = n_0 - 0.434 C_c (1-n)^2 \frac{d\sigma'}{\sigma'} \quad (2.35)$$

where m is a material-dependent coefficient; γ_w and β_0 are the specific weight and the compressibility of water; C_c is compression index and it is replaced by C_s whenever unloading occurs ($\sigma' < \sigma'_{pre}$); n and n_0 are porosity and initial porosity of the soil respectively; and K_{z0} and K_z are respectively the initial hydraulic conductivity and hydraulic conductivity.

A three-dimensional mesh, which is shown in Figure 2.13, is used to solve the flow equation. For one-dimensional vertical consolidation, the model is a combination of Terzaghi's effective stress concept and an empirical constitutive relationship. Assuming that there is a total water head change Δh occurring in a time step Δt , the one-dimensional governing equation for the settlement of an individual layer is as follows:

$$u_z(\Delta t) = \sum_{j=1}^N \sum_{k=1}^M 0.434 \gamma_w (1-n) \frac{C_c}{\sigma'} \Delta h_{kj} \Delta b_{kj} \quad (2.36)$$

where N is the number of aquitards; M is the number of vertical elements into which each aquitard is discretized; Δb_{kj} is the thickness of the k^{th} finite element of the j^{th} aquitard.

The method was demonstrated to give good results in prediction of land subsidence due to ground water withdrawal in the Venice Lagoon, Italy.

2.5.2 COUPLED METHODS

Lewis and Schrefler (1978) used a fully coupled finite element method where land subsidence and excess pore pressures are calculated simultaneously. The governing equation incorporates the three-dimensional equation of fluid flow and the one-dimensional consolidation equation. The non-linear stress-strain relationship equation of the soil skeleton is given by:

$$d(\vec{\sigma} - \vec{m}p) = \bar{D}_T (d\vec{\epsilon} - d\vec{\epsilon}_p) \quad (2.37)$$

where $\vec{\sigma}$ is total stress tensor; p is pore pressure; $\vec{m} = [1 \ 1 \ 1 \ 0 \ 0 \ 0]^T$; $\bar{D}_T$ is effective stress tangent modulus matrix; $d\vec{\epsilon}$ is total strain increment vector; $d\vec{\epsilon}_p$ is strain increment vector due to pore pressure change.

The equation of the continuity of flow of the pore fluid is demonstrated as follows:

$$-\vec{\nabla}^T \vec{v} = -\text{rate of fluid accumulation} \quad (2.38)$$

where $\vec{v}$ is velocity vector and $\vec{\nabla}^T = [\partial/\partial x \ \partial/\partial y \ \partial/\partial z]$.

The equilibrium equation is then expressed in terms of the nodal displacement and pore pressures in finite element form. The continuity of flow, with the flow boundary conditions applied, is also expressed in finite element form. Then both equations of equilibrium and continuity, expressed in the finite element form, are coupled with a time-step scheme used to calculate the values of deformation and of changes in pore

water pressure. The program was successfully applied to matching past behaviour and for predictive simulation in two regions (Venice and the Polesine region) in Northern Italy.

Narasimhan and Witherspoon (1976) presented a method that is a combination of a general three-dimensional fluid flow field and one-dimensional deformation of a porous medium. This governing equation is solved by applying the Integrated Finite Difference Method (Narasimhan and Witherspoon, 1976a). The only limitation of this method is that it ignores lateral deformation.

Sandhu and Wilson (1969) presented a general finite element method that was originally based on the 3-D coupled consolidation theory of Biot. The coupled general finite element method can solve the problem of land movement due to ground water extraction in two or three dimensions with non-linear and time dependent material behaviour. The method permits complex geometry, arbitrary time varying boundary conditions, non-homogeneity as well as anisotropy to be considered. Viscoelasticity and creep, temperature effects, residual stresses, plastic behaviour also can be allowed for. The basic equations of deformation and water flow are respectively the equilibrium equation and the generalization of Darcy's law that are presented as:

The equilibrium equation:

$$\sigma_{ki,k} + \delta_{ik} \pi_k + \rho F_i = 0 \quad (2.39)$$

where σ_{ki} are the components of the symmetric stress tensor for the solid phase; π is the hydrostatic stress in the pore fluid; ρ the total mass density of the saturated material; F_i are the components of the body force; δ_{ik} is the Kronecker delta.

The equation of flow:

$$u_{i,i} + \int_0^t \left[K_{ij} (\pi_{,j} + \rho_2 F_j) \right]_{,i} dt = 0 \quad (2.40)$$

where u_i are the components of the displacement vector; e_{ii} are the components of the strain tensor, for the solid phase; K_{ij} are the components of the permeability tensor and ρ_2 is the mass density of the fluid, assumed to be incompressible.

The equation of equilibrium that is written in terms of three components of displacements and the equation of flow are sufficient for calculating deformations and fluid pressures. The finite element method that replaces the actual continuum by a

finite number of discrete sub-regions called elements is applied to solve these equations. Figure 2.14 illustrates some typical land subsidence problems that can be dealt with by using this coupled finite element method.

Hsi *et al.* (1994) developed a finite element method that is based on the three-dimensional consolidation theory of Biot and solved the equations by applying the Sandhu and Wison (1969) numerical approach for consolidation of a porous media. This fully coupled method of calculating the surface subsidence due to fluid withdrawal considers the three-dimensional consolidation of the ground, and transient unconfined seepage due to pumping of ground water.

The governing equations to the coupled problem that consider the drawdown of the water table due to pumping of ground water are given by:

$$\begin{bmatrix} K_{sm} & -\gamma_w L_c^T \\ -\gamma_w L_c & -\gamma_w (1-\alpha) \Delta t \Phi_f - \gamma_w G^{FS} \end{bmatrix} \begin{bmatrix} \Delta \delta^{(i)} \\ \Delta h^{*(i)} \end{bmatrix} = \xi \quad (2.41)$$

$$\xi = \begin{bmatrix} \Delta f(i) \\ \gamma_w \alpha \Delta t \Phi_f h_t^* + \gamma_w (1-\alpha) \Delta t \Phi_f h_{t+\Delta t}^{*(i-1)} \\ -\gamma_w L_c \delta t + \gamma_w L_c \delta_{t+\Delta t}^{(i-1)} \\ + \gamma_w G^{FS} (h_{t+\Delta t}^{*(i-1)} - h_t^*) + \gamma_w \Delta t Q \end{bmatrix}$$

where K_{sm} is the incremental stiffness matrix; L_c is a coupling matrix; Φ_f is the flow matrix; Δt is increment in time; G^{FS} is a term to allow for release of fluid when the free surface (water table) falls; h is total water head; Q is volume of water pumped; and γ_w is the unit weight of water.

The equations may be solved for each time step to give values of increment in deflection and total water head at each node in the finite element mesh. The details of this method as well as the derivation of the finite element equations are given by Hsi *et al.* (1994) and Hsi and Small (1992a, and b).

Safai and Pinder (1979, 1980) presented a finite element method based on Biot's consolidation theory. The numerical methods to solve the governing equation were first developed by Schiffman *et al.* (1969). The equations of fluid flow are based on three-dimensional theory and the equation of consolidation is for two dimensions. However, Safai and Pinder (1979, 1980) did not consider the variability of total stress in their equations.

Kim and Parizek (1999) used a Galerkin finite element method as the solution to the fully coupled three-dimensional governing equations. The governing equations are based on Biot's consolidation theory.

Bear and Corapcioglu (1981a,b) and Corapcioglu and Bear (1983) developed a fully coupled elastic three-dimensional mathematical model for regional land subsidence due to pumping of ground water. The mathematical model is based on the three-dimensional consolidation theory developed by Biot. A special case of radial flow to a single pumped well (and vertical and horizontal movements of the ground) has been presented by obtaining a closed form analytical solution (using a Laplace transformation). In all the other cases, a numerical solution is required. From the Bear and Corapcioglu (1981a,b) and Corapcioglu and Bear (1983) study, it is shown that the effect of drawdown of the water table in aquifers cannot be ignored although the magnitude of excess total stress is quite small in comparison to excess pore pressure.

A summary of the different numerical methods that have been used by various researchers is presented in Table 2.1.

2.6 SUMMARY AND CONCLUSIONS

The literature review has presented some approaches that have been used to predict land subsidence due to ground water withdrawal all around the world. Basically most approaches have employed two theories: (1) The theory of consolidation or time dependent deformation of the ground. (2) The theory of fluid flow.

The available methods for calculating land subsidence due to ground water extraction can be classified into the following methods: (1) Analytical methods; (2) Transform methods; (3) Numerical methods.

Because it is difficult to obtain a closed form solution to a three-dimensional boundary value problem, the analytical methods have generally only been used to calculate the vertical subsidence of the land surface by applying the one-dimensional consolidation equation. The horizontal movement of soil is often not considered. The fluctuation of the water table is often assumed a priori in the solution.

Transform methods are generally a combination of analytical and numerical methods as numerical integration is used to invert the Fourier, Hankel, or Laplace transforms.

Three-dimensional behaviour, non-linear behaviour, and time dependent behaviour can be considered in transform methods. Apart from these advantages, transform methods still encounter some restrictions. The drawdown of the water table cannot be computed with these methods and so this effect cannot be included. The point sink approaches (Booker and Carter (1986, 1987), Small and Booker (1984), Booker *et al.* (1985), Nukulchai and Chau (1990)) have only considered the pore pressure reduction in the soil with the level of the water table unchanged. They can be applied to cases where the water table is not lowered.

Numerical methods are the most powerful tool and allow solution of more complex problems. Numerical methods can be classified as two types: Uncoupled and coupled methods. Uncoupled methods solve the fluid flow equation separately and then the pore pressures are used to compute the deformation of the ground. The deformation models are often one-dimensional or sometimes two-dimensional (Saxena and Mohan (1978)). Coupled methods allow pore pressure changes and soil deformations to be computed simultaneously in both the saturated and unsaturated zones. Coupled methods take into consideration the effects of the pore pressure changes on the deformation of the soil and vice versa. Most coupled methods employ Biot's theory of consolidation and can be used to solve three-dimensional problems.

In general, the last method or the fully coupled approach (after Hsi *et al.* (1994)) will be applied to the particular case in Hanoi, Vietnam because it allows for coupled effects and the drawdown of the water table to be computed.

Table 2.1: Numerical models for calculation of land subsidence due to ground water extraction.

Developer (date)	Application	Flow model		Deformation model			Coupled or Uncoupled method
		Dimension	Numerical method	Dimension	Numerical method	Constitutive relationship	
Helm (1975 and 1976)	Multi layer system	-	-	1-D	FD	Terzaghi	Uncoupled
Kashef and Chang (1976)	Two layer system	2-D	FD	1-D	FD	Chang (1976)	Uncoupled
Kamata, Harada, and Nirei (1976)	Multi layer system	2-D	FD	1-D	FD	Compressibility	Uncoupled
Gambolati and Freeze (1973)	Multi layer system	2-D	FE	1-D	FD	Compressibility	Uncoupled
Leake (1991)	Multi layer system	3-D	FD	1-D	FE	Terzaghi	Uncoupled
Bravo, Rogers and Cleveland (1991)	Multi layer system	3-D	FD	1-D	FD	Compressibility	Uncoupled
Sato, Daito, and Ueshita (1984)	Multi layer system	3-D	FE	1-D	FE	Compressibility	Uncoupled
Saxena and Mohan (1978)	Multi layer system	3-D	FE	2-D	FE	Compressibility	Uncoupled
Villanueva (1984)	Multi layer system	Quasi 3-D	BE	1-D	-	Compressibility	Uncoupled
Harada and Ymanouchi (1983)	Multi layer system	Quasi 3-D	FE	1-D	FE	Compressibility	Uncoupled
Teatini, Gambolati, and (1995)	Multi layer system	Non-linear Quasi 3-D	FD	Non-linear 1-D	FE	Terzaghi	Uncoupled
Lewis and Schrefler (1978)	Multi layer system	3-D	FE	1-D	FE	Compressibility	Coupled
Narasimhan and Witherspoon (1976)	Multi layer system	3-D	FD	1-D	FD	Terzaghi	Coupled
Bear and Corapcioglu (1981a,b and 1983)	Multi layer system	3-D	-	3-D	-	Biot	Coupled
Safai and Finder (1980)	Multi layer system	2-D	FE	2-D	FE	Biot	Coupled
Kim and Parizek (1999)	Multi layer system	3-D	FE	3-D	FE	Biot	Coupled
Sandhu and Wilson (1969)	Multi layer system	2 or 3-D	FE	2 or 3-D	FE	Biot	Coupled
Hsi, Cater and Small (1994)	Multi layer system	2-D	FE	2-D	FE	Biot	Coupled

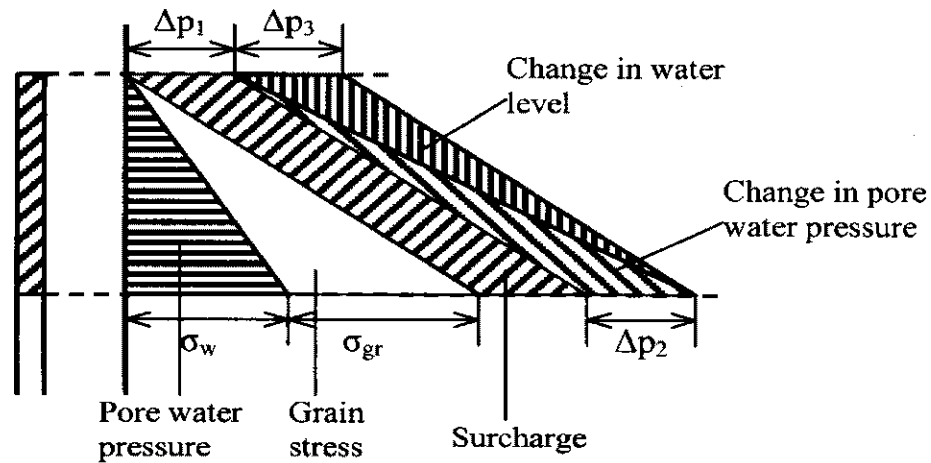


Figure 2.1: Pore water pressure distribution due to load or change in water level or change in pore pressure at the base (after Viergever 1991).

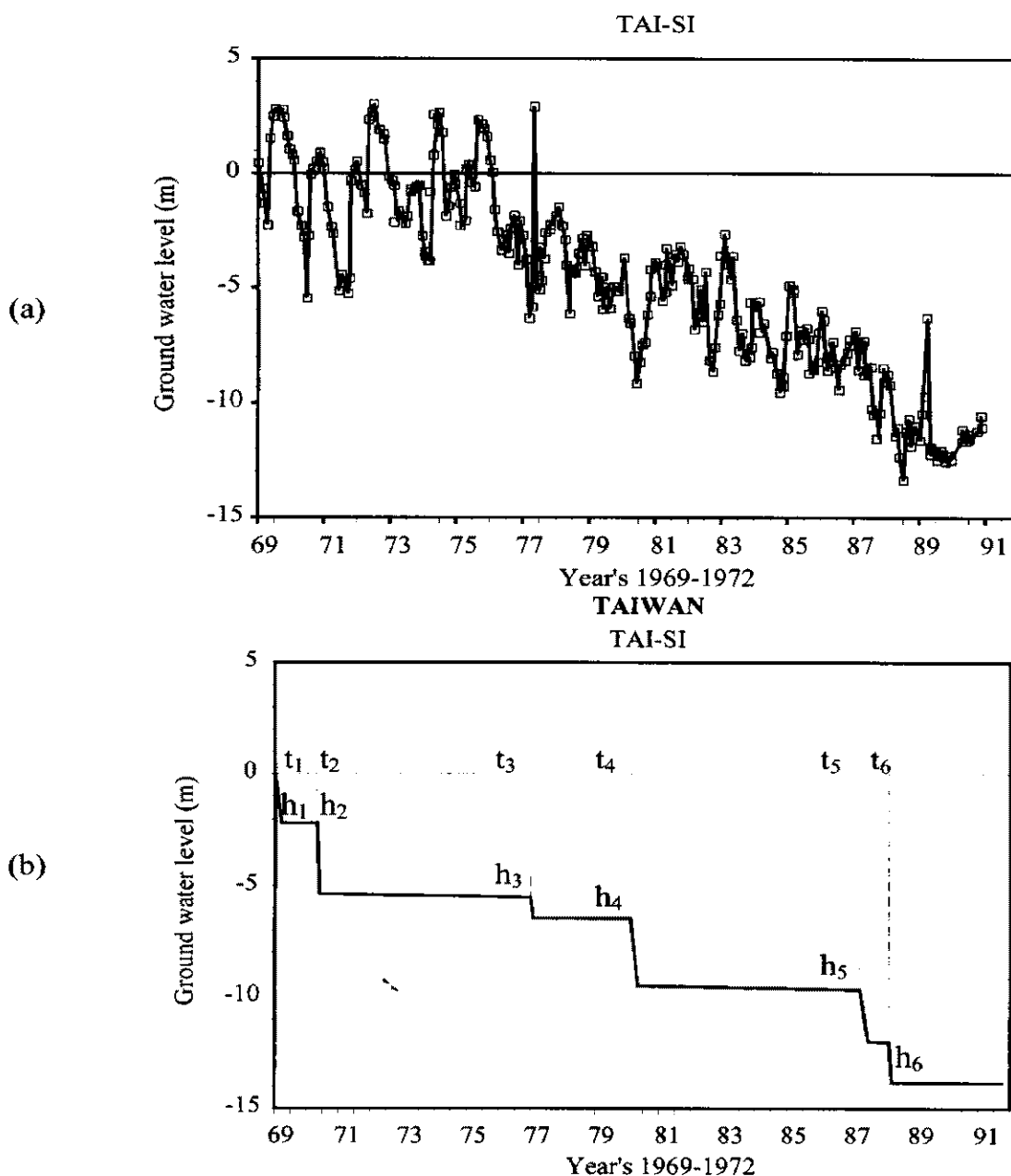


Figure 2.2: Water level variation with time (after Yaw and Jyh, 1995).

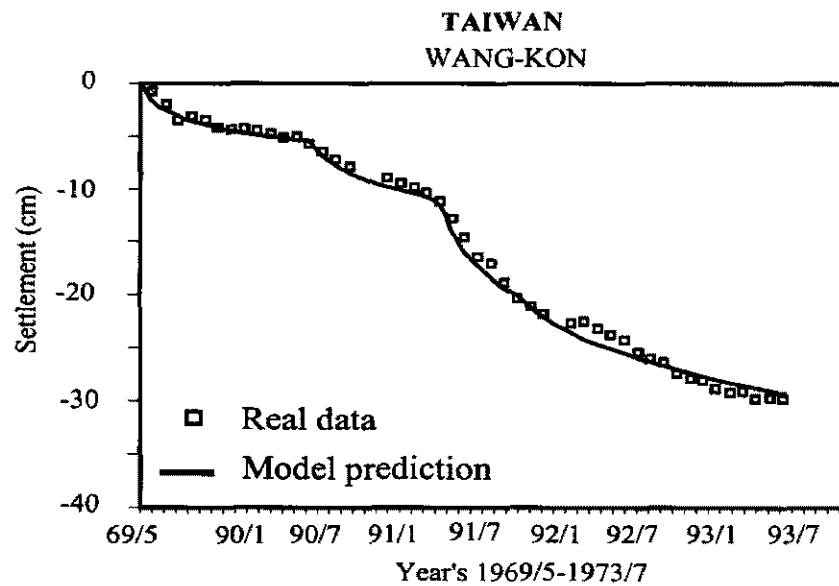


Figure 2.3: Total settlement at 44.5 - 125.5m depth in Wang-Kon (after Yaw and Jyh, 1995).

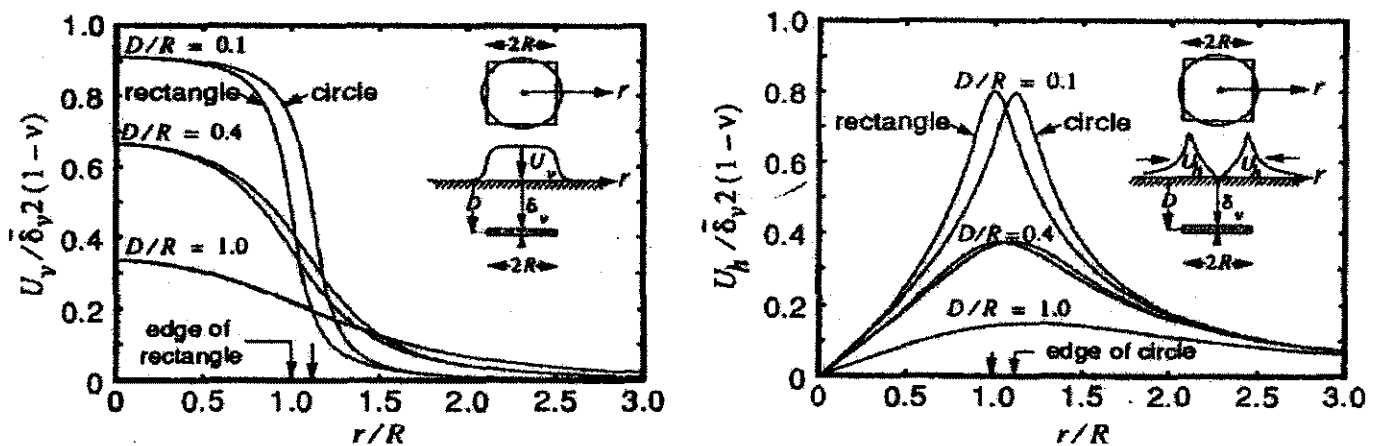


Figure 2.4: The vertical and horizontal displacements due to a uniform pore pressure decline within the rectangular and circular area (after Rothenburg *et al.*, 1995).

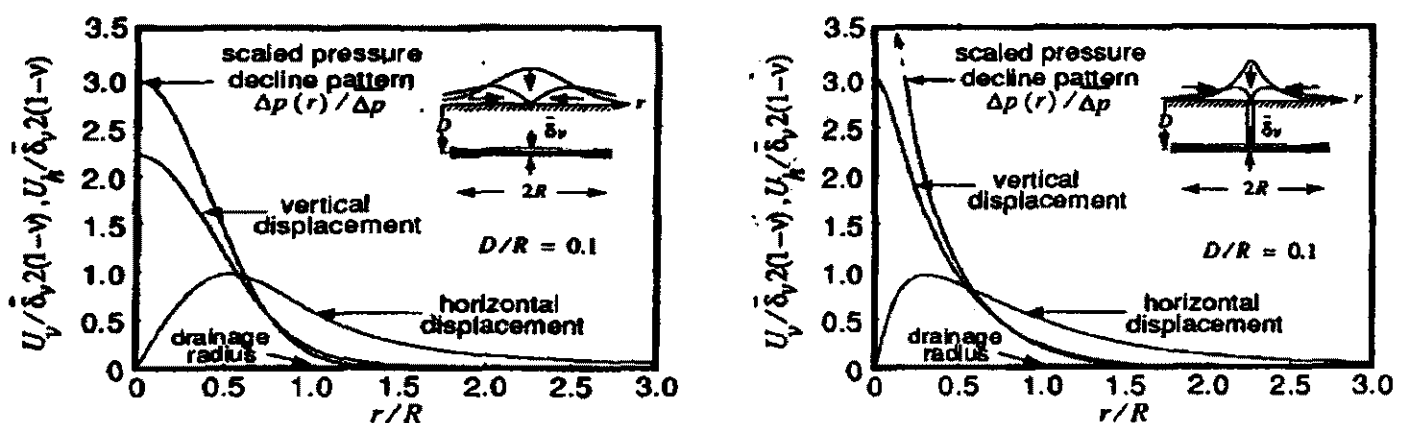


Figure 2.5: The vertical and horizontal ground movements due to a bell-shape pore pressure decline and pore pressure decline near a well (after Rothenburg *et al.*, 1995).

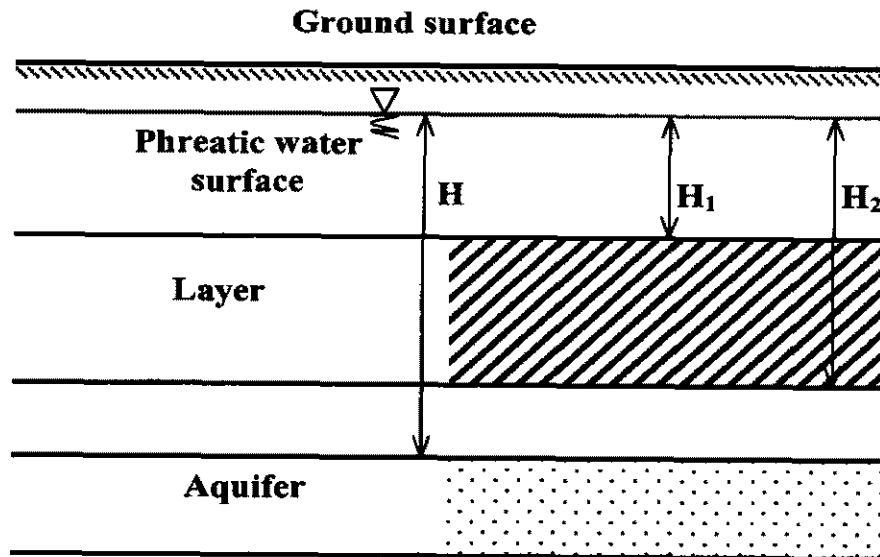


Figure 2.6: Schematic diagram of layer (after Shu *et al.*, 1991).

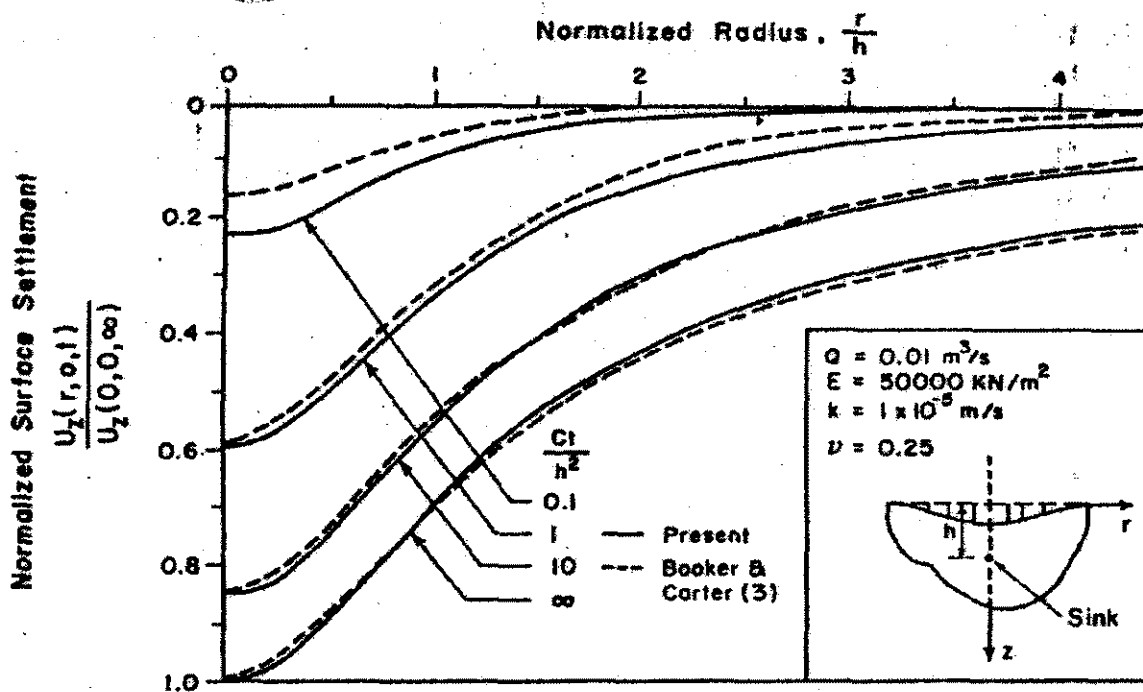


Figure 2.7: Surface settlement profile along radial direction due to point sink (after Nukulchai and Chau, 1990).

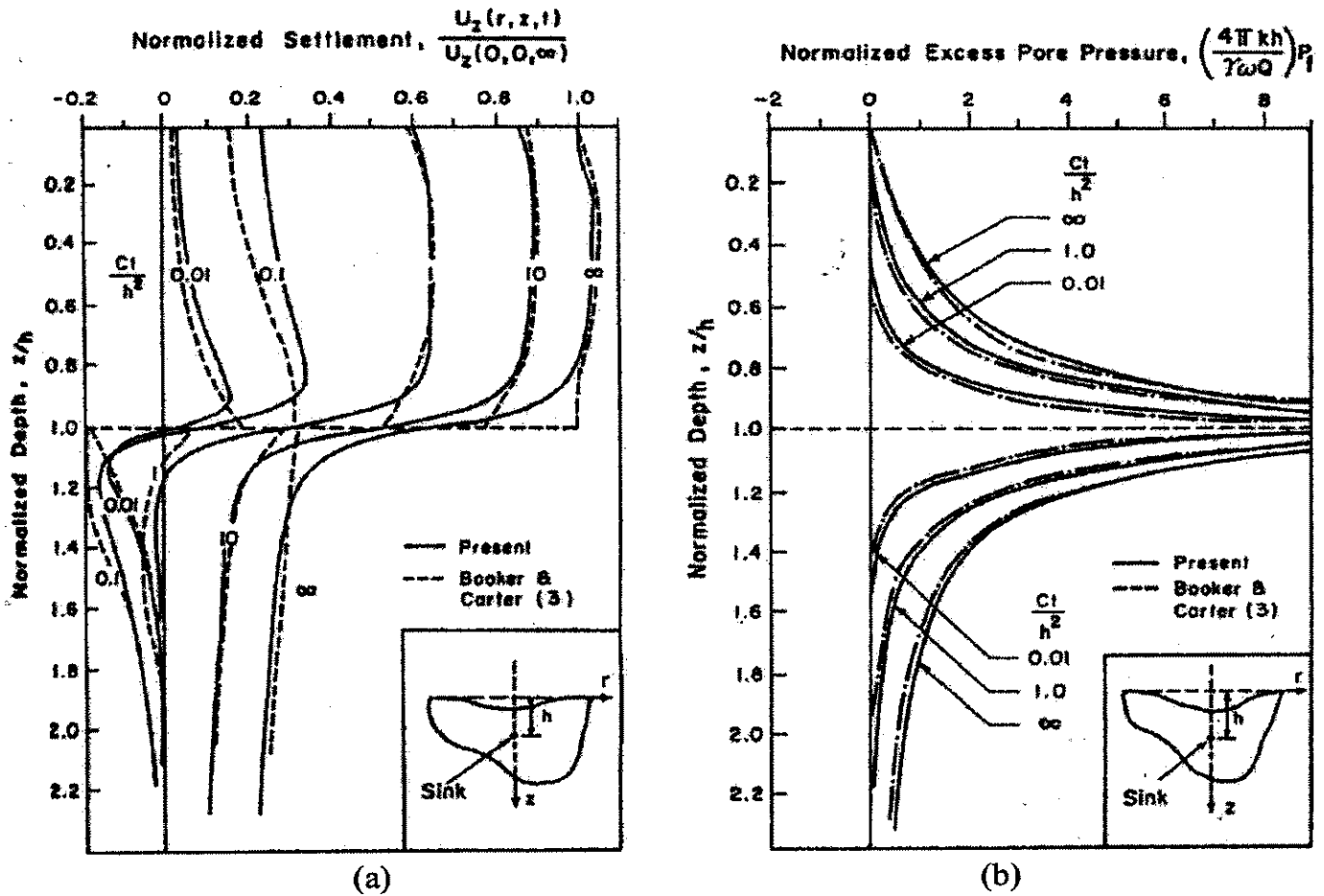


Figure 2.8: Settlement profile (a) and distribution of pore pressure (b) along vertical axis passing through applied point sink (after Nukulchai and Chau, 1990).

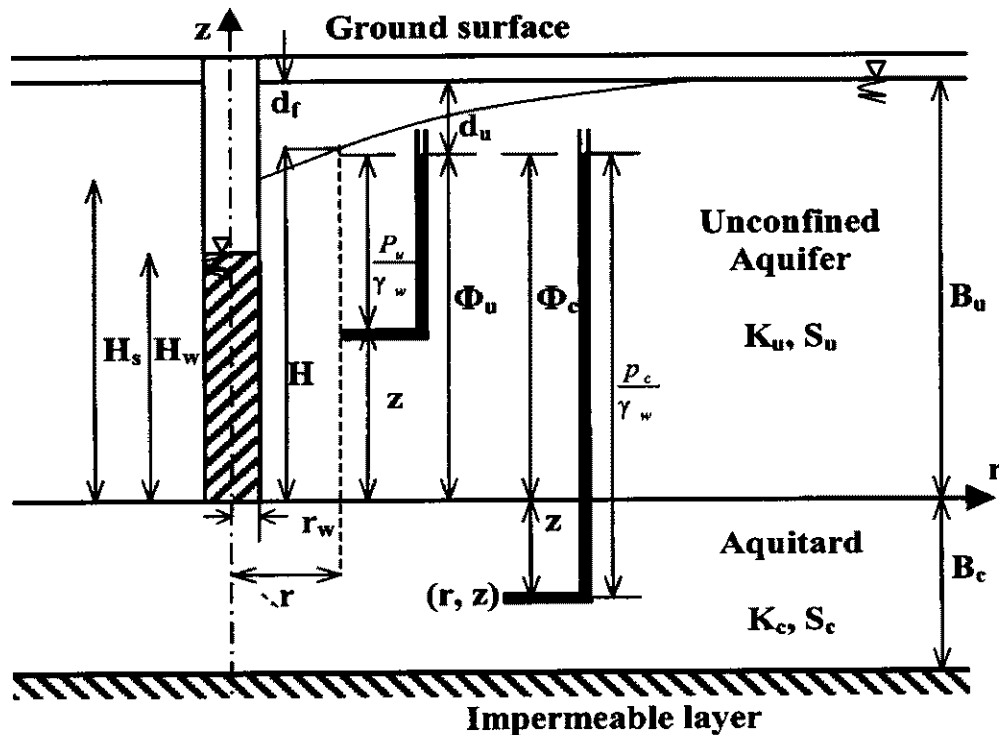


Figure 2.9: Diagrammatic sketch of the investigated system (after Kashef and Chang, 1976).

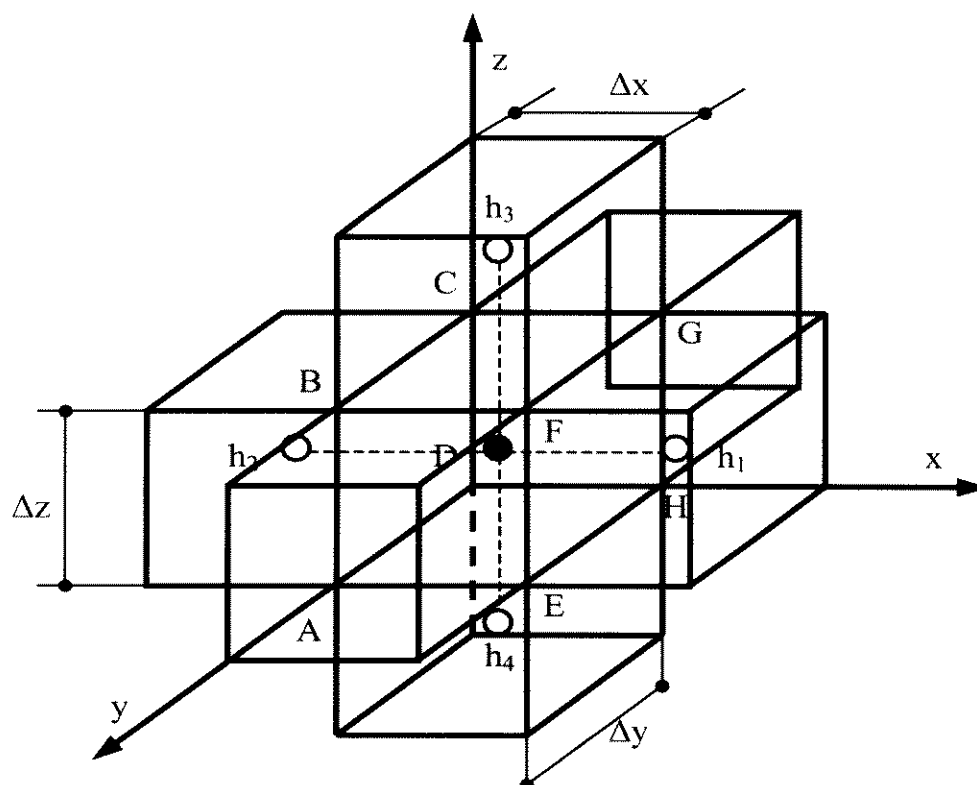


Figure 2.10: Ground water balance in unit cubic cell (after Kamata *et al.*, 1976).

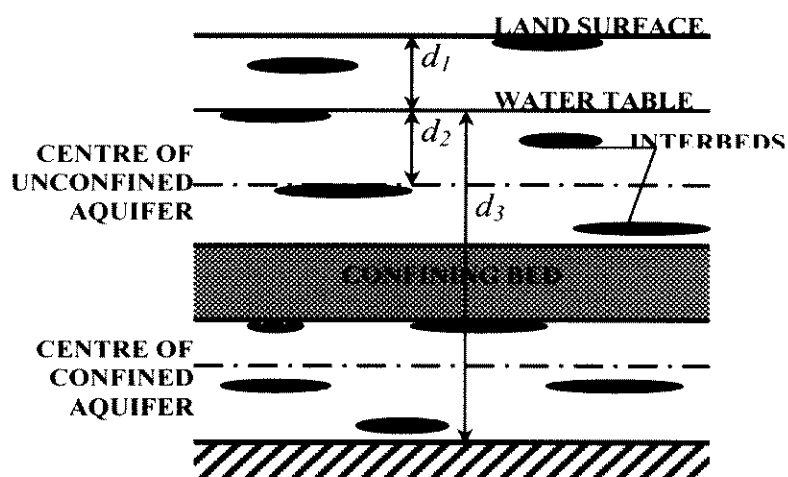
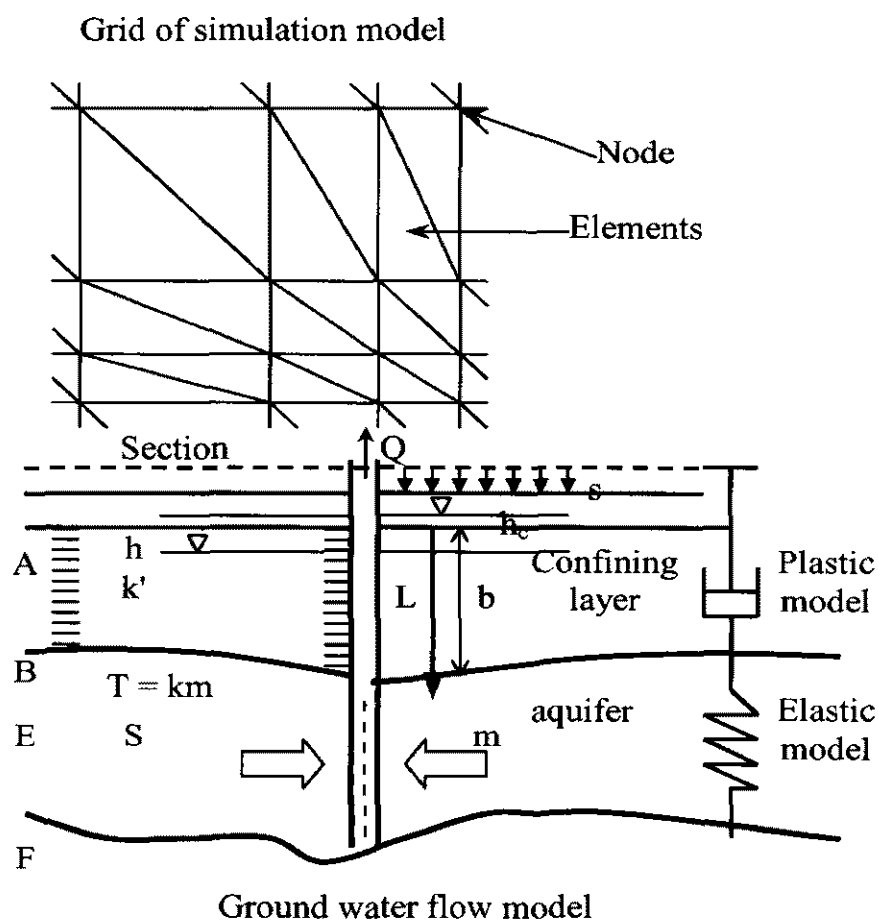


Figure 2.11: A system of discontinuous interbeds and extensive confining beds (after Leake, 1991).



Parameter

s: subsidence

k: permeability

S: storage

h : ground water head of aquifer

b: thickness of confining layer

T: transmissivity

Q: ground yield

h_c : ground water table

L: leakage

Figure 2.12: Concept and model of quasi-three-dimensions with deformation sub-model incorporated (after Harada and Yamanochi, 1983).

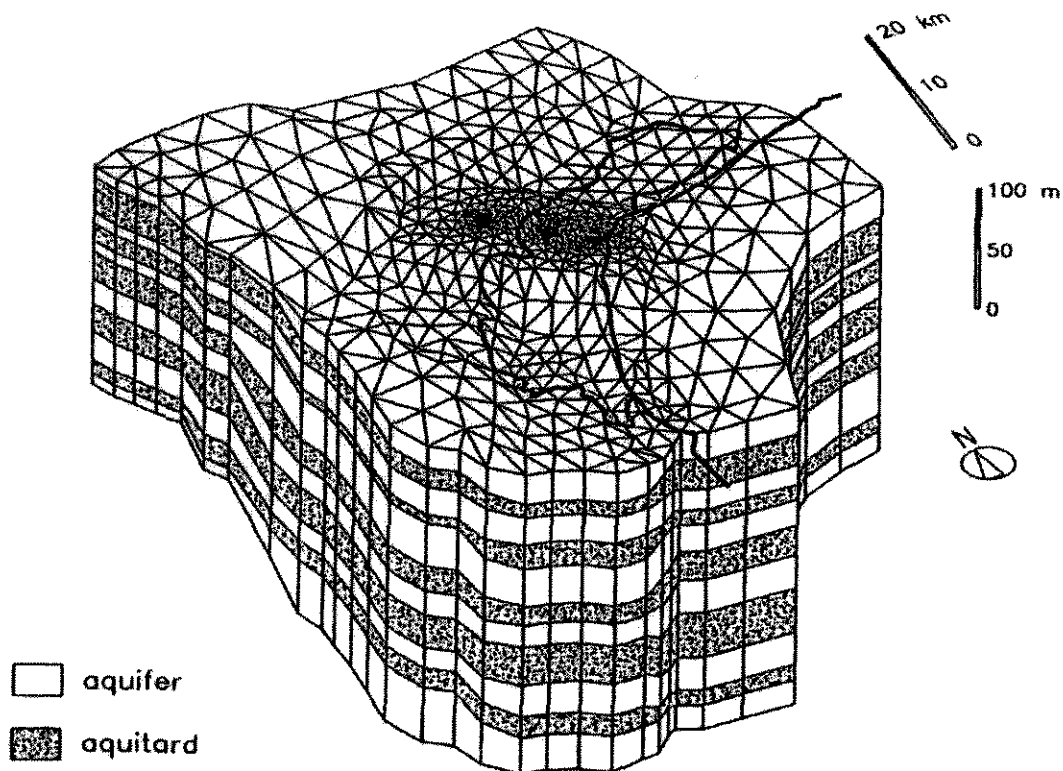


Figure 2.13: Three-dimensional view of the regional finite element model used to simulate the ground water flow in the Venice multi-aquifer system (after Teatini *et al.* 1995).

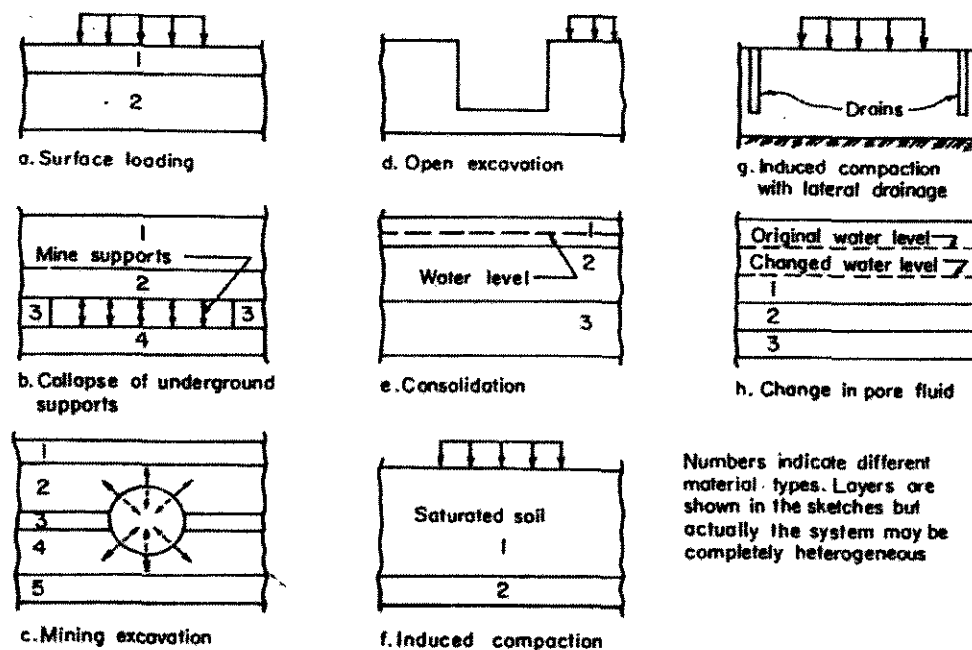


Figure 2.14: Typical land subsidence (after Sandhu and Wilson, 1969).

CHAPTER 3

GEOLOGICAL AND HYDROLOGICAL CONDITIONS IN HANOI

3.1 GENERAL INTRODUCTION

The Hanoi area is located in a fairly large delta, which is called the Red River delta. The Quaternary sediments, which are one of the youngest sediments of the earth's surface, are found all over the Red River delta and cover the Tertiary bedrock (Neogen). Because these Quaternary sediments are found at the surface, all construction-engineering activities such as excavation, mining etc. are carried out in it. Therefore, it is important to investigate the Quaternary sediments in the Hanoi area. The study of the Quaternary sediments in the Hanoi area will provide a clear picture and better understanding of the Hanoi subsoil structures and their distributions and the geological history of the Quaternary sediments.

This chapter will involve six parts:

- The introduction to Hanoi – Vietnam.
- The environmental and geotechnological infrastructure problems in the Hanoi area.
- The current state of ground water extraction and land subsidence in the Hanoi area.
- The study of the geological history and the depositional environments as well as formation of the Quaternary sediments in the Hanoi area.
- The study of the physical and mechanical properties of the Quaternary sediments that are of practical importance for numerical analysis.
- The hydrological properties of sediments in the Hanoi area.

3.2 HANOI-VIETNAM

3.2.1 VIET NAM

Vietnam is located in the centre of the Southeast Asian region. It lies in the eastern part of the Indochina peninsula bordering China to the north, Laos and Cambodia to the west and the South China Sea to the east and south. Vietnam's coastline is 3,260 kilometers long and its inland borderline measures 3,730 kilometers. The country's total length is 1,650 kilometers from the northernmost point to the southernmost. Its width, stretching from the eastern coast to the western border, not including the

continental shelf area, is 600 kilometers at the widest point in the north, and 50 kilometers at the narrowest part in the Quang Binh province on the central coast.

Three quarters of Vietnam's territory is made up of mountainous and hilly regions. Vietnam has four distinctive mountain zones each having its own unique features. The first zone is the northeastern zone or Viet Bac - a former revolutionary base between 1945 and 1954. The second zone is the North-western mountain range running from the northern area on the Sino-Vietnamese border to the western region in Thanh Hoa, Nghe An and Ha Tinh provinces on the central coast. The third mountain zone is the North Truong Son range, a long low mountain range running from the western part of Thanh Hoa province to the Hai Van Pass. The fourth and last mountain zone is the South Truong Son, a mountain range located in the western side of provinces situated on the southern end of the central coast. Forests and forestland account for 50% of Vietnam's total land area. These forests are home to many kinds of plants and animals not found elsewhere in Southeast Asia or in many other parts of the world. Beneath the forests and forestland are valuable mineral deposits most of which remain untapped. Vietnam has two major deltas: The Red River Delta in the northern part and the Mekong River delta in the south. In addition, it is criss-crossed by thousands of large and small rivers (there is a river mouth for every 20 kilometers of coastline). Vietnam's coastline is 3,260 kilometers long. The country is flanked by the East Sea. Offshore on Vietnam's continental shelf are thousands of islands and islets lying scattered from the northern to southern ends.

3.2.2 HANOI

Hanoi is the capital of Socialist Republic of Vietnam and it is the economic, political, cultural, scientific and technological centre as well as the heart of Vietnam.

Hanoi lies in the Red River Delta and is situated in a tropical area having a strong monsoon influence. It has a history of 1000 years of development. In 1961 Hanoi had only four inner precincts (Ba Dinh, Hoan Kiem, Dong Da, and Hai Ba Trung) and five suburbs. From 1996 Hanoi has been extended and now it includes five inner districts and five suburbs with an area of 2139 km² and a population of 2.5 million. Having tropical weather with a strong monsoonal influence, Hanoi has two different seasons: it is cold and dry during the winter, and hot and rainy during the summer. The winter

in Hanoi lasts from November to March of the next year. The average temperature during the winter is 16°C (lowest 5-7°C). It is very hot and wet during the summer, which starts in April and finishes in October. The highest temperature in the summer may reach 35-40°C. The rainfall accounts for 80-90% of the annual total rainfall that is 1657.1 mm and the annual evaporation is about 1025.5 mm. Hanoi lies in the Red River Delta so that it is rather flat and slightly slopes from Northwest to Southeast. The height above sea level is 8 - 12m in the Northwest area, 5 - 7m in the centre of the city and 3 - 4m in the Southeast area of Hanoi.

3.3 ENVIRONMENTAL, GEOTECHNOLOGICAL AND INFRASTRUCTURE PROBLEMS OF THE HANOI AREA

All the infrastructure, e.g. road systems, bridges, airports, underground structures, water supplies and drainage systems, of Vietnam in general and of Hanoi in particular still face many problems. They have not kept pace with the development of the economy, culture, science and technology and will not be able to cope with the expected growth of the capital from now until the year 2010 (the population now is about 2.5 million but, according to the predictions of the experts, it will be 5 million in 2010).

Plans for future infrastructure in Hanoi have not been officially approved. All the construction activities now are provisional. They have been developed for the present time in order to cope with the current problems. Consequently, the road-bridge systems are in very bad condition. Floods almost always happen during the rainy season. The ground water has been withdrawn in huge quantities for living and industrial purposes.

Moreover, the Hanoi area has a sensitive environmental geotechnology and is easily affected by human activities as well as the influence of nature. There are four main points concerning the geotechnical conditions in Hanoi:

- The soil beneath Hanoi consists mainly of soft organic alluvium and peat. Therefore they have the potential for undergoing large settlements when loaded and could easily cause damage to any structures that were built on this area (that includes infrastructure).

- The extraction of large quantities of ground water for living and industrial purposes without any control of the state government (at present 500000 m³/day) has caused a drawdown of water table in the whole area of urban Hanoi. The average drawdown of the water table is 10m in most of the Hanoi area, at some places the water level has been drawn down by 26m. There has been some information and initial predictions about the subsidence of the Hanoi surface that relates to the lowering of ground water level. However, until now, there has not been any research or project that considers carefully and clearly this problem, especially a model to predict the ground subsidence.
- The subsidence of the ground due to loading: almost all of the high-rise buildings (more than 5 storeys) in Hanoi were built on weak soils with high compressibility. Hence, many buildings have settled up to 2m. Besides this, some buildings not only settled but tilted or underwent large differential settlements so that they became a danger to other properties and people living inside or near those buildings.
- The influences of earthquake: earthquake is also a danger to the stability of structures and the life of inhabitants in the Hanoi area. Hanoi lies in an area of high seismicity.

In short, because of these reasons, planning for the exploitation and protection of the environmental geotechnology of Hanoi in particular and of the state in general is very important and needs to be considered in advance. Only if the effects of activities such as pumping of ground water are considered in advance, can the risks to structures and buildings be avoided; and then the economic and technological solutions can be found.

3.4 THE CURRENT STATE OF GROUND WATER EXTRACTION AND LAND SUBSIDENCE IN HANOI

In Hanoi, ground water is the only source of water for industrial and living purposes. The French first exploited the ground water in the beginning of the 20th century in Hanoi, although the pumping rate was rather small. First records show that the pumping rate was 20000 m³/day in 1909. This rate was doubled in 1954 and gradually increased to 164000 m³/day in 1978. From 1985, with the assistance of a Finnish

Government Organization, the number of water plants in Hanoi was increased to 11. At the same time, because of the open policies for economic development, the Vietnam economy started growing strongly. Therefore the pumping rate was increased significantly. It was 300000 m³/day in 1988, and after 2 years, in 1991 it increased to 400000 m³/day. However it was reported that it satisfied only 60% of water demand in Hanoi City (Raul *et al.* 1991). The pumping rate has been increasing heavily, and in 1998 it reached the rate of 650000 m³/d. For the reason that the Vietnam economy is still growing and the current pumping rate still does not meet the water demand, the pumping rate will be raised to 900000 m³/day by 2010.

According to the 1995 report of T. Q. Nguyen, and D. C. Helm about the current state of land subsidence in the Hanoi area, the land subsidence in Hanoi was first noticed during the 1988-1989 levelling of the land surface over the Hanoi urban area. Figure 3.1 illustrates the land surface subsidence in 1993 while Figure 3.2 shows the decline in piezometric head in the five years from 1988 to 1993. Maximum land subsidence in Hanoi reached about 25-30cm in the centre and about 5-10 cm in the adjacent areas. The land subsidence has formed a bowl, which coincides with the bowl shaped contours of decline of piezometric head.

Damage to some houses, factories, and schools has occurred in the southern part and in the centre of Hanoi. Most of the buildings in Hanoi were built on weak soils with high compressibility. Hence they settle easily under loading. As well, the land subsidence caused by lowering the water level will exaggerate the likelihood of building settlement and differential settlements. In the centre of Hanoi the surface rainwater can not be drained and this causes flooding during the rainy season or after heavy rain. This affects the traffic system in the city. Recently the appearance of fissures in the southwest part of Hanoi that have never been seen before indicates that severe land subsidence has occurred in the Hanoi area.

Due to the fact that there is an Upper Pleistocene clay layer (Q₃), with low permeability, lying above the Middle-Lower Pleistocene aquifer layer that is found almost all over the Hanoi area, the process of land subsidence will last longer than the process of pumping ground water. Even when water pressures decline no further in

the two aquifers, land subsidence still continues for years. In the coming years if the pumping rate is still increased without a wise management plan for using ground water in Hanoi, land subsidence will be more serious and can cause a lot of damage to the infrastructure of Hanoi City.

One typical example is the settlement and tilting of Building A5 at The Hanoi University of Education I. This building is just 500m away from the Mai Dich water plant. The four-story masonry building has 2 blocks (block II and I) that are separated by a settlement gap and reinforced concrete strip foundations. It was built in 1960 and has been in use from that time. From 1991 the A5 building became so deformed that it had a significant potential of collapse. The building was withdrawn from use in order to avoid the risk of injury for people working in this building. The reason for the damage to Building A5, Block II was the consolidation of soft soil layer number 4 (Figure 3.3), that was affected by the over pumping of the ground water in the Mai Dich water plant.

To transfer the load of the building to the relatively stiff soil layer below the silt layer, piles with a cross-section of 200x200 (mm) and different lengths have been used. The piles were driven into stiff soil layer number 5.

Before the piles were placed, during the period from 4 October 1992 to 14 February 1993 the settlements at measurement points of Block II - Building A5 were measured as: average velocity of settlement $S_{\min} = -0.9$ mm/month at mark M26; $S_{tb} = -12.4$ (mm) at mark M17; $S_{\max} = -25.2$ (mm) at mark M16.

After installing the piles, over the following period from 15 March 1993 to 30 August 1993 the settlements at measurement points were nearly equal and the average velocity of settlement had been reduced significantly to $S_{tb} = 2$ mm/month.

The contours of equal settlement of Block II, – Building A5 – The Hanoi University of Education I are shown in Figure 3.4.

3.5 THE QUATERNARY SEDIMENTS IN THE RED RIVER DELTA

3.5.1 GENERAL

To have a thorough understanding and a clear view of the Quaternary sediments in the Red River delta in general and the Hanoi area in particular (the research area) a geologic description of the delta is presented based on the Geological Investigation Report for Hanoi City, Report Number 2666, and the “Geological Maps for Construction Engineering for Hanoi City” drawn to a scale of 1/50000, which were published in 1993 (after M. Tran, T. V. Pham, 1993).

Generally, the north of Vietnam, where the Red River delta is located, has three types of land formations, which have been developed from the north west to the south east of the region. In the northwest is the mountainous area with an elevation of 400-500m to 1200m above mean sea level. Then there is the intermediate area that is at an elevation of from 20-40m to 300-350m, which continues to form the Pediment system. The low, flat delta, which is the major concern of this research, is about 3-4m to 12m above the current sea level.

In the Red River delta, the sediments mostly belong to the Quaternary age and are usually young, normally consolidated, uncemented sediments. These Quaternary sediments can be generally subdivided into two major types: the coarse-particle sediments lying below and the relatively fine particle sediments above, which are usually clay, sandy clay, clayey sand, peat and mud. Table 3.1 shows a general stratigraphy of the Quaternary sediments in the Red River delta, after Nguyen and Helm, 1995.

According to all the geologic maps and documents, the Quaternary sediments in the Red River Delta have the sequence that is presented in Table 3.1:

Table 3.1: General stratigraphy in the Hanoi area (after T. Q. Nguyen and D. C. Helm, 1995)

Geologic age		Approximate thickness (m)	Description
Cenozoic	Quaternary	Holocene (Q)	0-30 Fine and small grained sands, clayey sand, sandy clay, soft clay, peat and silt
		Upper Pleistocene (Q)	0-15 Clay
		Middle-Lower Pleistocene (Q)	40-70 Pebbles, gravel, coarse-grained sand and clay lenses
	Tertiary	Pliocene-Miocene (N)	>1000 Gravel, sandstone, claystone

3.5.2 THE RED RIVER DELTA AND THE SOUTH EAST ASIAN REGION

The south East Asian region is comprised of a group of countries, namely, Burma, Thailand, Cambodia, Laos, Vietnam, Malaysia, Singapore, Indonesia, and the Philippines. Figure 3.5a shows the distribution of the recent clays in the south East Asian area, after Cox, 1969. Figure 3.5b shows the map of the Red River Delta and Hanoi and suburbs. Originally, the south East Asian region was formed from an ancient massif centred in East Thailand, Cambodia and Vietnam by a time-orogenic movement from the northwest to the southeast. The youthful mountainous highlands and the flat, deltaic low lands are two major landforms in the South East Asian region. There is an intermediate landform in between two major ones. More detail on the distribution and description of the sediments in the South East Asian region can be found in the study of Cox (1970).

The Red River delta is a small flat deltaic plane lying in the North East of the North of Vietnam, on top of the south East Asian region, near the Chinese border. The

formations and the descriptions of the Quaternary sediments in the Red River delta will be discussed in the next sections.

Historical geology of the Red River delta

The Quaternary sediments were formed during the period between the ice age and present. Their characteristics were much influenced by the results of drastic climatic changes, the occurrence of the ice age, and the worldwide sea level fluctuation.

Before the Quaternary period, which was close to the end of the Tertiary age, the sea level was hundreds of meters higher than the current sea level. The sea invaded and flooded all the existing land area and at that time the Red River delta was still a gulf. The waves, which broke on shore, eroded the land by the force of their impact and by the impact of the debris they carried forward. The debris itself comprises rock fragments derived from the local cliffs. The fragments became rounded and reduced by the battering they received from the waves and formed the familiar pebble and sand beach deposits. The evidence that can be seen today is some white pebble layers located at a height of 25m to 45m above the present sea level located in the region from Pha Lai to Bac Ninh, Phuc Yen.

During the first period of the Quaternary age, which is also called the Pleistocene epoch, there were subzero temperatures worldwide and the sea level was 100m lower than the current level. Streams began to widen and deepen their valleys, and carried much material in suspension that was deposited downstream. This epoch occurred for hundreds of thousands of years and is also known as the ice age occurrence.

The second period of the Quaternary age was called the Holocene epoch that continues until now. The formation of the Red River delta was due to the fact that the velocity of the river current reduced as it reached the sea and consequently resulted in deposition of sediments at its mouth. The depositional environments required to form the Red River delta are complex. However, the fluvial and marine environments are the major influences.

3.5.3 THE LOWER PLEISTOCENE

This stratum is called The Lechi stratum (aQ_1^{lc}). The Lechi stratum is distributed over the centre of the Red River delta and there is no trace of it on the margin and the

adjoining marginal area. The Lechi stratum covers almost all of the area where subsidence has occurred in the delta region. It spreads from northwest to southeast and in the centre of the delta this stratum reaches 25m in thickness and gradually reduces in thickness in the northeast and southwest directions. In addition, the historical geologic formations of the Lechi sediments involved the disintegration and decomposition processes of the early exposed Pleistocene. It underlies the Hanoi stratum and is directly above the Tertiary bedrock (Neogen). In the vertical direction, from bottom to top, the sedimentary particles range from coarse to fine showing the sequence of the depositional process of fluvial sediments. The Lechi can be subdivided into three sub-strata:

- *Sub-stratum 1*: consists of pebbles, gravel and some butter clay. The pebbly particles are very clean and very rounded, 2-3cm (sometimes 3-5cm) in diameter. The pebbly mineral is usually quartzite.
- *Sub-stratum 2*: is mostly fine sand and sand that is grey in colour.
- *Sub-stratum 3*: grey, yellowish-grey, blackish-grey butter clay that contains freshwater weeds.

3.5.4 THE MIDDLE AND UPPER PLEISTOCENE

The Hanoi stratum ($ap-aQ_{II-III}^{I\text{hn}}$) lies directly above the Lechi stratum and is also covered by the younger sediments. But, unlike the Lechi stratum, the Hanoi stratum can be found exposed on the margins and the areas adjoining the margins of the delta. In the geological documents supplied by the Hanoi geological group, the Hanoi stratum is specifically characterized by two types of sections: (1) An exposed section and (2) a covered section.

For the covered section that can only be investigated with deep boreholes, there are three sub-strata that can be identified:

- *Sub-stratum 1*: consisting of pebbles, gravel and some fine sand.
- *Sub-stratum 2*: containing fine and coarse sand and some pebbles or gravel

- *Sub-stratum 3*: consisting of greyish-yellow, reddish-yellow, and greyish-brown butter clay containing vegetable humus. Clay minerals present are hydrous aluminium silicates which contain K, Ca, Mg and Na.

Furthermore, depending on the appearance of tektite pebbles in the Middle and Upper Pleistocene sediments, they can be defined as fluvial sediments. The maximum thickness reaches 55m.

3.5.5 THE UPPER PLEISTOCENE

In the research zone, the Vinh Phuc stratum ($a-lbQ_{III}^{2vp}$) exposes in Dong Anh and as far as Thang Long Bridge, Xuan Dinh, Co Nhue, Mai Dich and a part of the Nghia Do, Cau Giay zones. The Vinh Phuc also exposes southwest of the Hanoi area. Within the centre of Hanoi, in contrast, the Vinh Phuc stratum is almost covered by younger sediments. In the boreholes in this area these sediments may be encountered between 10 and 30m, while at places such as Le Truc market, Ngoc Khanh, or Phan Ding Phung they can be found at shallower depths of 4-5m. In the vertical direction, the Vinh Phuc sediment texture changes a lot. The Vinh Phuc stratum may be recognized by its laterized surface.

According to the geologic group number 204 and some other geologists, the Vinh Phuc sediments are supposed to be marine sediments. However, depending on mineral constituents, texture, and fossils contained in the sediments, the Hanoi geologic group suggested that the Vinh Phuc sediments are continental deposits. The Vinh Phuc stratum can be sub-divided into four sub-strata:

- *Sub-stratum 1*: composed of pebbles, gravel, and some greyish-yellow clay containing the imprint of freshwater weed. This is fluvial sediment and the maximum thickness is about 10m.
- *Sub-stratum 2*: mostly consisting of fine sand, yellow sand, and some clay, and lenses of yellow, greyish-brown pebbles. This sediment contains cross bedding structures, which is characteristic of shallow-water deposits and river deposits. The maximum thickness reaches 33m.

- *Sub-stratum 3*: consisting of white-grey kaolin, and yellowish-grey butter clay, which are known to be shallow-water deposits. It contains freshwater weed and animal fossils. The thickness changes from 2 to 10m.
- *Sub-stratum 4*: consisting of black clay, blackish-brown, and yellowish-grey butter clay, thought to be fluvial sediments. In some areas such as Soc Son, and Dong Anh, pebbly lenses of peat 0.5m thick containing vegetable matter are sometimes encountered in this layer. The thickness of this sub-stratum varies between 3 and 8m. In the area that is considered in this thesis there is no trace of this sub-stratum.

3.5.6 THE LOWER AND MIDDLE HOLOCENE

The Hai Hung stratum is sub-divided into three sub-strata, which are the lower Hai Hung sub-stratum ($lbQ^{1-2}_{IV^{hh1}}$); the lower Hai Hung sub-stratum ($lmQ^{1-2}_{IV^{hh2}}$); and the lower Hai Hung sub-stratum ($bQ^{1-2}_{IV^{hh3}}$).

- *The lower Hai Hung sub-stratum ($lbQ^{1-2}_{IV^{hh1}}$)*: these sub-stratum sediments are continental sediments that formed during the time before the sea covered the area. This is constantly met in the research area. The predominant constituents are clay, organic butter clay that is black and blackish-grey in colour. In some areas, there are peat layers, 1-3m in thickness lying above the surface of this sub-stratum. The thickness of this sub-stratum ranges from 2-6m to more than 20m. This sub-stratum contains imprints of vegetable matter.
- *The middle Hai Hung sub-stratum ($lmQ^{1-3}_{IV^{hh2}}$)*: These are both continental and marine sediments. The continental sediment includes clay, yellowish-grey, greyish-blue butter clay, and at the bottom of the layer there are some pebbles. The thickness is about 2-4m and it contains freshwater weed. The marine sediment is mostly clay, some butter clay, and bluish-grey and blue clay, there is some organic mud containing vegetation at the bottom. It contains marine fossils.
- *The lower Hai Hung sub-stratum ($bQ^{1-2}_{IV^{hh3}}$)*: these sediments are swampy deposits deposited after the sea level subsided. There is no trace of it in the research area. This

sediment consists of butter clay, and blackish-grey, to blackish-brown sand containing peat, consisting of vegetable matter, and freshwater weed.

3.5.7 THE UPPER HOLOCENE

The Thai Binh stratum (aQ_{IV}^{3tb}) contains the younger sediments and is distributed all over the research area. They are fluvial sediments and can be sub-divided into two sub-strata:

- *The lower Thai Binh sub-stratum (aQ_{IV}^{3tb1}):* which can be sub-classified into four layers

- Layer 1: small particle pebbles, gravel, sand and some light-brownish-grey butter clay, 1-9m in thickness.
- Layer 2: fine grey sand, containing vegetable humus, 3-18m thick.
- Layer 3: grey butter clay containing vegetable humus, 1-3m thick.
- Layer 4: greyish-brown clay, containing vegetable humus and shells, 1m in thickness, there is no trace of these sediments in the research area.

Layers 1,2,3 are fluvial sediments and layer 4 is swampy sediment. The total thickness of this Thai Binh sub-stratum is about 31m.

- *The upper Thai Binh sub-stratum (aQ_{IV}^{3tb2}):* These sediments are young alluvial deposits, distributed near the Red River dike. They can be subdivided into two layers:

- Layer 1: pebbles, gravel, sand and some yellow butter clay. The thickness varies between 3-10m.
- Layer 2: light grey butter clay containing shells and vegetable humus. The thickness varies from 2 to 5m.

3.5.8 FILL

Man made deposits can also be found within the research area. They mostly consist of clay, sandy clay, clayey sand and many other waste materials. Man made fill has very different colours and may have been in place for many years or may have been recently placed. Because man made fill is very different in mechanical properties, thickness and constituents it is not used as the foundation bearing layer for buildings or other construction. In general we will not be concerned with this man made soil layer in the following.

Table 3.2 gives a summary of the quaternary sediments that are found in the Red River delta. This table shows the age of sediments, the name of each stratum, the thickness, and a brief description for each kind of soil layer.

3.6 DISTRIBUTION OF SOFT SOIL LAYERS IN THE HANOI AREA

Figure 3.6 shows a distribution of the soft soil layers all over the Hanoi area and the Red River delta. It is important to note that the distribution of the soft soil layers presented here only extends to a maximum depth of 20m because of the fact that information about the soil layers lying deeper than 20m from the surface is not available or not adequate to classify the soil.

Though the quaternary sediments in the Red River delta include a variety of sediments from clay to sand, based on the mechanical and physical soil properties of these layers the sediments have been classified into three categories, namely, A, B, and C corresponding to one-layer, two-layer and three-layer systems. In each category there are sub-categories that are presented in Table 3.3.

Table 3.2 Stratigraphical column for the Quaternary sediments in the Red River delta.

Group	System	Geological period	Steps	Strata	Symbol	Geological column	Thickness (m)	Descriptions
CAINOZOIC	QUATERNARY	Holocene	Upper	Thai Binh	NT		1-3	Fill: clay, sandy clay, sand, clayey sand, waste and soil with vegetation. Medium to stiff
					aQ ³ _{IV} ^{tb2}		10-30	Bright brown sandy clay contains fresh-water fragments of shell and humus. Soft to medium. Coarse-grained sand and pebbles are below.
					aQ ³ _{IV} ^{tb1}		15-20	Pinky brown clay, yellow brown clay, black pinky brown sandy clay. Very soft to soft. Pinky brown clayey sand contains humus. Black brown sand and pebbles
			Lower and Middle	Hai Hung	mQ ¹⁻² _{IV} ^{hb2}		0.5-3	Blue brown or blue clay. Homogeneous. Soft to medium
					bQ ¹⁻² _{IV} ^{hb1}		2-20	Brown grey organic clay. Very soft. Grey black peat and silt.
					mb ² _{III} ^{vp3}		3-8	Sandy clay, pinky brown, yellow grey and black grey organic clay
		Pleistocene	Upper	Vinh Phuc	bmQ ² _{III} ^{vp2}		2-10	Kaolin, yellow grey sandy clay. Stiff to very stiff
					aQ ² _{III} ^{vp1}		5-20	Sandy clay, clayey sand, yellow sand containing pebbles. There are some organic lenses or silty clay lenses
					aQ ¹ _{II-III} ^{hn1}		55	Sandy clay containing humus Clayey sand Yellow sand and pebbles Pebbles, gravel, boulders.
			Middle	Ha Noi				
			Lower	Le chi	aQ ^{1c}		25	Grey clay containing fresh water-weed Grey fine sand Round pebbles and gravel, 2-5cm in diameter.

Table 3.3: The distribution of soft soil layers in the Hanoi area (20m depth).

Distribution types		Geological characteristics	
A	A-1	Q^2_{III}	Upper Pleistocene; Clay and sandy clay; the thickness is up to 20m
	A-2	aQ^4_{IV}	Upper Holocene; clay and sandy clay; the thickness is up to 20m
B	B-1	mQ^2_{III}	Upper Pleistocene; Sandy clay; The thickness is about 10m.
		$a-mQ^2_{III}$	Upper Pleistocene; Marine sandy deposit intercalated by continental deposits; Fine to medium sand, medium dense, thickness is about 10 to 15m.
	B-2	aQ^3_{IV}	Upper Holocene; clay or sandy clay; thickness is about 10m.
		aQ^3_{IV}	Upper Holocene; Medium sand and medium dense, thickness about 15m.
C	C-1	Q^2_{III}	Upper Pleistocene; sandy clay; Thickness is about 10m
		$a-mQ^2_{III}$	Upper Pleistocene; peat, sandy clay, and organic clay, thickness about 10m
		Q^2_{III}	Upper Pleistocene; Medium to coarse grained sand.
	C-2	aQ^3_{IV}	Upper Holocene; Sandy clay; Thickness is about 5m.
		mQ^2_{IV}	Middle Holocene; Peat and soft clay, marine deposit; Thickness is about 10m
		mQ^2_{III}	Upper Pleistocene; Clay and sandy clay
	C-3	aQ^3_{IV}	Upper Holocene; Sandy clay; Thickness is about 5m.
		$b-mQ^{1-2}_{IV}$	Middle Holocene; Very soft sandy clay, marine and continental deposits; Thickness varies from 3-4m to 15-20m
		mQ^2_{III}	Upper Pleistocene; Clay and sandy clay.

3.7 MECHANICAL AND PHYSICAL PROPERTIES OF THE HANOI SOILS

3.7.1 GENERAL

In the previous part of this chapter, the engineering geology (origin, and distribution of the sediments) has been discussed. In this part, the geotechnical characteristics, such as water content, compressibility, plasticity etc. of the sediments in the Hanoi area will be considered. However, there is still very little information on the mechanical properties of the soil.

3.7.2 THE HOLOCENE ANTHROPOGENIC GENESIS FILL (anQ_{IV})

These kinds of man made soils or fills are very complex: mostly clay, sandy clay, and a mix of clayey sand and other waste materials. Fills usually are of different colors, have different states and may have been formed recently or have been in place for many years. Therefore, their engineering properties are extremely variable.

Man made soils are usually found in the old or new residential areas. The thickness varies from 0.5m to 10m or more. Due to anisotropy and non-homogeneity they are not convenient for use as a foundation-bearing layer.

3.7.3 THE UPPER HOLOCENE-THE LOWER THAI BINH ALLUVIAL STRATUM (aQ_{IV}^{3tb1})

Based on detailed geologic sections, this alluvial stratum can be sub-divided into 6 sub-layers:

(a) SANDY CLAY LAYER

The sandy clay can be brown, dark brown, yellowish-brown, and bluish-brown striped in colour; it is medium-soft and gradually changes into a stiff material with depth: it consists of: sand ($d=2-0.05$ mm) 28%, silt ($d=0.05-0.005$ mm) 48%, clay ($d<0.005$ mm) 24%. The layer also contains small agglomerate particles $d=1-3$ mm, that are black in colour. The thickness changes from 0.2m to 9m. The average thickness is over 3m. The average mechanical and physical properties are listed in Table 3.4.

Because it lies near the ground surface, this stratum takes an important role in the design of shallow foundations.

(b) BROWN, YELLOWISH-BROWN SANDY CLAY

This clay is soft to very soft with particle size distributions: sand 26%, silt 50%, and clay 24%. It is distributed all over the Hanoi area. In some areas layer (a) covers it, but in other areas it is exposed. Its thickness is between 1m and 8m, with an average thickness of about 3m. The average mechanical and physical properties are shown in Table 3.4.

(c) GRAYISH-BROWN SANDY CLAY

This layer is very soft, containing coarse particles: sand 28%, silt 50%, and clay 22%. In some areas this layer consists of silty clay, silty sandy-clay, and silty clayey-sand of blackish-grey colour. At the bottom of the layer there are lenses of clay sand or fine sand. The lab-tests on samples from this layer show that:

- Before testing, the sample is very soft.
- During testing, consolidation occurs quickly and it becomes hard.

Permeability ranges between 10^{-3} to 10^{-5} cm/s. Thickness of the layer ranges from 1m to 8m, with an average thickness of 4m.

The average mechanical and physical properties are shown in Table 3.4.

(d) GREY, AND BROWNISH-GREY CLAYEY SAND

The layer consists of soft, clayey sand and sometimes loose sand. The particle distribution is sand 77%, silt 17%, and clay 6%. There are also some interbedded soft sandy clay or fine sand layers. As the layer contains many coarse particles, it consolidates quickly. The thickness is from 2m to 4m.

Table 3.4: Physical and Mechanical properties of the upper Holocene-the lower Thai Binh alluvial stratum (aQ³_{IV}^{tbl})

Properties	Symbol	Values		
		(a)	(b)	(c)
Moisture	w_o (%)	26.8	29.1	34.3
Unit weight	γ_s (kg/m ³)	1940	1880	1830
Unit weight of solids	γ_d (kg/m ³)	2700	2690	2680
Initial void ratio	e_o	0.77	0.842	0.97
Porosity	n_o (%)	43	46	49
Degree of saturation	G (%)	96	92	96
Plastic index	I_p (%)	13	11	11
Liquid index	I_s	0.24	0.6	0.86
Undrained cohesion	c (kPa)	34.0	21.0	14.0
Undrained friction angle	φ°	11°35'	11°00'	9°19'
Coefficient of compressibility	a_v (MPa ⁻¹)	0.027	0.036	0.052
Young's modulus	E_o (MPa)	6.5	5.1	3.8

(e) GREY, BLUISH-GREY FINE SAND

In the upper part of this layer, the silt particle content is higher than in the lower part, but it contains mostly clean sand. However, in some areas there are thin interbedded sandy clay or clayey sand lenses. It was once saturated but at present due to the effects of ground water pumping activities in some areas, especially the area along the Red River dike, the upper part of this layer is completely dry. The average thickness is between 10m and 15m. The particle size distribution is shown in Table 3.5.

Table 3.5: Grading of grey fine sand ($aQ_{IV}^{3\text{tbl}}$)

Properties		Symbol	Values
Grain-size distributions (%)	2.0 (mm)	-	$2/6-2$
	2.0-1.0 (mm)	-	$34/56-2$
	1.0-0.5 (mm)	-	$5/9-1$
	0.5-0.25 (mm)	-	$11/29-2$
	0.25-0.1 (mm)	-	$35/60-4$
	0.1-0.05 (mm)	-	$3/8-1$
Initial void ratio		e_0	Not Available
Unit weight of solids		γ_d (kg/m ³)	$2680/2700-2640$
Dry friction angle		φ°	$22^\circ/26^\circ55'-18^\circ00'$
Saturated friction angle		φ°	$27^\circ/36^\circ45'-18^\circ00'$

Note: the numerators are average values and the denominators are the maximum and minimum values

(f) CLAYEY SAND

Usually this layer consists of clayey sand, and fine sand. The thickness is 5m-10m. In general this alluvial stratum can be found in most regions in the research area. However, the complete section may be seen in the West Lake area running along the Red River dike toward the lower section of the Red River, the Sky Market area, the Lake Side Hotel, the Polytechnic University, Kim Lien, Trung Tu, Bach Mai airport, Nga Tu So, North Thanh Xuan, Bieu bridge, Van Dien, and Ha Dong. In other areas there is only clay covering the older sediments.

3.7.4 THE MIDDLE AND LOWER HOLOCENE-THE UPPER HAI HUNG STRATUM ($mQ_{IV}^{1-2\text{hh2}}$)

This stratum consists of two different clay layers:

(a) BLUISH-GREY AND PINKISH-GREY SMOOTH CLAY

This clay is a soft to very soft deposit. Its grading is sand 20%, silt 34%, clay 46%. It is marine clay, not frequently met in the research areas and is usually found in the lower Hai Hung stratum. The average thickness is 2m. The average mechanical and physical properties are shown in Table 3.5.

(b) BLUISH-GREY AND LIGHT PINKISH DRAB CLAY

The clay in this layer is of soft to medium stiffness. It is soft at the lower part of this stratum and stiffer towards the top. Its grading is sand 31%, silt 30%, and clay 39%. The major minerals are kaolinite, a mix of Hydromica - montmorillonite and some chlorite. The average mechanical and physical properties are shown in Table 3.6.

Table 3.6: Physical and Mechanical properties of the middle and lower Holocene-the upper Hai Hung stratum ($mQ^{1-2}_{IV^{hh2}}$).

Properties	Symbol	Values	
		(a)	(b)
Moisture	w_o (%)	47.5	31.2
Unit weight	γ_s (kg/m ³)	1710	1890
Unit weight of solids	γ_d (kg/m ³)	2700	2710
Initial void ratio	e_o	1.328	0.888
Porosity	n_o (%)	57	47
Degree of saturation	G (%)	97	95
Plastic index	I_p (%)	19	17
Liquid index	I_s	0.71	0.31
Undrained cohesion	c (kPa)	18	35
Undrained friction angle	ϕ°	6°30'	9°23'
Coefficient of compressibility	a_v (MPa ⁻¹)	0.073	0.032
Young's modulus	E_o (MPa)	3.2	5.9

3.7.5 THE ALUVIAL-SWAMPY MIDDLE AND LOWER HOLOCENE-THE LOWER HAI HUNG STRATUM ($bQ^{1-2}_{IV^{hh1}}$)

This stratum may be composed of silty clay, sandy clay or clay, that is soft to very soft, and contains organic matter. In some areas, soils are partially peat or completely peat. In the research area, these sediments are distributed in the Mai Dich, Giay Bridge, Nghia Do, Thu le, Ngoc Khanh, Thanh Cong, and Giang Vo areas. It is also found in other areas such as south of Hoan Kiem Lake, Ba Mau lake, Quynh Loi, Thanh Nhan, Thanh Tri, Ha Dong, and Gia Lam. The thickness varies over a wide range from 0.2-0.5m to several tens of metres. This stratum can be sub-divided into two classes:

(a) CLAY OR SILTY CLAY

The stiffness of clay varies from soft to very soft. It appears at Thanh Cong, Giang Vo, Ngoc Khanh, Lang Ha, and Thu Le. It contains the following three sub-classes:

- (i) Blackish-grey organic silty clay with about 8% organic matter. The thickness of this sub-class is about 1m.
- (ii) Clayey sand that contains peat, grey or blackish grey in colour. Organic matter ranges from 35-52%, and in some other places the peat content reaches 60-100%. Its thickness ranges between 1m and 3m.
- (iii) Organic silty clay, for which organic matter is 7-19%, blackish-grey or grey in colour, the thickness is between 5-13m, and the average thickness is 8-10m.

The soils belonging to this stratum have a high moisture content of about 100% and porosity is from 2-3 and sometimes reaches 4. The mechanical and physical properties are very poor in general.

The average mechanical and physical properties are presented in Table 3.7.

(b) GREY OR BLACKISH-GREY SILTY SANDY-CLAY, SOFT SANDY CLAY

These types of soils are usually found in some places such as Lang Trung, Thanh Cong, and Ha Dong. Organic content is less than 10%. The average mechanical and physical properties are presented in Table 3.7.

Table 3.7: Physical and Mechanical properties of the alluvial-swampy middle and lower Holocene-the lower Hai Hung stratum ($bQ^{1-2}_{IV^{hh1}}$).

Properties	Symbol	Values	
		(a)	(b)
Moisture	w_o (%)	31.2	35.7
Unit weight	γ_s (kg/m ³)	1890	1760
Unit weight of solids	γ_d (kg/m ³)	2710	2670
Initial void ratio	e_o	0.888	1.054
Porosity	n_o (%)	47	51
Degree of saturation	G (%)	95	92
Plastic index	I_p (%)	17	11
Liquid index	I_s	0.31	0.98
Undrained cohesion	c (kPa)	35	12
Undrained friction angle	ϕ^o	9°23'	6°53'
Coefficient of compressibility	a_v (MPa ⁻¹)	0.032	0.066
Young's modulus	E_o (MPa)	5.9	3.1

3.7.6 THE ALLUVIAL SEDIMENT-UPPER PLEISTOCENE, THE VINH PHUC STRATUM ($aQ^{2_{III^{vp1}}}$)

This stratum consists of 5 sub-strata that are described in the following.

(a) REDDISH-GREY, YELLOWISH GREY, GREY OR BLuish GREY CLAY

This clay layer is of medium strength, and its thickness ranges between 10m and 1m, with an average thickness is of 4m. The average physical properties of this soil are presented in Table 3.8.

(b) YELLOWISH GREY, REDDISH GREY, OR LIGHT VIOLET SANDY CLAY

These clays are soft to firm and consist of: sand ($d= 2-0.05\text{mm}$) 42%, silt ($d= 0.05-0.005\text{mm}$) 38%, clay ($d <0.005\text{mm}$) 20%. The maximum thickness and minimum

thickness are respectively 6m and 1m and the average thickness is 4m. The average physical parameters are presented in Table 3.8.

(c) PINKISH BROWN, GREYISH BROWN SANDY CLAY

The strength of the clays is low and the grading of the sand is ($d = 2-0.05\text{mm}$) 28%, silt ($d = 0.05-0.005\text{mm}$) 46%, clay ($d < 0.005\text{mm}$) 26%. The thickness changes between 1m and 6m. The average physical properties are shown in Table 3.8.

(d) REDDISH BROWN, GREYISH BROWN SANDY CLAY

This clay is soft to very soft. The thickness varies between several meters to several tens of meters. The average physical properties are presented in Table 3.8.

Table 3.8: Physical and Mechanical properties of the alluvial sediment-upper Pleistocene - the Vinh Phuc stratum ($aQ_{III}^{2\text{vpI}}$)

Properties	Symbol	Values			
		(a)	(b)	(c)	(d)
Moisture	w_o (%)	26.8	28.6	32.1	25.5
Unit weight	γ_s (kg/m^3)	1950	1910	1850	1890
Unit weight of solids	γ_d (kg/m^3)	2710	2700	2680	2690
Initial void ratio	e_o	0.76	0.812	0.914	0.781
Porosity	n_o (%)	43	45	48	44
Degree of saturation	G (%)	96	94	93	87
Plastic index	I_p (%)	17	12	12	6
Liquid index	I_s	0.25	0.52	0.76	0.35
Undrained cohesion	c (kPa)	42	28	21	19
Undrained friction angle	φ^o	$11^o45'$	$11^o10'$	$11^o57'$	$18^o29'$
Coefficient of compressibility	a_v (MPa^{-1})	0.025	0.032	0.049	0.029
Young's modulus	E_o (MPa)	0.7	5.7	3.9	6.1

(e) YELLOWISH GREY, YELLOWISH BROWN, YELLOWISH BLUE SAND

This layer consists of medium dense sand of thickness varying from 5-10m to several tens of meters.

Table 3.9: Physical and Mechanical properties of brown, yellowish brown sand ($aQ_{III}^{2\text{vpI}}$)

Properties		Symbol	Values
Grain-size distributions (%)	2.0 (mm)	-	$\frac{9}{18-2}$
	2.0-1.0 (mm)	-	$\frac{26}{31-7}$
	1.0-0.5 (mm)	-	$\frac{8}{17-2}$
	0.5-0.25 (mm)	-	$\frac{9}{19-1}$
	0.25-0.1 (mm)	-	$\frac{9}{18-1}$
	0.1-0.05 (mm)	-	$\frac{31}{40-3}$
Initial void ratio		e_o	0.5 – 1.3
Unit weight of solids		γ_d (kg/m ³)	$\frac{2660}{2690-2630}$
Dry friction angle		φ°	$\frac{31^\circ}{36^\circ 48'-30^\circ 18'}$
Saturated friction angle		φ°	$\frac{27^\circ}{32^\circ 54'-21^\circ 37'}$

Note: the numerators are average values and the denominators are the maximum and minimum values.

3.7.7 THE MIDDLE AND UPPER PLEISTOCENE COMPLEX-THE HANOI STRATUM ($aQ_{II-III}^{1\text{hnl}}$)

The Hanoi layer is mainly pebbles and gravel, the diameter of the pebble particles ranges from 2-5cm, and sometimes pebbles of diameter 10cm may be found. In the Hanoi area the thickness of this layer is usually around 30m or more and it lies at a depth of 10-15m up to 35-40m. This layer is the main water source for Hanoi. It is also the foundation-bearing layer for high-rise buildings in the Hanoi area. However, at the moment, Hanoi does not have appropriate geotechnical and hydrological instruments for thoroughly investigating this layer. Hence information for this layer is

still insufficient and further geotechnical investigation of the layer is necessary to establish its engineering properties.

The pebbles, gravel, and boulder layers are usually poorly graded, and range from 3-5 to 10 cm in diameter. The results from 21 samples, which were taken under the Thang Long Bridge foundation, show that there is 5.2 to 63.3% of gravel (average 25.6%), and about 24.6 to 76.6% (average 51.6%) of pebbles.

The yellow sand and pebble material is quartzite. It is characterized by an average gravel percentage of 12.8% and pebbles of 24.6 %. The rest of the particle percentage is clean sand. The average initial void ratio is between 0.6 and 1.04. The dry friction angle is $35^{\circ}07'$ and the average saturated friction angle is $32^{\circ}50'$.

3.8 DETAILS OF SOIL PROPERTIES AT TWO SPECIFIC LOCATIONS

3.8.1 GENERAL

In this part, the detailed soil properties at two locations within the research area are presented and summarized in Figures 3.7 through 3.24. The previous sections have discussed the age of the soil, and the typical mechanical and physical properties of each soil type. In this part the detailed soil properties at specific locations, especially those areas close to the water pumping stations, is presented in order to clearly show the soil properties that have not been presented in the general part. These specific soil properties can also be used in the finite element analysis in the next part of the thesis.

3.8.2 THE UNIVERSITY OF EDUCATION, CAU GIAY, HANOI

This area is located between the Ngoc Ha and the Mai Dich water pumping station. It lies in the area classified as A2 soil type in Figure 3.6b. The soil tests that were carried out include laboratory tests (direct shear tests, triaxial tests, unconfined compression tests, and consolidation tests) and field tests (such as CPT, SPT).

A summary of the soil properties are shown in Figures 3.7 to 3.18.

3.8.3 THE PHAP VAN WATER PUMPING STATION

From the surface down to a depth of 40m, the soil at the Phap Van pumped water station can be generally divided into 6 layers. Figure 4.15 shows the description of the soil layers and their sequences.

- Layer 1 is man made soil with thickness that varies between 2.0 and 3.5m.
- Layer 2 is soft to medium brown clay. The colour of this clay is actually brownish-yellowish at the top of the layer, and gradually changes to a greyish-blue colour at the bottom of the layer. The soil consists of 28% sand sized grains ($d=2-0.05\text{mm}$), 48% of silt ($d=0.05-0.005\text{mm}$), and 24% of clay ($d<0.005\text{mm}$). The depth of this layer varies between 3.0m and 7.0m.
- Layer 3 is very soft silt and silty clay. The depth of this layer is from 7.0m to 14.0m.
- Layer 4 is soft to medium light brownish sandy clay. The depth of this layer is from 14.0m to 33.0m.
- Layer 5 is light brownish clay. It is medium to firm clay and lies at a depth varying between 33.0m and 40m.
- Layer 6 is fine to medium sand. It is medium dense sand with a thickness of about 3m
- Layer 7 is a gravel and pebble layer. The thickness of this layer is about 28 to 30m.

Figure 3.19 shows average values of static cone resistance at the Phap Van water station (0 to 30m depth).

3.8.4 SPT VALUES AT SOME SPECIFIC LOCATIONS IN THE HANOI AREA

Figures 3.20 to 3.25 present SPT information of some specific locations around the Hanoi urban area. Figure 3.20 shows the approximate locations of the specific points where SPT blowcount data were collected. Figures 3.21 to 3.25 show the SPT blowcount values.

3.9 HYDROLOGICAL CONDITIONS

3.9.1 GENERAL

Based on the Hydrological Investigation report for Hanoi City, Report Number 2674, and the Hydrological maps for Hanoi City, drawn to scale of 1/10000, which were published by Geological Group 64, Hydrological Division II, Viet Nam Geological Survey Institute (After M. Tran, T. V. Pham, 1993), the Quaternary sediments in Hanoi (or in the Red River delta in general) can be divided into three significant water-bearing complexes (qh, qp2, qp1) and two impervious layers (Table 3.11).

3.9.2 THE HOLOCENE AQUIFER (QH)

The Holocene aquifer is distributed widely within the Hanoi area. It can be found from the West Lake to the Red River dike and then to the south of Hanoi. In the centre of the city, it appears at the West Lake bank, the Sky Market, Polytechnic University, Kim Lien, Trung Tu, Bach Mai airport, Kim Giang, Thanh Xuan, and Ha Dong. The Holocene aquifer (qh) lies between two impermeable layers:

- Above this Holocene aquifer (qh) is an impervious layer of clay or sandy clay. The thickness of this impervious layer is between 2m and 21m, with an average thickness of about 6.73m.
- Below this Holocene aquifer is a clay, sandy clay and clayey silt aquitard, which belongs to the Hai Hung stratum.

The thickness of this aquifer varies from 3 to 29m, with an average thickness of about 14m. The constituents of this aquifer are clayey sand, fine sand, fine to medium pebbles and gravel that may be encountered at the bottom of this stratum. These sediments belong to the Thai Binh alluvial deposit.

There is no gauge pressure or only local weak pressure in the piezometers. Transmissivity varies between 20 m²/day and 790 m²/day. The average value of transmissivity is about 200-400 m²/day. The initial pressure-head level of this aquifer is usually 2-4m below the ground surface. The water recharge sources of this aquifer are from surface water such as rainfall, lakes and ponds, and rivers, and the water level depends a lot on environmental and geologic conditions. In some areas, due to the hydraulic connection with some aquifers, the water exploitation activities from

those deeper aquifers have caused a significant lowering of this aquifer's piezometric level. The water level has been depressed in a cone shape around water pumping plants such as Ngo Si Lien, the army museum, Buoi, and Ha Dong.

3.9.3 THE IMPERVIOUS COMPLEX PLEISTOCENE-HOLOCENE

This impervious layer covers a large area of the research region and lies contiguously below the Holocene aquifer (qh) and above the upper Pleistocene aquifer (qh²). However in some areas it exposes due to the disappearance of the Holocene aquifer (qh) such as in the Mai Dich-Co Nhue region. The components of this layer include clay, sandy clay, silt and clayey silt containing black or greyish black vegetable remnants. They are lake, swamp and shallow marine sediments. The average thickness of this impervious layer is about 11.19 m in the Hanoi urban region and from 8 to 10 m in the surrounding areas.

3.9.4 THE UPPER PLEISTOCENE AQUIFER (QP²)

The upper Pleistocene aquifer is found all over the Delta area. Hence it can be found in most of the research area. This aquifer is distributed between two water-bearing layers, with the Holocene aquifer qh on top and the upper Pleistocene aquifer qp¹ underneath. The thickness changes from 2.5m to 24m and the average thickness is 11.06m over the whole area.

This aquifer is the Vinh Phuc alluvial layer that mainly consists of sandy clay, medium sand, and small pebbles, with gravel at the bottom. It exhibits artesian pressure and in some places the hydraulic transmissivity reaches 5000 m²/day. The water flow rate per meter head drawdown from the wells is from 15.2 (m³/day/m) to 462.2 (m³/day/m). The original pressure head in the aquifer was at about 3-4m below the ground surface. In the centre of the city, especially the areas that have been mainly affected by the water extraction, the phreatic surface of this aquifer is 10-13m or more below the ground surface. This aquifer has a hydraulic connection with the aquifers

water sources for it are mainly rainfall, lake and river water with some water coming from the Holocene aquifer (qh).

3.9.5 THE UPPER-LOWER PLEISTOCENE AQUITARD

This upper-lower Pleistocene aquitard is located below the upper Pleistocene aquifer (qp^2) and above the lower Pleistocene aquifer (qp^1). It covers most of the area of the Quaternary sediment zone. Nevertheless in many regions it has been completely eroded or it is very thin. Therefore, in those regions, there is virtually no separation between the two aquifers qp^2 and qp^1 . In that case both aquifers become only one aquifer (qp). The thickness varies between 0.5m and 16m, with an average thickness of 4.11m. The aquitard constituent consists of sandy clay interbedded with thin sandy layers and clayey sand.

3.9.6 THE LOWER PLEISTOCENE AQUIFER (qp^1)

This lower Pleistocene aquifer (qp^1) is the main and only water source supplying water for the whole Hanoi area. The aquifer covers most of the Red River delta. Therefore it is distributed all over the research area. The thickness of this aquifer ranges between 3 and 40m. It is very thin in areas such as Dong Anh, Soc Son and increases in thickness towards the centre of Hanoi.

This aquifer contains pebbles and gravel that are alluvium-pluvium of the Hanoi stratum and alluvium of the Lechi stratum. However, the appearance of clay and sandy clay sediments in this aquifer make its water storage vary. Pebbles and gravels of this aquifer are usually composed of quartzite constituents, sometimes calcareous, igneous, or tektites. Average particle sizes range from 2-5cm, and sometimes reach 10cm. In some areas this aquifer is separated from the above aquifer by an impermeable layer (the lower Pleistocene aquitard), but in other areas due to the lack of the aquitard there is virtually no separation between them. Consequently there is the existence of a hydraulic connection between the two aquifers, qp^1 and qp^2 . Hence, the two aquifers have the same pressure-head. In the past, the initial piezometric

surface was usually from 2 to 4m beneath the ground surface. But now, as a result of the increase in water extraction activities, the piezometric level has been decreasing.

Research into the lowering of the pressure head on 7/12/1991 showed that the pressure head was lowered in the whole urban area for example: Ha Dinh (-16m); Tuong Mai (-15m); Phap Van (-8.22m); Ha Dong (-3.8m); Ngoc Ha (-7.5m); Ngo Si Lien (-9.14m). The main sources of water supply for this aquifer are from the Red River and some from infiltration from the upper aquifers. Transmissivity is from 600 m²/day to 2900 m²/day.

Table 3.10: Division of aquifer-aquitard system in the Red River Delta.

Group	System	Geological period	Steps	Strata	Symbol	Geological column	Thickness (m)	Name and symbol of the hydrological strata
CAINOZOIC	QUATERNARY	Holocene	Upper	Thai Binh	$aQ_{IV}^{3\text{ }tb2}$		10-30	The Upper Holocene aquifer No pressure qh
					$aQ_{IV}^{3\text{ }tb1}$		15-20	The Upper Holocene aquifer No pressure qh
			Lower and Middle	Hai Hung	$m-lbQ_{IV}^{1,2\text{ }ht2}$		0.5-3	The impervious complex Pleistocene-Holocene
					$lbQ_{IV}^{1,2\text{ }ht1}$		2-20	The impervious complex Pleistocene-Holocene
		Pleistocene	Upper	Vinh Phuc	$lbQ_{III}^{2\text{ }VP3}$		3-8	The impervious complex Pleistocene-Holocene
					$lQ_{III}^{2\text{ }VP2}$		2-10	The impervious complex Pleistocene-Holocene
					$aQ_{III}^{2\text{ }VP1}$		5-20	The upper Pleistocene aquifer Low pressure qp ²
			Middle	Ha Noi	$aQ_{II-III}^{1\text{ }hn1}$		55	The upper-lower Pleistocene aquitard The lower Pleistocene aquifer. High pressure qp ¹
			Lower	Le chi	$aQ_I^{1\text{ }lc'}$		25	The lower Pleistocene aquifer. High pressure Qp ¹

3.10 SUMMARY AND CONCLUSIONS

In this chapter, a brief introduction to the geological and hydrological conditions of the Quaternary sediments in the Hanoi area is presented. In the first part of the chapter the sediment types, ages and their distributions are gradually introduced. The youngest sediments, Holocene, are normally fine and small grained sediments that are found all over the research area. In this layer of the Holocene sediments, there are sub-divisions of the sediments that are from the Lower and Middle Holocene, and the Upper Holocene. Pleistocene sediments are also divided into Upper Pleistocene sediments and Middle and Lower Pleistocene. The former includes four sub-strata and is thought by Geological Group 204 and some other geologists to be marine sediments, but the Hanoi Geological Group suggested that the Upper Pleistocene sediments are continental deposits. The later Middle and Lower Pleistocene is the final layer in the Quaternary sediment age scale. This sediment lies above the Tertiary formations.

The general stratigraphy of the Quaternary sediments is given in Table 3.1. Table 3.2 provides more detail of the Quaternary sediment in the Red River delta with descriptions and layer thicknesses shown. The distribution of the soft soil layers that mostly fall into the Holocene sediment categories is presented in Table 3.3 and Figure 3.6b. Unfortunately only the upper 20m of the soft soil layer near the ground surface has been studied in detail. Below 20m depth there is not enough information to assist in providing a detailed distribution map of the Quaternary sediments in the Hanoi area.

The mechanical properties of the Hanoi sediments are also presented. The typical values of the physical properties of the sediments are given in Tables 3.4 to Table 3.9. The soil properties of a typical location close to two ground water pumping stations, the Ngoc Ha and Mai Dich water pumping stations, are introduced. In these locations, the soil properties are studied in detail to a depth of 40m from the ground surface. These typical soil properties give a clear picture of the Hanoi aquifer and aquitard system.

Data is also provided for the hydrological conditions of the Hanoi soils. Table 3.11 presents a division of the aquifer and aquitard systems in the Red River delta. The system includes three water-bearing layers, namely qh , qp^1 , and qp^2 , and two impervious layers. Among these aquifers, the Lower Pleistocene Aquifer (qp^1) is the only main water source supplying water for the whole Hanoi area. It covers most of the Red River delta with thickness ranging between 3m and 40m.

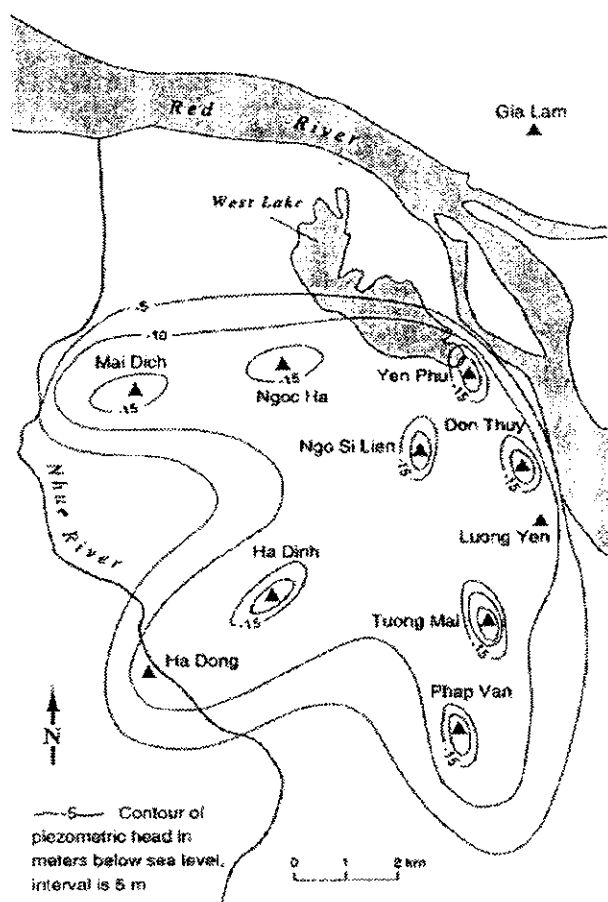


Figure 3.1 shows the Piezometric head in 1993 in the Hanoi urban area. Contour lines show piezometric head in m below sea level, interval 5m. ▲: Location of water plants (After T. Q. Nguyen and D. C. Helm, 1995).

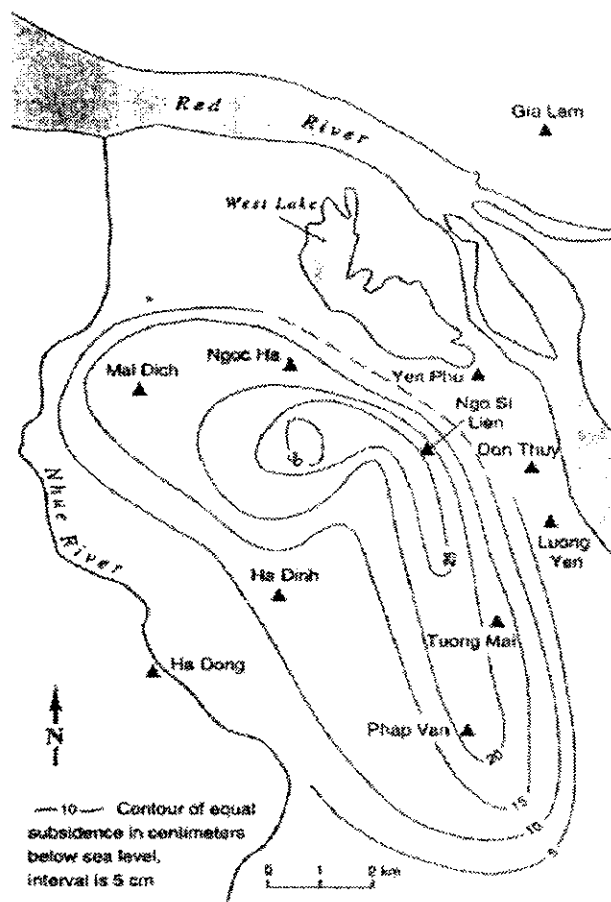


Figure 3.2 shows land surface subsidence in the Hanoi urban area, contour lines show land subsidence in cm, interval 5cm. ▲: Location of water plants (After T. Q. Nguyen and D. C. Helm, 1995).

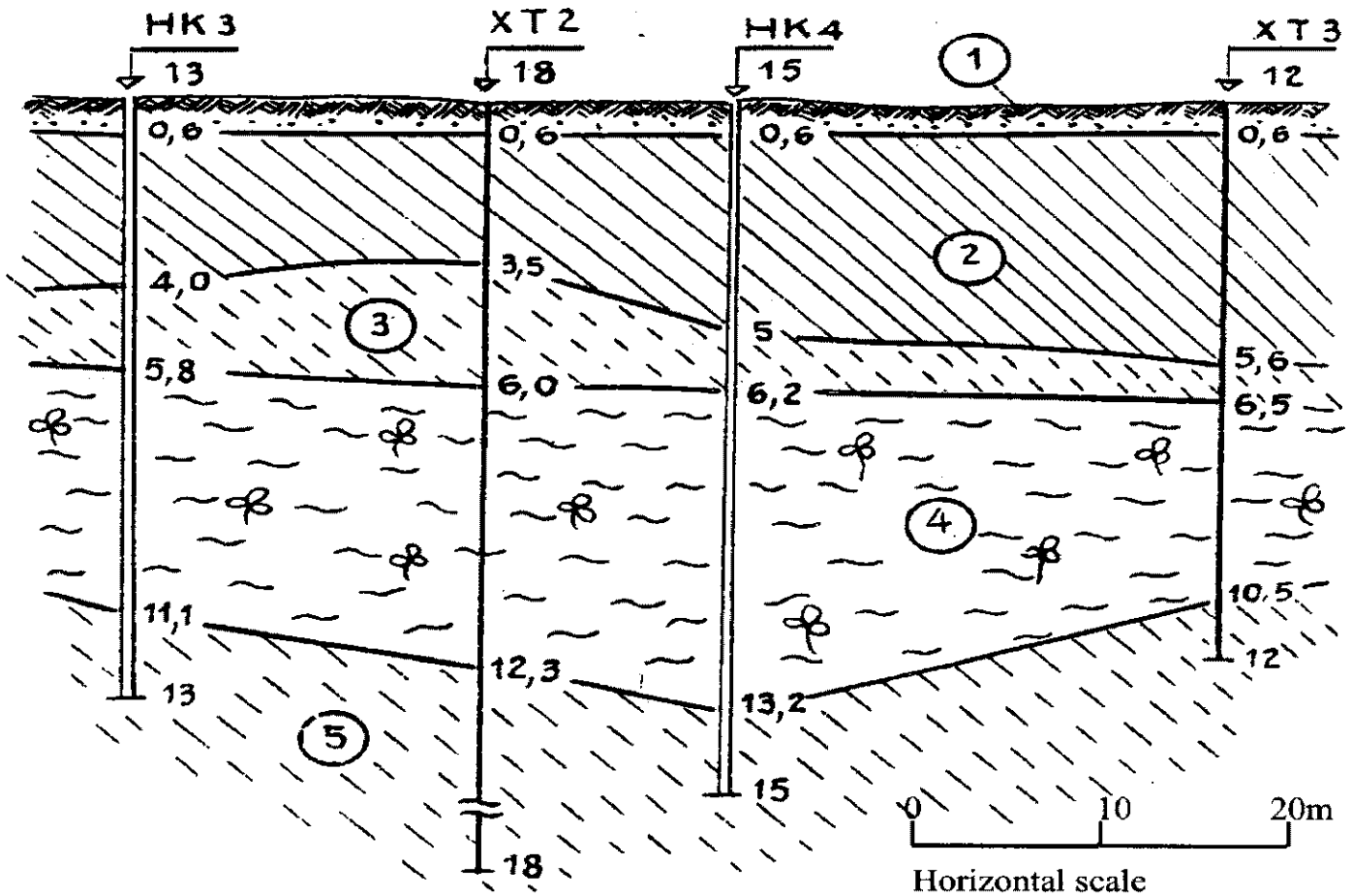


Figure 3.3: Soil layer profile beneath Building A5, Block II, – The Hanoi University of Education I.

HK3: bore hole number 3; XT2: cone penetration test number 2; (1): soil layer 1.

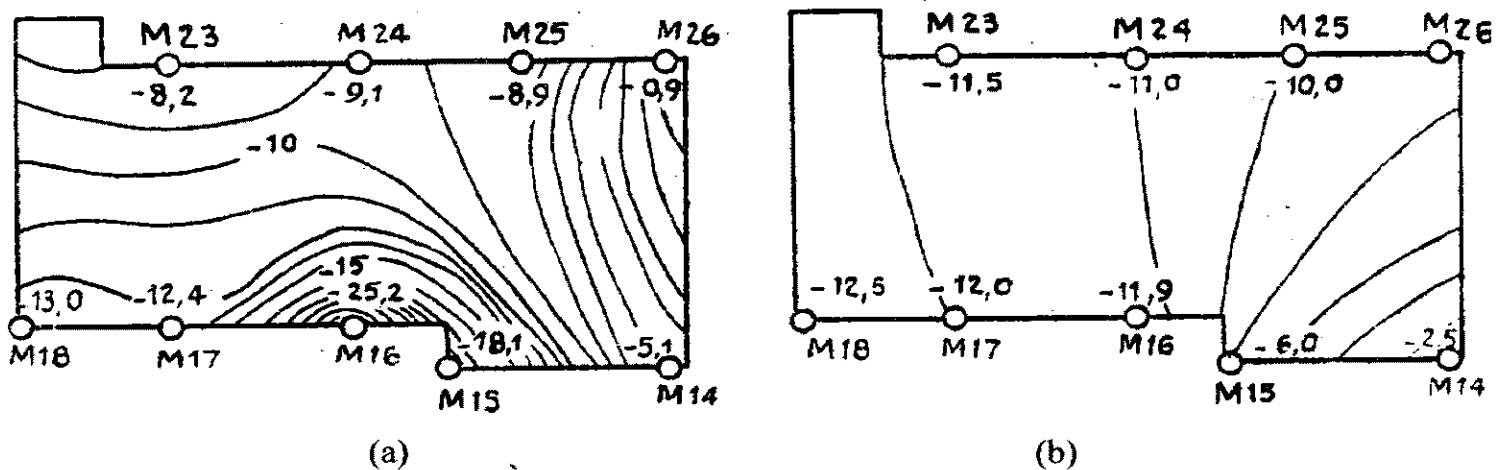


Figure 3.4: Contours of equal settlement (mm) of Block II, – Building A5 – The Hanoi University of Education I before (a) and after (b) the piles were placed.

M23: settlement measurement point number 23.

Figure 3.5a: The distribution of recent clay deposits in South East Asia (After Cox, 1970).

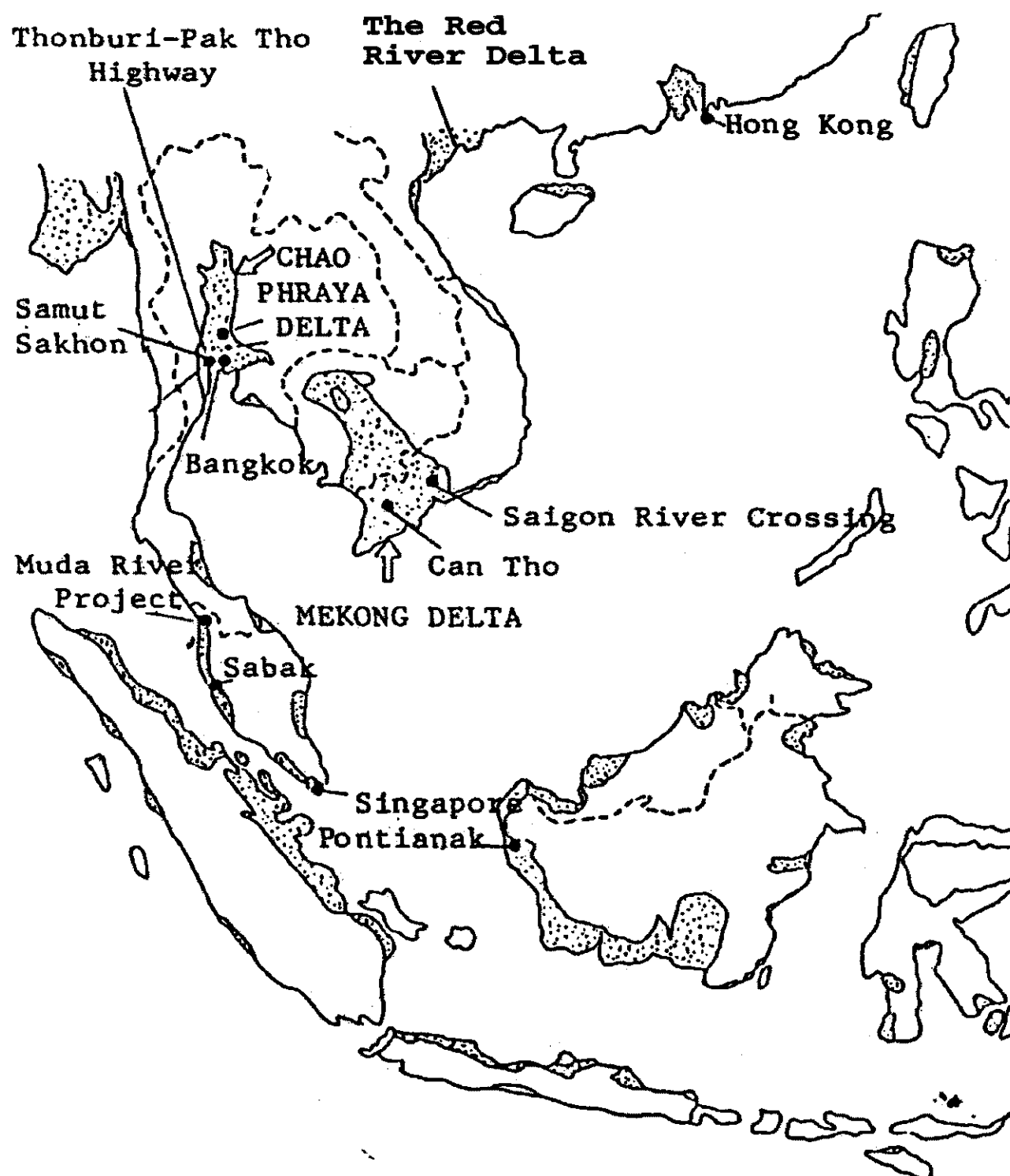


Figure 3.5b: Map of Hanoi and suburbs.

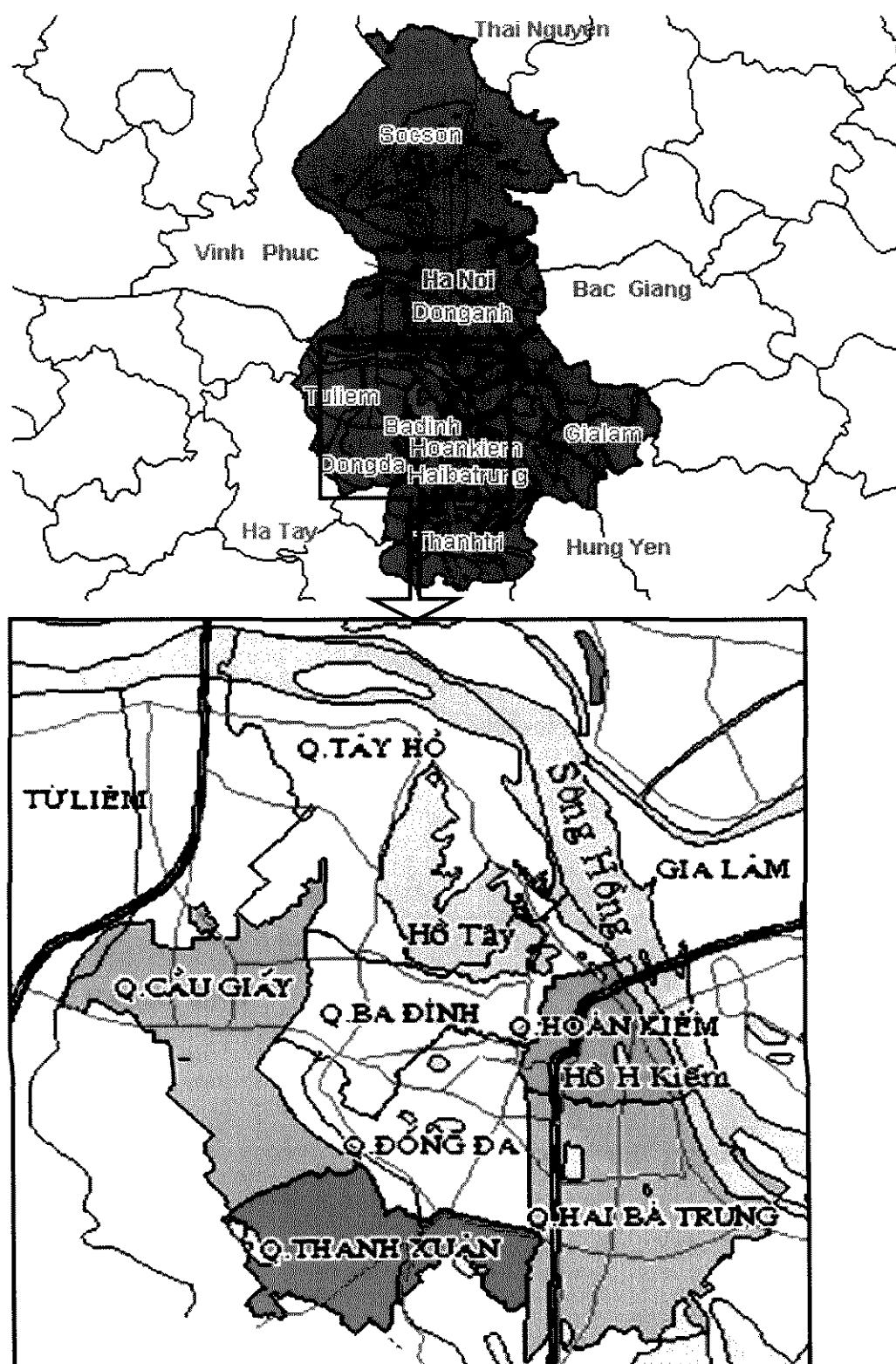
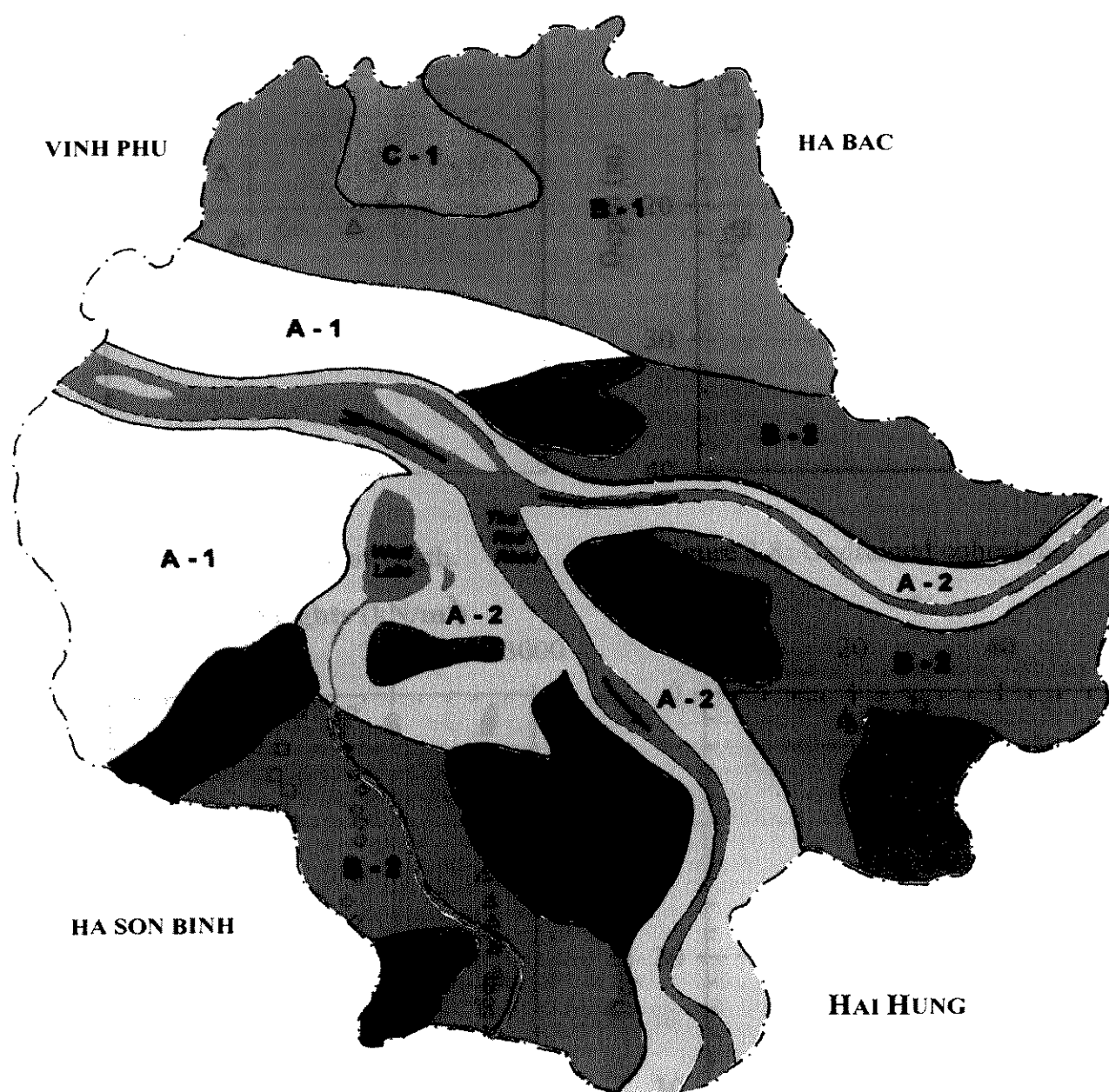


Figure 3.6b: Distribution of the soft soil layers to a maximum of 20m depth from the surface.



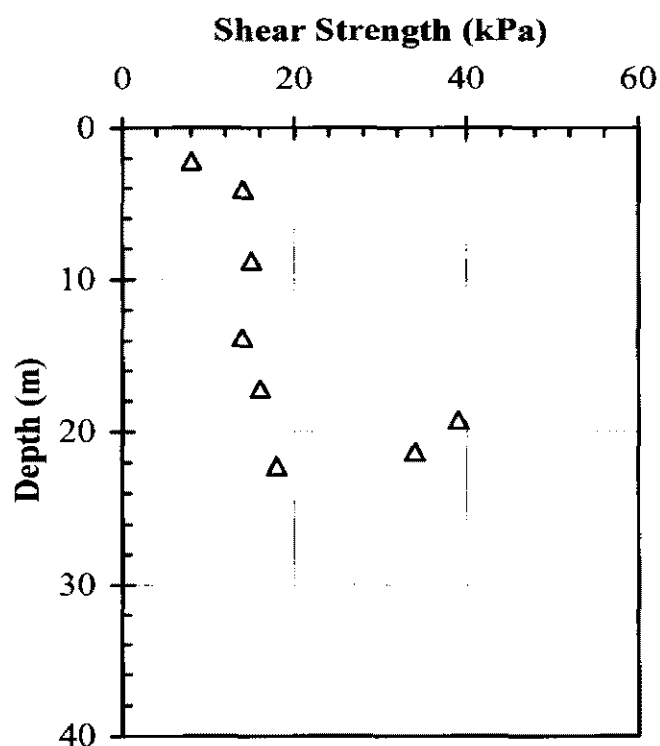


Figure 3.7: Undrained shear strength.

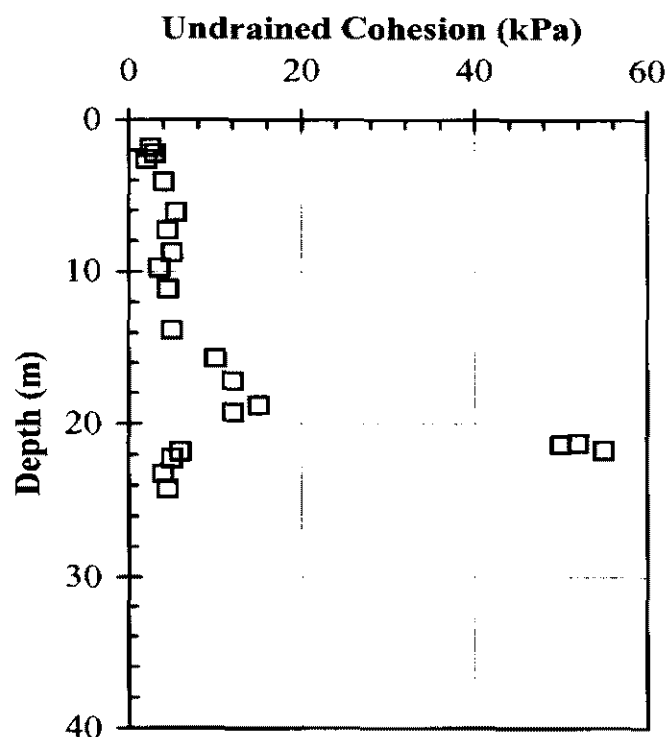


Figure 3.8: Undrained cohesion.

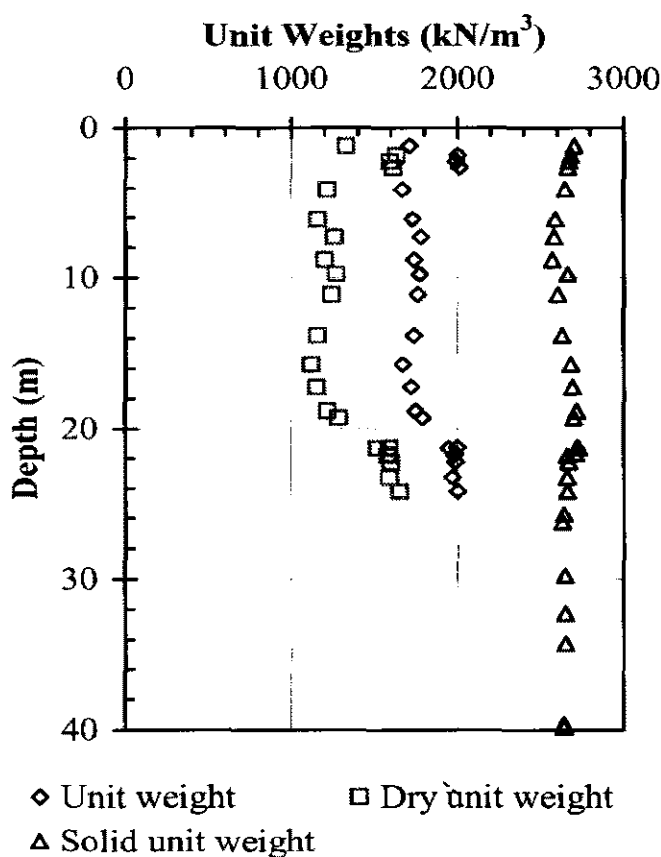


Figure 3.9: Unit weight of soils.

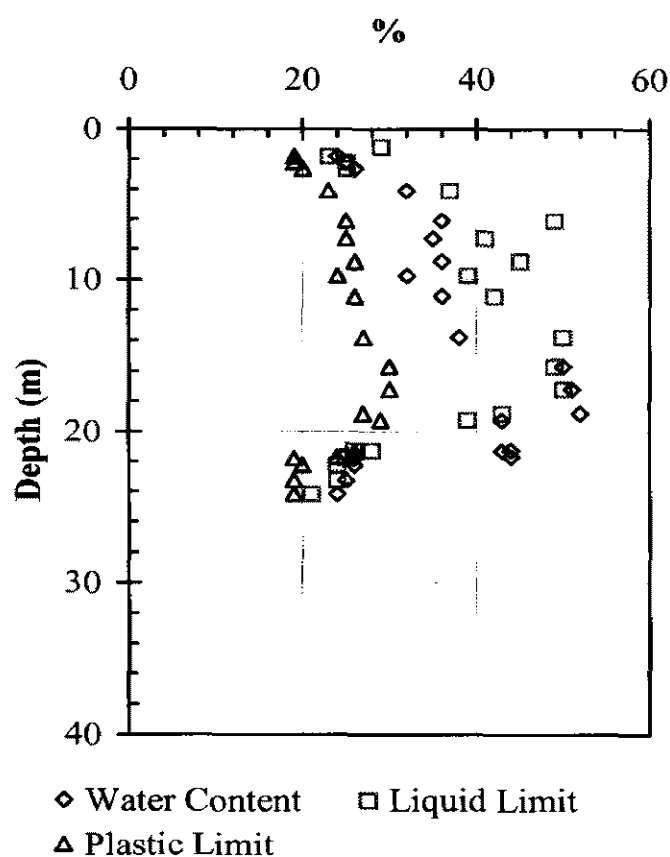


Figure 3.10: Atterberg Limits.

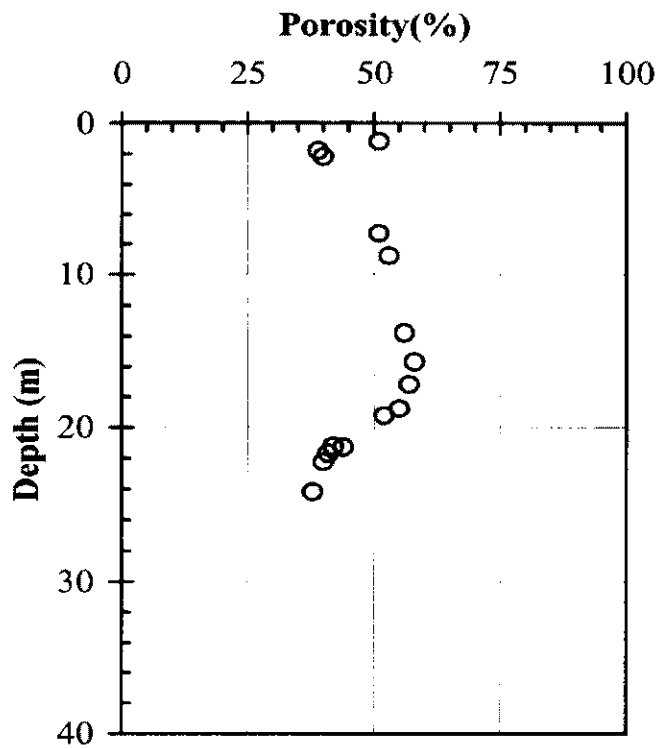


Figure 3.11: Porosity.

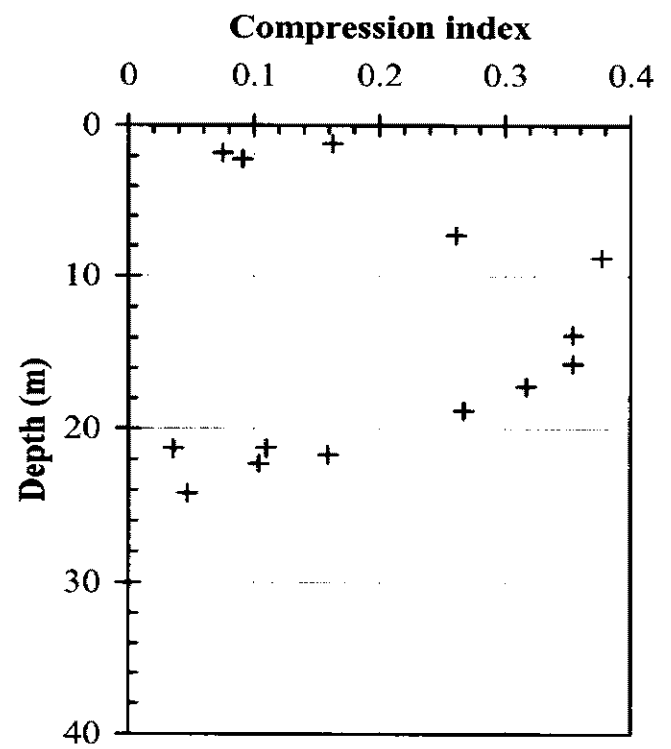


Figure 3.13: Compression index.

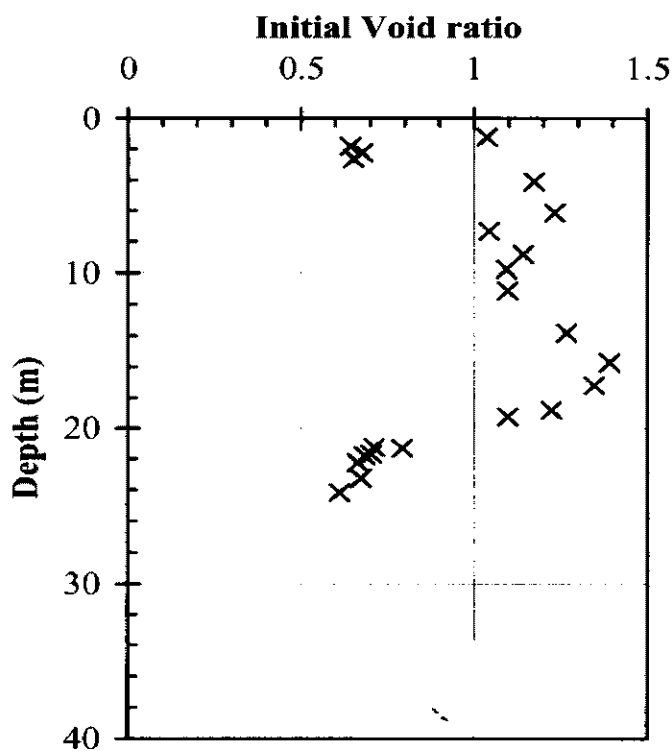


Figure 3.12: Initial void ratio.

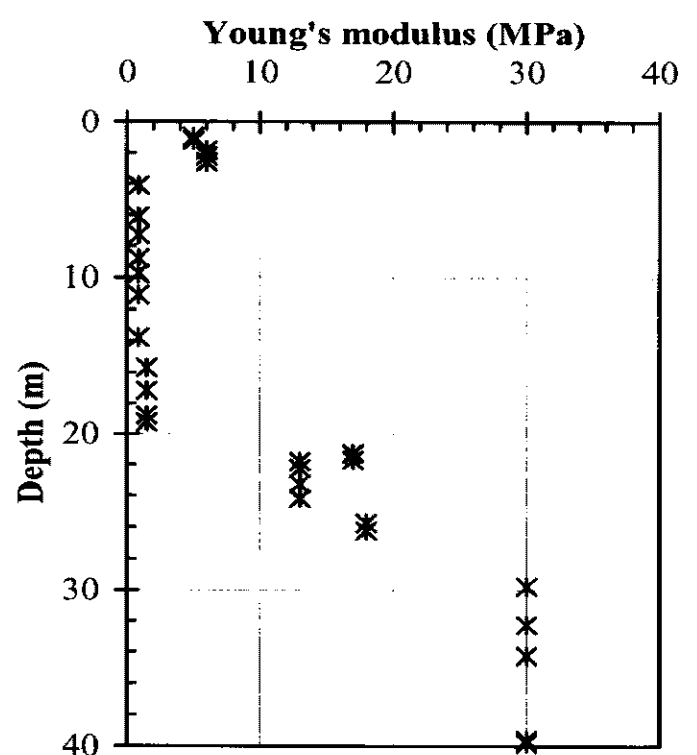


Figure 3.14: Average Young's modulus.

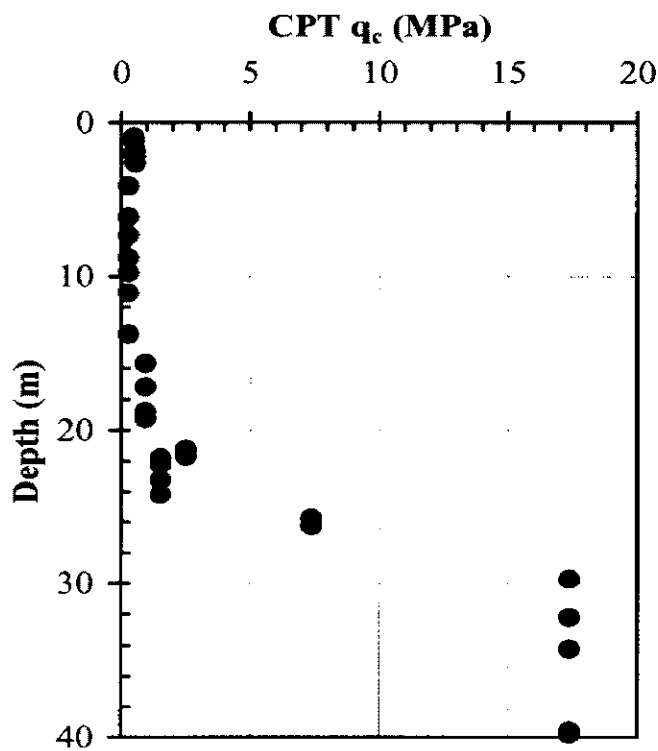


Figure 3.15: Average cone resistances.

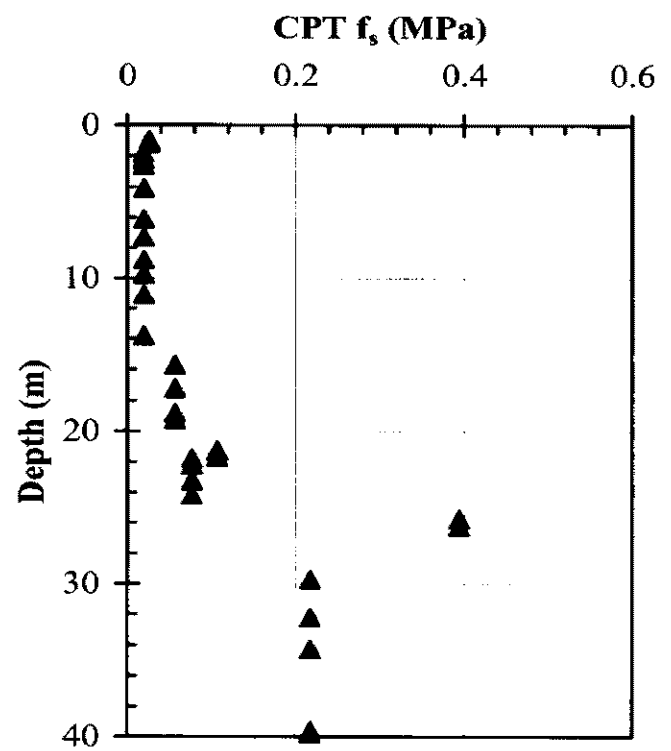


Figure 3.16: Average cone friction.

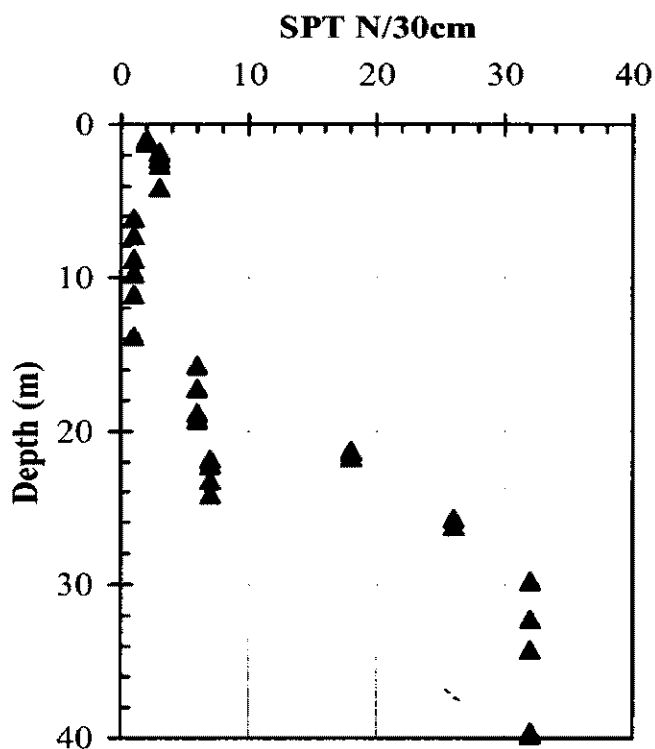
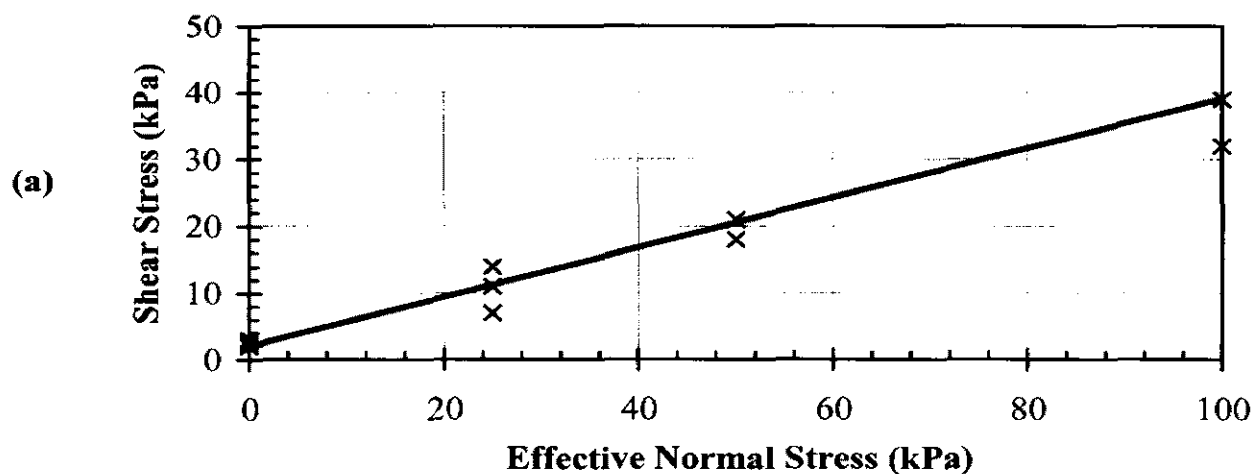


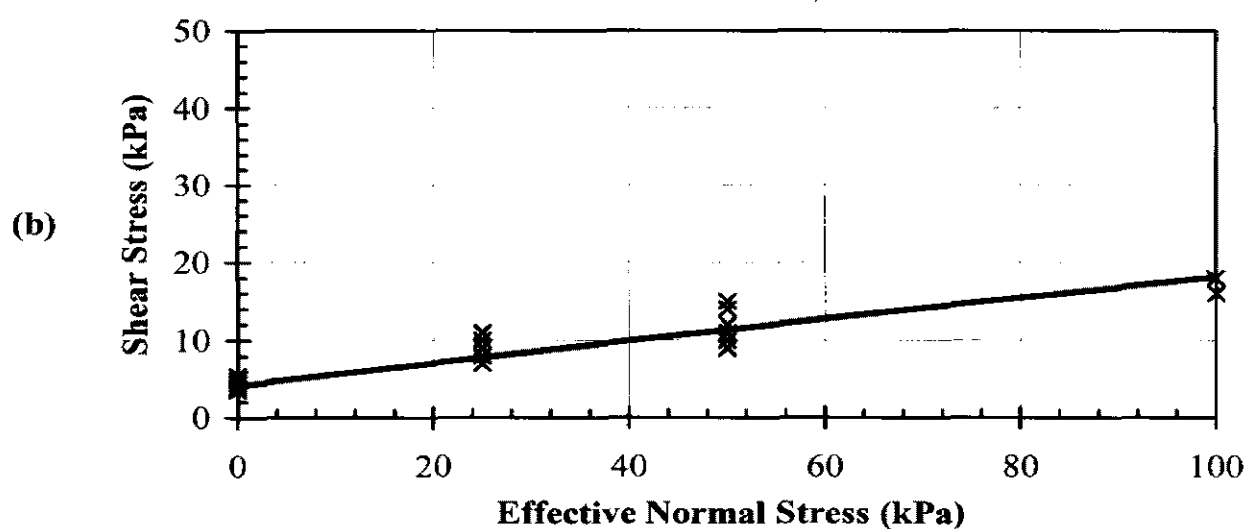
Figure 3.17: Average blowcount.

Note

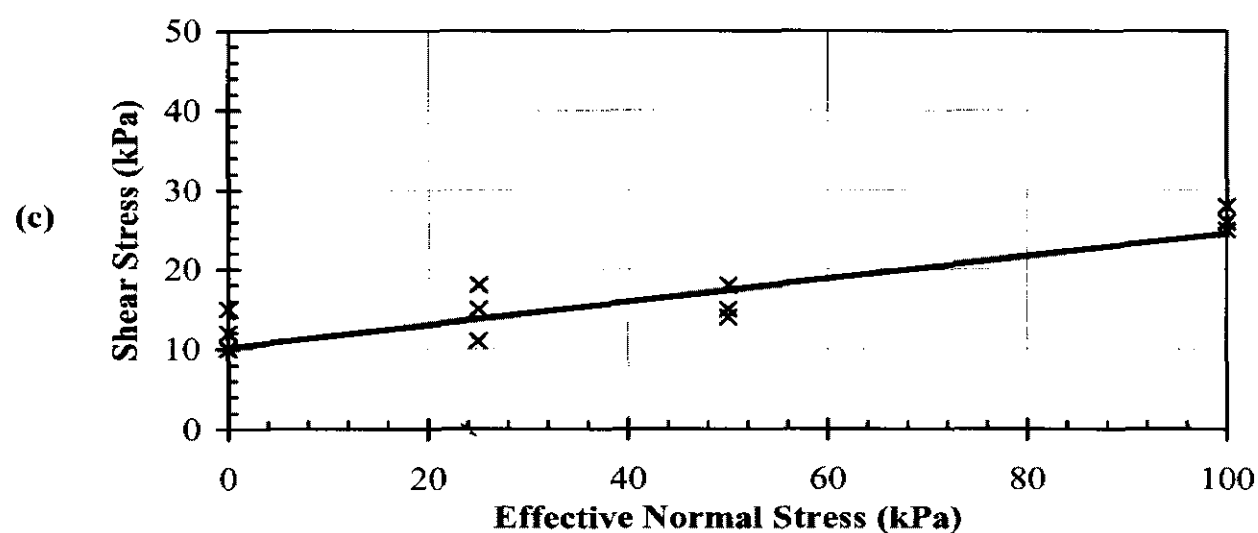
The values of E , q_c , f_s , and $N/30$ are average values and similar in each layer



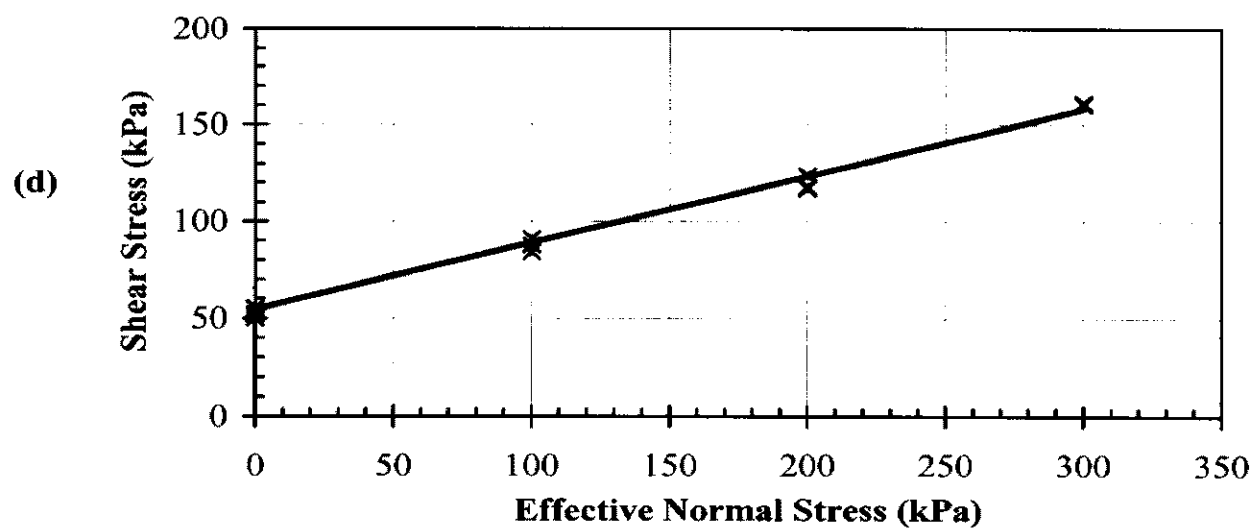
Clayey sand layer (Depth = 1.2 - 2.6m)



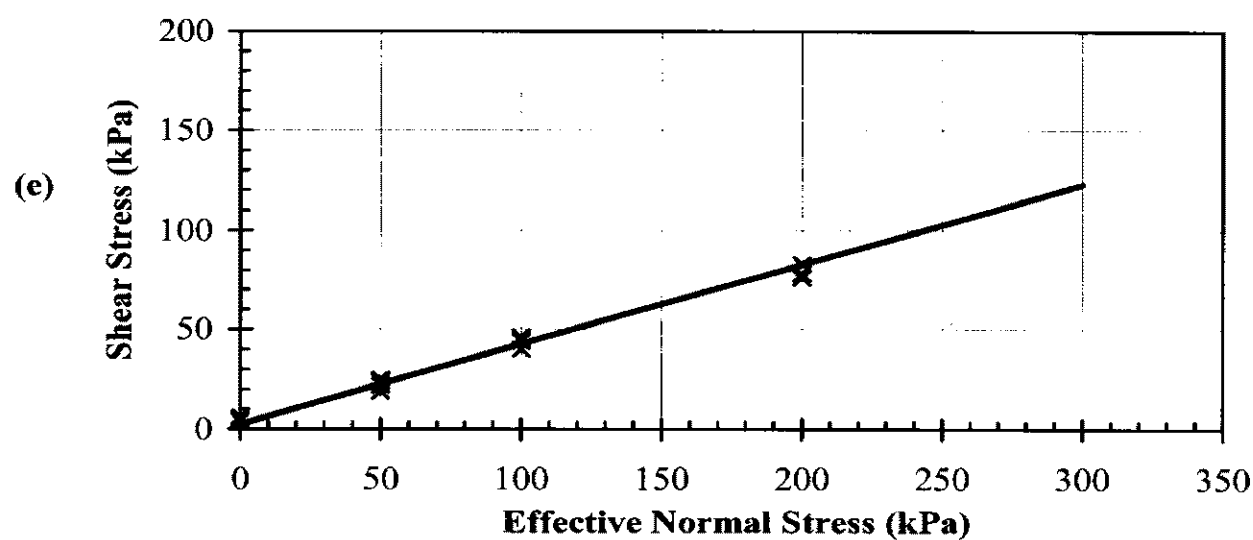
Soft sandy clay layer (Depth = 2.6 - 13.8m)



Firm clay layer (Depth = 13.8m - 19.5m)



Stiff clay layer (Depth = 19.5m - 21.7m)



Firm clayey sand layer (Depth = 21.7m - 24.2m)

Figure 3.18: Direct shear test results.

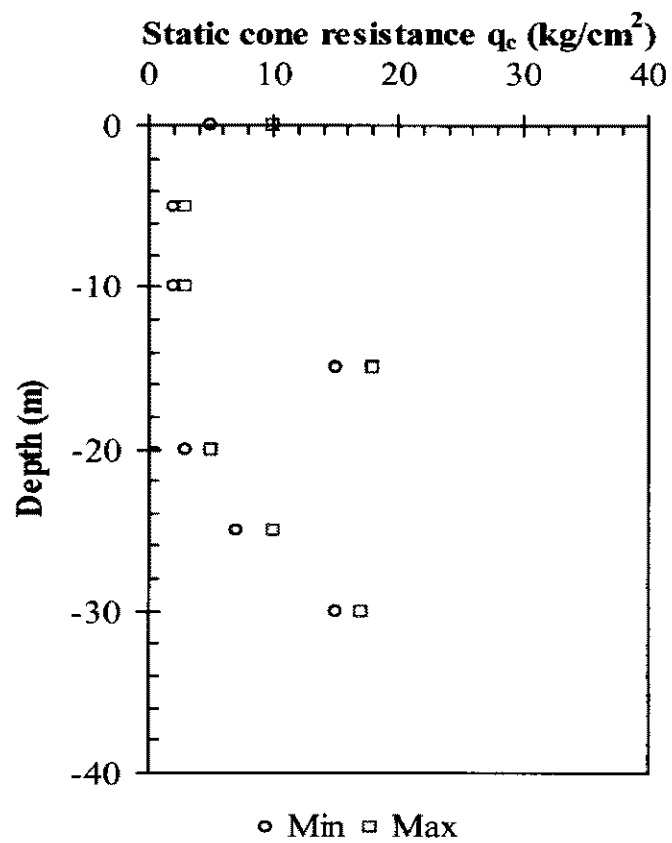


Figure 3.19: Average cone resistance at the Phap Van water station.

Figure 3.20: The specific locations for which SPT data is available.

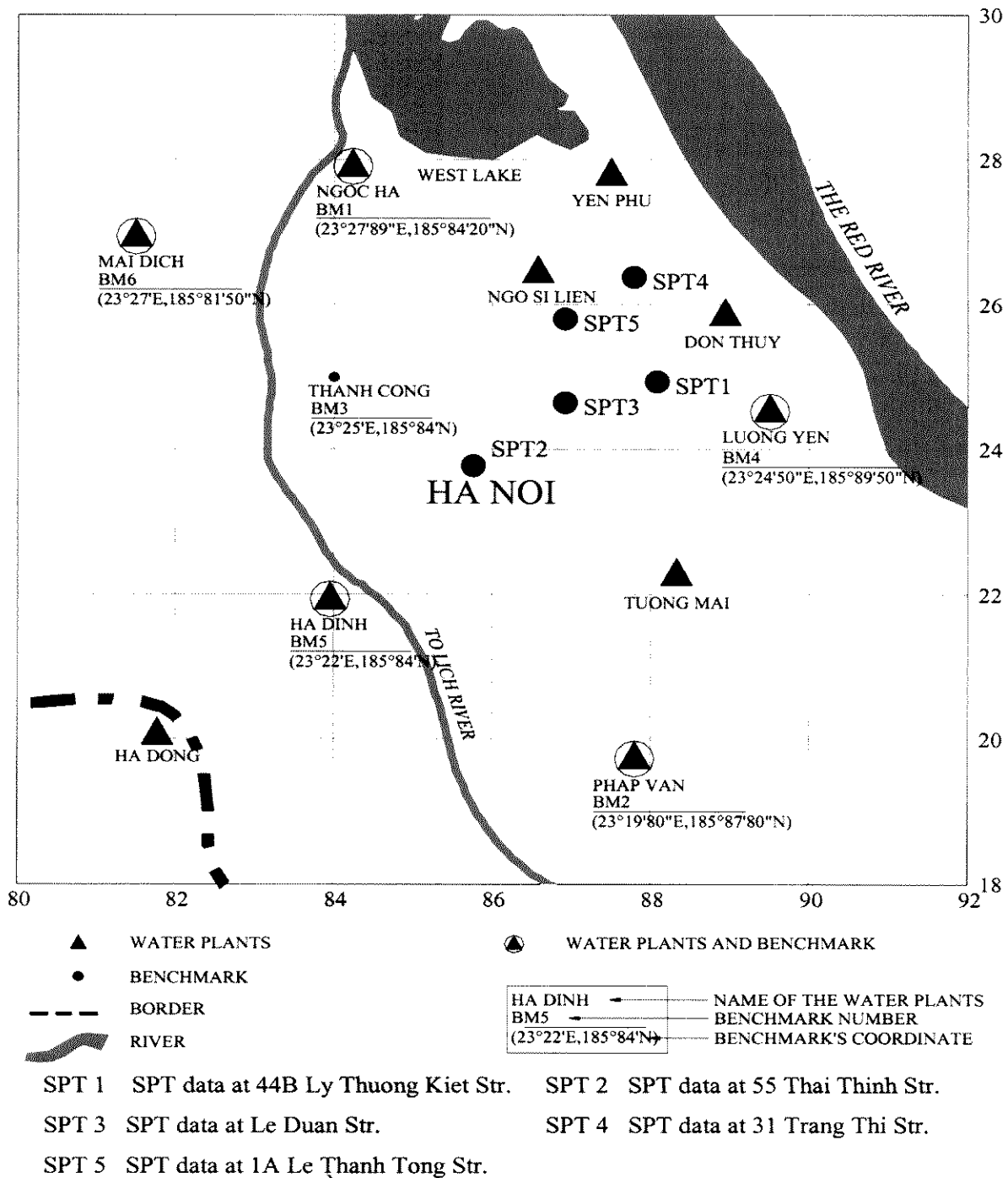


Figure 3.21: SPT data at 44B Ly Thuong Kiet Street, Hanoi.

Soil and Rock Engineering - Borehole No. 4

Project: Hanoi Plaza Hotel

Location: 44B Ly Thuong Kiet street, Hanoi

Layer 1: Fill layer

Layer 2: Soft to firm silty clay with organic content

Layer 3: Firm silty clay with thickness of 9.0m

Layer 4: medium silty sand

Layer 5: Medium dense to dense, fine to medium sand.

Layer 6: Very dense, medium to coarse sand mixed with gravel and cobbles.

Layer 7: Very dense, medium to coarse sand with high gravel and cobbles content.

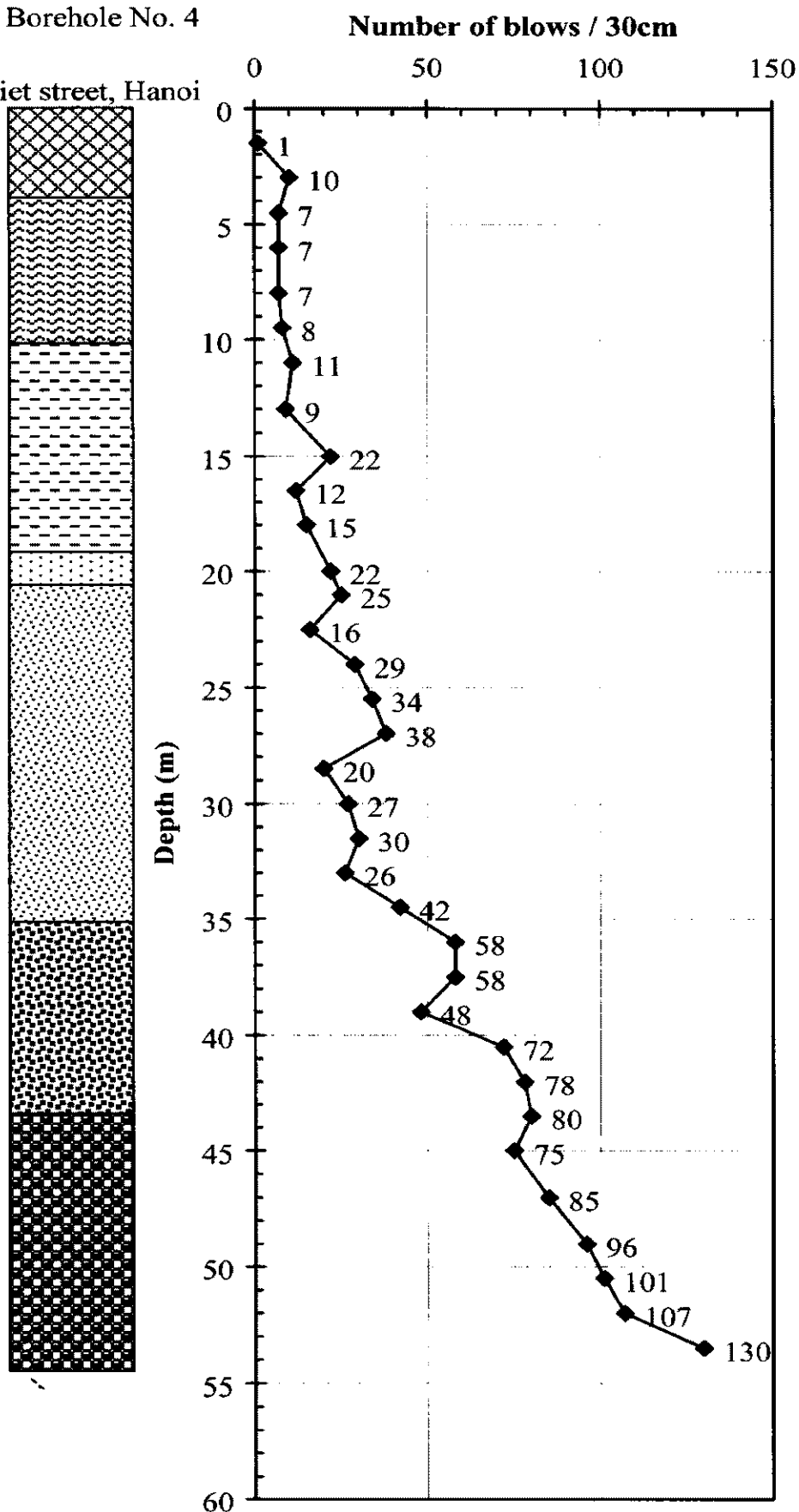


Figure 3.22: SPT data at 55 Thai Thing, Dong Da, Hanoi.

Project: Logitem Vietnam Corp.

Location: 57 Thai Thinh, Dong Da, Hanoi

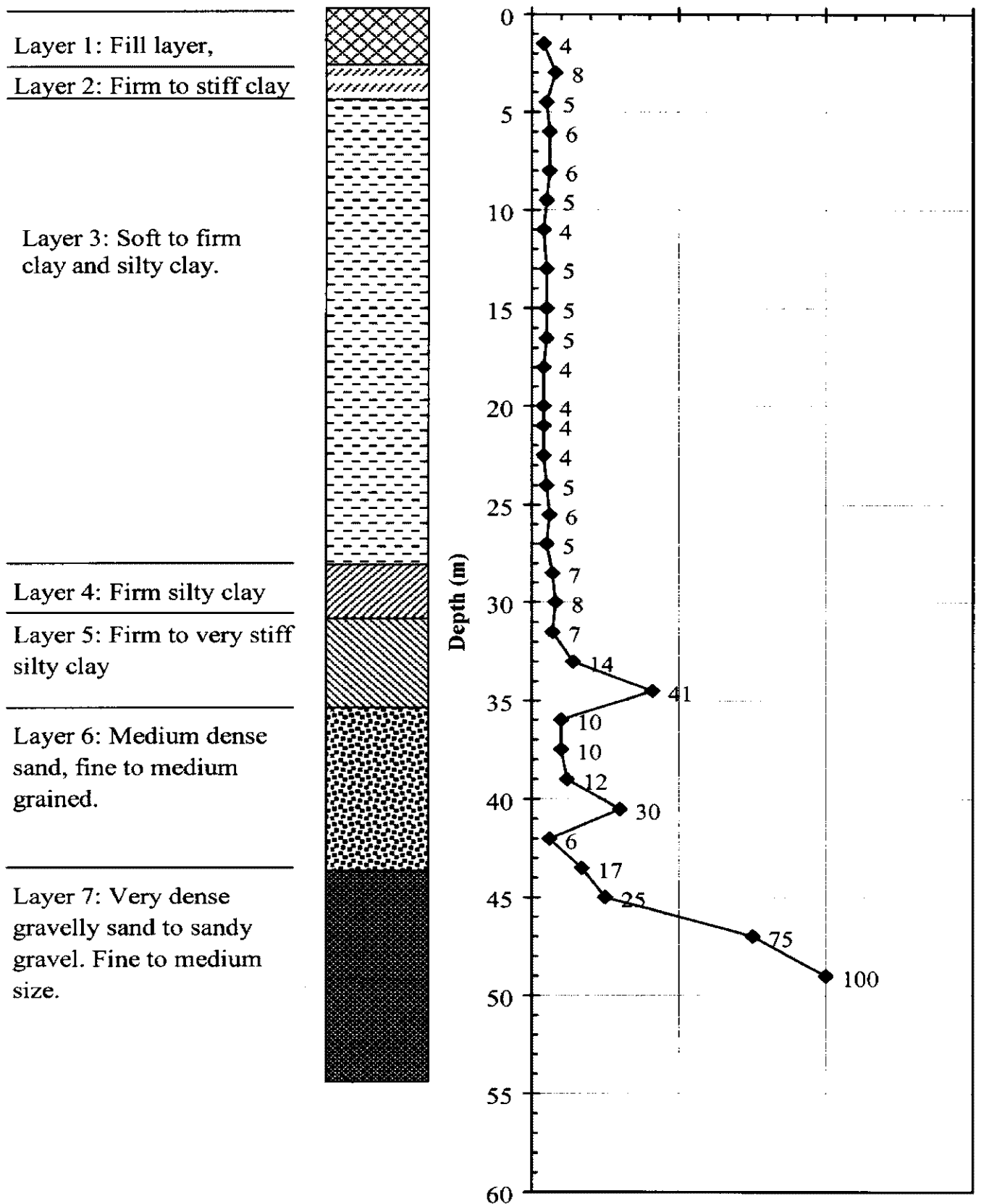


Figure 3.23: SPT data at Le Duan Street, Hanoi.

Project: SAS Royal Hotel

Location: Le Duan street, Hanoi

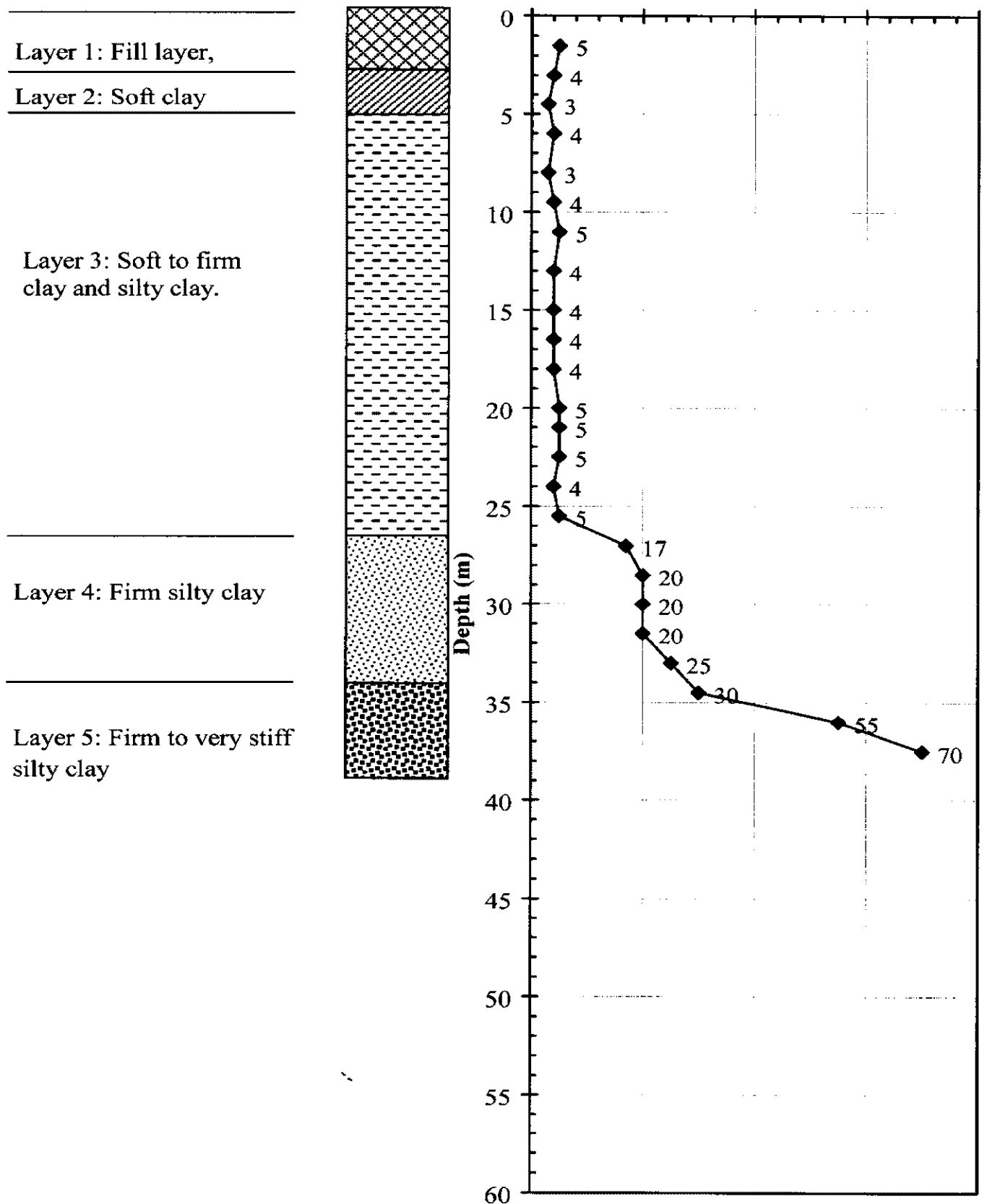


Figure 3.24: SPT data at 31 Trang Thi Street, Hanoi.

Project: National Library

Location: 31 Trang Thi, Hanoi

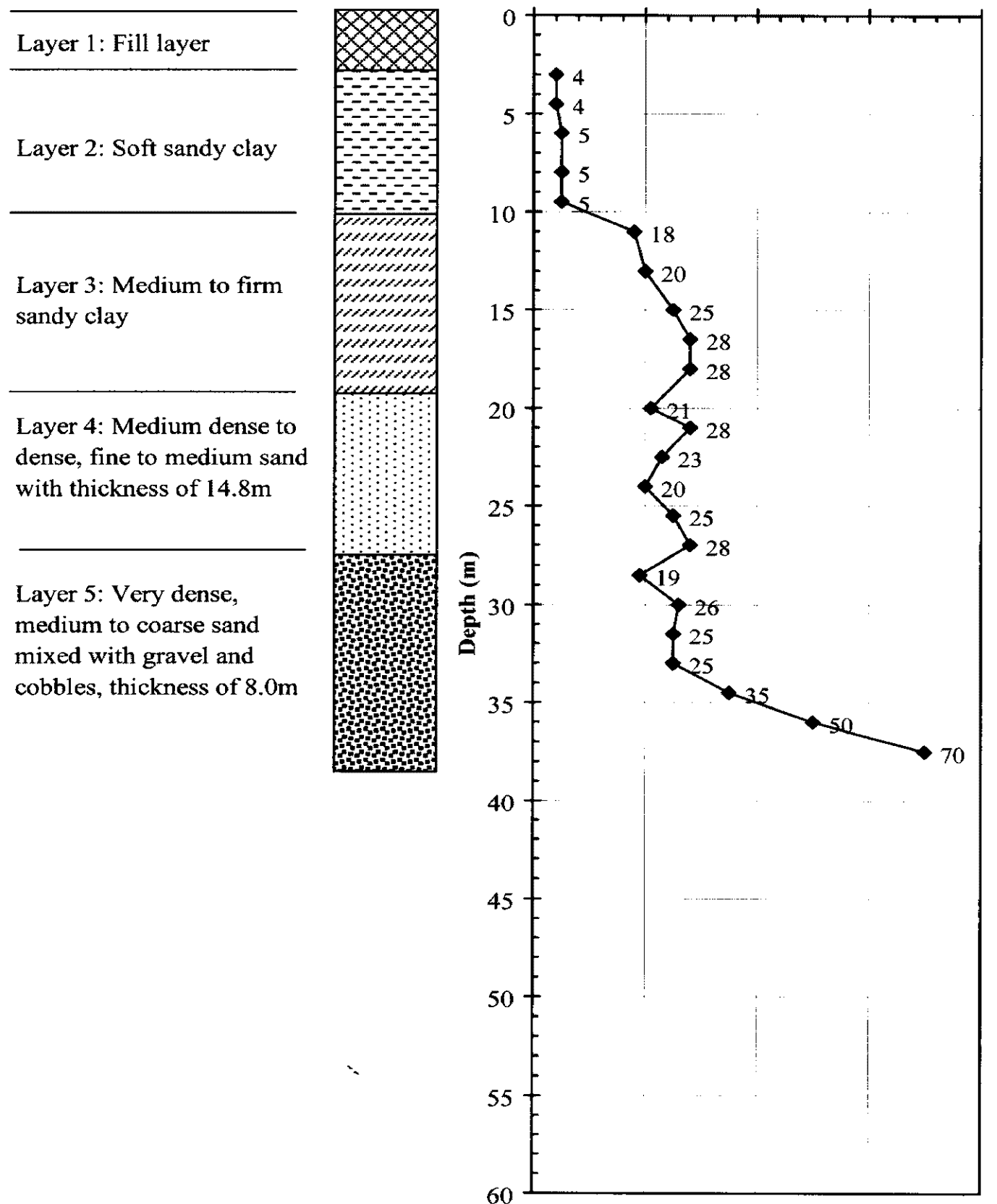
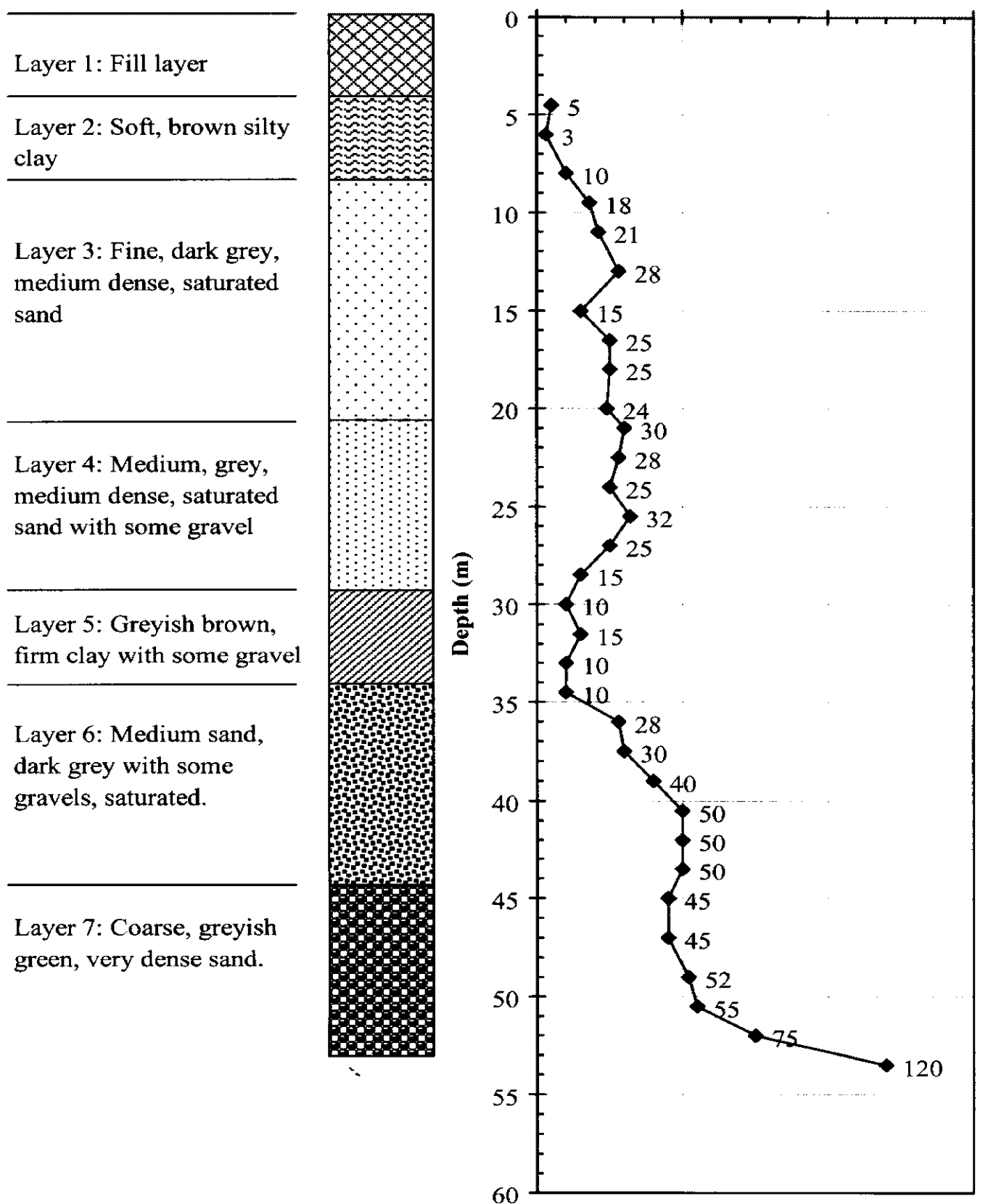


Figure 3.25: SPT data at 1A Le Thanh Tong Street, Hanoi.

Project: Hanoi Opera Hilton Hotel

Location: 1A Le Thanh Tong street, Hanoi



CHAPTER 4

LAND SUBSIDENCE

MEASUREMENT

4.1 GENERAL INTRODUCTION

According to M. Tran and T. V. Pham (1993) the first levelling over the Hanoi urban area was carried out in 1988, and involved a system of 32 measurement points over the area influenced by the piezometric depression cone. Measurement operations were performed once a year until 1991. However, inaccurate and out of date devices and lack of high precision levelling resulted in inaccurate results for land subsidence. In order to obtain a better result, a system of 6 benchmark stations was built in the areas where the water table drawdown might cause large deformation of the ground. Normally, the benchmark stations were put inside the ground water pumping plants. The name and the coordinate of each benchmark station are shown in Figures 4.1 and 4.2. At each benchmark station, the benchmark was installed and fixed to the Tertiary bedrock, usually about 90m below the surface. The station was surrounded by a fence to protect it from mechanical and environmental damage. The design and installation of those measurement instruments as conducted by Dr. V. H. Tran, Dr. N. D. Tran, Engineer V. H. Nguyen and N. A. Truong, who are from a division of the Hanoi Marine Institution, the National Technological and Science Centre, Vietnam. Six benchmarks, which were installed around the Hanoi area for monitoring tasks, were supported by Research Project number 01-36-04 of the Hanoi People's Committee.

The general purpose of this chapter is to review the design and the installation process of 6 benchmarks around the Hanoi area. The main purpose of this instrumentation program in the Hanoi area is (a) to monitor the total soil settlements and the lowering of the ground water level due to the pumping of ground water; (b) to investigate the subsoil layers underlying the area, where water is pumped from; (c) to provide sufficient data to permit a comparison between the numerically predicted results for the ground surface subsidence and the drawdown of the water table due to ground water pumping.

4.2 DESIGN AND INSTALLATION OF INSTRUMENTS FOR GROUND MOVEMENTS

4.2.1 GENERAL

Two types of ground movement measurement devices were used, which were a benchmark and measurement points. A benchmark is used for measuring the total soil settlements. Hence it is usually located in sound bedrock, where is not influenced by the movements of the ground. The measurement points were positioned at different depths, where the ground is subject to movements due to ground water pumping, for monitoring the different compressions of each soil layer. Design and installation of the ground movement measurement instruments and the measurement devices for pore water pressure will be discussed next.

4.2.2 DESIGN AND INSTALLATION OF BENCHMARKS

4.2.2.1 DESIGN OF BENCHMARKS

The benchmark and the components discussed below are shown schematically in Figure 4.3 with a detail of the top of the benchmark shown in Figure 4.4.

Central pipe

The central pipe is a steel pipe with a 73 mm outside diameter, a 9 mm wall thickness and is 93.00 m long. It is installed 92.35 m deep into the ground. From a depth of 80 m to 92.35 m it is firmly cemented into the sandstone layer (Neogen layer) in order to make it become a permanent benchmark.

The benchmark is used for three purposes:

- Indicator observations.
- Surveying measurements.
- Water table position measurements.

Box casing

The box casing is a tube that has an outside diameter of 108 mm. It is placed vertically into the ground on top of the sandstone layer's surface. Along the shaft of

the box-casing pipe, between 40 m to 45 m below the surface, there are drilled holes for the purpose of water table observation.

The spacers placed between the box casing and the central pipe at 5-m intervals protect the central rod from down drag and keep it permanently stable.

Settlement pipe

The settlement pipe is a tube of 146 mm (outside) diameter and is 3m long. It is placed 2.5 m into the ground and attached to a concrete pedestal (650 mm in diameter and 70 mm in thickness). It therefore allows about 500mm of settlement pipe to protrude above the ground surface.

This settlement pipe allows measurement of the total settlement of the ground surface at a depth of 3m (excluding the man-made soil layer on top).

Indicator

The indicator is installed on top of the benchmark to measure the differential movements between the benchmark and the upper casing and hence measures the total settlement of the ground surface. An accuracy of ± 0.01 mm is reported. In addition, the direct measurement of the ground settlement taken from the indicator can be compared with the ground settlement obtained from surveying measurements.

Protection pipe

The protection pipe has a diameter of 400mm and projects 850mm above the ground surface. It covers the whole top section of the benchmark, indicator, and box casing. It protects them from any natural and human activities on the ground surface that may affect the accuracy of the measurement results.

4.2.2.2 INSTALLATION OF BENCHMARKS

The benchmark is installed so that the inner rod reaches a solid sandstone layer (that is found at depths of between 75m and 80m). The method of installation is as follows:

- A hole of diameter of 650 mm is drilled from 0 m to 3 m in the sandstone bedrock and the bottom of the hole is washed by pumping bentonite (10%-15% NaOH) into

the hole. Then a concrete base is placed at 3 m depth and the settlement pile is placed. The rest of the hole is back filled and compacted with sand that is compacted into the hole.

- From 3 m to the surface of the sandstone layer a hole of 132mm diameter is drilled. Then the bottom of the hole is washed and the casing pipe ($\phi = 108$ mm) is placed. The foot of the casing pipe is installed at the surface of the sandstone layer. A hole is drilled to the required depth into the sandstone layer (Neogen) and the bottom of the hole washed by bentonite and clean water.
- The bored hole is then cleaned with compressed air.
- The hole is re-drilled and washed again by clean water to remove all drill-dust in the bored hole.
- The volume of the bored hole in the sandstone layer is then measured.
- The central pipe ($\phi = 73$ mm) is lowered to the design depth and the foot of the central pipe is cemented into the sandstone.
- The indicator and protection box are installed.
- The indicator is placed on top of the settlement pipe ($\phi = 146$ mm). Its accuracy is $\pm 1/100$ mm.

4.2.3 DESIGN AND INSTALLATION OF MEASUREMENT POINTS

4.2.3.1 DESIGN OF MEASUREMENT POINTS

In an observation station, apart from the benchmark there are three other measuring points on the ground surface that are called M1, M2, and M3.

M1 is used for measuring the total settlement of the ground surface excluding the man made layer on top. M2 is used for measuring the settlement of those layers lying bellow 11 m. Settlement of those layers lying below 25 m (below the water table level) is measured with an M3 type gauge.

The structure of the three types of measurement points is almost the same (see Figures 4.5 and 4.6) and detail of a typical measurement point, that includes three main parts, is presented below:

Central pipe

The central pipe is a tube that has a 50 mm (outside) diameter and a wall thickness of 5.5 mm. A spiral anchor is attached at the end of the central pipe tip. The spiral anchor is 110 mm in diameter, and is 300 mm long. It is screwed at least 0.5 m into the layer where settlement is to be measured. On top of the central pipe is placed a hemispherical stainless steel knob.

Outer protection pipe

This is a 108 mm (outside) diameter tube with a length that depends on the required length of each measurement point. There are two sealed rings at the two ends of the tube. Oil is poured into the space between the central and outer pipe, providing protection from corrosion, reducing shaft friction, and increasing the stability of the central pipe.

Surface protection pipe

Like the protection pipe of the benchmark that has been presented previously, this steel pipe is 400 mm in diameter and protrudes 0.6 m above the ground surface. It surrounds and protects the top section of the measurement point from damage.

4.2.3.2 INSTALLATION OF MEASUREMENT POINTS

The process of installing the measurement point is similar to the process of benchmark installation. In addition, there are some minor differences between the methods of installation of measurement points because of the differences in the design depth. The main process of installation is as follows:

- Drill a hole 500mm in diameter to the design depth. Install the measurement point by either screwing or pushing it into the ground
- Pour sand and compact it around the protection pipe ($\phi = 127$ mm).
- Install the protection box.

4.3 DESIGN AND INSTALLATION OF INSTRUMENTATION FOR PORE WATER PRESSURE MEASUREMENT

4.3.1 GENERAL

The numbers of piezometers at each measurement station are different. However there are two typical types of piezometers that have been used to measure the pore pressure, namely, hydraulic piezometers and pneumatic piezometers. A pneumatic piezometer is used for measuring pore pressure in the clayey layers, while the hydraulic piezometer is used for measuring pore pressures in the sandy layers and the saturated sandy layers. A sketch showing details of a hydraulic piezometer system and a hydraulic piezometer tip are shown in Figures 4.7 and 4.8. The design of each kind of piezometer that has been used to measure pore pressure in the Hanoi region will be presented below.

4.3.2 DESIGN AND INSTALLATION OF HYDRAULIC PIEZOMETERS

4.3.2.1 DESIGN OF HYDRAULIC PIEZOMETER

Hydraulic piezometer tip

This is the main part of the system. It is actually a porous element, connected to two plastic tubes filled with de-aired water. The tip is placed in a bed of coarse clean filter sand. This bed of sand is carefully compacted and separated from the backfill grout by a plug of bentonite granules so that the backfill cannot contaminate the sand filter.

Twin plastic tubes

Twin plastic tubes are used to connect the readout system and the hydraulic head. De-aired water is circulated through these two tubes until the porous tip is completely filled.

Protection tube

In order to protect the twin plastic tubes from damage that may occur during installation or use, a protection tube or conduit is used to cover the two plastic tubes.

Readout station

The readout station includes several parts: a water supply tank, pump, valve, and a range of measuring systems including a mercury manometer, transducers and bourdon gauges.

Protection box

This box is designed to protect the readout station from any possible mechanical damage on the surface.

4.3.2.2 INSTALLATION OF HYDRAULIC PIEZOMETERS

Firstly, a hole with a diameter of 76 mm is drilled to the design depth and washed by bentonite and clean water. Then the piezometer is installed. The tip of the piezometer is usually located in a bed of coarse, clean filter sand. On top of the piezometer tip, in the borehole, there is a plug of bentonite in order to provide protection from contamination of the sand filter with the back-fill grout. Twin plastic tubes then are connected to the piezometer tip and the readout station. They are put inside a protection tube, called a conduit. The readout station is installed and protected by a protection box.

4.3.3 DESIGN AND INSTALLATION OF PNEUMATIC PIEZOMETERS**4.3.3.1 DESIGN OF PNEUMATIC PIEZOMETERS**

The design of the pneumatic piezometer is more or less the same as the design of the hydraulic piezometer. However the operating system and general principles for operation are different. The design of the pneumatic piezometer is shown in Figures 4.9, 4.10 and 4.11. The pore water pressure enters through the porous element and acts against the flexible diaphragm sealing off the pressure line. Gas pressure is applied to the diaphragm through the pressure line and when it is equal to the water pressure, the diaphragm will deflect allowing gas to flow up the return line to an air indicator in the readout station. The components of the pneumatic piezometer are shown schematically in Figures 4.9 and 4.10, and consist of:

- A pneumatic piezometer tip
- Two plastic tubes
- A readout station
- A protection tube
- A protection box

4.3.3.2 INSTALLATION OF PNEUMATIC PIEZOMETERS

The installation process for a pneumatic piezometer is similar to the installation process for a hydraulic piezometer. A hole with a diameter of 76 mm is drilled to the design depth and washed by bentonite and clean water. The piezometer tip is then installed with care into a bed of filter sand. The twin tubes connecting the piezometer tip and the readout station are then placed in a protection tube. The readout station is also protected from mechanical damage by a protection box.

4.4 THE NGOC HA BENCHMARK-BM1

The Ngoc Ha benchmark – BM1 is the first benchmark which was built in the Hanoi area for taking ground surface settlement and water level measurements. It was located inside the Ngoc Ha water plant that has been operating for about 30 years. The global longitude and latitude coordinates of the Ngoc Ha benchmark are 23°27'89"E and 185°84'20"N. During the 70's and 80's, the water level in the area was at about 3m below the ground surface. In the time period from the beginning of the 80's to the end of the 90's, due to the increase in the number of pumping plants and the pumping rate, the water level in the area was significantly lowered and caused settlement of the ground surface. The soil profile and distribution of piezometers (shown as U), the benchmarks, and measurement points (shown as M) are as shown in Figures 4.12 and 4.13.

4.5 THE PHAP VAN BENCHMARK-BM2

The Phap Van benchmark ($23^{\circ}19'80''$ and $185^{\circ}87'80''$) has been set inside the Phap Van water plant where there is a strong influence from the pumping of ground water from the Phap Van water plant. The Phap Van water plant was built and put into operation in 1988. It is located in the southern area of Hanoi City. This is a flat area, 3-4m above sea level, and the soil usually contains soft clay and peat. The stratigraphy of the Phap Van water plant area and the distributions of piezometers, benchmarks, and measurement points are as shown in Figures 4.14 and 4.15.

4.6 THE THANH CONG BENCHMARK-BM3

Thanh Cong and Giang Vo were swampy areas at the beginning of the 20-century. In the 70's it was decided to level (raze to the ground) existing buildings and to build high-rise buildings for residential living. In the period from 1988 to the present, the increase in ground water pumping in the Hanoi area has caused some of the old buildings, that had settled already and in use for about 30 years, as well as new buildings to start tilting and settling. Some reasons have been suggested to explain the phenomenon, and the drawdown of ground water level in the Hanoi area is one of the factors that may have affected the subsidence of those buildings. The Thanh Cong benchmark has longitude and latitude coordinates of $23^{\circ}25'E$ and $184^{\circ}84'N$. Figures 4.16 and 4.17 show the layout and the depth where the measurement instruments were installed.

4.7 THE LUONG YEN BENCHMARK-BM4

The Luong Yen benchmark ($23^{\circ}24'50''$ and $185^{\circ}89'50''$) is located inside the Luong Yen water plant that is near the Red River as shown in Figure 3.2. The Luong Yen water plant has existed for more than 10 years and water is pumped at a rate of 63000 m^3/day . The Luong Yen benchmark is shown in Figures 4.18 and 4.19.

4.8 THE HA DINH BENCHMARK-BM5

The Ha Dinh benchmark ($23^{\circ}22'00''$ and $185^{\circ}84'00''$) is located in the Southwest of Hanoi City where the industrial and cultural area lies (tobacco, rubber, electric bulb and filter factories, and the National University). The Ha Dinh water plant has existed for 30 years. The initial capacity of the plant was $18000\text{m}^3/\text{day}$ in 1970. It was $26000\text{m}^3/\text{day}$ in 1980, $37000\text{m}^3/\text{day}$ in 1990 and $43000\text{m}^3/\text{day}$ in 1995. Due to the increase in the pumping rate, the ground water level has been lowered gradually from -3m below the surface in 1970 to -27m in 1998 (Figures 4.20 and 4.21).

4.9 THE MAI DICH BENCHMARK-BM6

The Mai Dich benchmark number 6 ($23^{\circ}27'$ and $185^{\circ}81'50''$) is in the Northwest of Hanoi city where there are many important centers such as the Language Teaching University, Teaching University, Commercial College, and North Thang Long Advanced-Industry Complex. The Mai Dich Water plant was built and put into operation in 1988. Immediately after its began operating, private wells around the area were found to have dried out. After that, some buildings in this area tilted and subsided. (Figures 4.22 and 4.23)

4.10 MEASURED RESULTS

Figures 3.1 and 3.2 present the contours of the measured land subsidence and decline in piezometric heads in the five years from 1988 to 1993 in the Hanoi area (Nguyen and Helm, 1995).

Table 4.1 presents the summary of the depths to which measurement points were installed at six benchmarks in the Hanoi area . Table 4.2 shows the depths and types of piezometers that were used at the benchmarks.

In the next chapter measured results for water table drawdown and surface subsidence will be presented. Figure 5.9 shows the measured result for water table drawdown between 1988 and 1999 at the Phap Van water station. Figure 5.10 shows the observed water table level in 1992. The measured ground surface settlement at the

Phap Van water station between 1988 and 1999 is illustrated in Figure 5.11. At the Ngoc Ha water station, the measured results of the ground surface settlement and the water table drawdown are provided in Figures 5.18 and 5.19.

4.11 SUMMARY AND CONCLUSIONS

Information of measurement equipment used to monitor soil settlements and water levels in Hanoi have been presented. For ground movement measurements, two types of devices were used, namely, measurement points and benchmarks. A measurement point is actually used to monitor the movements of typical soil layers while a benchmark is used to observe the total movement of the ground surface. The different functions of measurement points and benchmarks lead to differences in design and installation of the two measurement devices. A benchmark is usually installed into a relatively deep and hard soil layer that can be considered to have no influence from any settlement of the soil layers lying above. In contrast, a measurement point is bedded into the soil layer that is required to be investigated.

For pore water pressure measurements there are also two types of devices, namely, hydraulic and pneumatic piezometers. The former is usually used for measuring pore pressure in sandy soil types and the latter is used to monitor pore pressure in clayey soil types. The designs and installation processes for the two types of equipment are discussed.

For the purpose of protection and to ensure these devices are working properly, a station with a high surrounding fence was provided for each benchmark. There are total of 6 standard benchmarks in Hanoi. Each benchmark is presented here with global coordinates as well as its location relative to a nearby water pumping station, and the layout of the benchmarks, measurement points, and pore pressure-measuring equipment. Information of the operation history of each benchmark station is also given. The location of soil layers beneath each benchmark station and their thickness are also introduced with the depth where these measurement devices were installed.

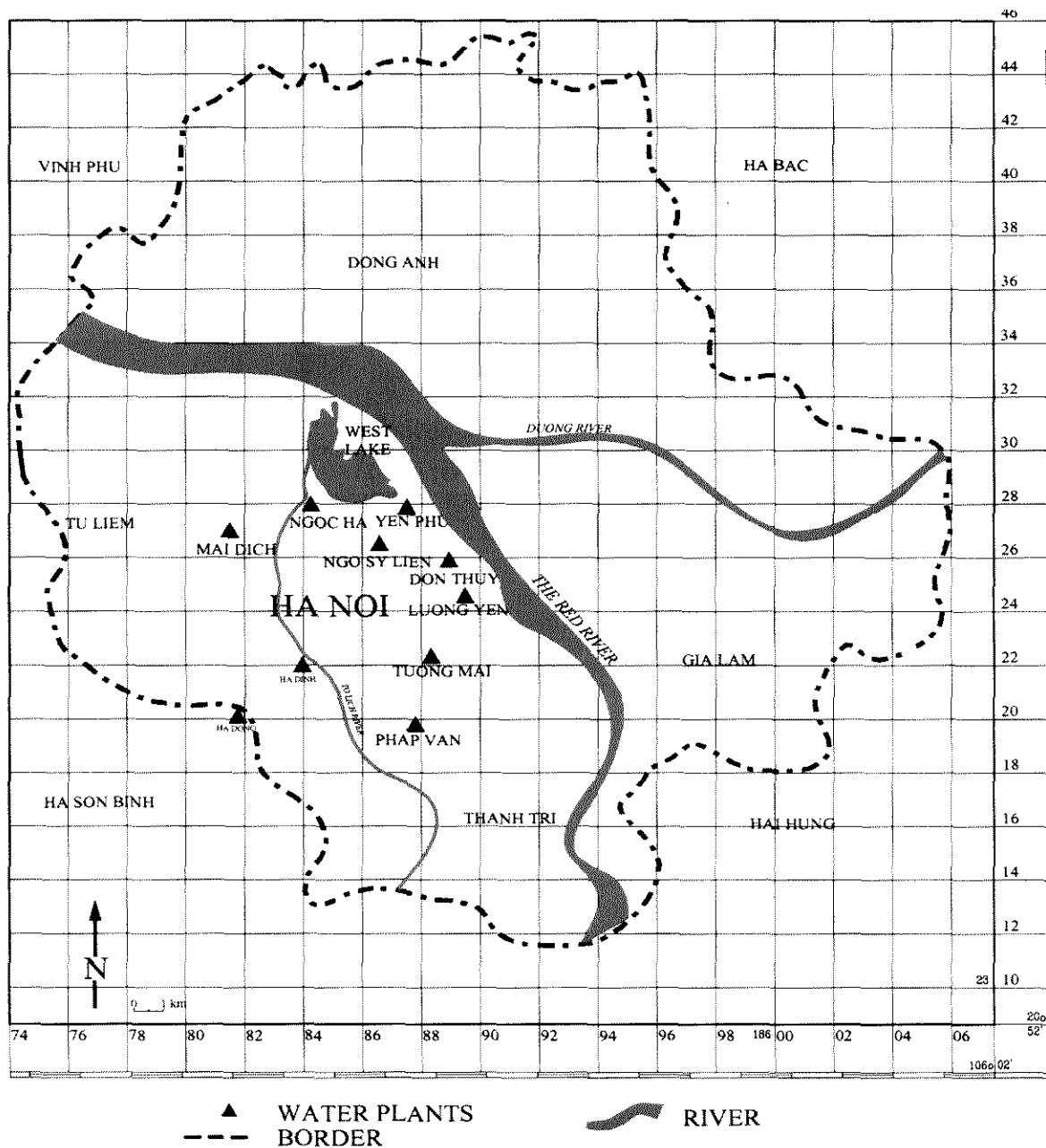


Figure 4.1: The Hanoi water plants.

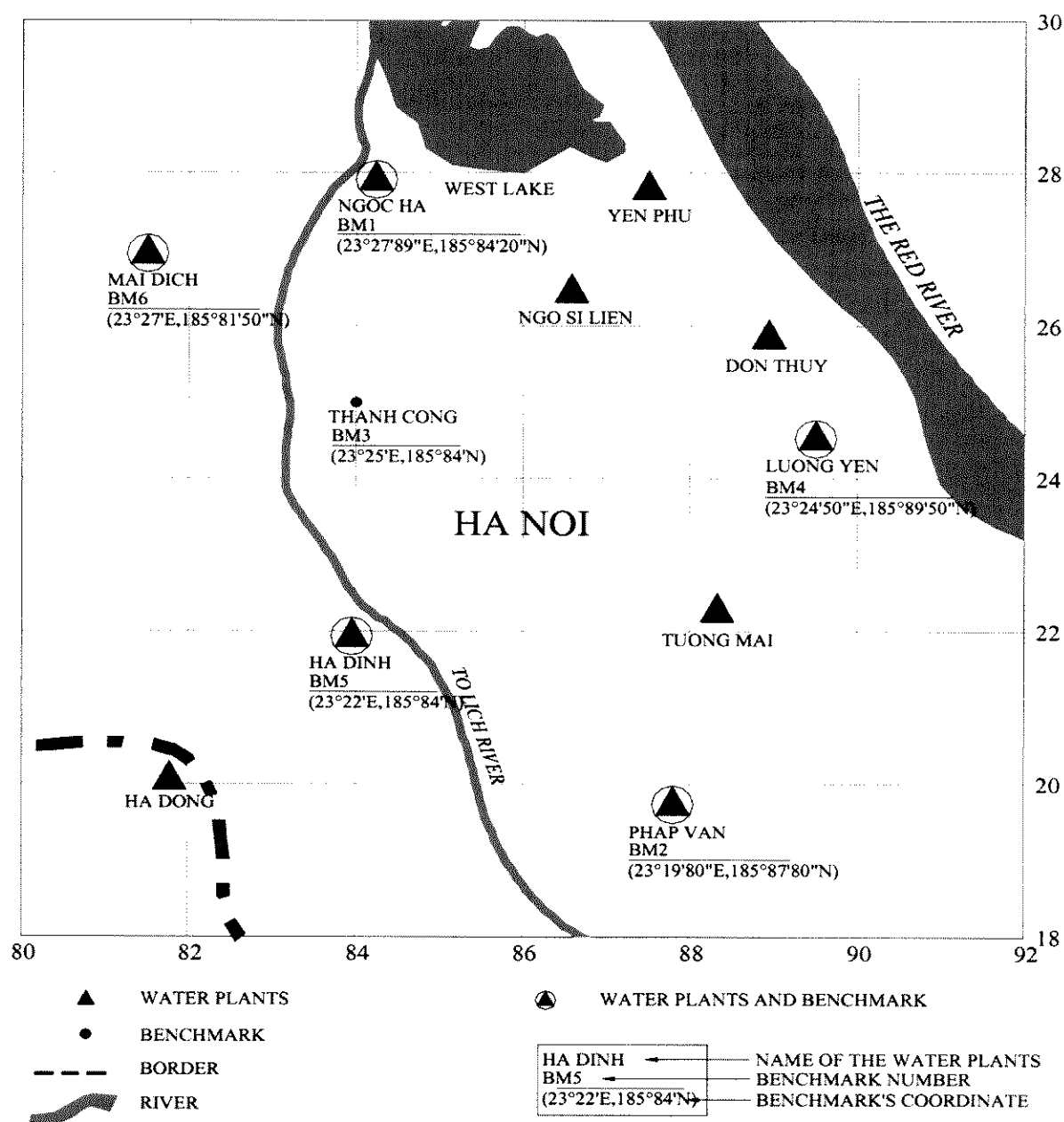


Figure 4.2: Hanoi benchmark stations.

SCHEMATIC OF BENCHMARKS

Figure 4.3

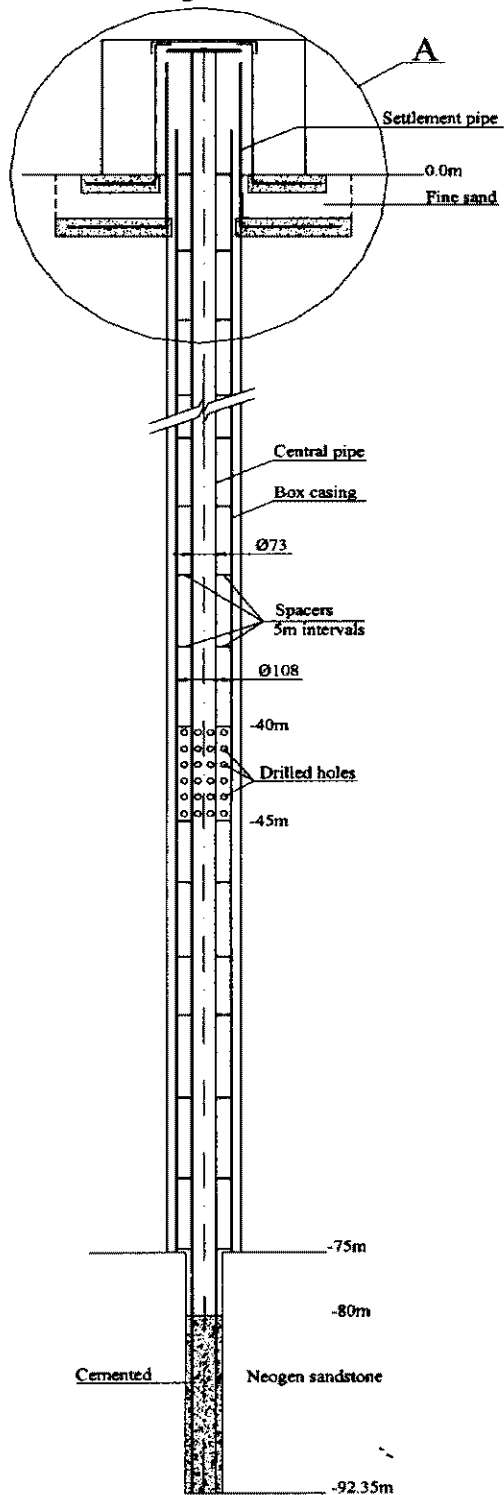


Figure 4.4

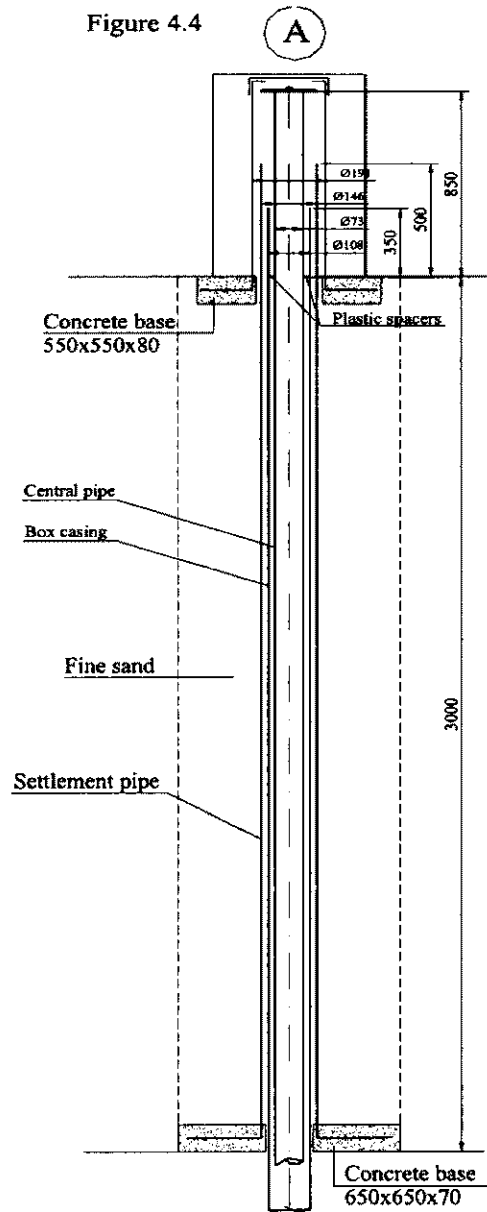


Figure 4.6 MEASUREMENT POINT - M3 and M4

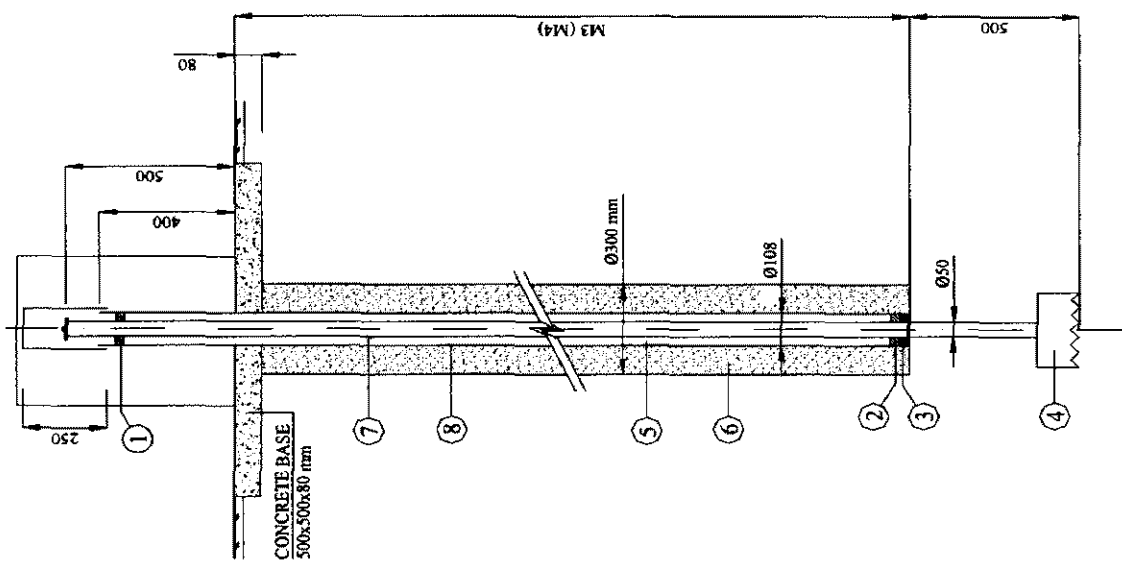


Figure 4.5 MEASUREMENT POINT - M1 AND M2

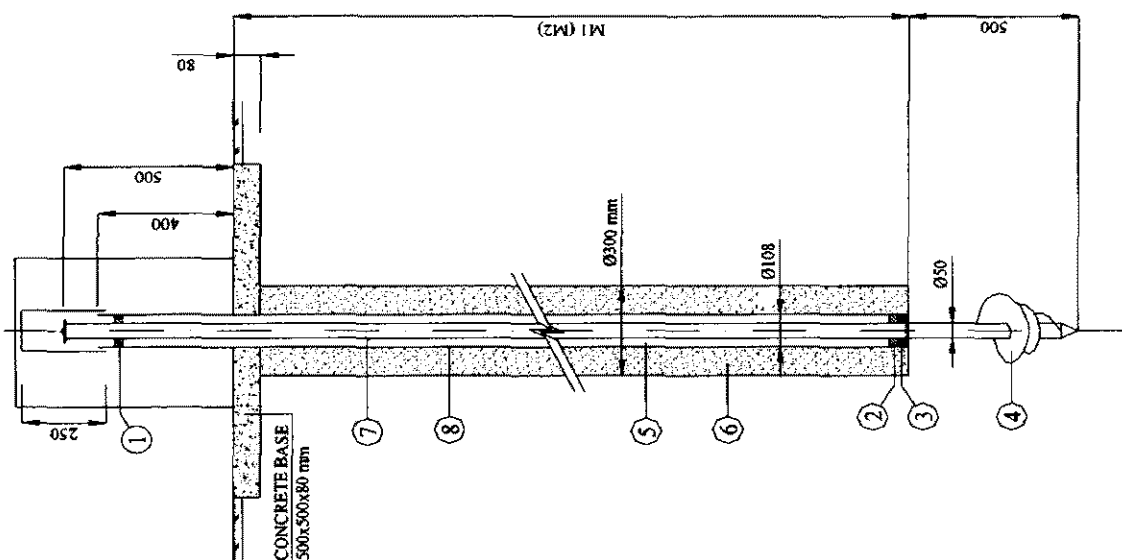


Table 4.1

Bench Mark	M. POINTS	PIPE Ø108 (mm)	PIPE Ø50 (mm)
BM1	M1	2400	2700
	M2	10900	11200
	M3	24900	25500
BM2	M1	7400	7700
	M2	14400	14700
	M3	32900	33200
BM3	M4	3400	3900
	M1	3200	3700
	M2	10200	10700
BM4	M3	21200	21700
	M4	33200	33700
	M1	3200	3700
BM5	M2	10200	10700
	M3	20200	20700
	M4	24200	24700
BM6	M1	2200	2700
	M2	14200	14700
	M3	26200	26700
	M4	35200	35700
	M1	2200	2700
	M2	24200	24700
	M3	30200	30700

Note:
1,2 : Collar
3 : Sealed
4 : Anchor
5 : Oil
6 : Sand
7 : Central pipe
8 : Outer pipe

Figure 4.7: Piezometer.

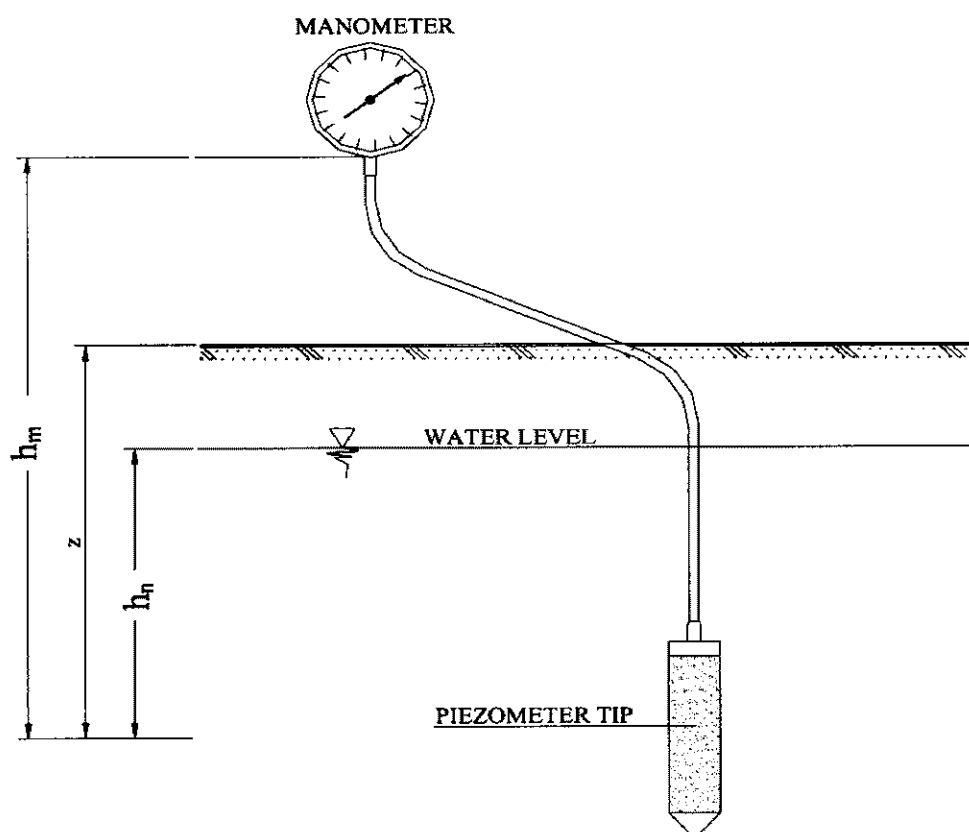
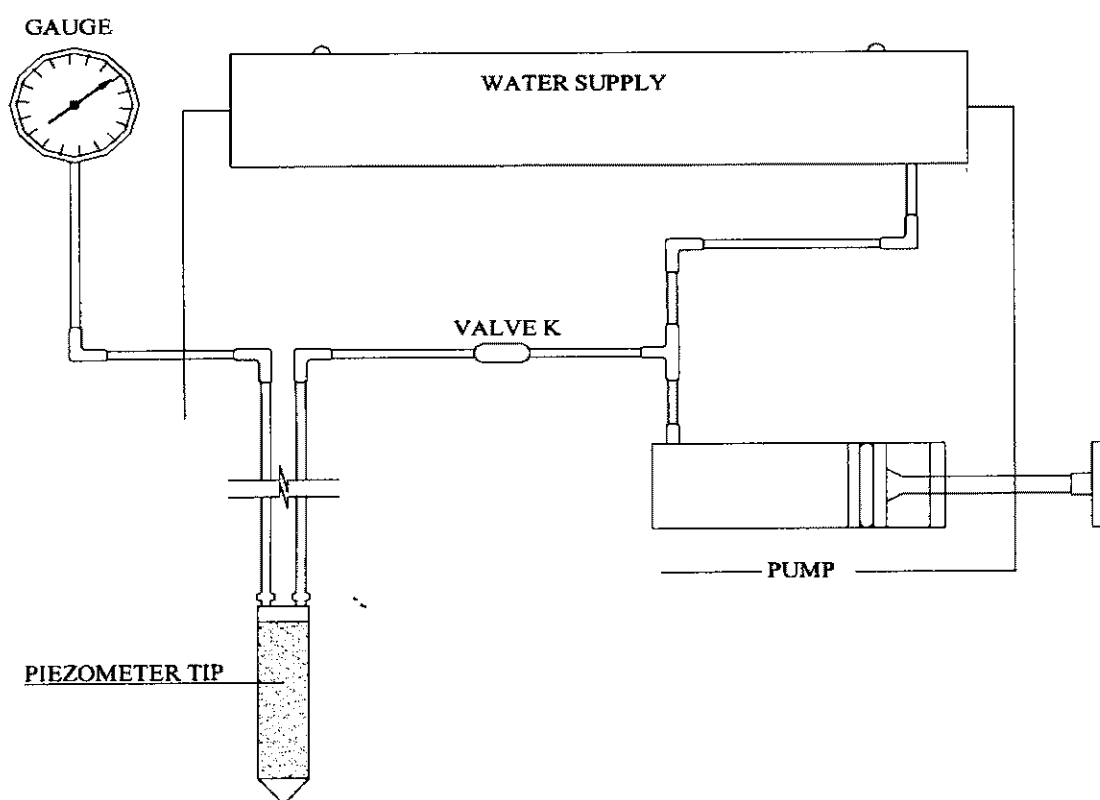


Figure 4.8: Hydraulic piezometer system.



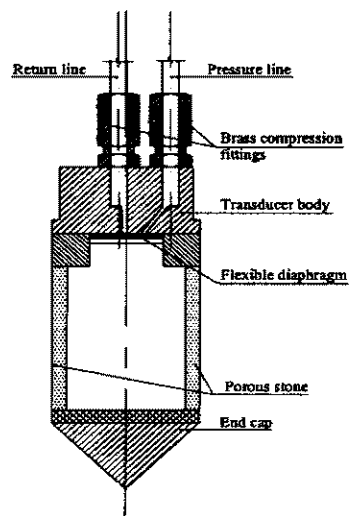


Figure 4.9: Schematic of pneumatic piezometer tip.

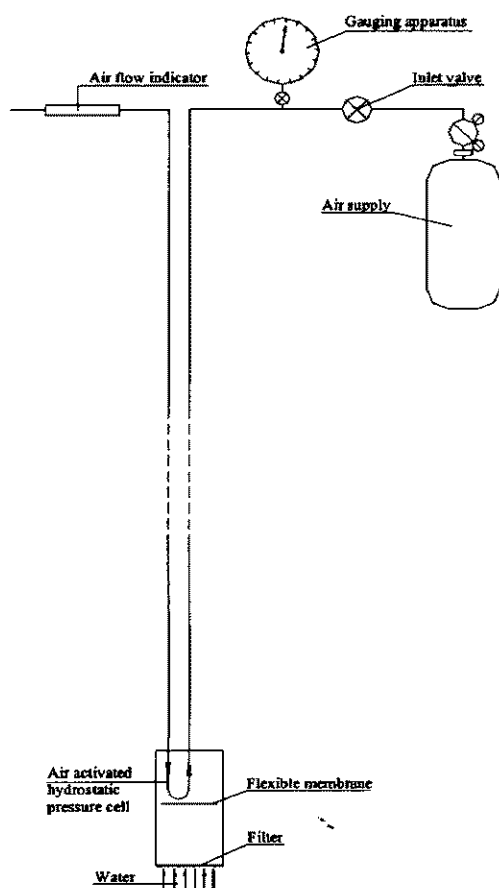


Figure 4.10: The pneumatic piezometer

Figure 4.11

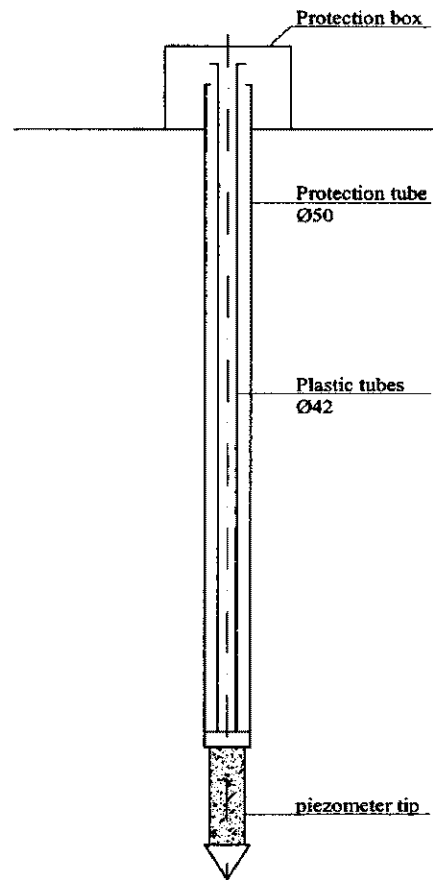


Table 4.2: Piezometer lengths and types.

BENCHMARK	PIEZOMETER	LENGTH (mm)	TYPES
BM1	U1	8000	Pn. P.
	U2	24000	H. P.
BM2	U1	5000	H. P.
	U2	10000	H. P.
	U3	23000	H. P.
BM3	U1	4000	Pn. P.
	U2	11000	Pn. P.
	U3	15000	Pn. P.
	U4	23000	H. P.
BM4	U1	4000	H. P.
	U2	10500	H. P.
	U3	20500	H. P.
	U4	24500	H. P.
BM5	U1	3000	Pn. P.
	U2	15000	Pn. P.
	U3	27000	H. P.
	U4	35500	H. P.
BM6	U1	3000	Pn. P.
	U2	25000	Pn. P.
	U3	30000	H. P.

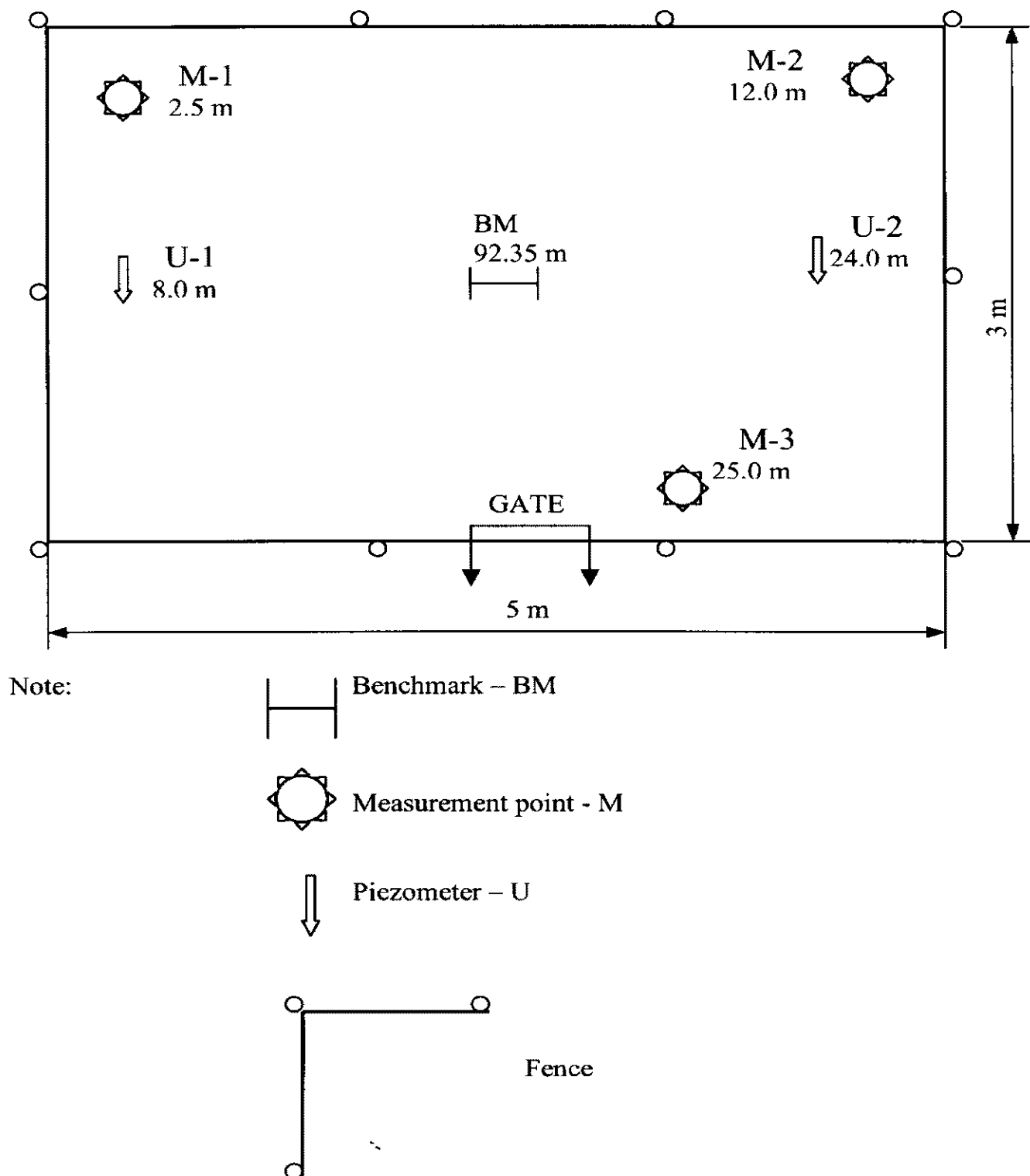


Figure 4.12: The Ngoc Ha benchmark – BM1.

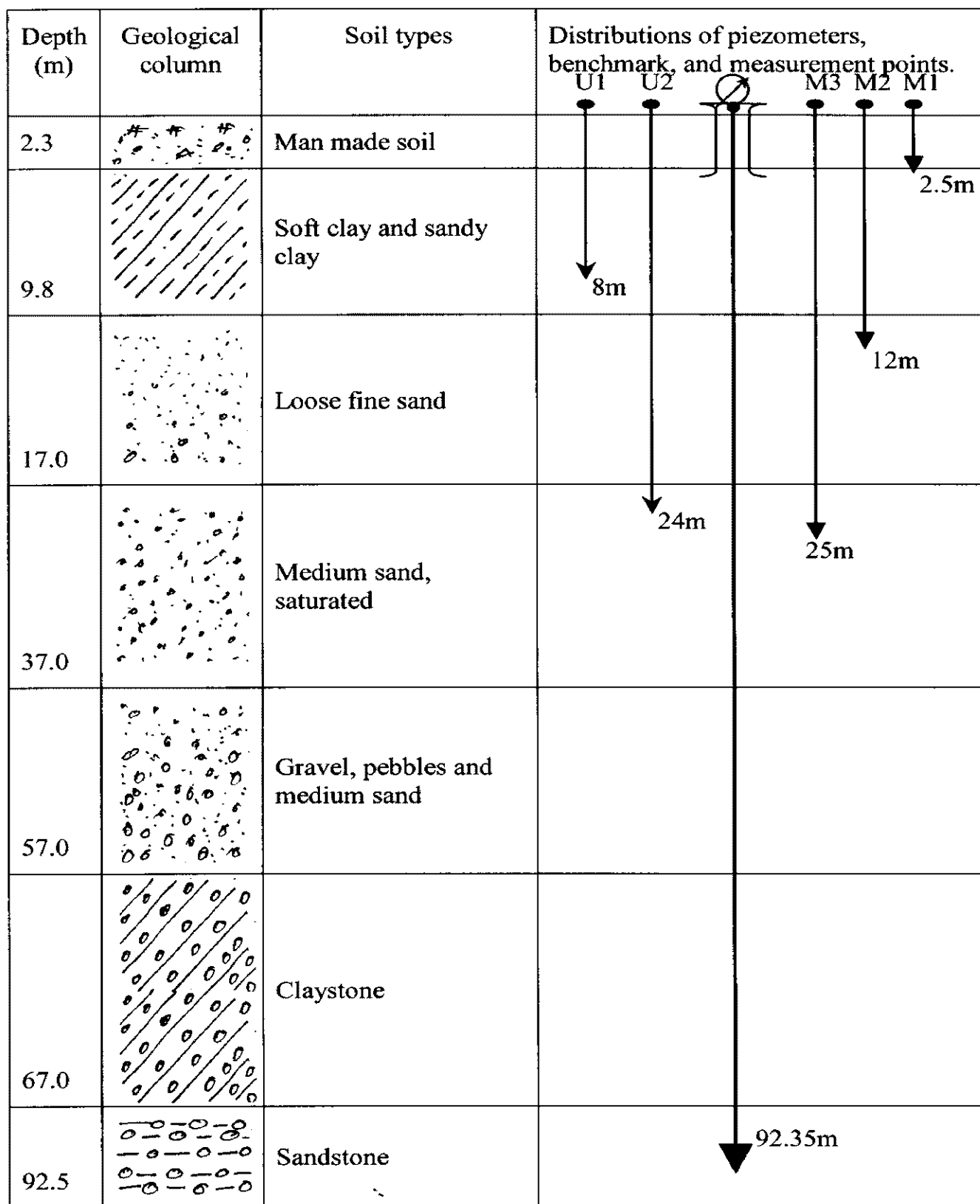


Figure 4.13: Soil Profile and distribution of the piezometers at the Ngoc Ha benchmark.

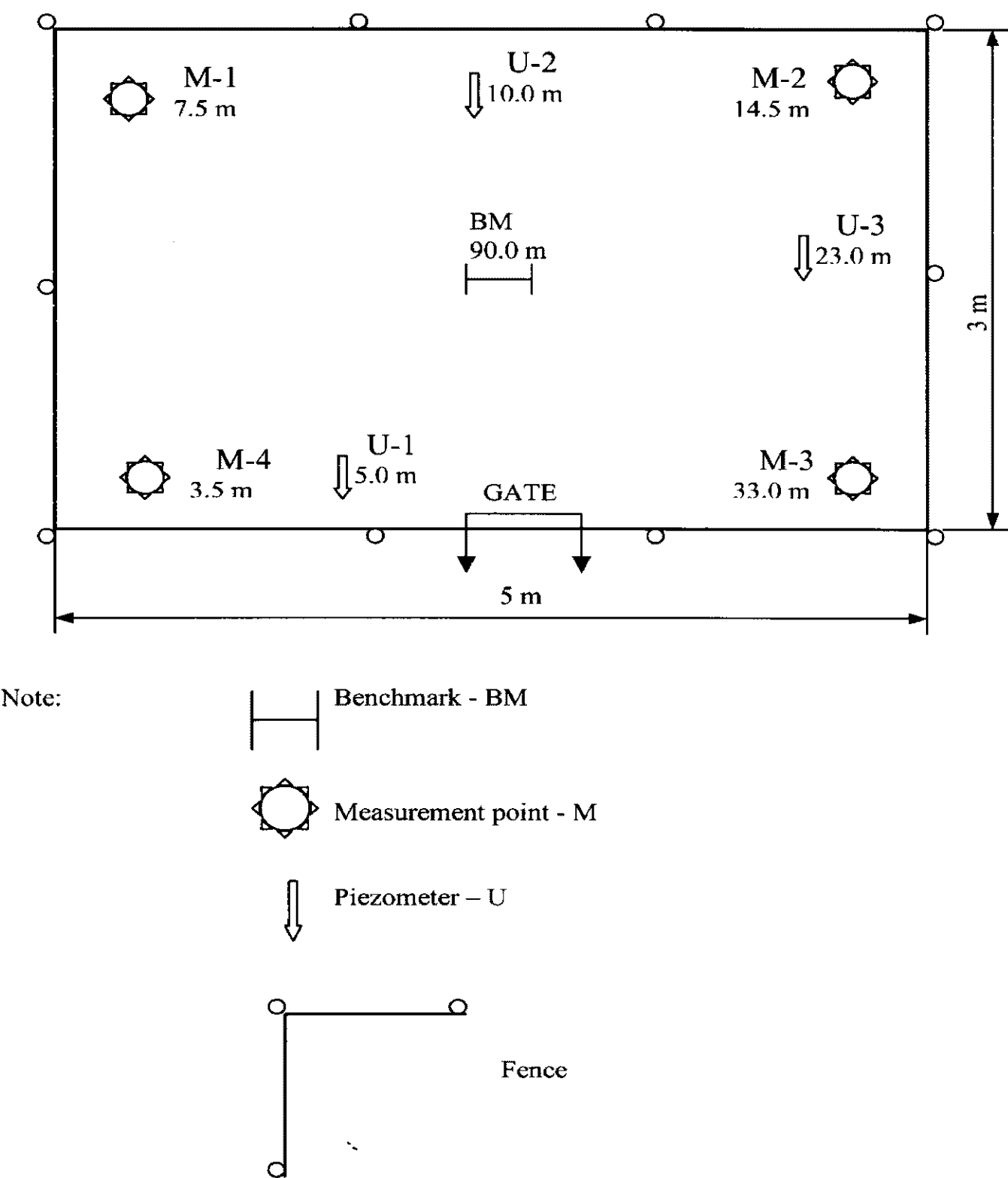


Figure 4.14: The Phap Van benchmark – BM2.

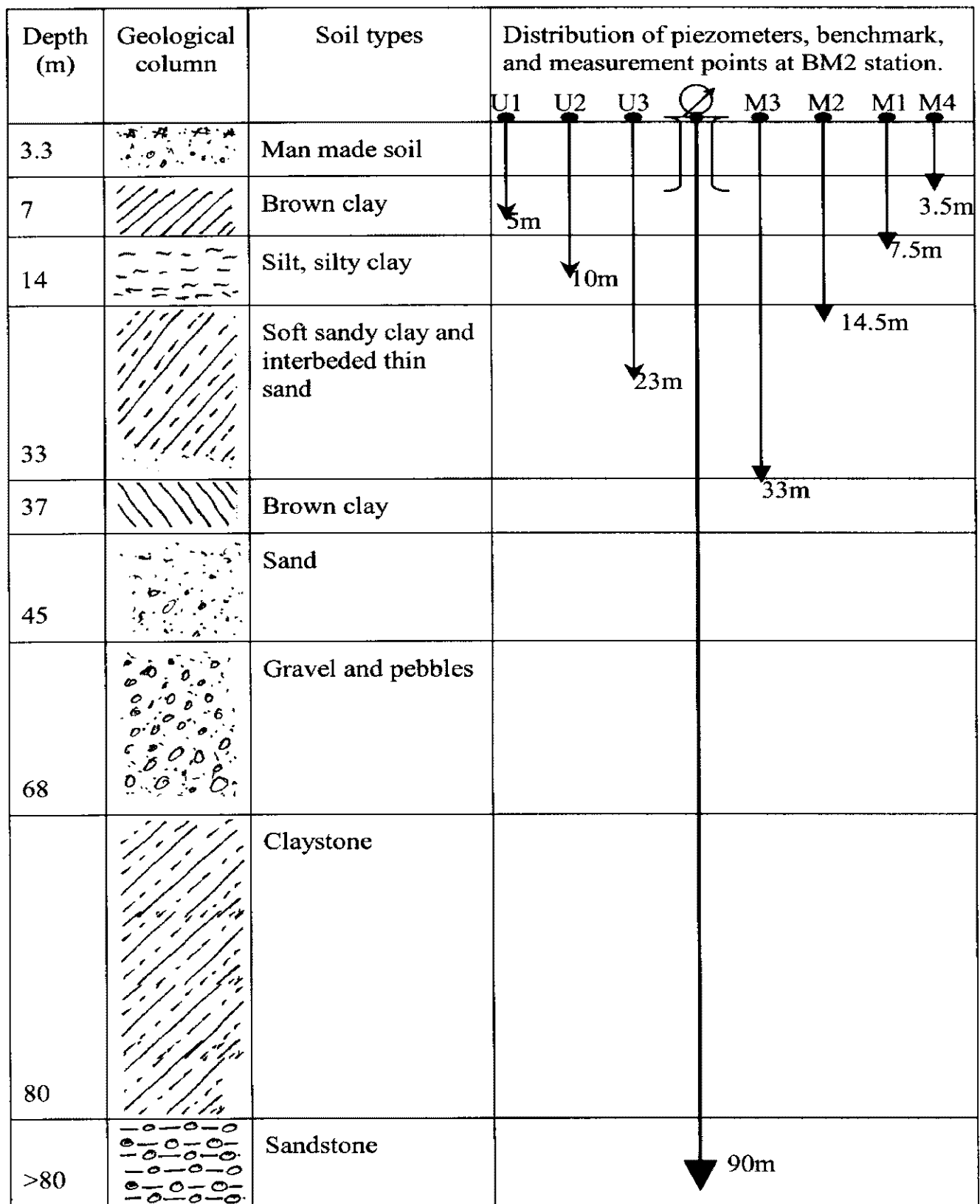
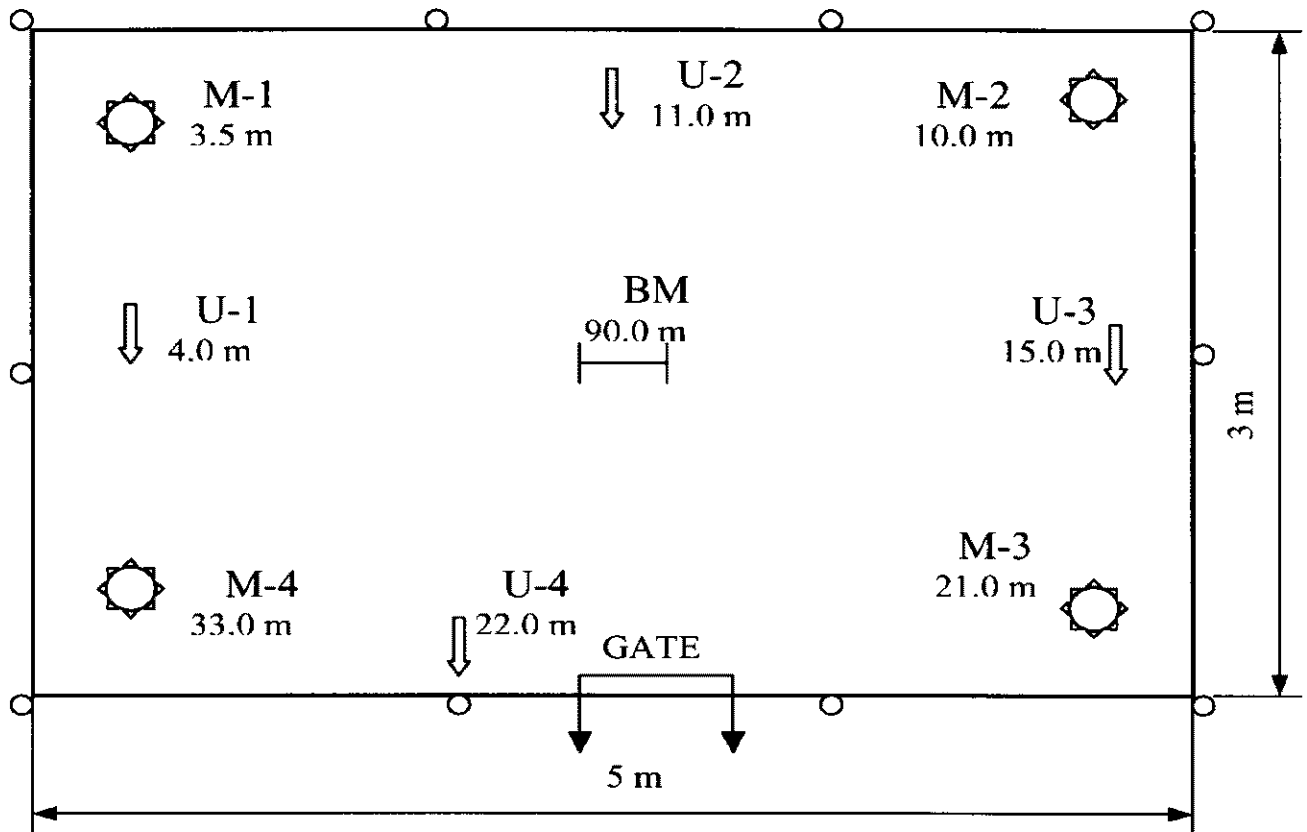


Figure 4.15: Soil Profile and distribution of the piezometers at the Phap Van benchmark.



Note:

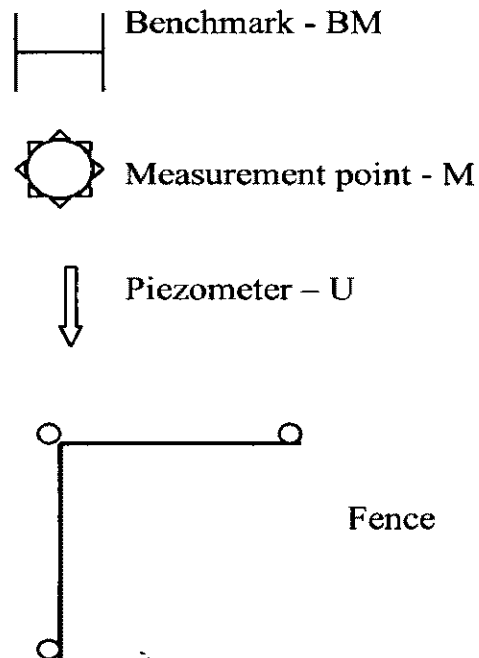


Figure 4.16: The Than Cong benchmark – BM3.

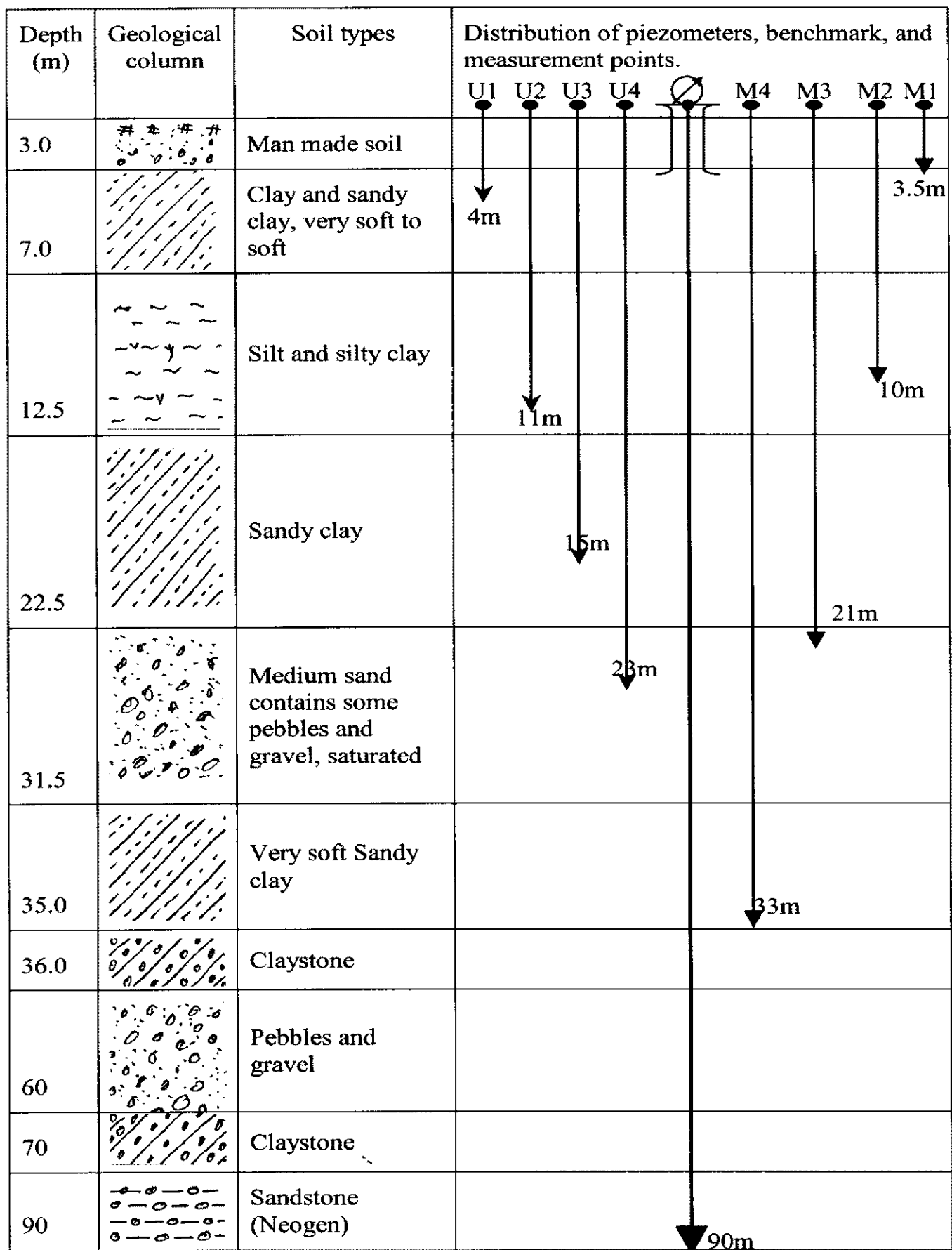
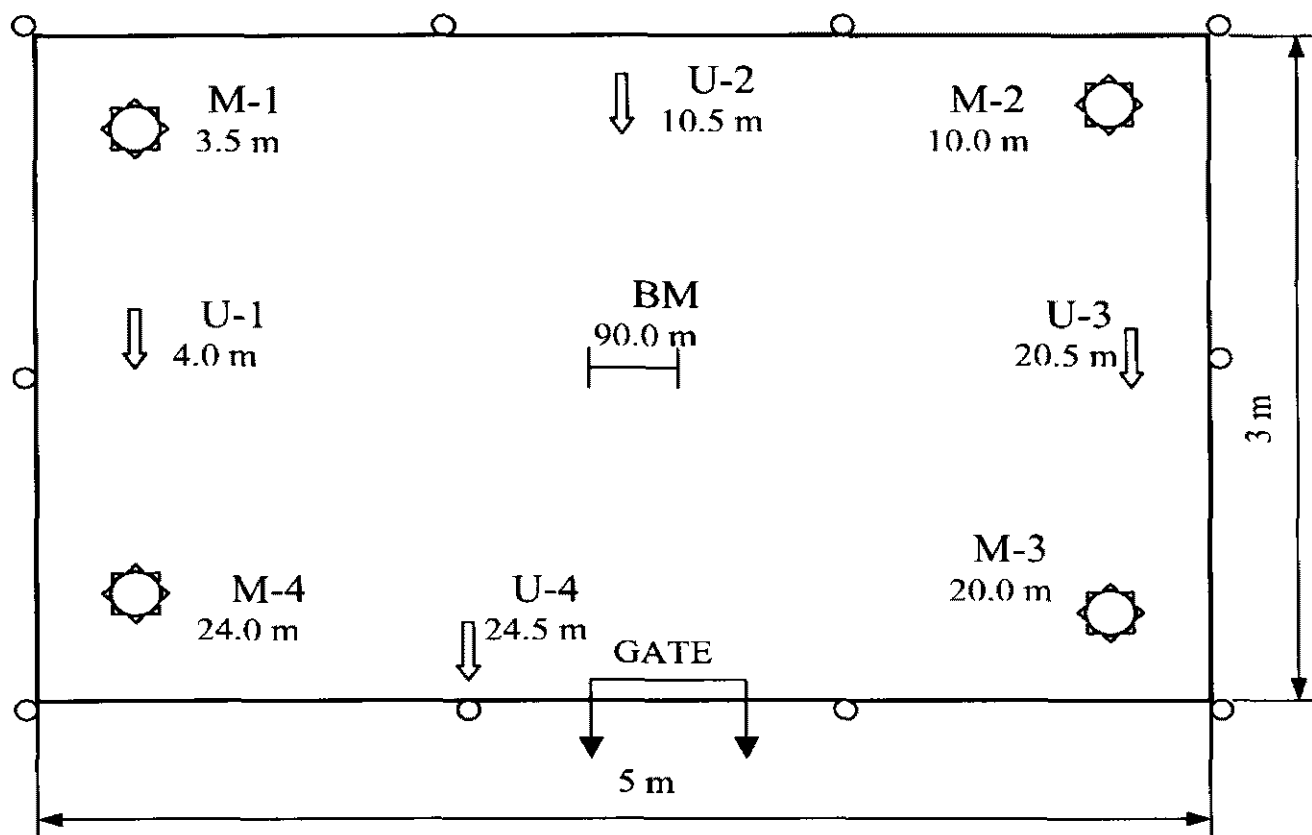


Figure 4.17: Soil Profile and distribution of the piezometers at the Thanh Cong benchmark.



Note:

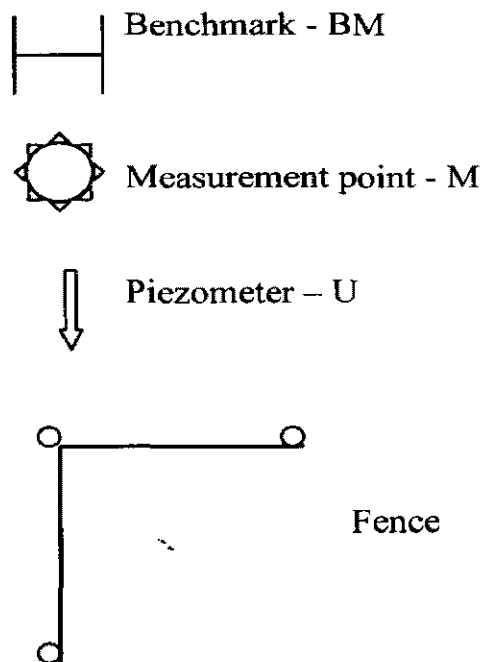


Figure 4.18: The Luong Yen benchmark – BM4.

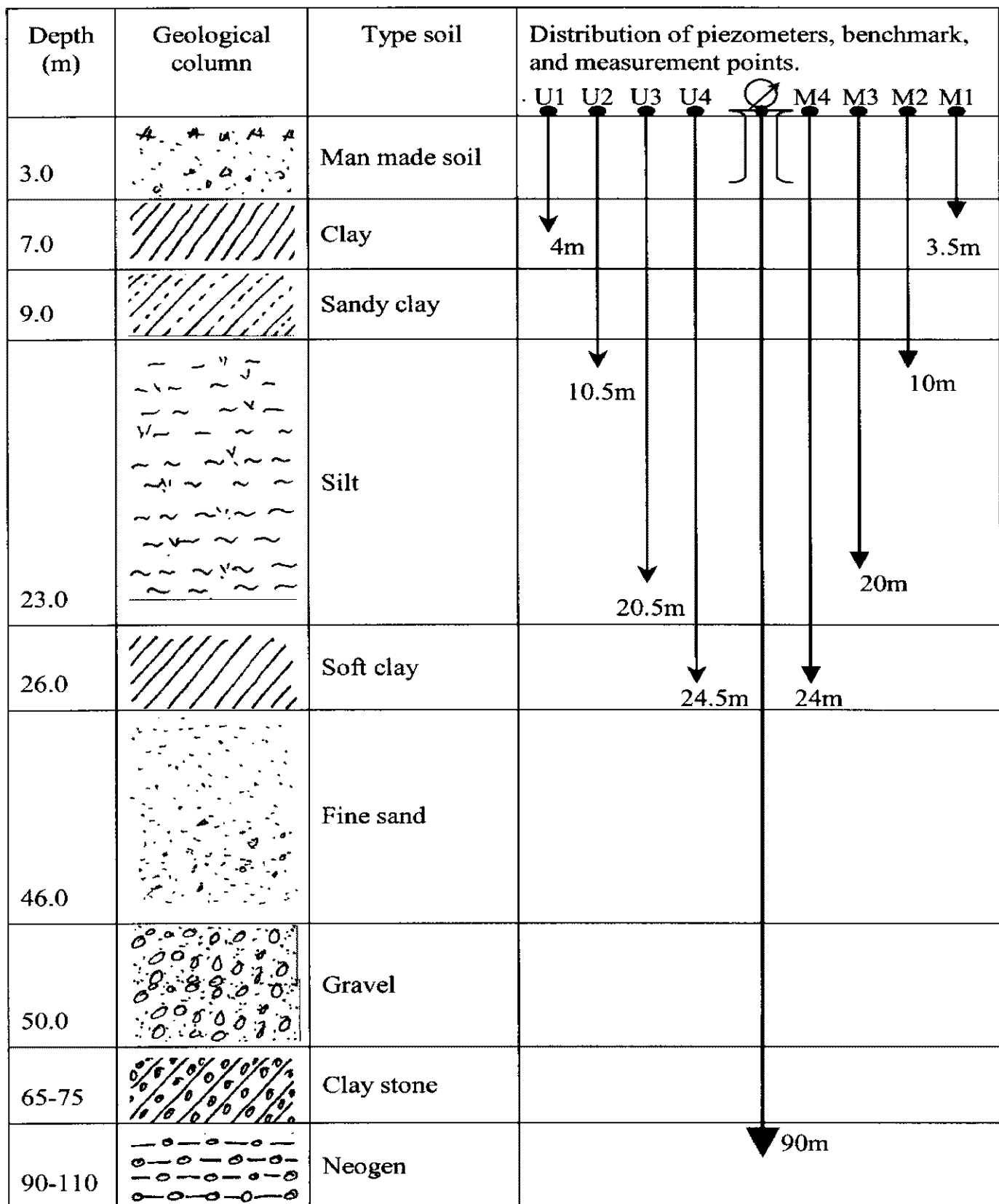


Figure 4.19: Soil Profile and distribution of the piezometers at the Luong Yen benchmark.

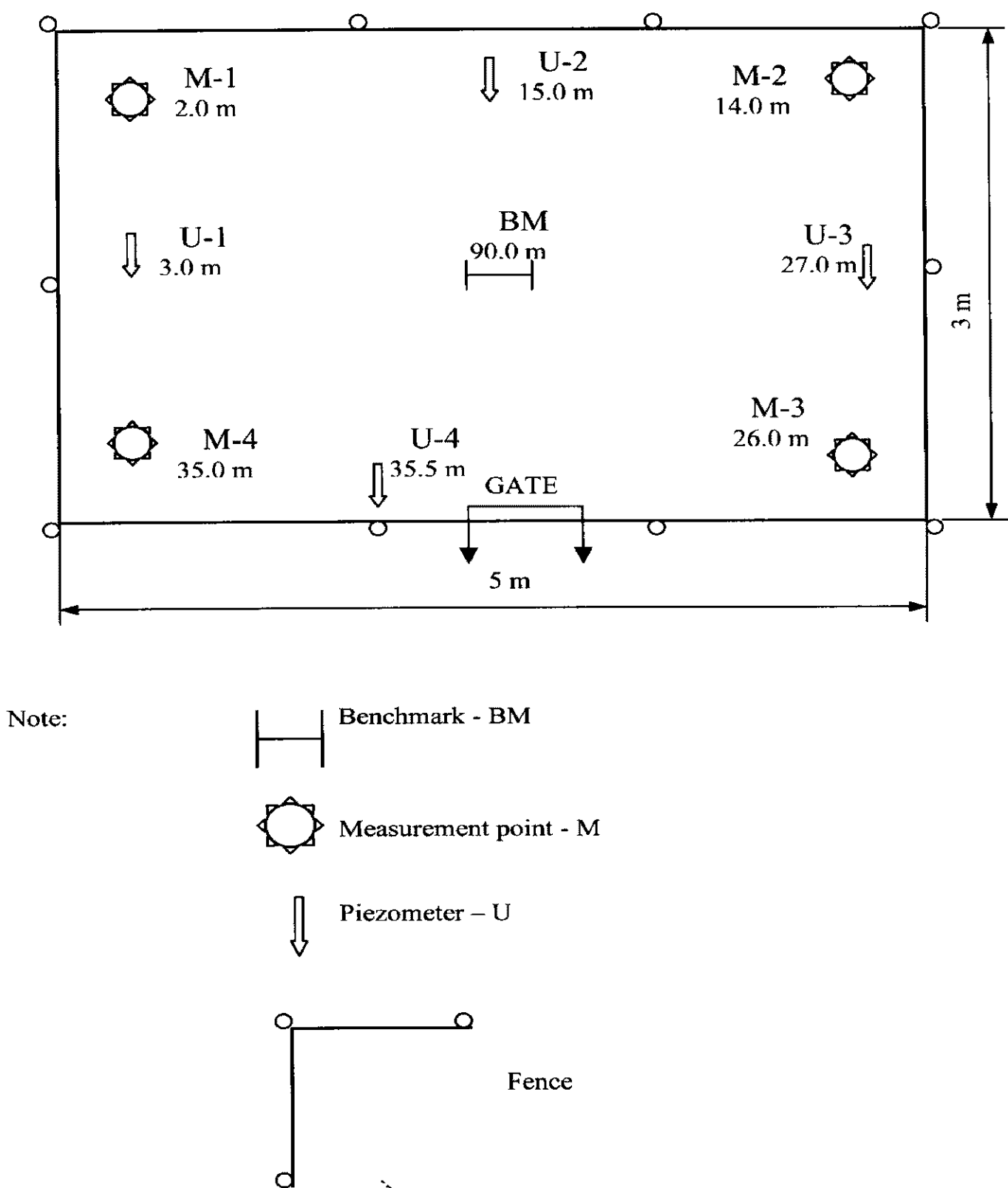


Figure 4.20: The Ha Dinh benchmark – BM5.

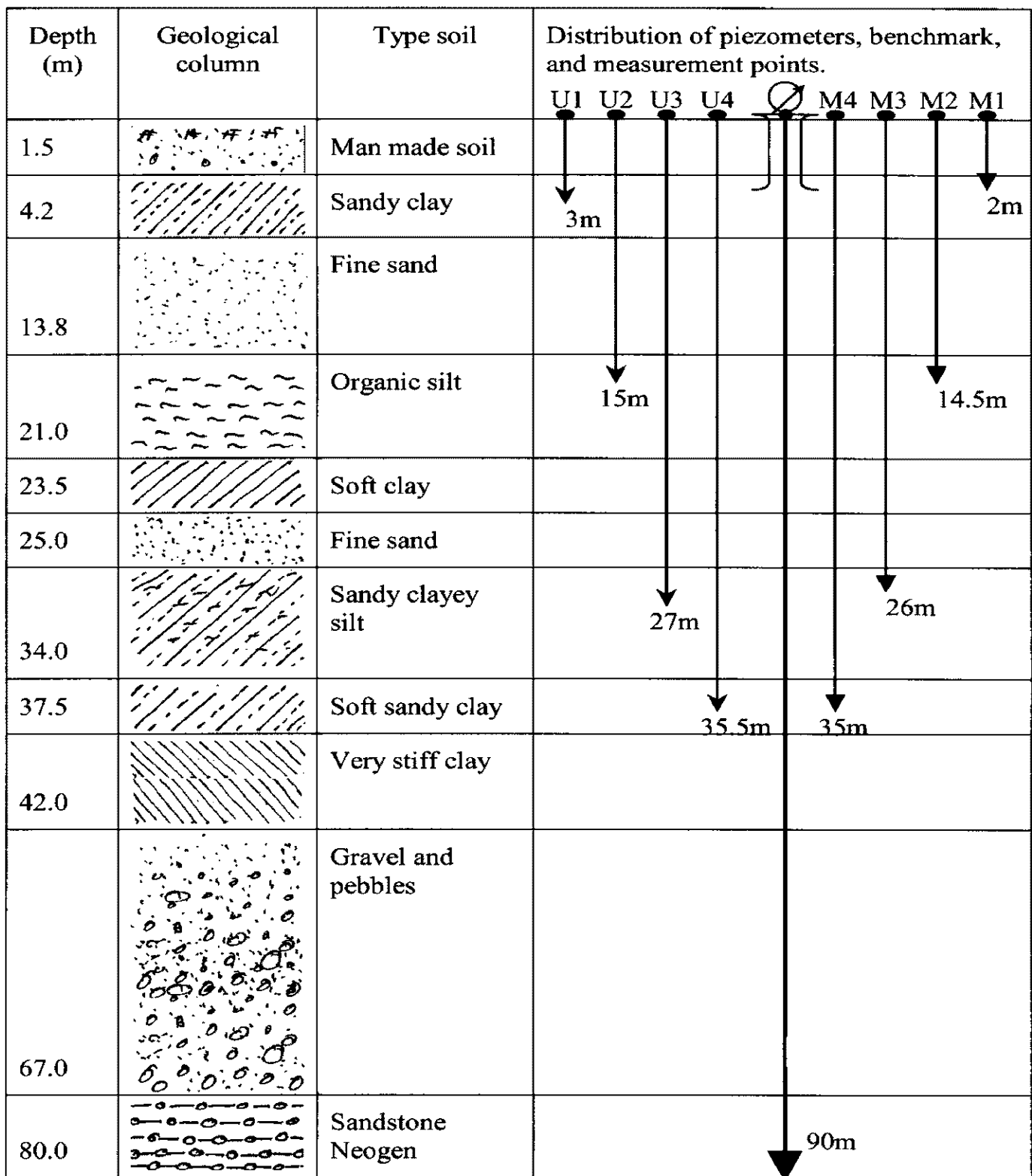
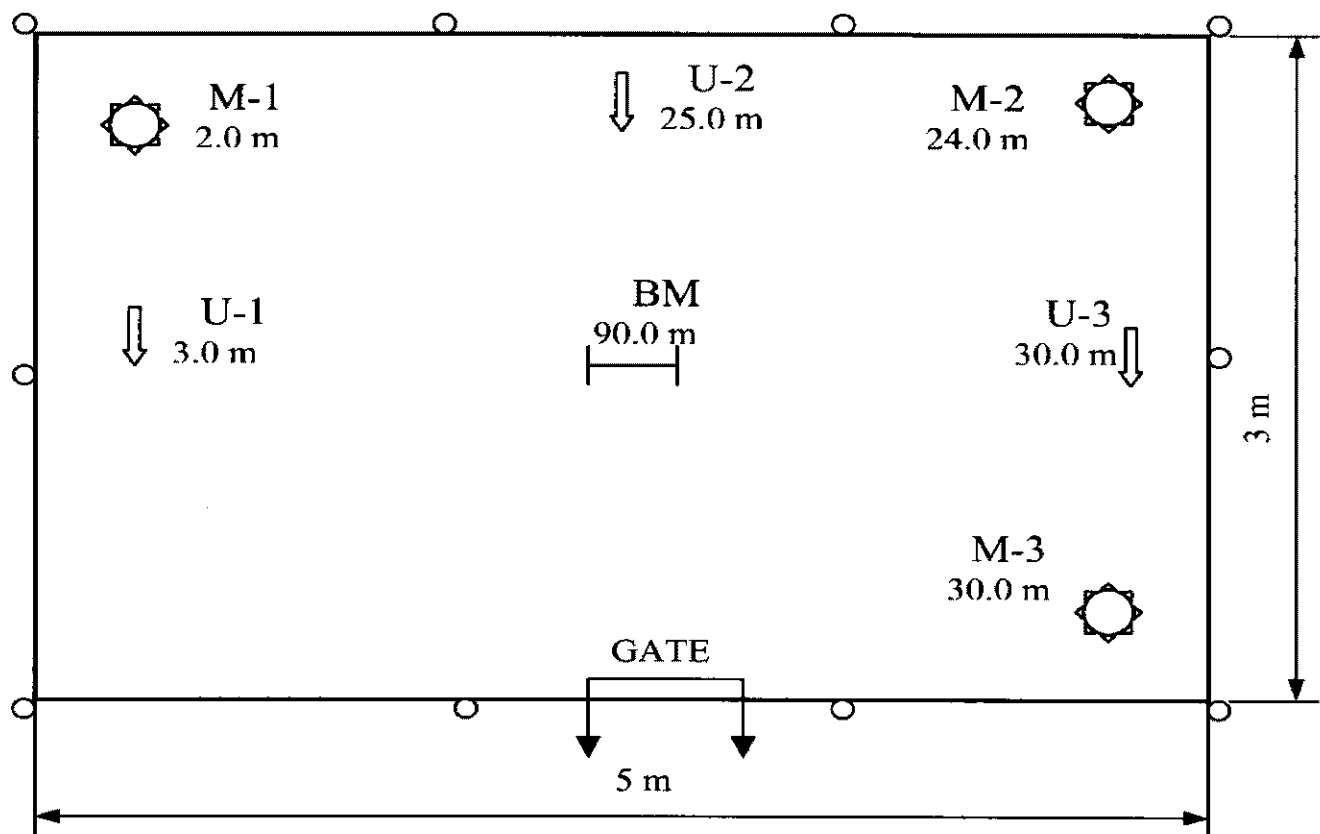


Figure 4.21: Soil Profile and distribution of the piezometers at the Ha Dinh benchmark.



Note:

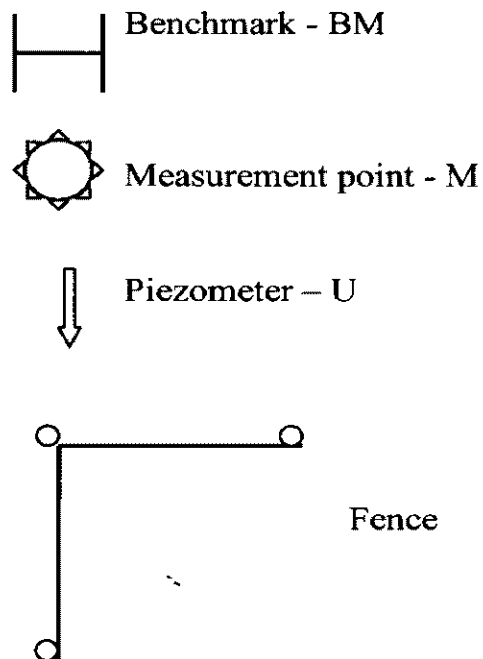


Figure 4.22: The Mai Dich benchmark – BM6.

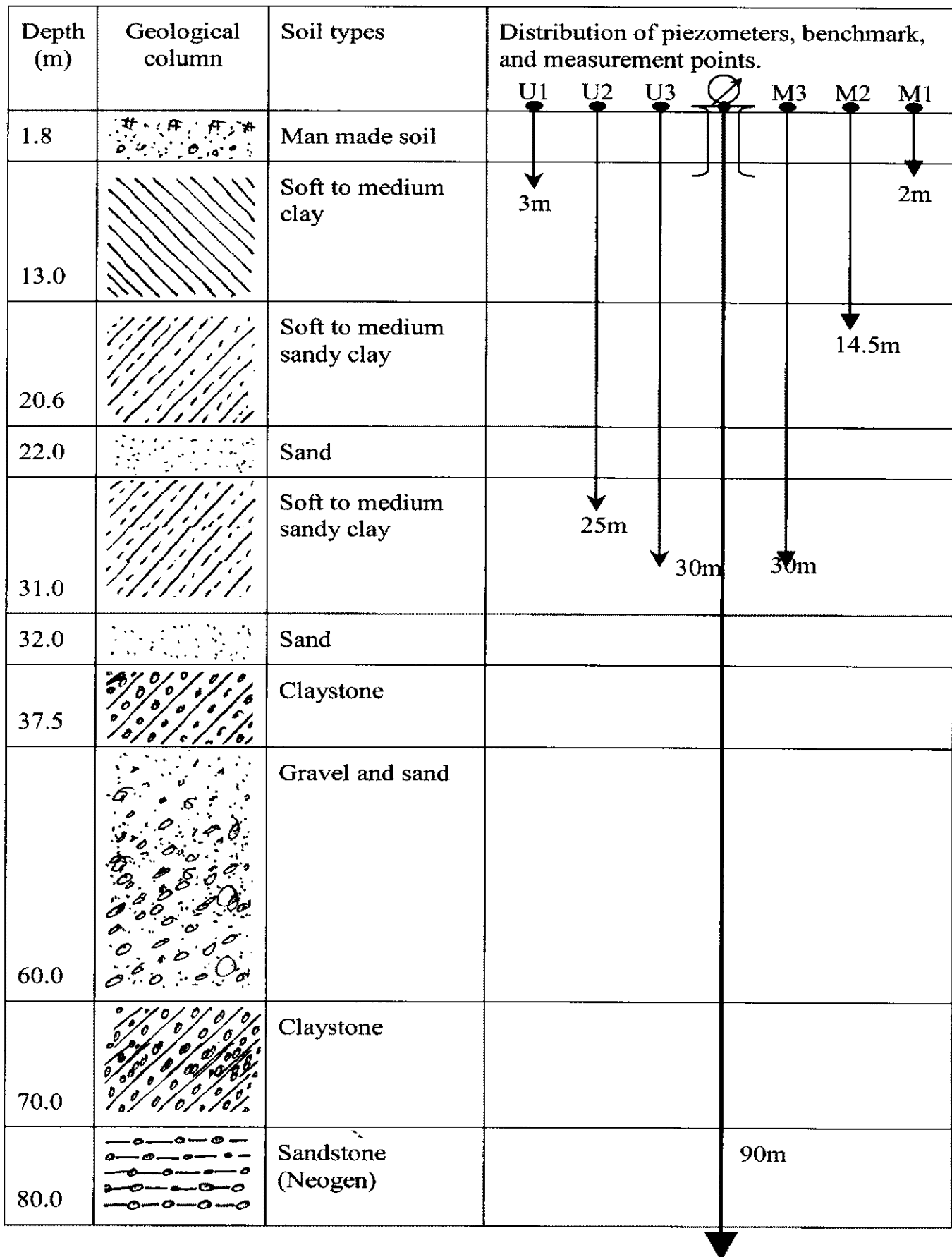


Figure 4.23: Soil Profile and distribution of the piezometers at the Mai Dich benchmark.

CHAPTER 5

NUMERICAL ANALYSIS OF GROUND WATER PUMPING

5.1 INTRODUCTION

A number of methods have been developed to calculate the ground movement due to excessive pumping of the ground water such as analytical methods, transform methods, and numerical methods.

Numerical methods are widely used because of the fact that they are able to describe the problem in more detail than the other methods. This makes solutions from numerical methods closer to reality.

In this chapter, two aspects will be presented:

- A general numerical study of the behaviour of the ground due to ground water pumping.
- A parametric study using the computer program EF4.

5.2 INTRODUCTION TO THE PROGRAM EF4

The finite element program EF4 (Hsi, 1992) was developed at Sydney University to solve fully coupled problems involving excavation and ground water pumping. This computer program uses an 8-noded isoparametric element for interpolating both the displacement and the pore pressure field. The program is able to perform axis-symmetric and plain strain analysis in either elastic or elasto-plastic materials. (Hsi and Small, 1992a, 1992b). Only the elastic model for the soil behaviour was considered in the analysis presented in this chapter.

This finite element program is able to calculate soil displacements, pore pressure and the drawdown of the water table. Soil displacement due to changes in effective stresses and pore pressure is based upon Biot's consolidation theory (Biot, 1941, 1956) and the finite element formulation is based upon that of Sandhu and Wilson (1969). Drawdown of the water table is simulated by applying the theory of transient free surface seepage (Desai, 1976, Bathe *et al.*, 1982, Desai and Li, 1983, Cividini and Gioda, 1984).

5.3 NUMERICAL MODELLING OF THE PROBLEM

5.3.1 GENERAL

The numerical modelling of the problem includes specifying the most suitable modelling geometry, i.e. the size and number of soil elements used in the computer analysis. In this chapter, the negative pore pressure limit parameter is also investigated as this parameter has a large effect on the numerical stability of the drawdown analysis.

5.3.2 MODELLING GEOMETRY

To model the case studies, an 8 noded isoparametric element was used to represent the soil medium. Figure 5.1 shows the finite element mesh used in the axi-symmetric analysis of a typical case study. The number of elements, which depends on the capability of the computer, is 1200. The smallest size of elements is usually 2x5m in the area where the water is pumped and these elements extend 200 to 500m away from the axi-symmetric axis. The width of elements gradually increases at a larger distance from the symmetrical axis.

Due to the approximately symmetrical character of the cases analyzed, there are two types of boundary conditions that need to be considered at the axis of symmetry and the remote boundary. Both axial and remote boundaries have horizontal movements fixed but vertical movements are unrestrained. At the bottom of the soil model, both vertical and horizontal movements are restrained as it is considered to be a rough rigid base of Tertiary bedrock. The remote boundary of the mesh is also assigned the water-recharge condition, hence at this boundary, the water levels that depend on time are controlled by the observed water levels at that point. Soil properties are then assigned to the mesh in a series of horizontal layers.

The water plant coordinates, the pumping rates and also the pumping history of each water plant were also collected from research project number 01-36-04 of the Hanoi City People's committee (1993-1994) and the Hanoi Clean Water Supply company

(in Vietnam). The depth where water is usually pumped from is about 50m below the ground surface.

5.3.3 GOVERNING PARAMETERS AND MODELLING ISSUES:

Parameters governing analysis:

As the ground water is being pumped, the water table drops. The soil is divided into two regions. The region above the water table is the unsaturated or negative pore water pressure area, in contrast to the positive pore pressure area below the water table. The water table itself is considered to be a boundary condition where there is zero pore pressure. In the saturated soils below the water table, the flow of pore pressure can be modeled by Darcy's law. For the unsaturated soils, the flow of water is reduced with reduction in saturation. In order to model this small flow, the permeability of soil above the water table is reduced with a reduction in pore water suction. Pore pressures are allowed to decrease until they reach a specified value P_{limit} in the unsaturated zone. Once the unsaturated pore pressures reach the specified value, they are maintained at the limit pressure value.

Because the permeability-pore pressure relationship in the negative water pressure area above the free surface is a property of the soil, which depends on the type of soil concerned, the relationship can be determined from experiments, or if unknown, the permeability may be allowed to be reduced to a small value at a small negative pressure. Bouwer (1964) gave some examples of pore pressure-permeability relationships for different kind of soils.

In the numerical analysis, due to the fact that compressibility of the soil can significantly influence the convergence of the solution process, in typical analyses, the value of P_{limit} is usually set equal to about -20 kPa.

Modelling issues:

In numerical analysis, the task of preliminarily modelling is important. A good model will provide accurate results without the excessive use of elements that would lead to

inefficiency. In order to choose the best model, three meshes have been used in preliminary investigations:

- Mesh (1): a coarse mesh of 400 elements (20 rows x 20 columns of elements).
- Mesh (2): a fine mesh of 1200 elements (20 rows x 60 columns of elements).
- Mesh (3): a mesh of 1200 elements (30 rows x 40 columns of elements).

The geometry of the preliminary models is maintained (80x1000m). The soil is homogeneous with soil properties: Young's modulus 10000 kPa; permeabilities in both horizontal and vertical directions are 10 m/day; Poisson's ratio is 0.3 and the storage coefficient is 0.001; the pumping rate is 30000 m³/day. Water can be pumped at a point on the axis of symmetry or a number of pumping points can be spread out equally from the symmetric axis along a radius. Usually, water is pumped from 50m depths below the ground surface.

Figures 5.2 and 5.3 show the results for both the computed water table drawdown and the soil movements for the 3 trial meshes, when water is pumped from a point on the axis. Mesh (1) and mesh (2) give identical results for the water table drawdown while mesh (3) predicts slightly less drop of the water table.

Figure 5.4 shows the results for the water table drawdown as the value of P_{limit} is changed -10, -20, -50kPa. Two typical soil meshes have been used. In both cases, the soil meshes have the same dimensions but have a different number of elements. With P_{limit} equal to -10kPa, the water table drawdown levels in both cases are totally difference. This is because of the fact that the finite element program give non-convergent results with P_{limit} equals to -10kPa. With P_{limit} equal to -20kPa or -50kPa, the water table drawdown levels are similar in both cases. It can be said that the finite element program gives stable results of the water table drawdown levels with the P_{limit} value is equal or higher that -20kPa.

Figures 5.5 and 5.6 indicate the water table drawdown results, if the pumping points have been spread out radially (not at the centre). The pumping rate has been kept constant at 30000 m³/day. The number of pumped wells is 6 or 11 and they are equally distributed over a range of 100m, 200m and 500m away from the centre. As can be seen from the two figures, the results given by mesh (2) are far more stable than the results from mesh (3). The drawdown of the water table and the soil surface settlements do not depend on the number of wells but the pumping rate and the range over which the pumped wells are distributed.

Figures 5.7 and 5.8 show the computed water table drawdown and the horizontal soil movements due to the influence of the location of the remote boundary at either 1000m or 1500m from the centre line. As the lateral boundary is moved further away, the water table drops more but the horizontal soil movements are not influenced much. The water table drawdown will continue to increase as the side boundary is moved away from the centre line. However, in reality there is a distance at which the water table does not drop and this is taken as the boundary in any numerical analysis.

From the comparisons of the results of different meshes, mesh (2) has been chosen for analysis of the next group of case studies because it gives more stable and reasonable results than mesh (3). It is also finer than mesh (1), which gives similar results.

5.4 CASE STUDIES

5.4.1 GENERAL

Comparison between the numerical results of the finite element program EF4 and the collected field data have been carried out. Two typical case studies have been presented for areas having different soil properties.

5.4.2 PHAP VAN CASE STUDY

The Phap Van benchmark, for which the global longitude and latitude coordinates are 23°19'80"E and 180°87'80"N, has been set inside the Phap Van water plant where there is a strong influence from the pumping of ground water from the plant. The

Phap Van water plant was built and put into operation in 1988. It is located in the southern area of Hanoi City. This is a flat area, 3-4m above sea level, and the subsoil usually contains soft clay and peat.

The data for the elastic properties of 8 layers of the subsoil in the Phap Van case study is shown in Table 5.3. From the ground surface down to a depth of 37m, there are a series of soft soil layers. Two layers of soil on top with a total thickness of about 7m have been assigned the same value of Young's modulus. Underlying the two layers is a soft layer of silt and silty clay with a thickness of about 7m. Then below, there are two layers of medium to stiff clay, and sandy clay that are overlying permeable strata of sand, gravel and pebbles. The permeable or water bearing strata are the layers where water is usually pumped from in the Hanoi area. In numerical modelling, this strata is divided into two layers: (1) the thin layer of fine to medium sand on top; (2) the layer of gravel and pebbles that has very high permeabilities, a high water yield, and a high value of Young's modulus. Underneath is an impermeable layer of claystone with a thickness of 12m. The whole soil layer is about 80m in thickness and overlies Tertiary bedrock, which is considered to be a rough, rigid boundary in the finite element analysis. Figure 4.15 shows the descriptions of the soil layers and their sequence.

From Figures 3.6 and 3.19 the soil around the Phap Van area is classified as C3 type (Table 3.3). This C3 soil type generally consists of three subsoil layers, which have geological age in the range from Upper Pleistocene to Holocene. For the Upper Holocene sediments, the values of Young's modulus for the clay soil types vary between 3.2 and 5.9 MPa (Table 3.6). For the Middle Holocene sediments, the average values of Young's modulus for the clay soil types are from 3.8 to 5.9 MPa (Table 3.7). The Upper-Pleistocene sediments have Young's modulus values that vary between 0.7 and 6.1 MPa (Table 3.8). Figure 3.19 shows the static cone resistance values, q_c , at the Phap Van water station (Koponen, 1992). According to Vu and Nguyen (1995), the undrained soil Young's modulus can be obtained from the CPT resistance results by employing the equation:

$$E = \alpha^{qc} \cdot q_c \quad (5.1)$$

where q_c is the static cone penetration resistance, α^{qc} is an empirical ratio, given in Table 5.1.

Table 5.1: Values of ratio α^{qc} (After Vu and Nguyen, 1995).

Soil type	q_c (kg/cm ²)	α^{qc}
Soft to medium clay or sandy clay	> 15	5 – 8
	< 15	3 – 6
Very soft clay or sandy clay	> 7	4.5 – 7.5
Clayey peat	< 7	3 – 6
Sandy peat	< 7	2 – 4
Clayey sand	$10 < q_c < 35$	3 – 5
Sand	> 20	1.5 – 3

Layer 6 is loose, fine to medium sand with a thickness of 3.0m. The typical value of Young's modulus of loose normally consolidated sand is between 8.4 and 20.0MPa (SMDS, 1998). An average drained Young's modulus value of 15.0 MPa for the soil layer number 6 is selected.

Layers 7 and 8 are a gravel and pebbles layer and a claystone layer. The soil Young's moduli of the two soil layers are unknown as there is not sufficient information about the mechanical properties of these two soil layers that has been collected. Hence, It is very difficult to select proper Young's moduli for these two soil layers. The Young's moduli of these two layers were chosen to be 200MPa and 100MPa respectively (SMDS, 1998). As the Young's moduli of soil layers 7 and 8 are relatively large compared to the soft soil layers near the ground surface, the soil movements due to

ground water pumping mostly occur in the soil layers above the gravel and pebbles soil layers.

The index and mechanical soil properties of soil layer 2 to soil layer 5 are provided in Table 5.2.

Table 5.2: The index and mechanical soil properties of soil layer 2 to soil layer 5 at the Phap Van water pumping station.

Properties	Symbol	Layers			
		2	3	4	5
Moisture	w_o (%)	38	74.42	31.37	24
Unit weight	γ_s (kg/m ³)	1850	1500	1830	1860
Unit weight of solids	γ_d (kg/m ³)	2700	2540	2680	2690
Initial void ratio	e_o	1.015	1.953	1.393	0.79
Porosity	n_o (%)	50.37	66.14	58.21	44
Degree of saturation	G (%)	100	96	60	82
Plastic index	I_p (%)	18	22	11	19
Liquid index	I_s	0.34	1.1	0.86	0.71
Undrained cohesion	c (kPa)	0.034	0.009	0.014	0.018
Undrained friction angle	ϕ°	11°00'	5°00'	9°14'	7°00'
Coefficient of compressibility	a_v (MPa ⁻¹)	0.027	0.163	0.052	0.073
Average static cone resistance	q_c (kg/cm ²)	7-10	2-3	10-12	10
Correlation ratio for Young's modulus from CPT test	α^{qc}	4.5-7	3-6	4.5-7.5	4.5-7.5
$E = \alpha^{qc} \cdot q_c$	E (MPa)	3.1-7	0.6-1.8	4.5-9.0	4.5-9.0
Young's modulus used in the finite element analysis	E' (MPa)	6.5	2.5	7.0	8.0
Average permeability	k (m/d)	2.0E-04	6.5E-04	2.5E-04	2.0E-04

Table 5.3: Phap Van soil properties for the finite element analysis.

No.	Material type	Thickness (m)	E' (kPa)	ν'	s	k_x (m/day)	k_y (m/day)
1	Man made soil	3.3	6500	0.35	0.0001	0.00192	0.00192
2	Medium brown clay	3.7	6500	0.35	0.0001	0.00020	0.00020
3	Silt, Silty clay	7.0	2500	0.35	0.0010	0.00065	0.00065
4	Medium sandy clay	19.0	7000	0.35	0.0010	0.00025	0.00025
5	Medium to stiff brown clay	4.0	8000	0.30	0.0001	0.00020	0.00020
6	Fine to medium sand	3.0	15000	0.30	0.0020	1.0	1.0
7	Gravel and pebbles	28.0	200000	0.30	0.0025	45	45
8	Claystone	12.0	100000	0.30	0.0001	0.00001	0.00001

In Table 5.3, E' is drained soil Young's modulus, ν' is drained Poisson's ratio, s is the specific yield of water for the soil, k_x and k_y are respectively the soil permeabilities in the x and y directions.

At the Phap Van water plant there are approximately 9 pumped wells, and the current gross pumping rate of the Phap Van water plant is about 30000 m³/day. The numerical analysis is divided into three time periods: 1988 – 2000; 2000 – 2010; and 2010 – 2020 (Table 5.4). In the first period, the pumping rate was the current pumping rate (30000m³/day), in the second period the pumping rate was increased to 45000m³/day, and in the third period the pumping rate was assumed to be double the current pumping rate (60000m³/day). The number of wells and their positions are fixed throughout the three time periods, but the water pumping rate for each well is relatively changed to match the assumption of the water pumping rate increases. Usually, water is pumped at a depth of -50m down from the ground surface. In the analysis, a pumped well is represented by a point located at 50m (in the gravel and pebbles layer) from the surface.

Table 5.4: Total pumping rate versus time at the Phap Van water station.

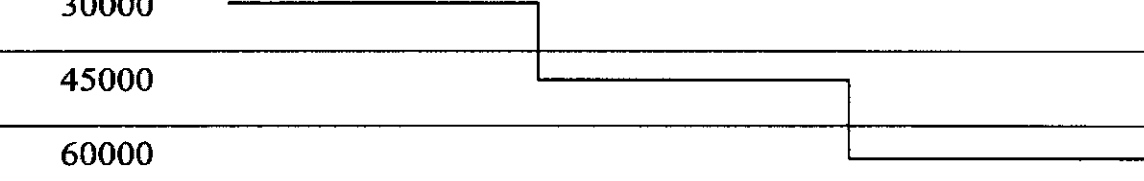
Pumping rate (m ³ /day)	Time		
	1988 – 2000	2000 – 2010	2010 - 2020
30000			
45000			
60000			

Figure 5.9 presents a comparison of the calculated drawdown of the water table and the measured water table for three periods of time. During the first period (1988-2000) there is a reasonable agreement between the measured data and the calculated result for the drawdown of the water table. Usually, the actual water table fluctuates somewhat. However, the trend of the measured water table level is close to the calculated water table. The comparison was also carried out with the numerical results of the study of Trinh and Fredlund (2000). In the period from 1988 to 1995 the prediction of the water table drawdown by these authors was close to the numerical results and also the actual water table. During the time period from 1995 to 2000 their prediction underestimated the drawdown of the water table compared to the numerical results by 30 to 40 %. Figure 5.10 shows the comparison of the calculated water table drawdown result and the measured one in 1992. Reasonably good agreement between the predicted surface subsidence and the observed one in the first period of calculated time (1988-2000) can be seen in Figure 5.11. In the period from 1988 to 2000 the computed surface subsidence is similar to Trinh and Fredlund's (2000) results and both of the results are about 10 to 20% greater than the measured values. The fair agreement between two predictions of the surface settlements while there is a large difference in the prediction of water table drawdown is due to the fact that Trinh and Fredlund (2000) used lower values of the soft soil Young's moduli than the values adopted in this study.

If it is supposed that the ground water pumping rate will be kept constant in the future (during the second and the third periods 2000-2010, 2000-2020), and the recharge

conditions will also not be changed, then the water table level and the total predicted surface subsidence numerical results level off as shown in Figure 5.11. This means the soil may have already reached a steady state. As the soil is assumed to be elastic, no further drawdown of the water table and the ground surface will develop once the ground reaches a steady state, even if water pumping is still continued without increasing the pumping rate.

Due to the fact that the current ground water pumping rate has only satisfied 60% of the need in the Hanoi area, there is a proposal to increase the pumping rate to double the current rate by the year 2010. If it is supposed that the pumping rate will be gradually increased from 2000 to 2010, it may be assumed that in 2000 the pumping rate will be raised by 50% of the current rate, and in 2005 the pumping rate will be double the current one. The numerical prediction of the water table drawdown and the surface settlement are also given in Figures 5.12 and 5.13. The predicted water table drop will be from -15m in 2000 down to -28m in 2010, while the prediction of the Trinh and Fredlund's (2000) water table drawdown is just slightly increased from -10m in 2000 to about -14m in 2010. Although there is such a large difference between the two water table drawdown predictions, there is closer agreement of the two surface settlement results (the numerical results and the Trinh and Fredlund's prediction). The total predicted surface settlements show an increase from 350 mm in 2000 to 600 mm in 2020.

The total horizontal and vertical soil movement profiles versus depth are also predicted in Figures 5.14, 5.15 and Figures 5.16, 5.17. As can be seen from these figures, the 40m thick soft soil layer near the surface contributes most to both vertical and horizontal movements of the ground. The hard layer (gravel and pebbles), where the water is pumped from, does not seem to contribute much to the total movements of the ground surface. Soil movements are plotted along vertical sections at different positions that are 310m, 400m, and 500m away from the symmetrical axis of the finite element soil mesh. Generally, the further away from the centre, the smaller the soil movements, as may be expected. As there was not any measured data for horizontal soil movement and the vertical soil movements and watertable drawdown were well predicted, the calculated horizontal soil movements were assumed to be reasonably correct.

5.4.3 NGOC HA CASE STUDY

The Ngoc Ha benchmark is located inside the Ngoc Ha water plant that has been operating for about 30 years. The global longitude and latitude coordinates of the Ngoc Ha benchmark are $23^{\circ}27'89''\text{E}$ and $185^{\circ}84'20''\text{N}$.

The elastic properties of subsoil layers, which have been used to model the Ngoc Ha case study, are given in Table 5.5. The difference from the Phap Van case study in this case, is the soft clay layers which are low in permeability and are only about 8m in thickness (they are 40m thick in the Phap Van case). Underlying these soft clay layers are sand layers with higher permeability. A layer of loose fine sand with a thickness of 7.4m overlies a thick medium sand layer (20m in thickness). Then lying below is the hard, high permeability gravel and pebbles layer, from where the water has been pumped. Beneath the gravel and pebbles layer is an impermeable layer of claystone. The Tertiary bedrock lies directly under this layer. The total thickness of the whole soil layer is about 70m.

Figure 4.13 shows the soil profiles at the Ngoc Ha pumped ground water station. The Ngoc Ha pumped ground water station lies in the area that is classified as having A2 soil type in Figures 3.6. According to Table 3.3, from the ground surface to the depth of maximum -20m, soil type at the Ngoc Ha pumped ground water station is clay or sandy clay, aQ^4_{IV} . Table 3.4 shows the Young's soil modulus values of the aQ^4_{IV} sediments that are in a range between 3.8 and 6.5 MPa. The drained Young's soil moduli that are used in the finite element analysis of the Ngoc Ha case study are as follows:

- Layer 1 is man made soil with thickness of 2.3m. The drained Young's modulus of this soil layer is unknown, however a value of 6.5MPa, which is equal to the Young's modulus of the man made soil layer at the Phap Van pumped ground water station, has been used in the finite element analysis.
- Layer 2 is soft clay with a thickness of about 1.9m. The average drained soil Young's modulus of this soil layer was chosen to be 2.5MPa. This value of Young's

modulus is arbitrarily chosen from the typical value range of the drained Young's soil modulus suggested by SMDS (1998) for soft normally consolidated clay.

- Layer 3 is soft to medium clay and sandy clay with a thickness of 5.6m. The averaged drained Young's modulus of this soil layer is chosen as an average soil Young's modulus value in the range of 3.8 to 6.5MPa (Table 3.3).
- Layer 4 is loose fine sand with thickness of 7.4m. The typical value of the drained Young's modulus of loose normally consolidated sand is between 8.4 and 20.0MPa, (SMDS, 1998). An average drained Young's modulus value of 15.0 MPa is chosen. The chosen value of the drained Young's modulus of the soil layer number 4 is also similar to the drained Young's soil modulus of a fine sand layer at a depth of between 20m and 30m at the Hanoi University of Education location, Figure 3.14.
- Layer 5 is medium dense, sand with a thickness of 20m. The typical value of the drained Young's modulus of medium normally consolidated sands is between 20.0MPa and 40.0MPa. A value of the drained Young's soil modulus of 20MPa is chosen for this soil layer.
- Layers 6 and 7 are a gravel and pebbles layer and a claystone layer. The drained Young's moduli of soil layers 6 and 7 are chosen to be similar to those of the Phap Van case study.

At the beginning, during the 70's, the pumping rate was as low as 18000 m³/day and the water table was about 5-6m below the ground surface. From 1988 there was a sharp decline of the water table inside and around the Ngoc Ha water plant. This decline was obviously the result of the doubled increase of the ground water pumping rate. Due to the fact that there was only the measured data that dated from 1988, the analysis of this case study has been carried out from that time. Hence, the pumping rate was taken to be 35000 m³/day from 1988 to 1995, and in 1995 45000 m³/day. It was assumed that it would be raised to a much higher rate in 2000 (or the current pumping rate), which will be 60000 m³/day. The process of calculation is also similar to the Phap Van case study.

Table 5.5: Ngoc Ha soil properties.

No.	Material type	Thickness (m)	E' (kPa)	ν'	s	k_x (m/day)	k_y (m/day)
1	Man made soil	2.3	6500	0.35	0.0001	0.0010	0.0010
2	Soft clay	1.9	2500	0.35	0.0001	0.0005	0.0005
3	Medium clay	5.6	5000	0.35	0.0001	0.0002	0.0002
4	Loose fine sand	7.4	15000	0.30	0.001	0.1000	0.1000
5	Medium sand	20.0	20000	0.30	0.002	1.0000	1.0000
6	Gravels, pebbles, and medium sand	20.0	200000	0.30	0.0025	35.0000	35.0000
7	Claystone	10.0	100000	0.30	0.0001	0.00001	0.00001

The numerical results for the water table drawdown and the surface subsidence are shown in Figures 5.18 and 5.19. During the first period 1970-1988, there was no comparison between the numerical and the observed result because there was not any measured data in this period. In the second period, from 1988-2000, rather close agreement may be seen between the calculated and the measured results.

Compared to the Phap Van case study, in this case, the water table drops faster. As the pumping rate doubled in 1988 the water table dropped at a very high rate, which is illustrated by the steep slope of the water table drawdown at the symmetric axis in Figure 5.18. Accordingly, the soil movements also developed with the same rate as the water table drop. This may be explained by the thickness of the soft clay layer near the surface. In the Phap Van case study, there is a thick layer (about 40m thick) of soft soil on top of the water-bearing layer, but in contrast, it is only about 8 m thick in the Ngoc Ha case study. The total horizontal soil movement profiles versus depth at different times and locations are also provided in Figures 5.20 and 5.21.

5.5 PARAMETRIC STUDY

Figure 5.22 illustrates the general problem in which ground water is pumped from a system of two soil layers. The system of two soil layers has a soft soil layer with low permeability lying on top of a stiff soil layer from where the water is pumped. The stiff soil layer is assumed to have a constant Young's modulus and permeability. The analyses of the soil movements due to pumping of the ground water were carried out by using the finite element program EF4. The ground water is assumed to be pumped from a region that is centred on the axis of symmetry.

The assumed values for the geometries of the soil mesh and for the pumping rate as well as the soil properties used in the parametric study are given in the following:

- The first soft soil layer has Young's modulus, $E = 2500\text{kPa}$, 5000kPa , or 10000kPa , soil permeability is equal in both horizontal and vertical directions, $k_1 = 0.1\text{m/day}$, 0.01 m/day and 0.0001m/day , the soil thickness, $H_1 = 20\text{m}$, 30m , 40m .
- The second soil layer has fixed soil properties where Young's modulus, $E_2 = 200\text{MPa}$, soil permeability $k_2 = 40\text{m/day}$, and the thickness varies, $H_2 = 20\text{m}$, 30m , 40m . As the first soil layer modulus is relatively much smaller than the second layer, then the soil deformations should be expected to occur mostly in the first soil layer and the deformation of the second soil layer will be negligible.
- The pumping rate has values of $Q = 20000\text{m}^3/\text{day}$, $40000\text{m}^3/\text{day}$, $60000\text{m}^3/\text{day}$ with pumping spread over a distance of $r_p = 100\text{m}$, 250m , 500m .

Figures 5.23, 5.24, 5.25 show the variation of the maximum ground water drawdown over the thickness of the first soil layer with the normalised ground water pumping rate, $Q / R \cdot H_2 k_2$. It is obvious that the ratio of $\text{Max } d_w / H_1$ increases linearly with the increases in the normalised value $Q / R \cdot H_2 k_2$ and it also depends on the radius of the pumping area. As the pumping rate is constant over the pumped region, the ratio of $\text{Max } d_w / H_1$ decreases as the pumping distance r_p increases.

The maximum drawdown of the water table does not depend on the permeability of the first soft soil layer. On the other hand, the permeability of the soft soil layer k_1 only influences the period of time that the ground water table needs to reach the maximum value. Figure 5.26 shows the ratio of maximum value of the ground water drawdown $Max d_w'$ at any time t to the final value of the maximum ground water drawdown, $Max d_w$, depends on time. The lower the value of the permeability value of the first soil layer, the longer the time until the ground water table reaches the steady state. Figure 5.27 is a similar plot but shows the settlements of the ground surface with time. The similarity between the two graphs indicates as may be expected, that any drawdown of the ground water table will result in settlement of the ground surface.

The relationship between the maximum horizontal soil movements and the maximum vertical soil movements is illustrated in Figures 5.28 to 5.36. This relationship is plotted against the ratio of the upper to lower soil layer thickness. From these graphs the following points are observed:

- The ratio of the maximum horizontal soil movement to the maximum vertical soil movement strongly depends on the ratio of the upper soil layer thickness to the second soil layer thickness and it also depends on the thickness of the lower soil layer itself though the lower soil layer is considered not to deform much compared to the upper layer.
- As the ratio of H_1/H_2 increases the ratio $Max d_h \cdot R / Max d_v \cdot H_2$ increases linearly.
- Generally, the ratio $Max d_h \cdot R / Max d_v \cdot H_2$ slightly reduces as the ratio of the pumped water area increases or the pumping rate increases, the differences are about 10% to 20%. However this is not the case when the pumped water area is increased up to 500m (ratio $r/R = 0.5$) the ratio $Max d_h \cdot R / Max d_v \cdot H_2$ is seen to be only marginally affected as the pumping rate is increased, especially when the ratio H_1/H_2 is less than 1.
- The ratio $Max d_h \cdot R / Max d_v \cdot H_2$ does not depend on the Young's modulus of the upper soil layer. When the Young's modulus of the upper soil layer changes, the ratio $Max d_h \cdot R / Max d_v \cdot H_2$ may be seen to change by less than 5%.

The relationship between the maximum vertical soil movements and the maximum ground water drawdown is presented in Figures 5.37 to 5.45. The relationship of the maximum vertical soil movements and ground water table drawdown is produced by the normalized ratio $Max d_v * R^{3/2} / Max d_w * H_1 * (H_1 + H_2)^{1/2}$. This ratio then is plotted against the normalized ratio of the upper soil layer to the second soil layer thickness H_1/H_2 . As the value of R is kept constant throughout the calculation then this normalized ratio depends on the thickness of the upper soil layer and also the thickness of the whole soil system. From these graphs the following points are observed:

- When the pumping rate increases, the ratio $Max d_v * R^{3/2} / Max d_w * H_1 * (H_1 + H_2)^{1/2}$ reduces. At different water pumping rates the rate of reduction also varies.
- The increase in the pumped water area (r_p) results in an increase in the normalized ratio $Max d_v * R^{3/2} / Max d_w * H_1 * (H_1 + H_2)^{1/2}$, as r_p varies from 100m to 500m the value of the normalized ratio $Max d_v * R^{3/2} / Max d_w * H_1 * (H_1 + H_2)^{1/2}$ increases from 10%-15% (average 12%) to 15%-25% (average 20%).
- Though the increases in the upper soil layer Young's modulus causes the normalized ratio $Max d_v * R^{3/2} / Max d_w * H_1 * (H_1 + H_2)^{1/2}$ to reduce. At higher values of E_1 , the effect of pumping rate is not as large.
- The normalized ratio $Max d_v * R^{3/2} / Max d_w * H_1 * (H_1 + H_2)^{1/2}$ does not depend on the ratio of the relative thickness between the upper soil layer and the lower soil layer (H_1/H_2). As the ratio H_1/H_2 increases, the value of the normalized ratio $Max d_v * R^{3/2} / Max d_w * H_1 * (H_1 + H_2)^{1/2}$ experiences a slight decline. However this reduction is approximately less than 5%.
- As the Young's modulus of the lower layer is very high compared to the Young's modulus of the upper soil layer and the soil movements of the lower soil layer are insignificant, then an increase in the Young's modulus of the upper soil layer causes the value of the ratio $Max d_v * R^{3/2} / Max d_w * H_1 * (H_1 + H_2)^{1/2}$ to reduce approximately proportionally.

Figure 5.46 shows ranges of the ratio of the maximum ground water table drawdown at a distance r from the axi-symmetrical axis over the maximum ground water table

drawdown, which is at the symmetric centre ($r = 0$), $Max d_w^r / Max d_w$, plotted against the ratio of the distance r over R (r/R). The ratio $Max d_w^r / Max d_w$ depends on the radius of the pumped water area, r_p . When the radius of water pumping is small, $r_p = 100\text{m}$, the range of the ratio $Max d_w^r / Max d_w$ is narrow; that is about 5% difference, and it reduces rapidly within the distance from the symmetrical axis and it reaches 50% when the ratio r/R is 0.18. As the radius over which water is pumped increases to $r_p = 250\text{m}$ or 500m , the ranges of the ratio $Max d_w^r / Max d_w$ widens and the differences can be from 10% up to 15%. The ratio $Max d_w^r / Max d_w$ reduces gradually as the ratio r/R rises. In the case where $r_p = 250\text{m}$, the ratio $Max d_w^r / Max d_w$ decreases 50% as the ratio r/R reaches 0.35 and decreases 50% as r/R reaches 0.45 if $r_p = 500\text{m}$.

Figure 5.47 presents a similar plot to that presented in Figure 5.46 but it illustrates the maximum vertical soil movements instead of the ground water drawdown. The ranges of ratio $Max d_v^r / Max d_v$ are more or less similar in that there is a variation of about 15% to 20% for all of the pumping radii considered. As the ratio r/R increases, the ratio $Max d_v^r / Max d_v$ decreases and it gets down to 50% when the ratio r/R is equal to 0.2 to 0.3 when $r_p = 100\text{m}$, or when r/R is equal to 0.35 to 0.45 when $r_p = 250\text{m}$ or r/R is equal to 0.45 to 0.55 when $r_p = 500\text{m}$.

Figures 5.48, 5.49, 5.50 also show the ranges of the ratio of the maximum horizontal soil movements at a distance r from the symmetrical axis over the maximum value of the horizontal soil movements which are not at the symmetrical centre in this case. Normally the maximum horizontal soil movements are located in the vicinity of edge of the pumped water area. Due to variation of the location of the maximum horizontal soil movements, it is very difficult to pin point the location of the maximum horizontal soil movements at a certain time.

5.6 SUMMARY AND CONCLUSIONS

This chapter consists of two main parts. The first part of the chapter is a general numerical study of the behaviour of the ground due to ground water pumping. Two case studies have been analysed and presented by using the finite element program EF4, which was developed by Hsi (1992). Two typical case studies have been chosen so as to compare results for two typical ground conditions in the city of Hanoi. In the Phap Van case study, up to 36m of clay soil with low permeability lies on top of a layer containing gravel and pebbles from which water is pumped. In contrast, for the second case study, the Ngoc Ha case study, nearly 30m of sand covers a layer of gravel and pebbles from where ground water is pumped, while there is only a layer of clay with a thickness of around 6m lying above the sand layer. With two different types of soil conditions, one covered with a thick layer of clay soil and the other covered with a thick layer of sand, the behaviour of the ground due to the ground water pumping is obviously different as may be expected. Comparisons of the analysis results with the measured data have been carried out. The prediction of the ground movements in the future due to an assumed ground water pumping rate were also performed and presented. It was shown that reasonably good predictions of the drawdown of the water table and of surface settlements could be made for the Hanoi area. In fact the finite element analysis shows that apart from the vertical soil movement there is a significant horizontal soil movement due to ground water pumping. In two case studies, the maximum horizontal soil movement is approximately from 8% to 12% of vertical soil movement.

The second part of the chapter introduced a parametric study of the ground behaviour due to ground water pumping. In this study, the relationships between parameters such as ground water pumping rate, radial distribution of the pumped wells, and soil layer thicknesses of a system of two soil layers was investigated. Time dependent soil behaviour was also taken into account. The behaviour of the ground due to ground water pumping was investigated, however only maximum values or the final behaviour is considered.

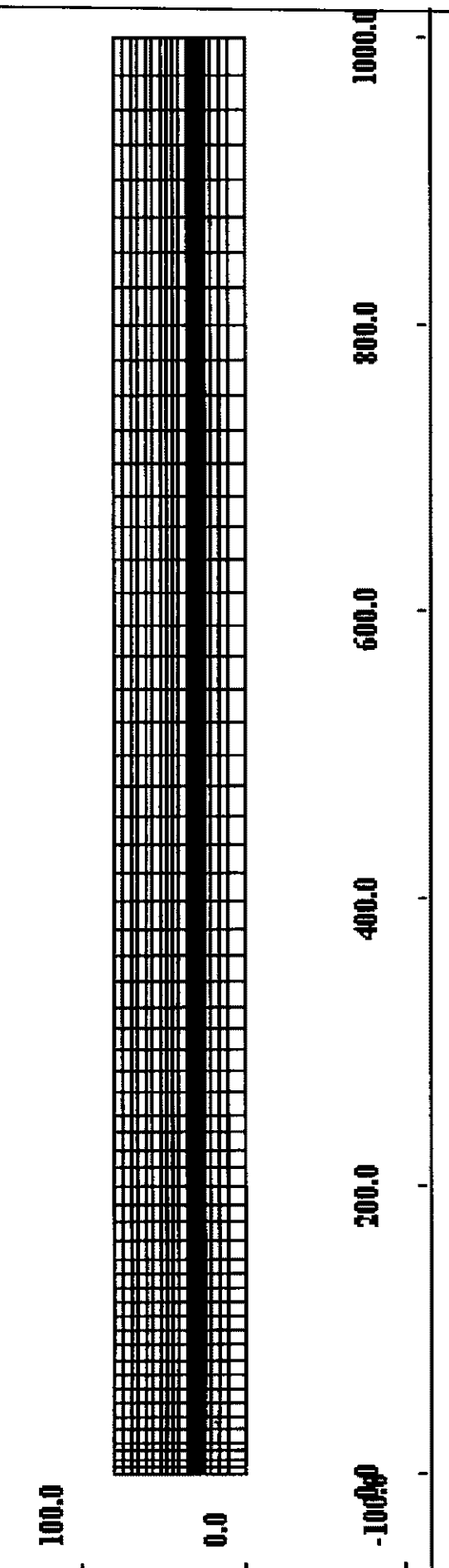


Figure 5.1: Typical finite element soil mesh (20x60).

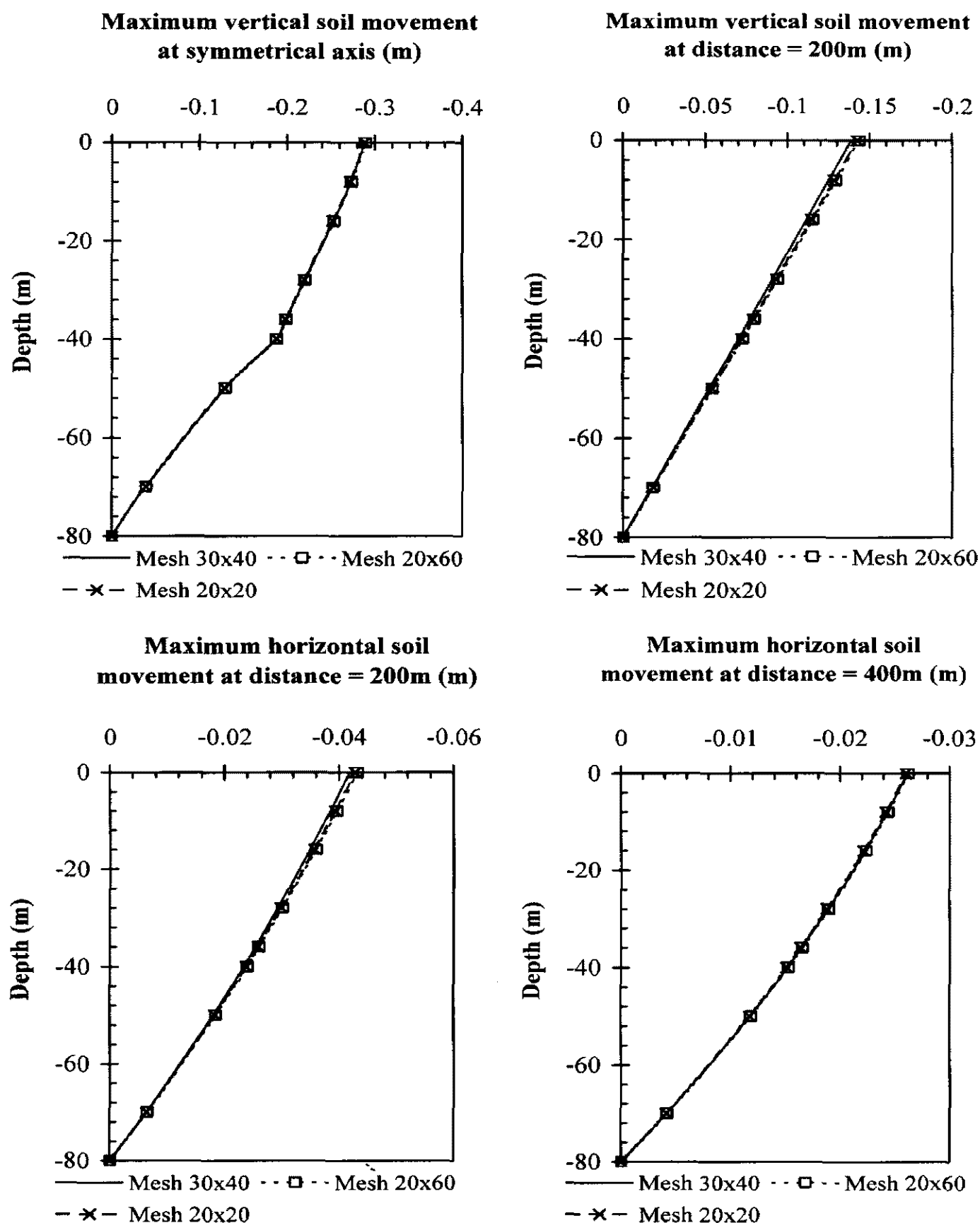


Figure 5.2: Comparisons of the results for soil movements using different finite element meshes.

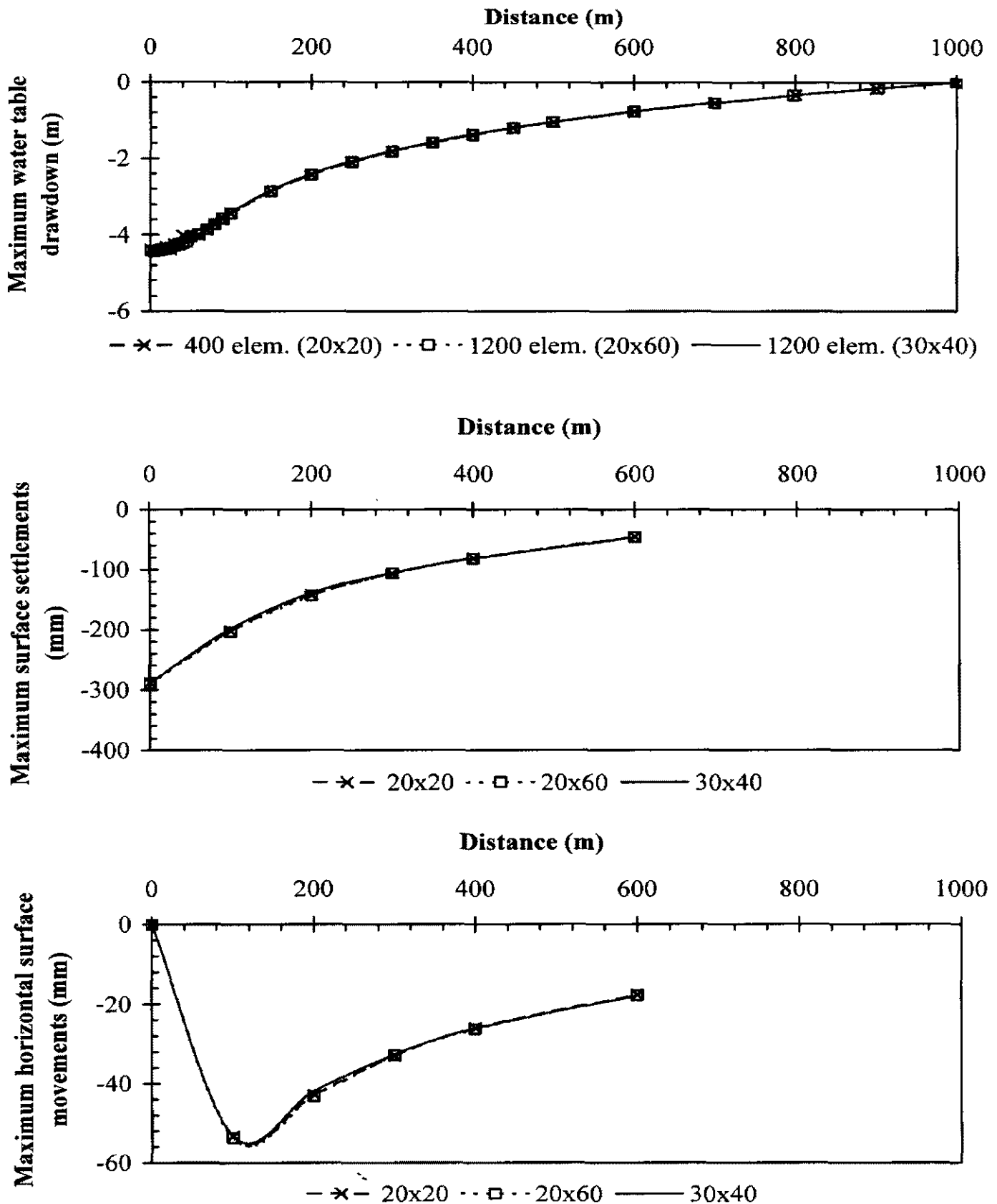


Figure 5.3: Comparisons of the water table drawdown and the ground surface movement for different finite element meshes.

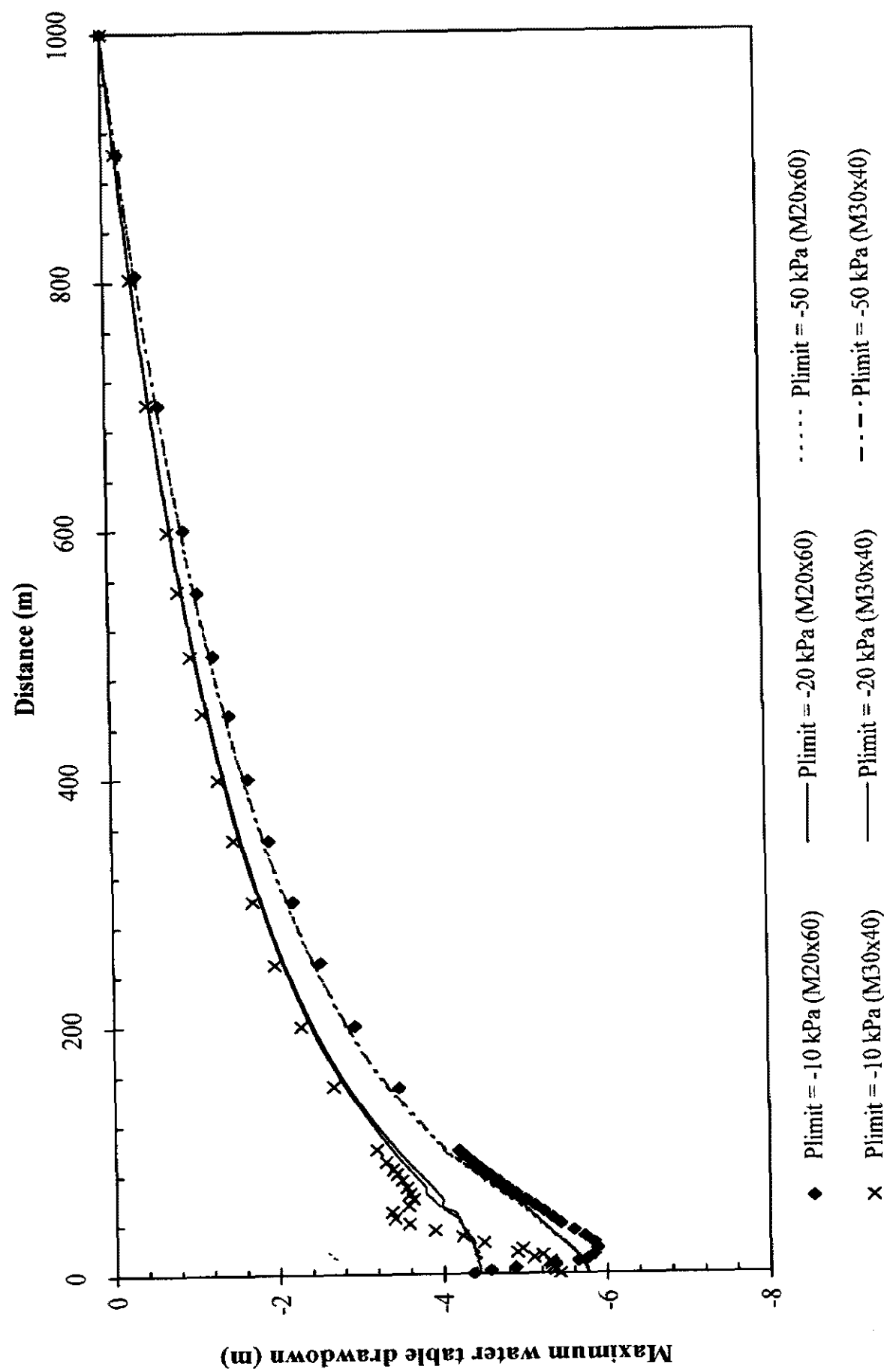


Figure 5.4: Maximum water table drawdown for different values of P_{limit} .

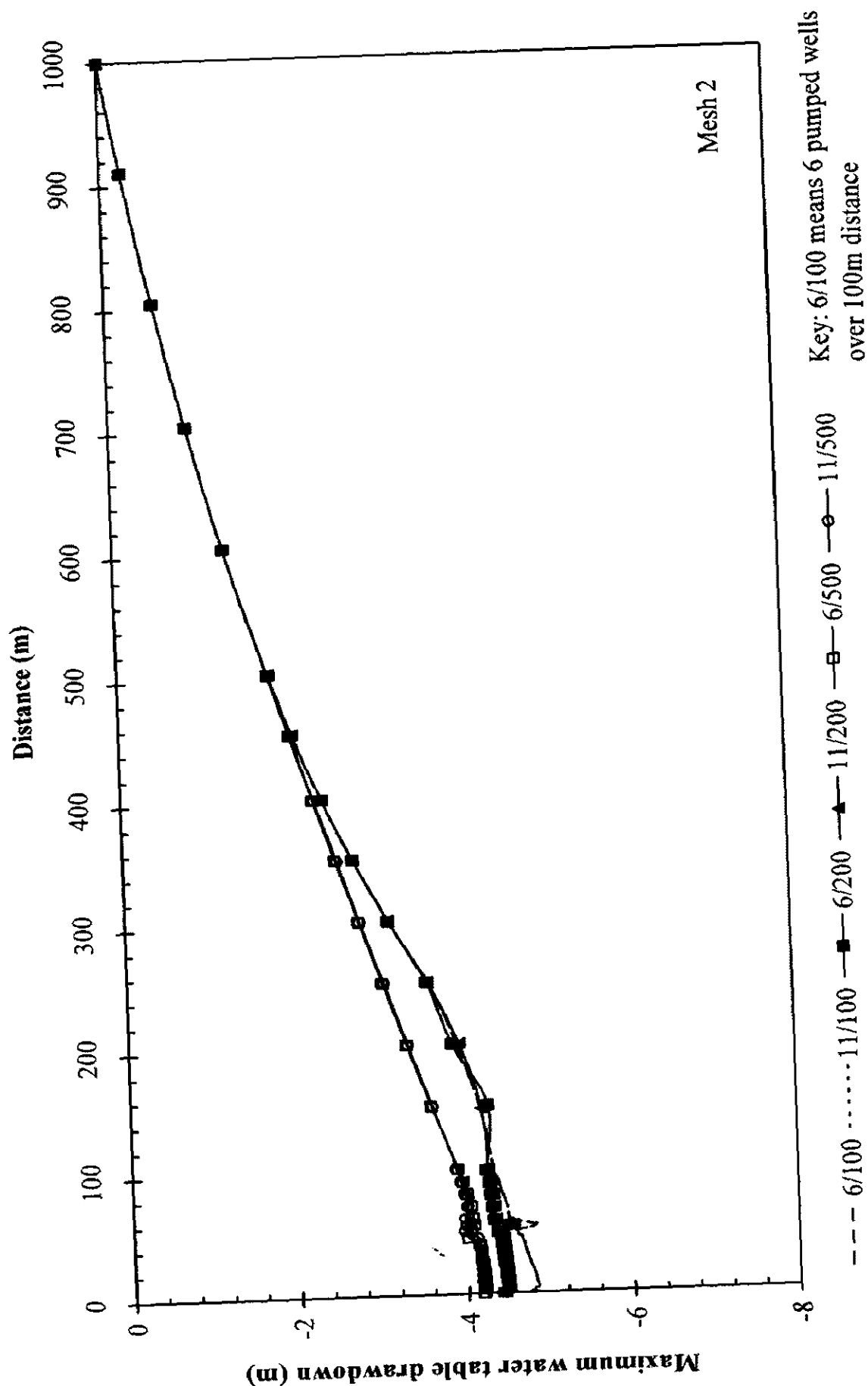


Figure 5.5: Maximum water table drawdown for pumped wells spread out radially (Mesh 1200 elements 30x40).

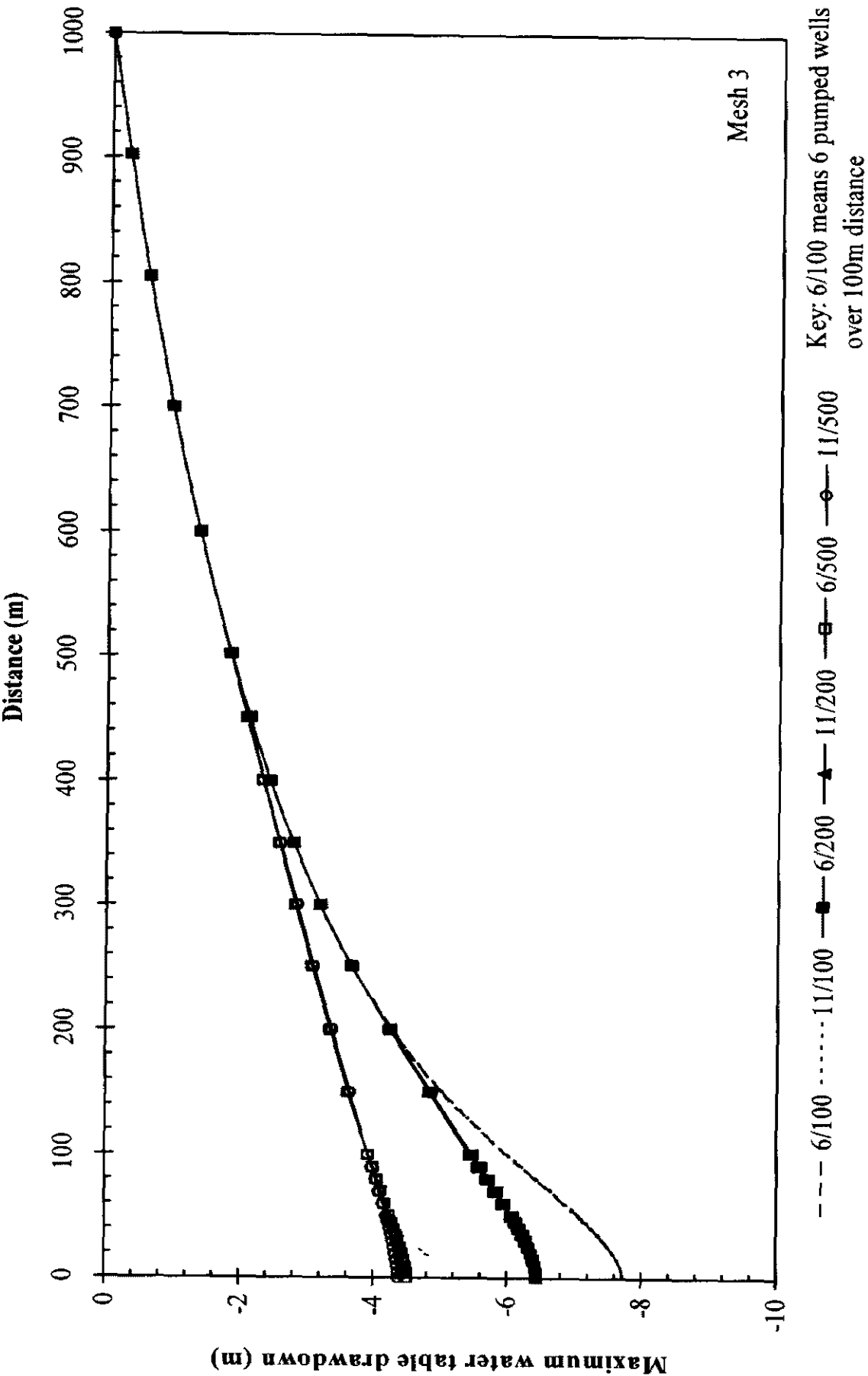


Figure 5.6: Maximum water table drawdown for pumped wells spread out radially (Mesh 1200 elements 20x60).

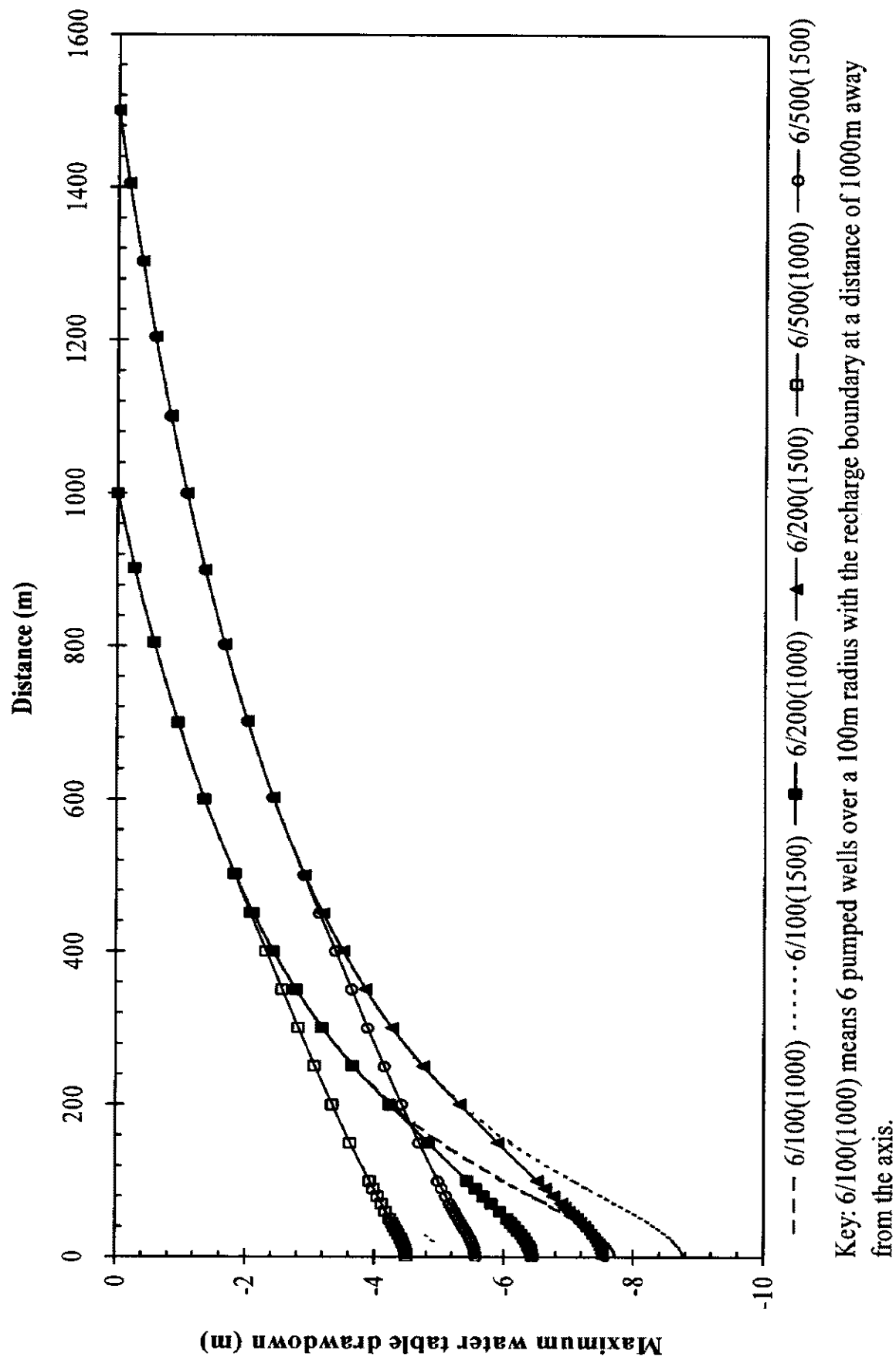
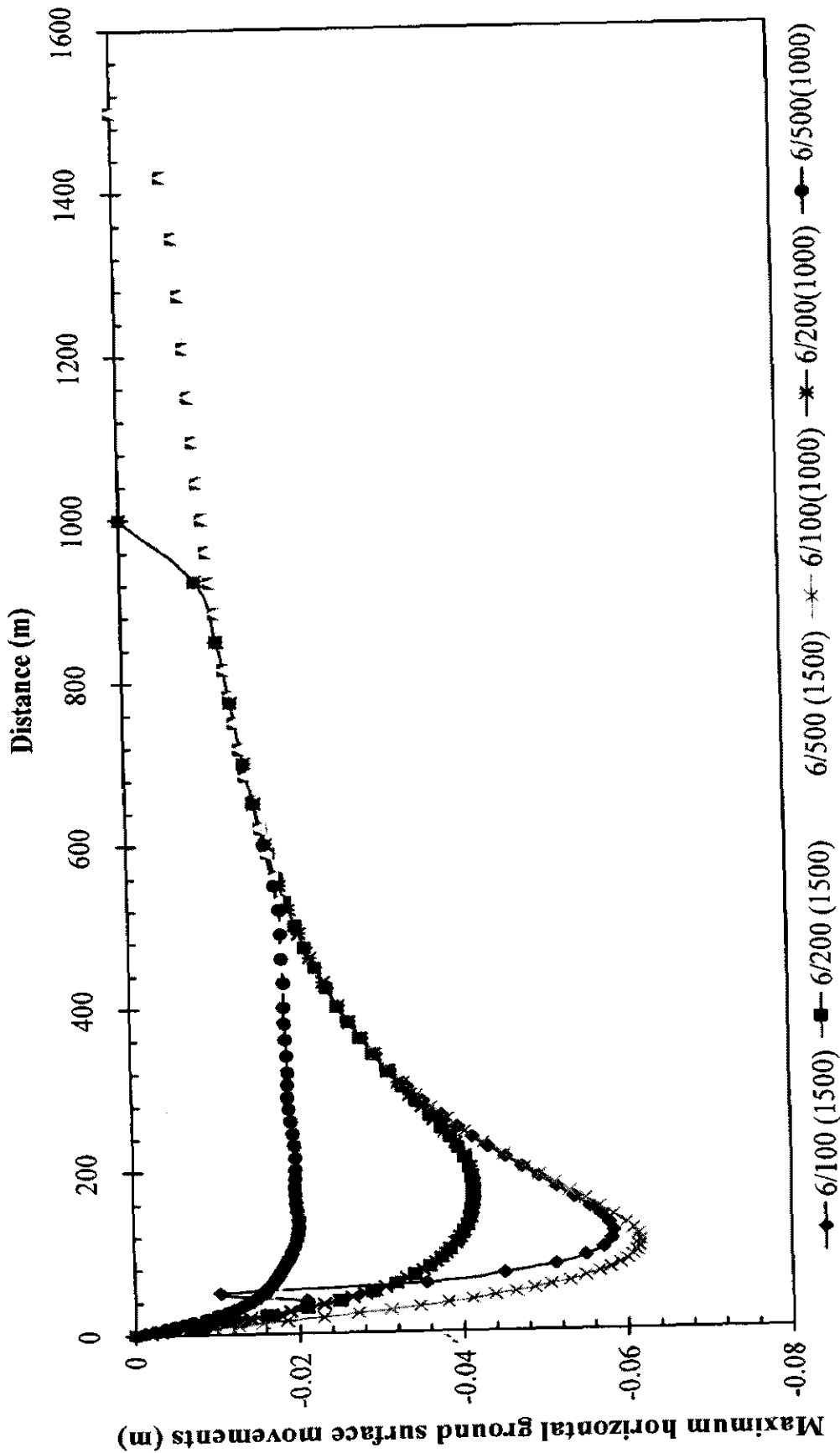


Figure 5.7: Maximum water table drawdown for wells spread out radially with the boundary at two different locations

(Mesh 1200 elements 20x60).



Key: 6/100(1000) means 6 pumped wells over a 100m radius with the recharge boundary at a distance of 1000m away from the symmetrical axis.

Figure 5.8: Maximum horizontal ground surface movements as both the water pumping locations and the boundary extend further from the axis.

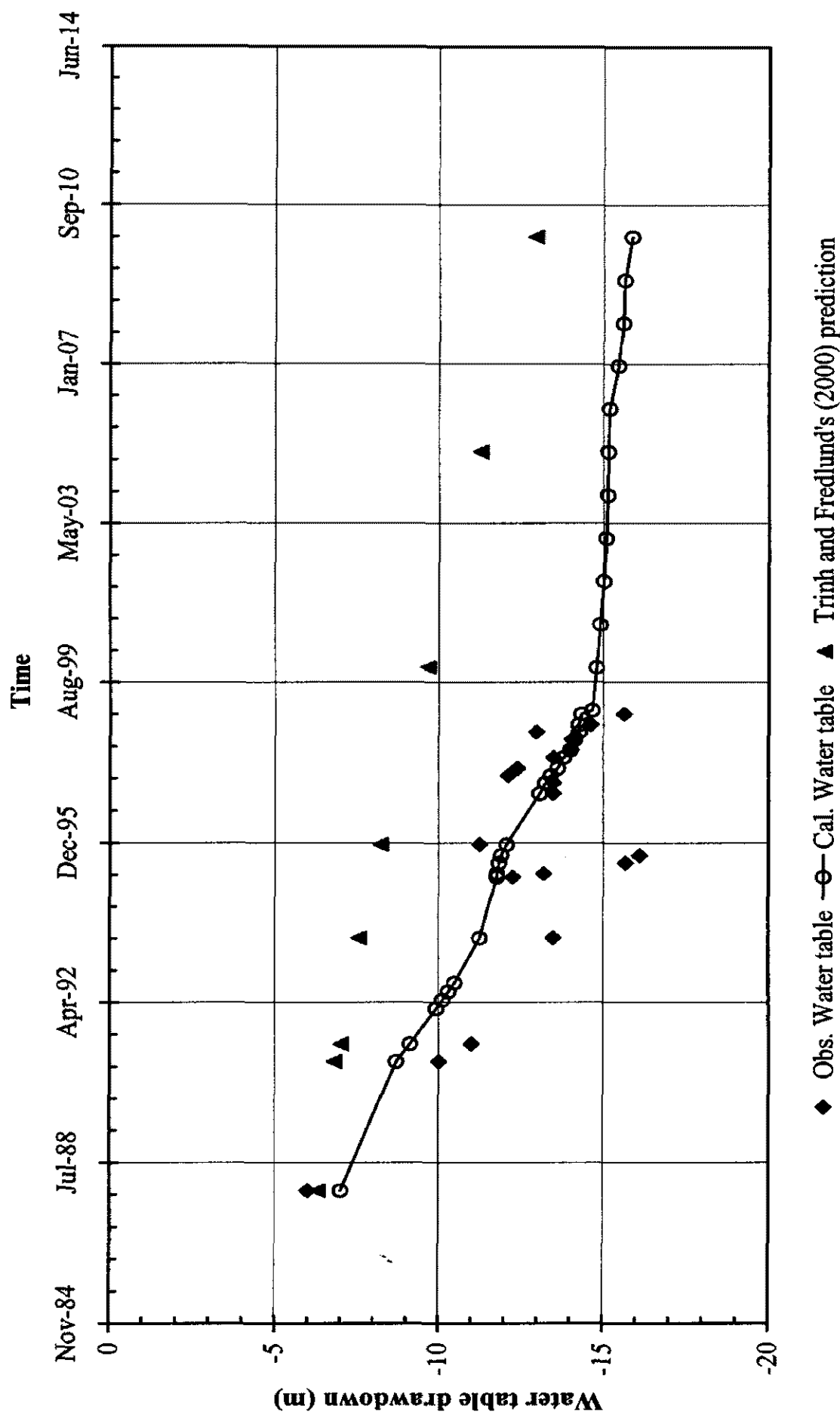


Figure 5.9: Water table drawdown with time (Phap Van).

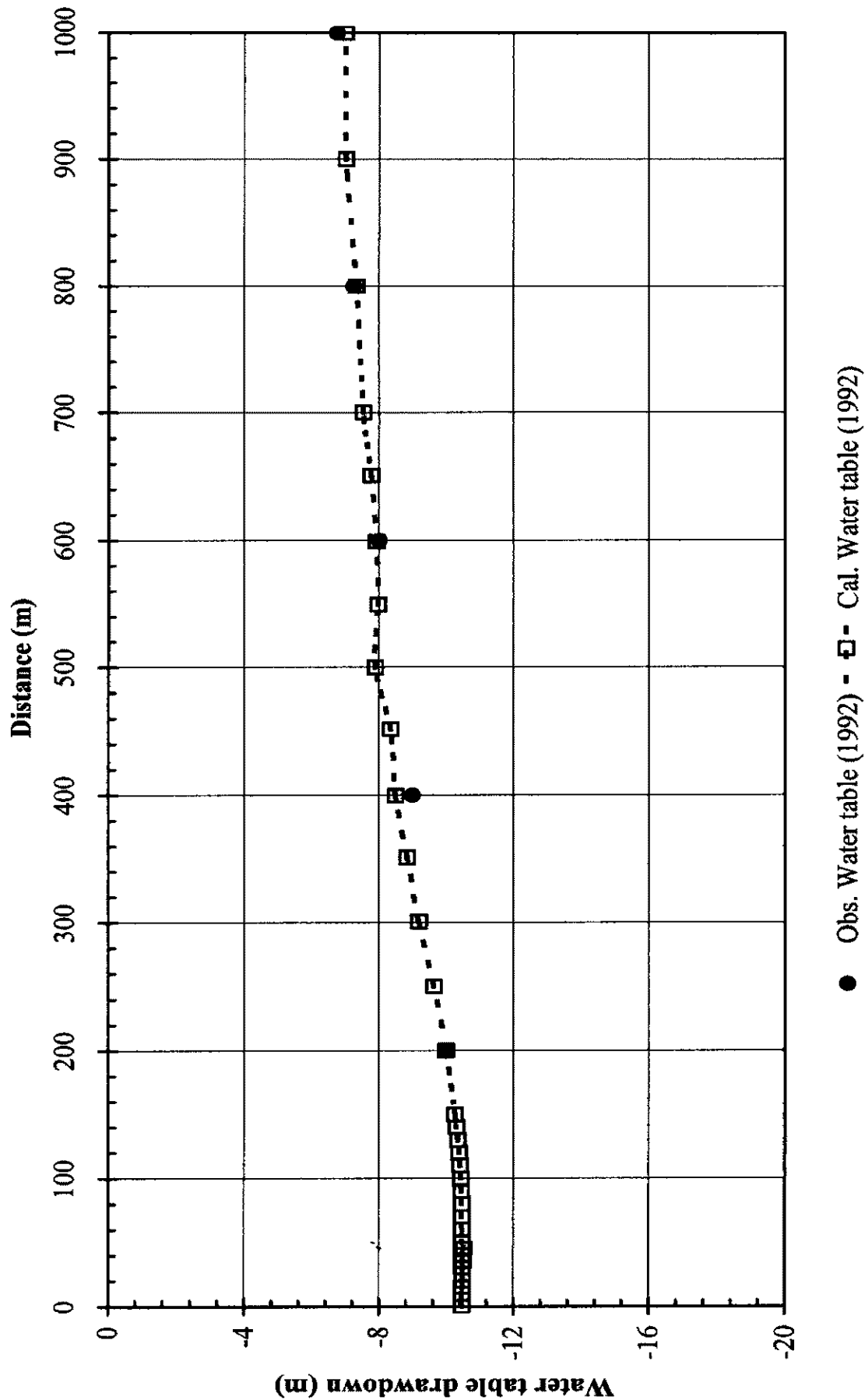


Figure 5.10: Drawdown of water table with distance from centre line in 1992 (Phap Van)

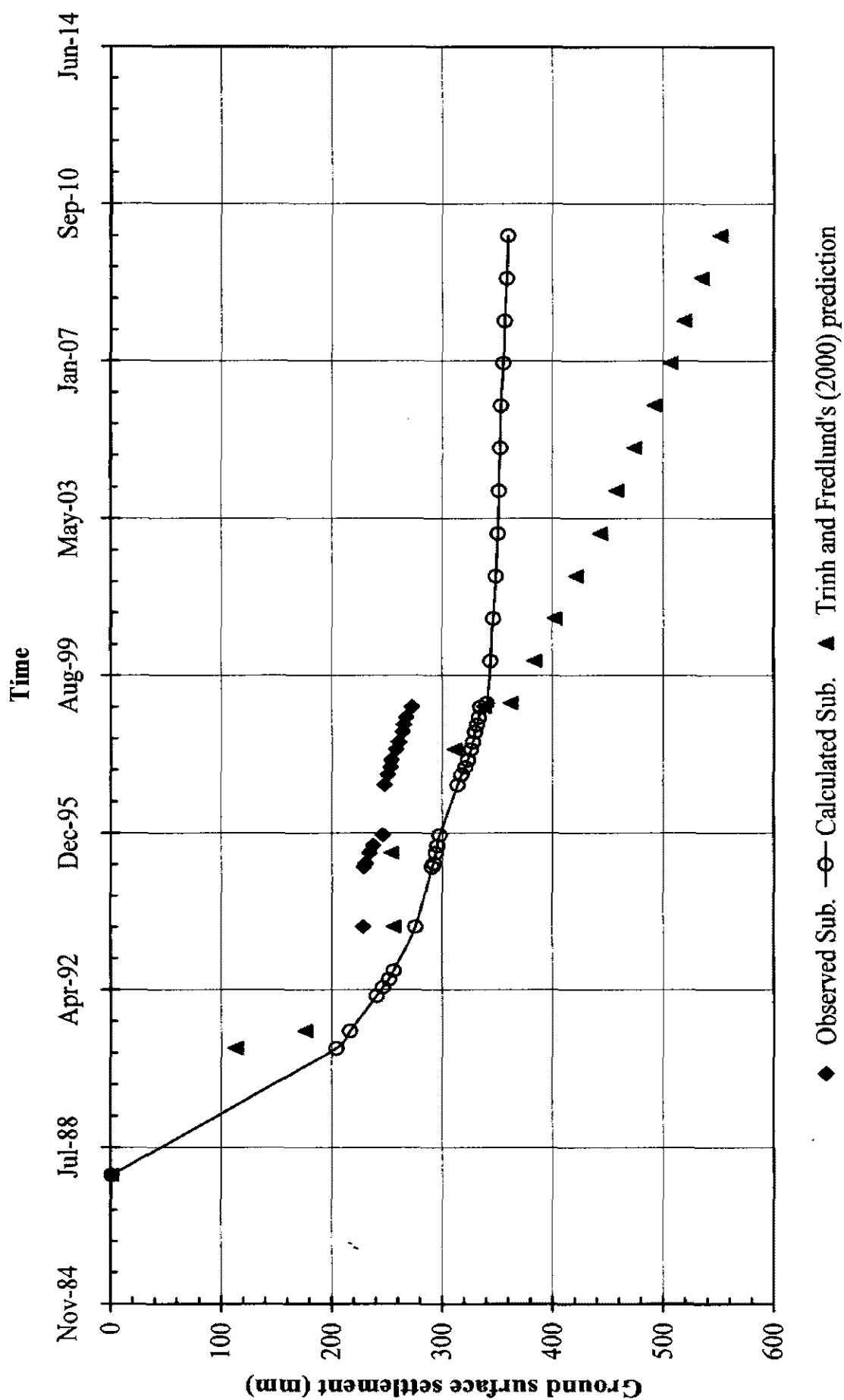


Figure 5.11: Maximum vertical soil movements (Phap Van).

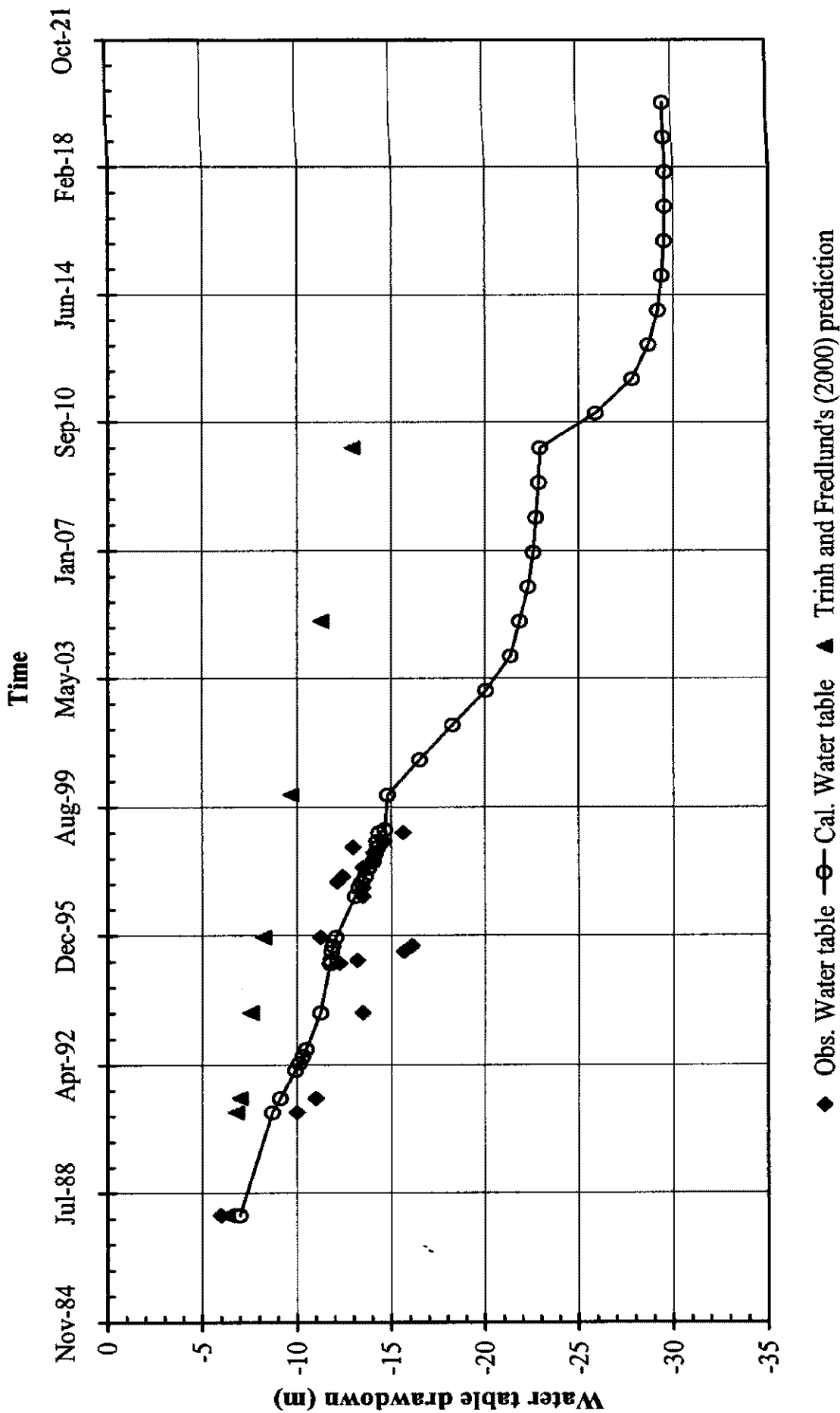


Figure 5.12: The predicted water table drawdown at Phap Van based on the assumption that the water pumping rate is doubled during the period from 2000 to 2020.

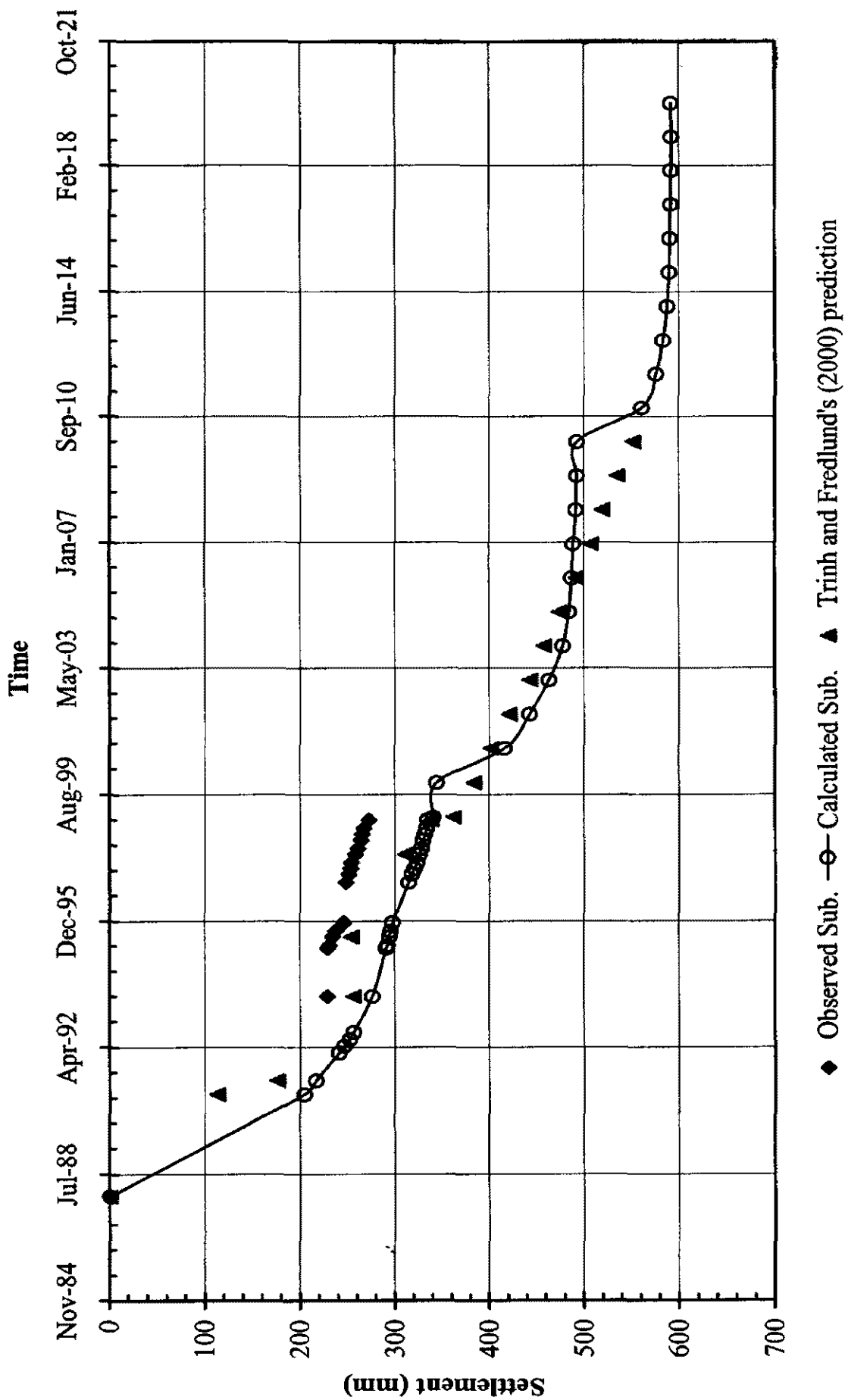


Figure 5.13: The predicted ground surface settlements at Phap Van based on the assumption that the water pumping rate is doubled during the period from 2000 to 2020.

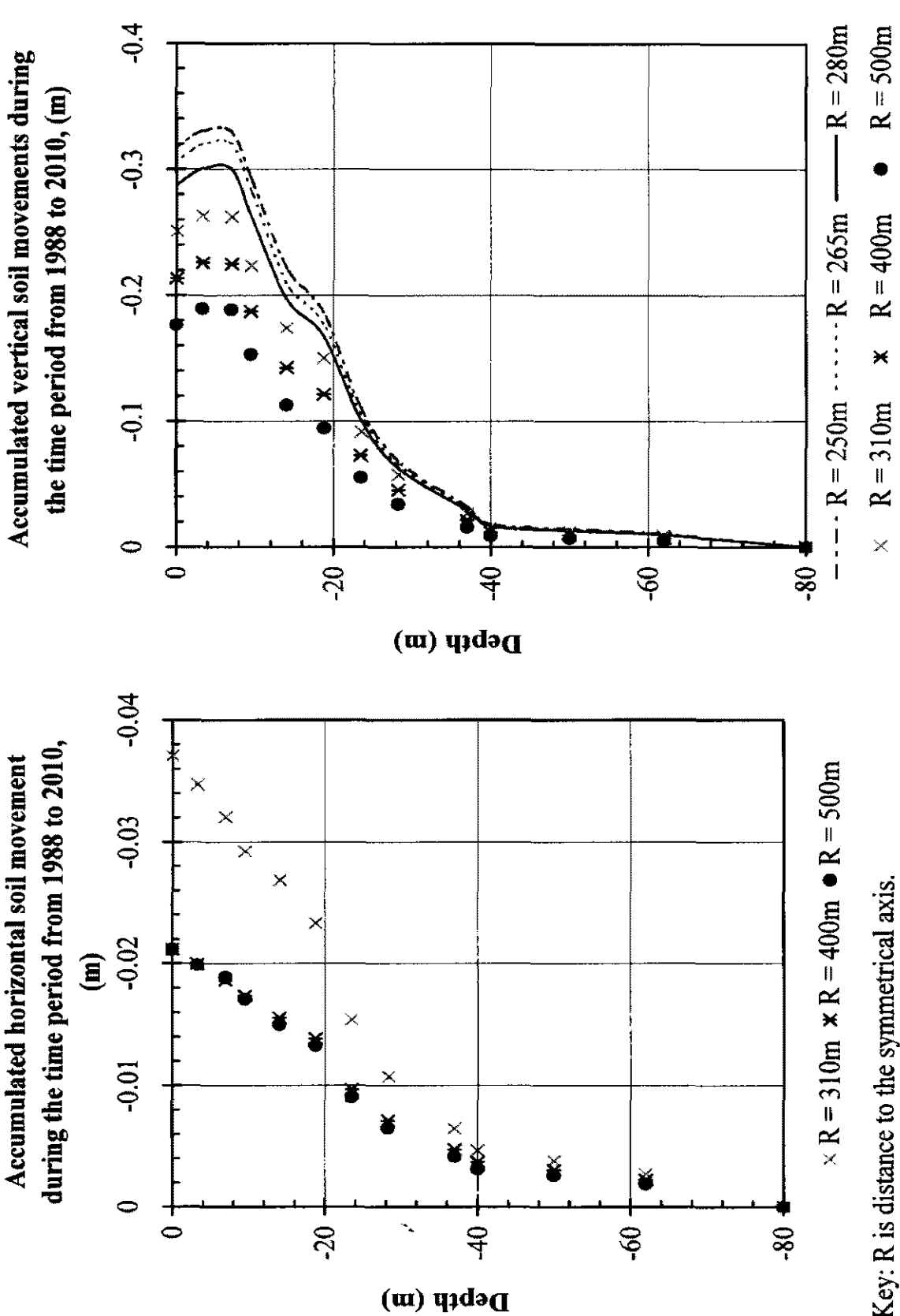


Figure 5.14: Accumulated soil movement profiles versus depth, predicted at Phap Van during the period from 1988 to 2010 with the assumption that the ground water pumping rate is kept constant over this time period.

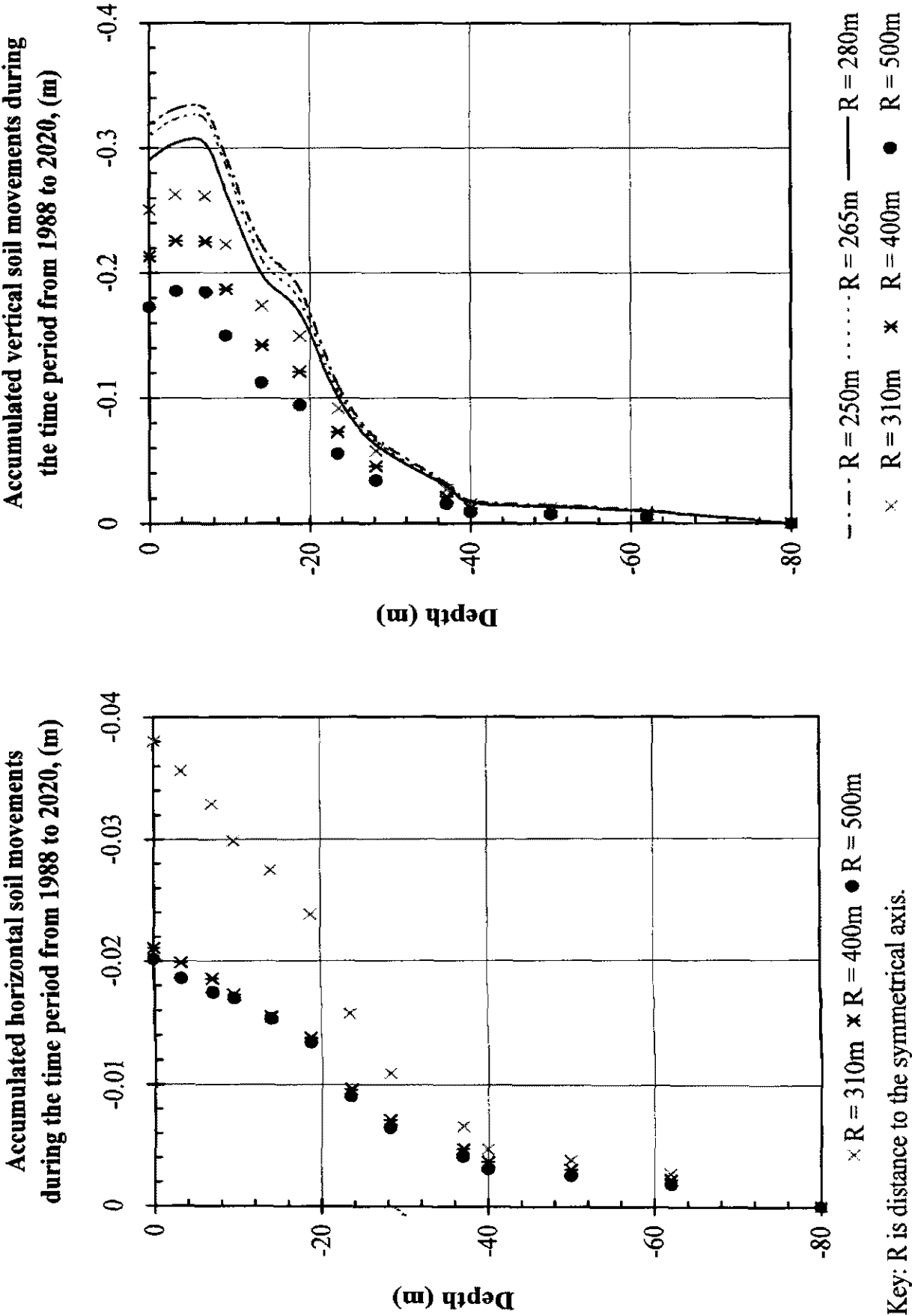


Figure 5.15: Accumulated soil movement profiles versus depth, predicted at Phap Van during the period from 1988 to 2020 based on the assumption that the ground water pumping rate is kept constant over this time period.

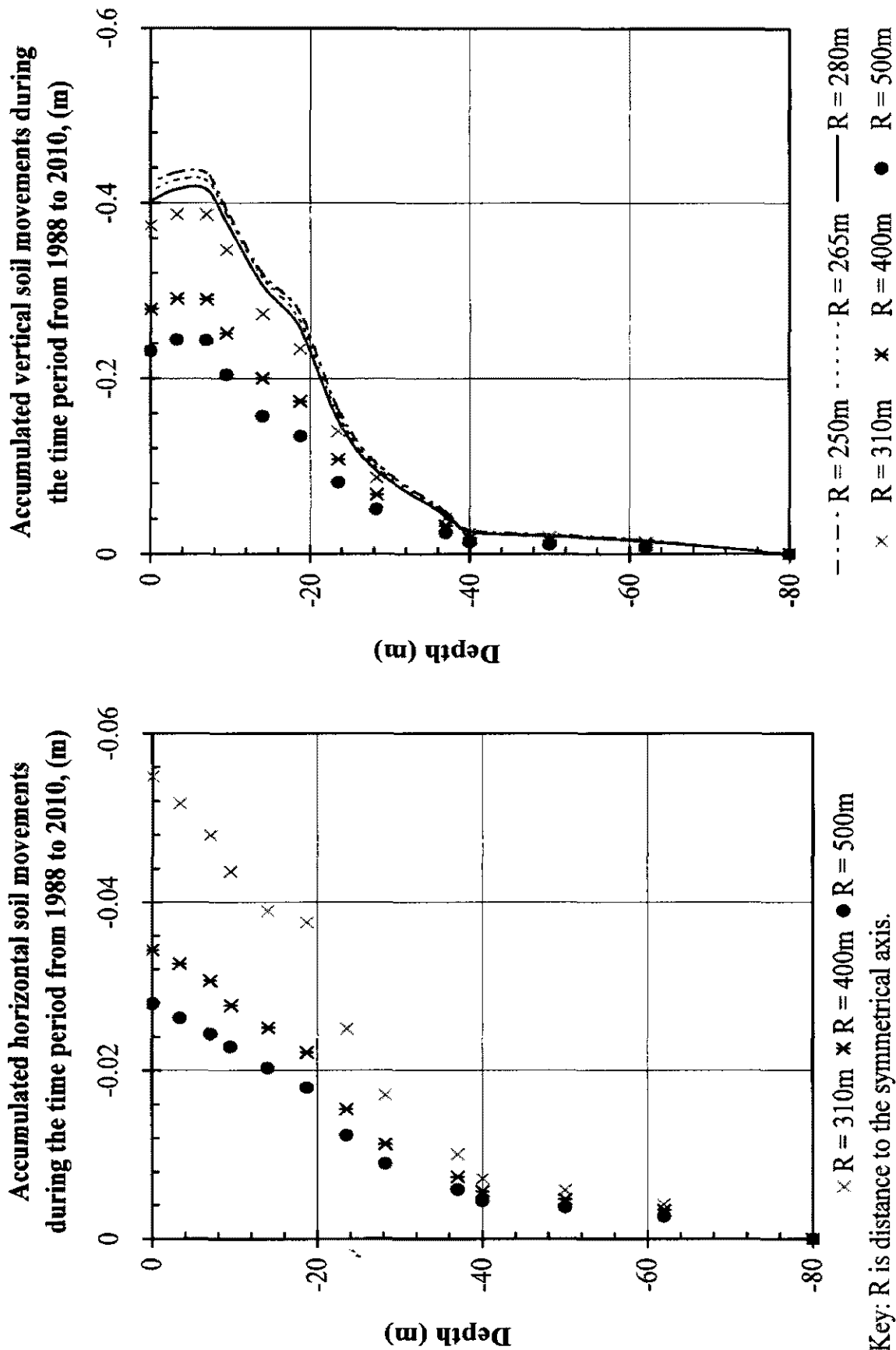


Figure 5.16: Accumulated soil movement profiles versus depth, predicted at Phap Van during the period from 1988 to 2010

based on the assumption that the ground water pumping rate is doubled over this time period.

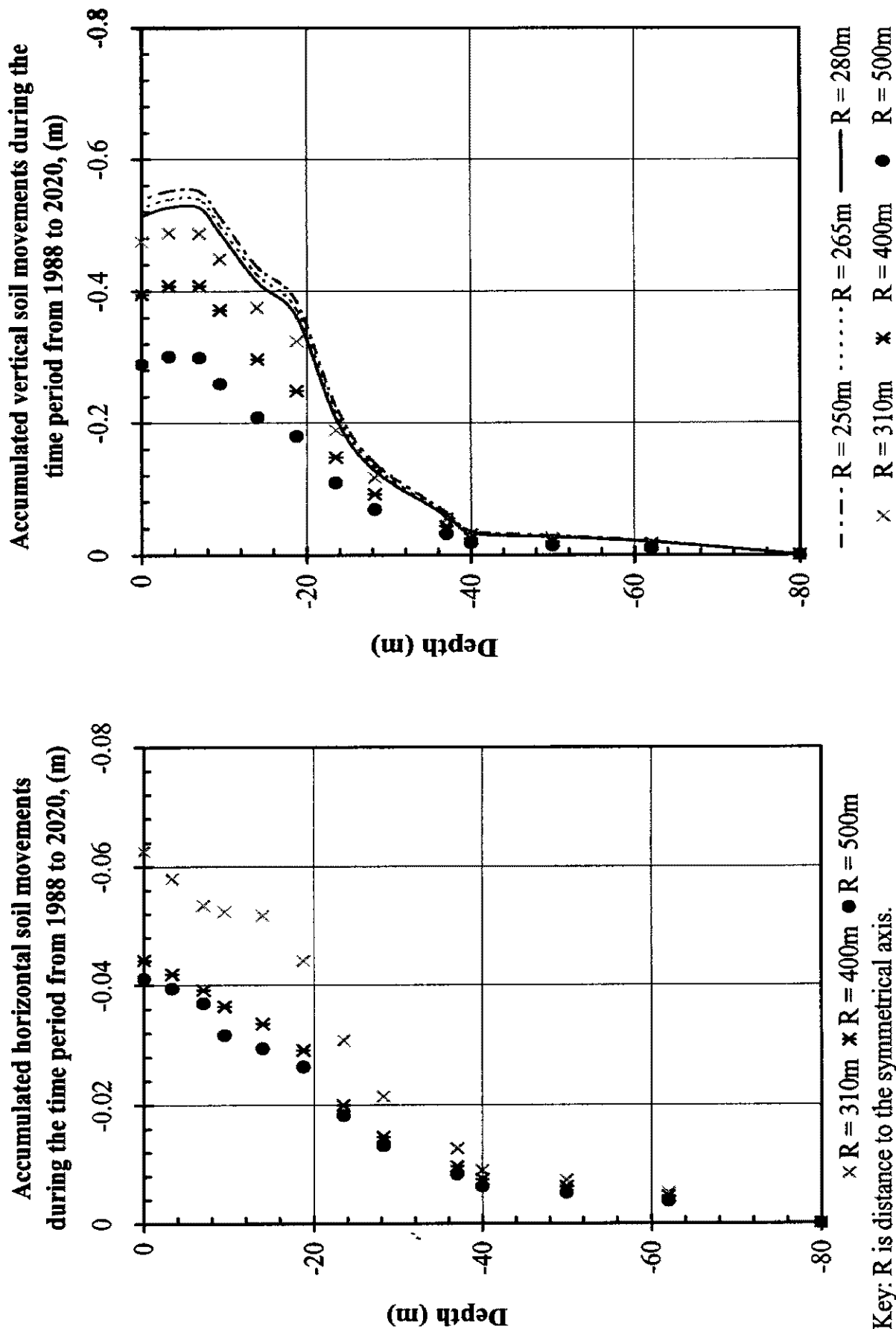


Figure 5.17: Accumulated soil movement profiles versus depth, predicted at Phap Van during the period from 1988 to 2020

based on the assumption that the ground water pumping rate is doubled over this time period.

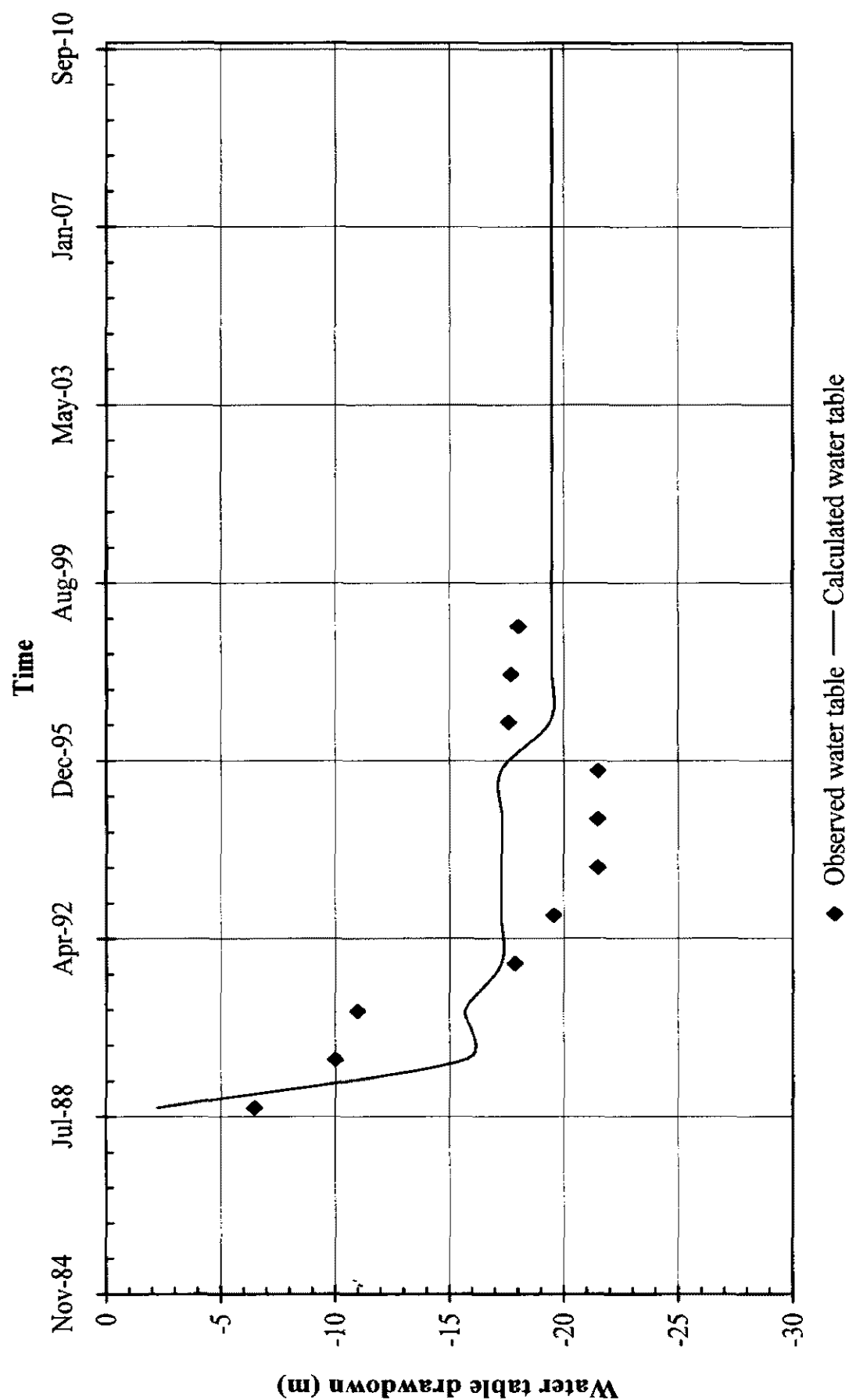


Figure 5.18: Ngoc Ha water table drawdown.

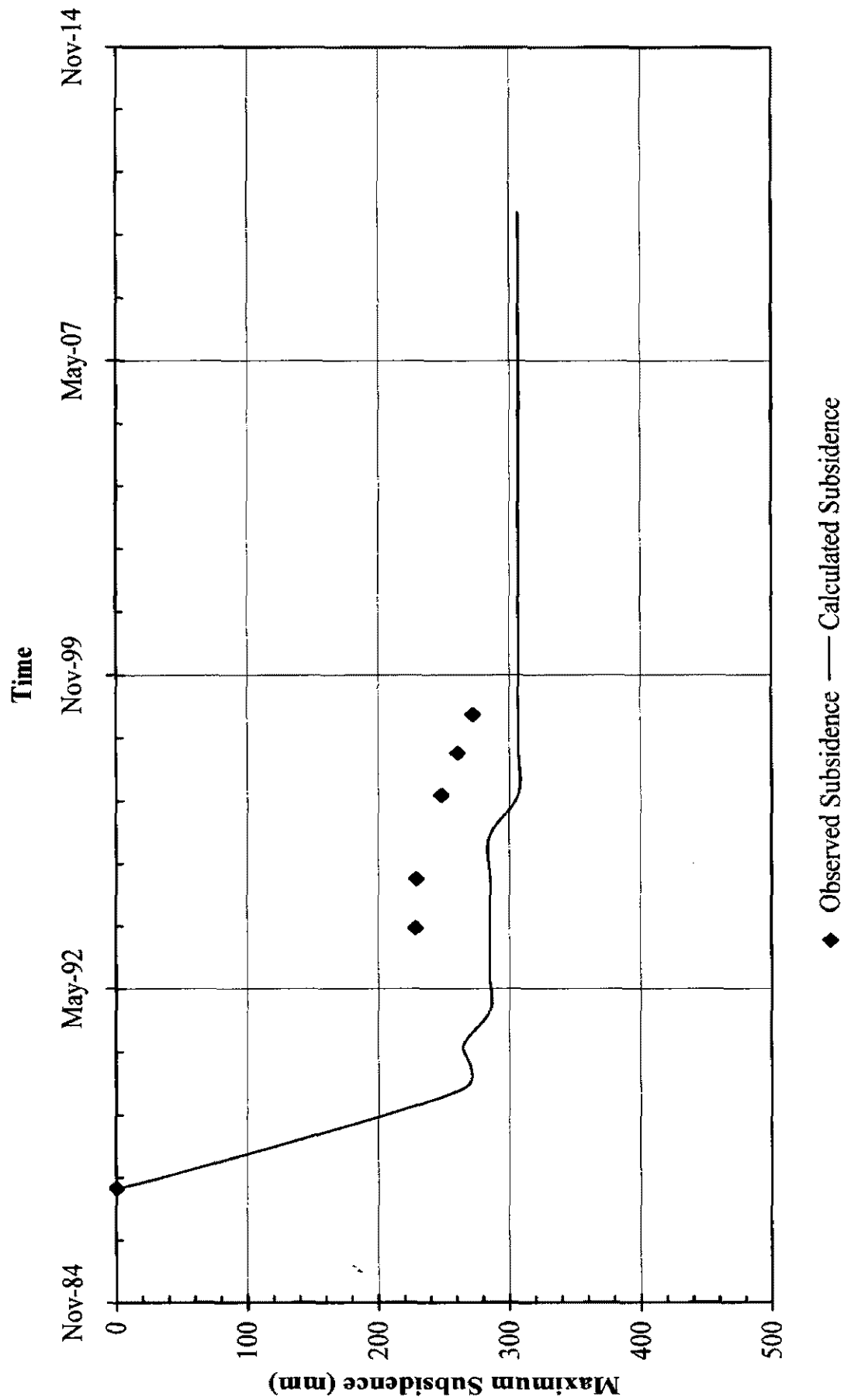


Figure 5.19: Ngoc Ha maximum vertical soil movements.

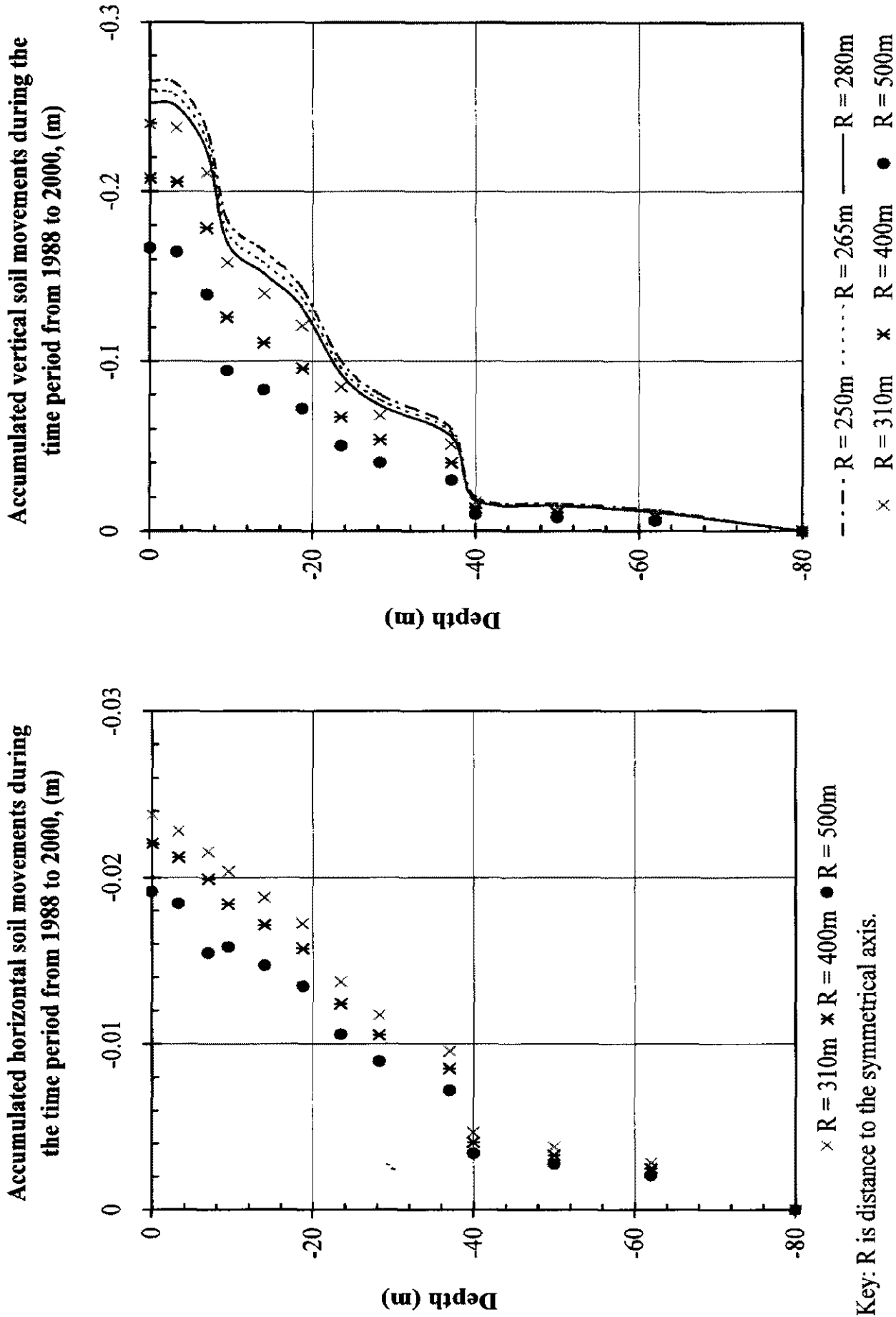


Figure 5.20: Accumulated soil movement profiles versus depth, predicted at Ngoc Ha during the years from 1988 to 2000 based on the assumption that the ground water pumping rate is kept constant over this period.

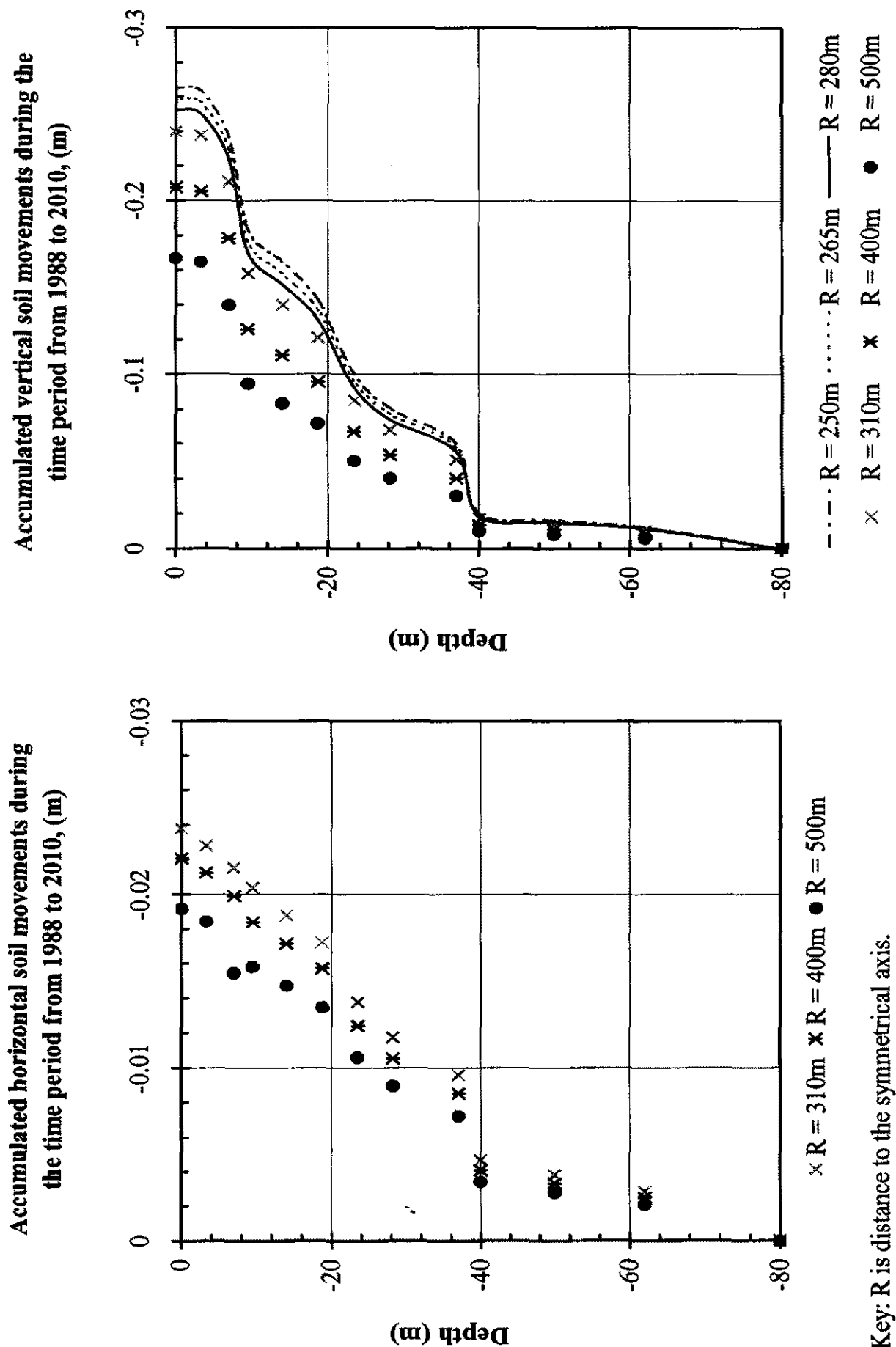


Figure 5.21: Accumulated soil movement profiles versus depth, predicted at Ngoc Ha during the years from 1988 to 2010 based on the assumption that the ground water pumping rate is kept constant over this period.

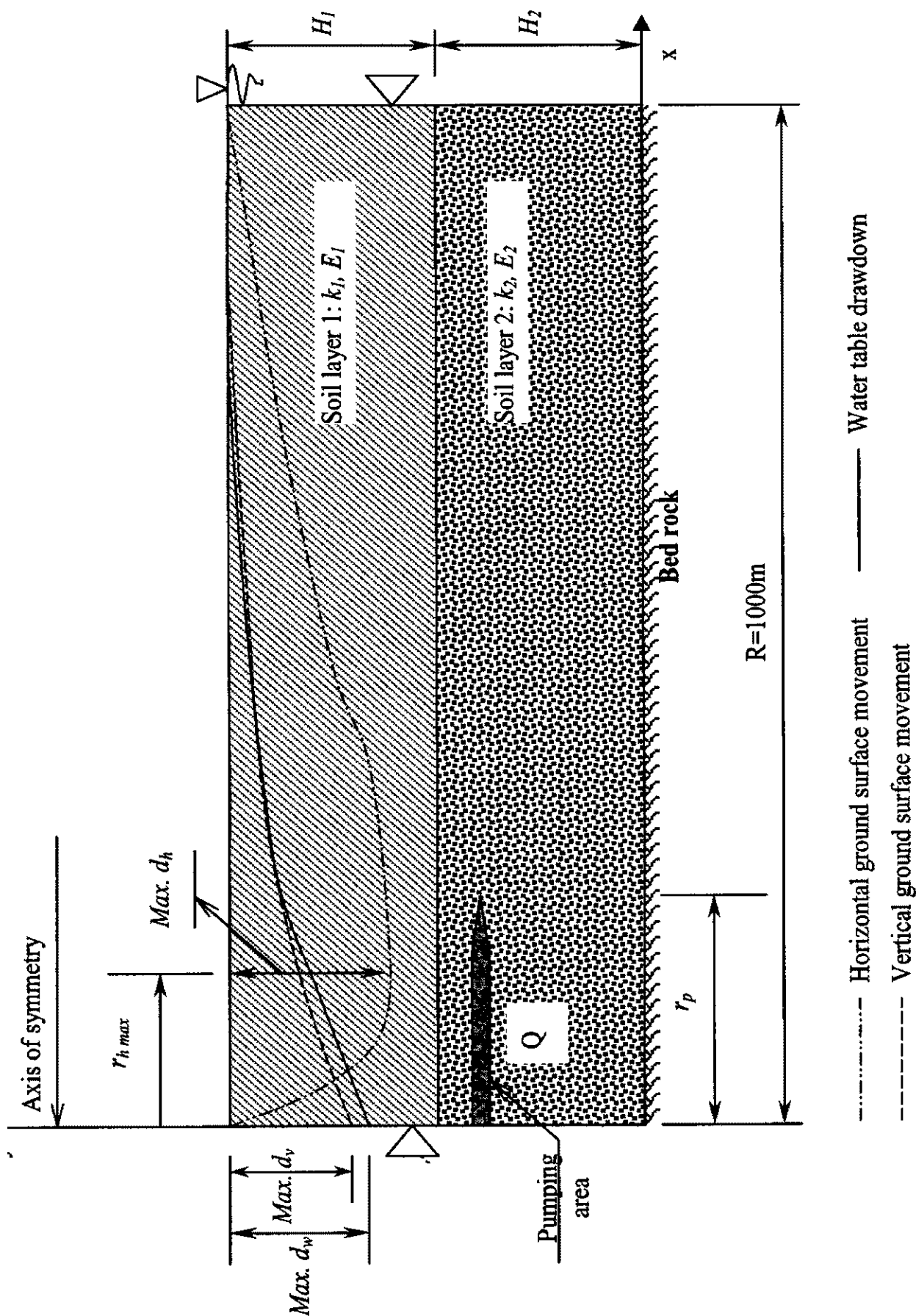


Figure 5.22: Notation used for parametric study of soil movement due to pumping ground water.

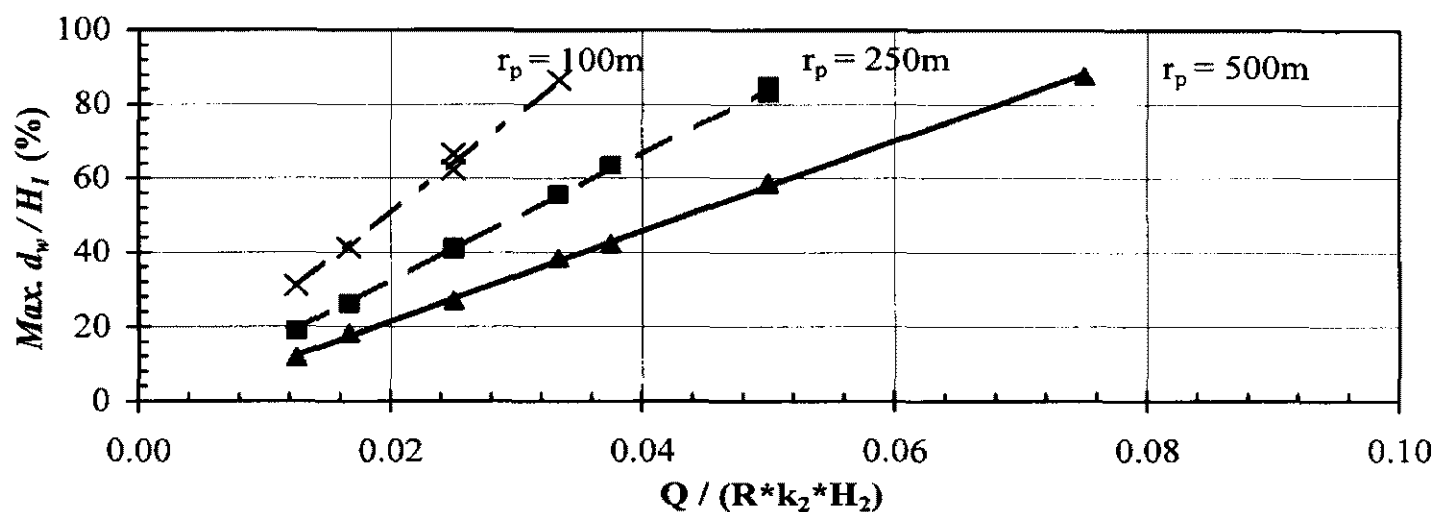


Figure 5.23: Maximum ground water drawdown over the thickness of the soft soil layer ($\text{Max. } d_w / H_1 (\%)$) with $H_1 = 20\text{m}$.

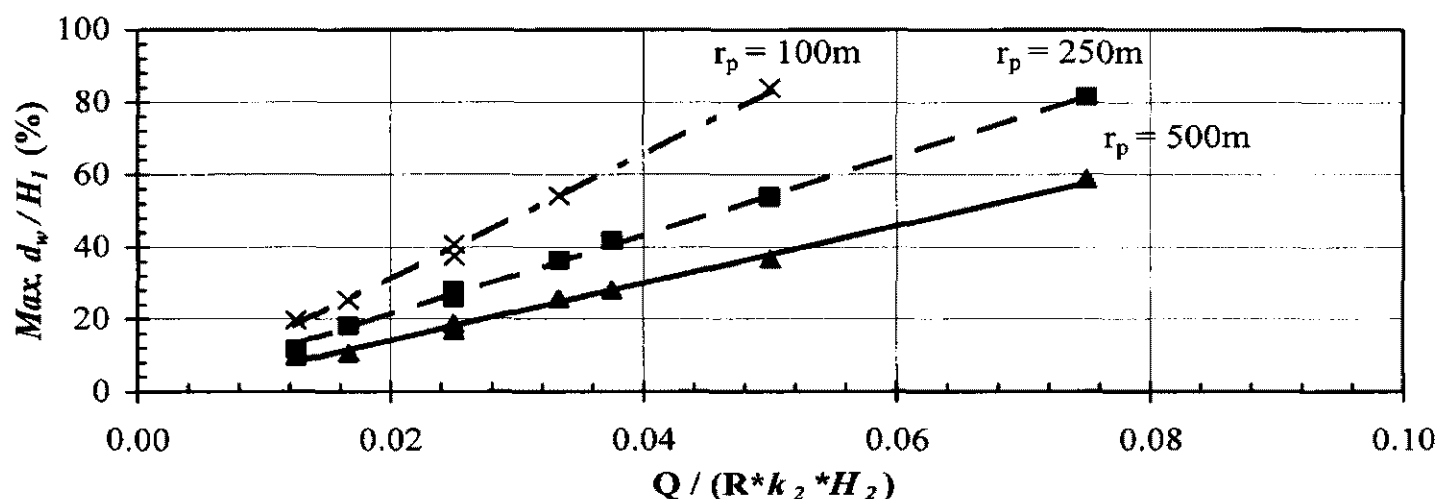


Figure 5.24: Maximum ground water drawdown over the thickness of the soft soil layer ($\text{Max. } d_w / H_1 (\%)$) with $H_1 = 30\text{m}$.

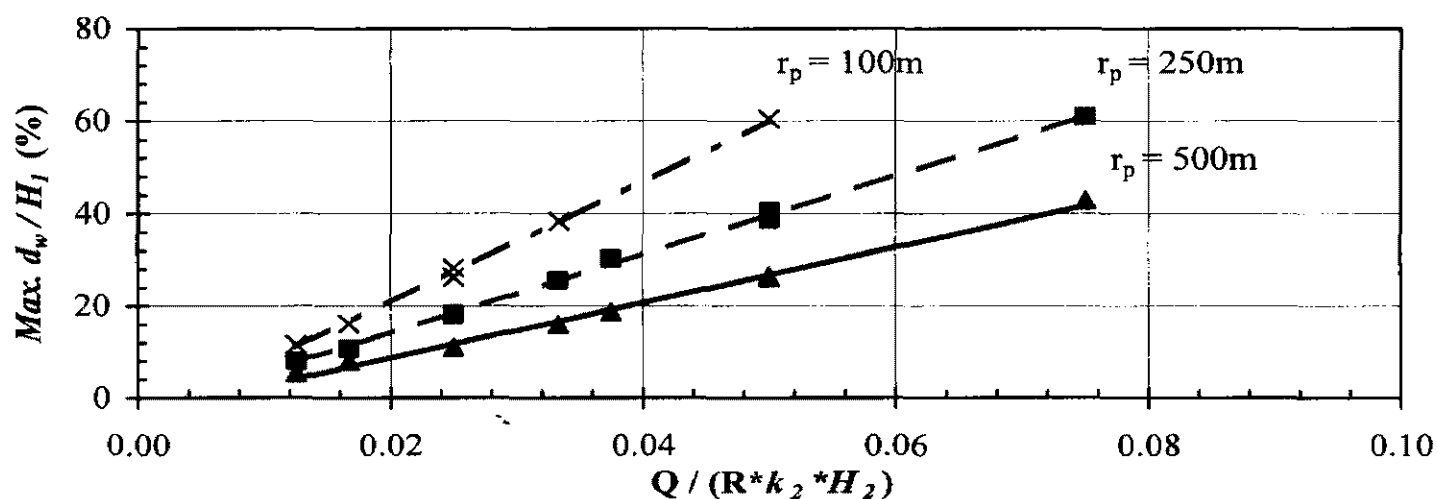


Figure 5.25: Maximum ground water drawdown over the thickness of the soft soil layer ($\text{Max. } d_w / H_1 (\%)$) with $H_1 = 40\text{m}$.

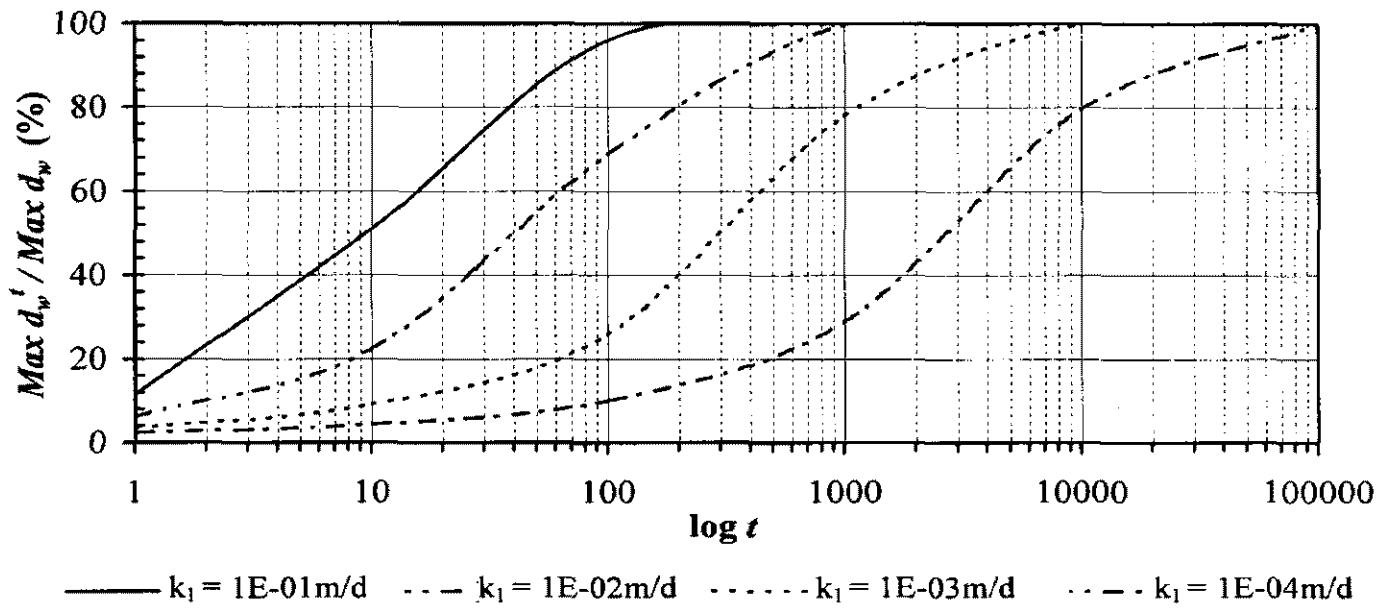


Figure 5.26: Maximum ground water table drawdown ratio versus time.

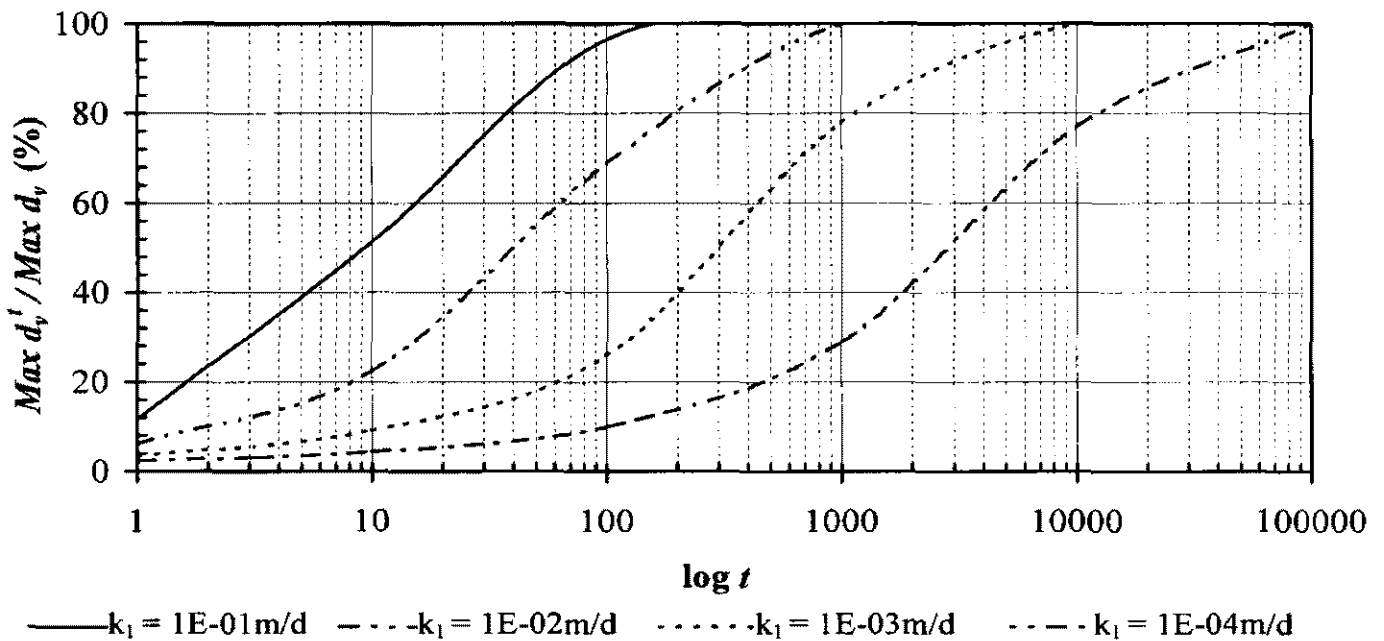


Figure 5.27: Maximum ground surface settlement ratio versus time.

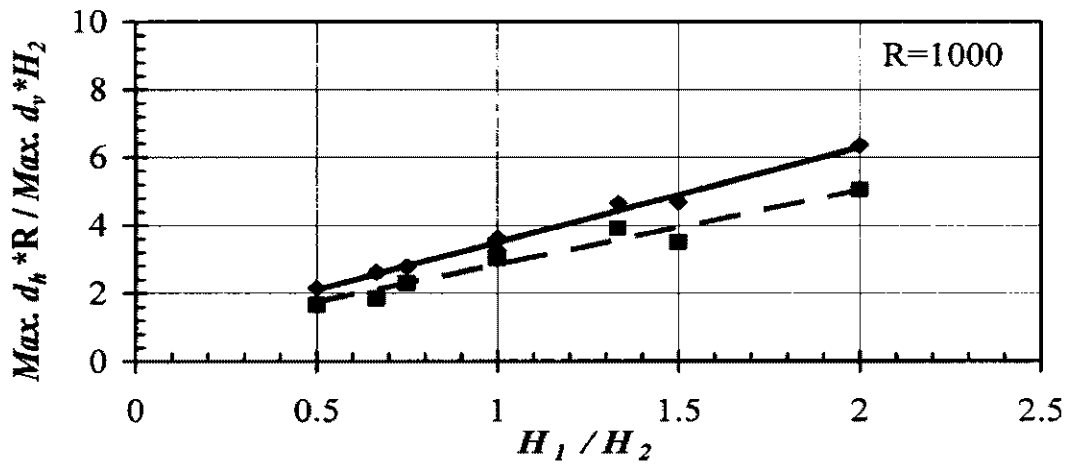


Figure 5.28: Ratio of maximum vertical surface movement over maximum horizontal surface movement with $r_p = 100\text{m}$, and $E_l = 2.5\text{MPa}$.

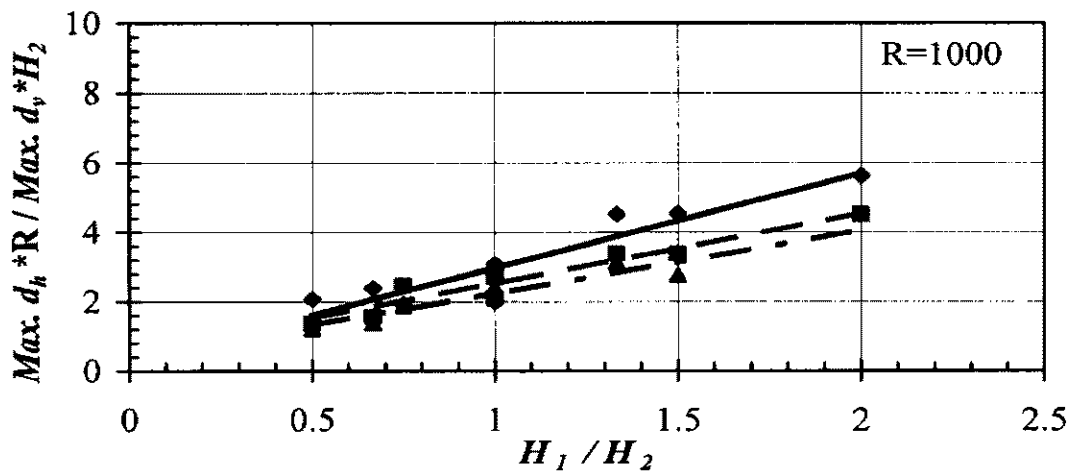


Figure 5.29: Ratio of maximum vertical surface movement over maximum horizontal surface movement with $r_p = 250\text{m}$, and $E_l = 2.5\text{MPa}$.

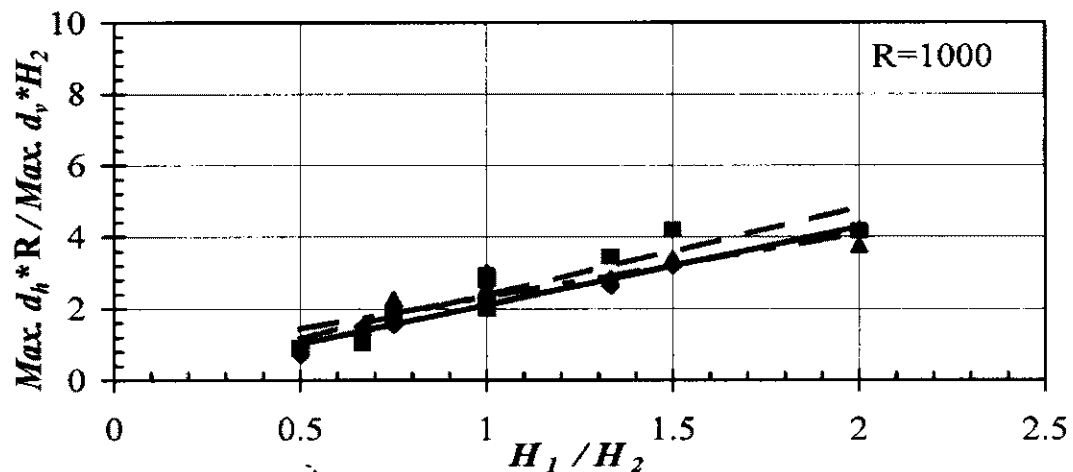
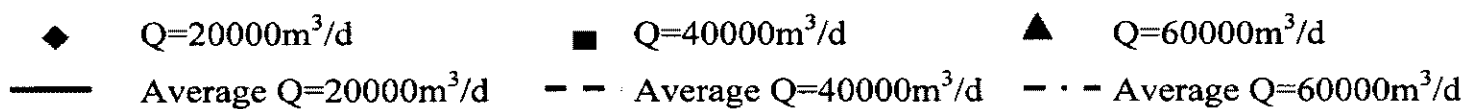


Figure 5.30: Ratio of maximum vertical surface movement over maximum horizontal surface movement with $r_p = 500\text{m}$, and $E_l = 2.5\text{MPa}$.



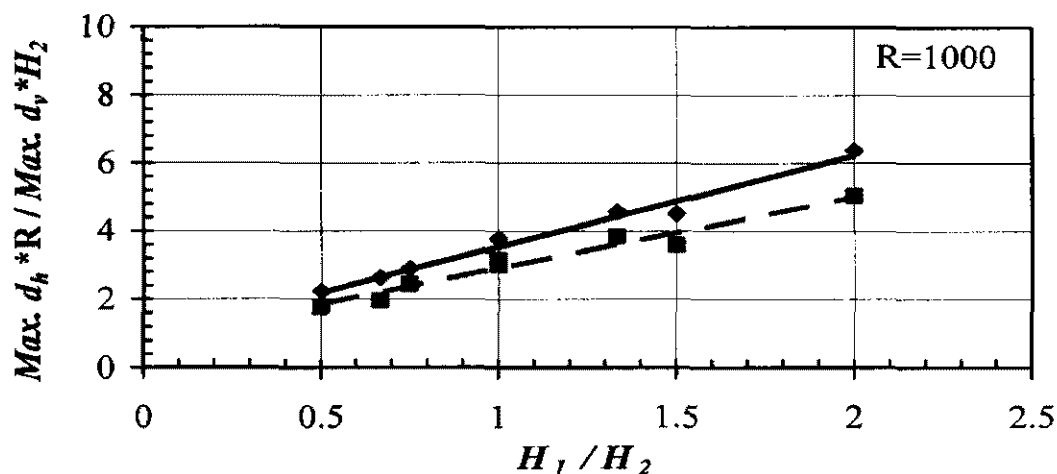


Figure 5.31: Relationship between the maximum vertical surface movement and the maximum horizontal surface movement with $r_p = 100\text{m}$, $E_l = 5.0\text{MPa}$.

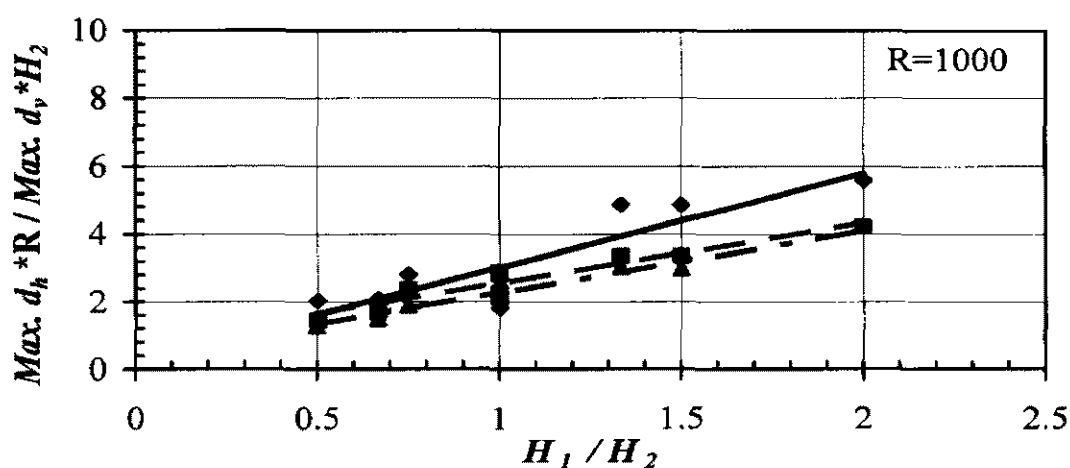


Figure 5.32: Relationship between the maximum vertical surface movement and the maximum horizontal surface movement with $r_p = 250\text{m}$, $E_l = 5.0\text{MPa}$.

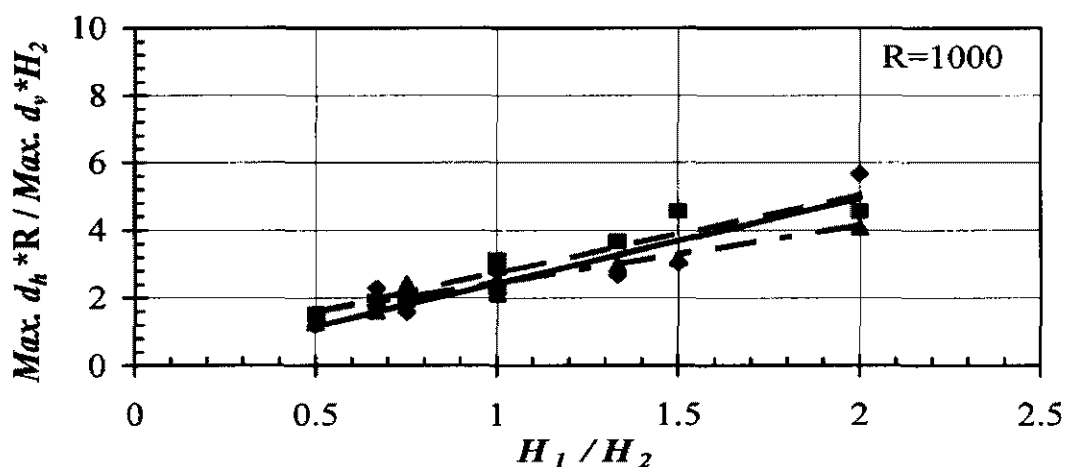
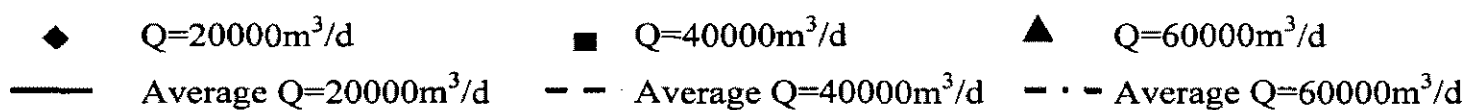


Figure 5.33: Relationship between the maximum vertical surface movement and the maximum horizontal surface movement with $r_p = 500\text{m}$, $E_l = 5.0\text{MPa}$.



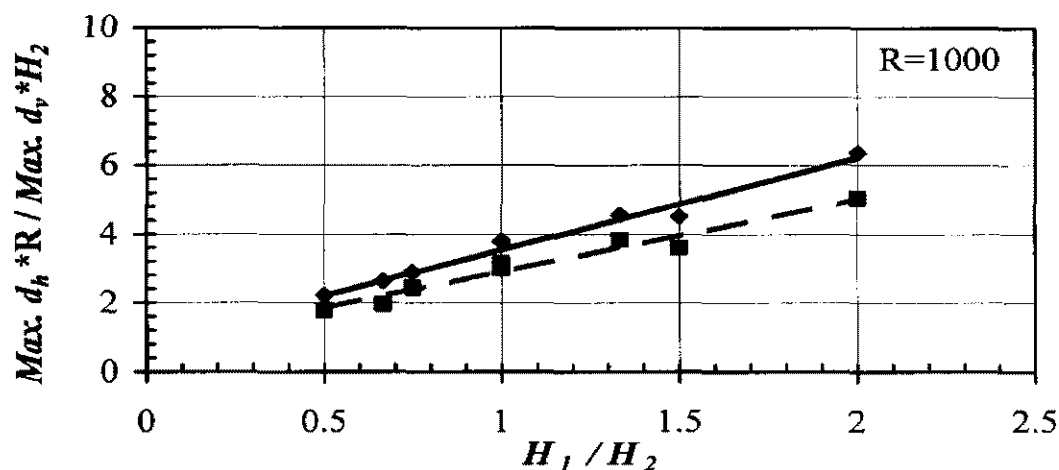


Figure 5.34: Relationship between the maximum vertical surface movement and the maximum horizontal surface movement with $r_p = 100\text{m}$, $E_l = 10.0\text{MPa}$.

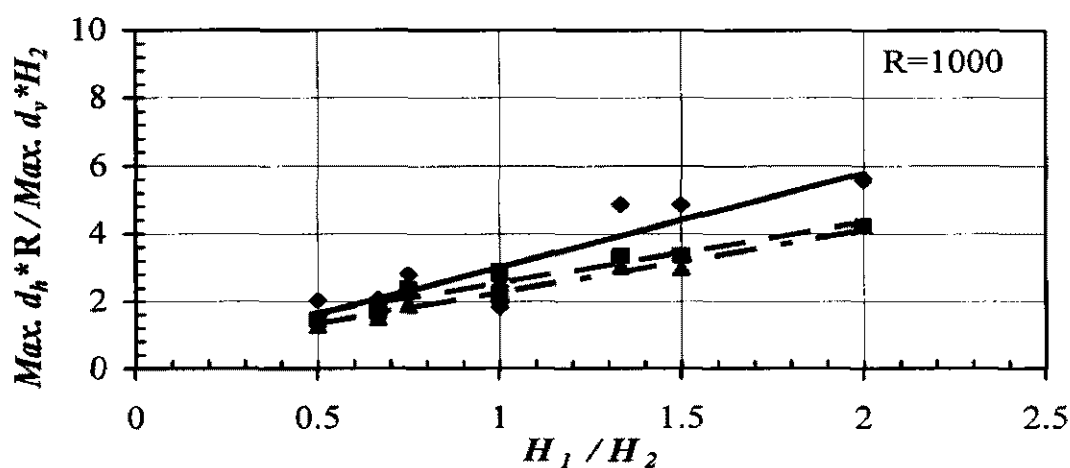


Figure 5.35: Relationship between the maximum vertical surface movement and the maximum horizontal surface movement with $r_p = 250\text{m}$, $E_l = 10.0\text{MPa}$.

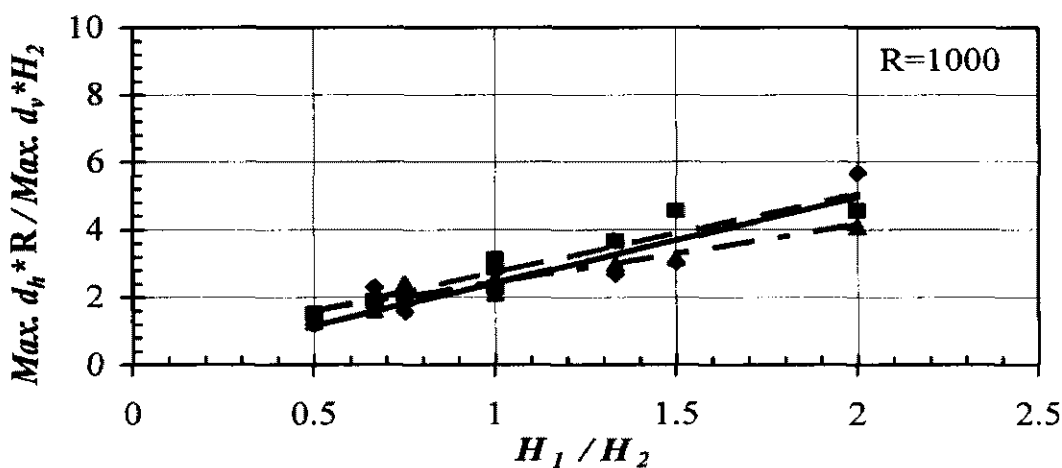
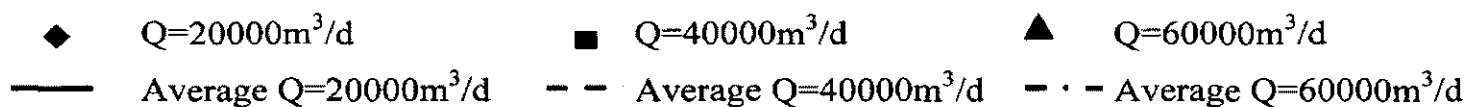


Figure 5.36: Relationship between the maximum vertical surface movement and the maximum horizontal surface movement with $r_p = 500\text{m}$, $E_l = 10.0\text{MPa}$.



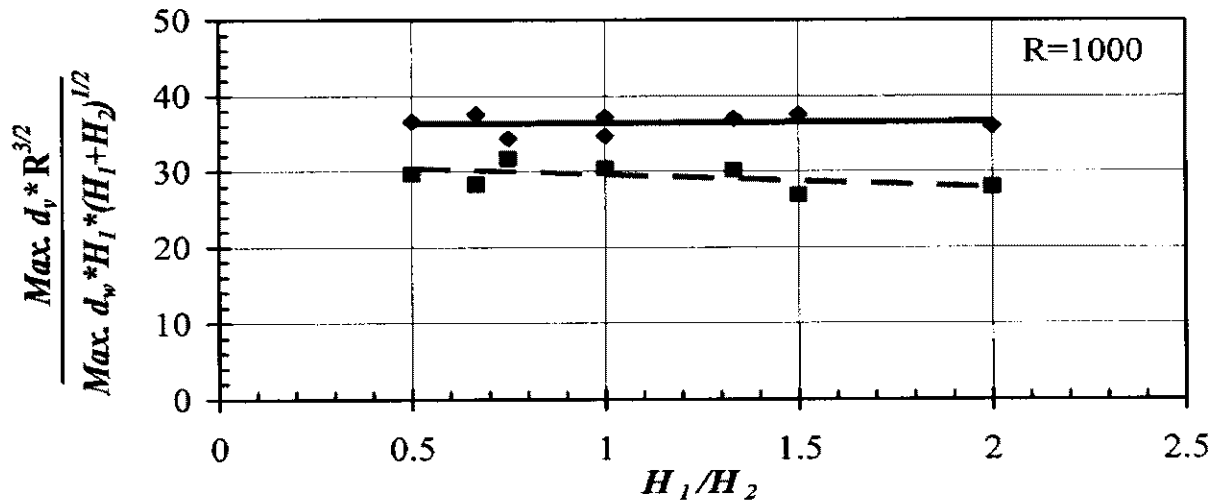


Figure 5.37: The relationship between maximum vertical movement and maximum water table drawdown with $r_p = 100\text{m}$, and $E_l = 2.5\text{MPa}$.

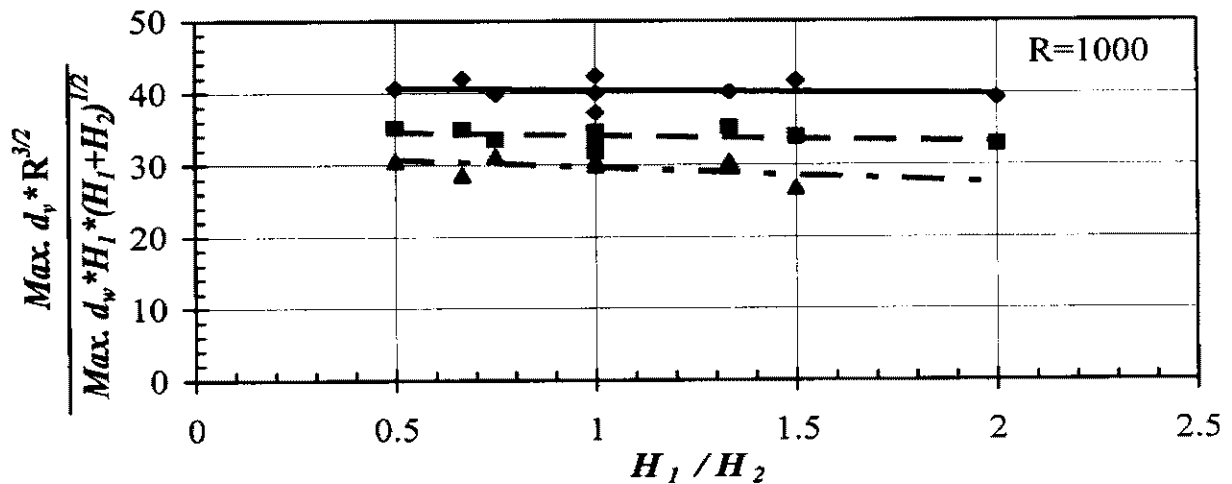


Figure 5.38: The relationship between Maximum vertical movement and maximum water table drawdown with $r_p = 250\text{m}$, and $E_l = 2.5\text{MPa}$.

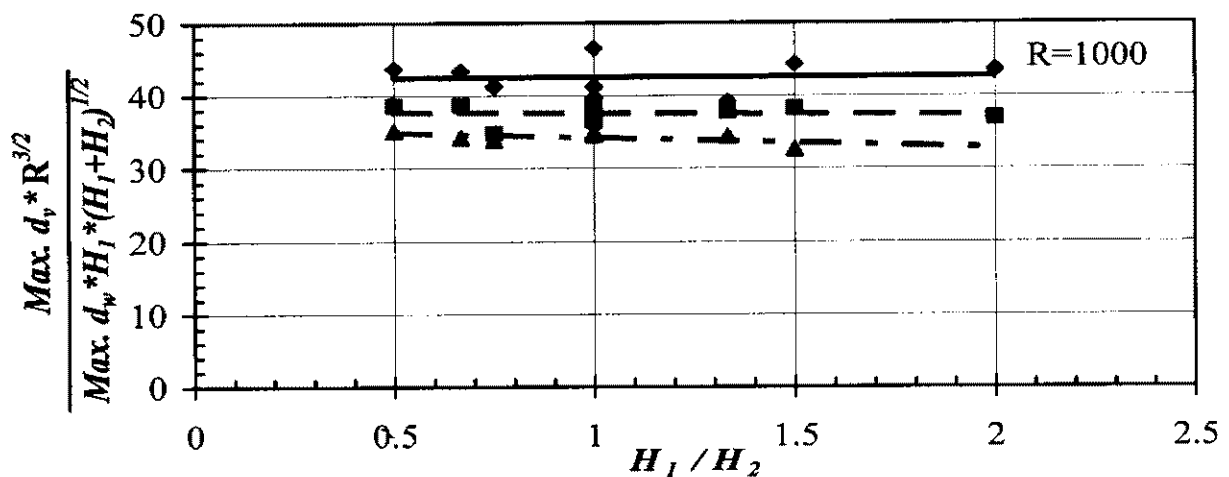
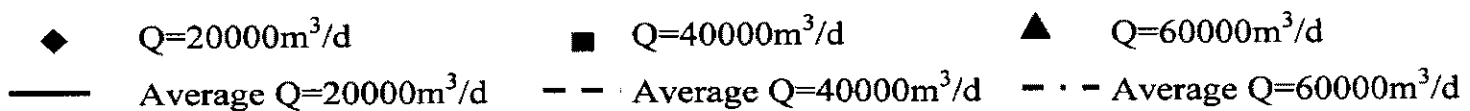


Figure 5.39: The relationship between Maximum vertical movement and maximum water table drawdown with $r_p = 500\text{m}$, and $E_l = 2.5\text{MPa}$.



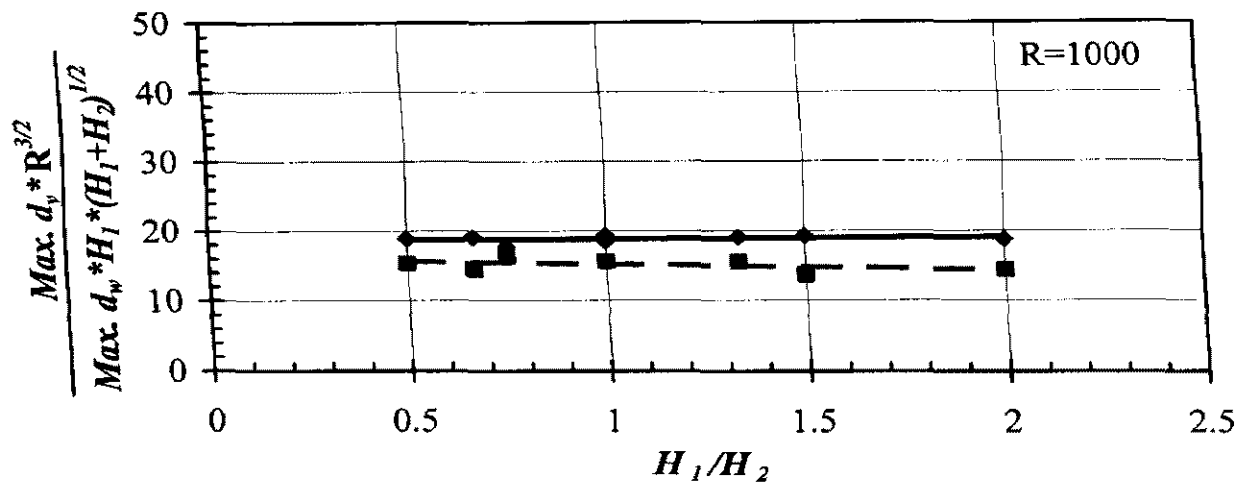


Figure 5.40: The relationship between the maximum vertical movement and the maximum water table drawdown with $r_p = 100\text{m}$, and $E_l = 5.0\text{MPa}$.

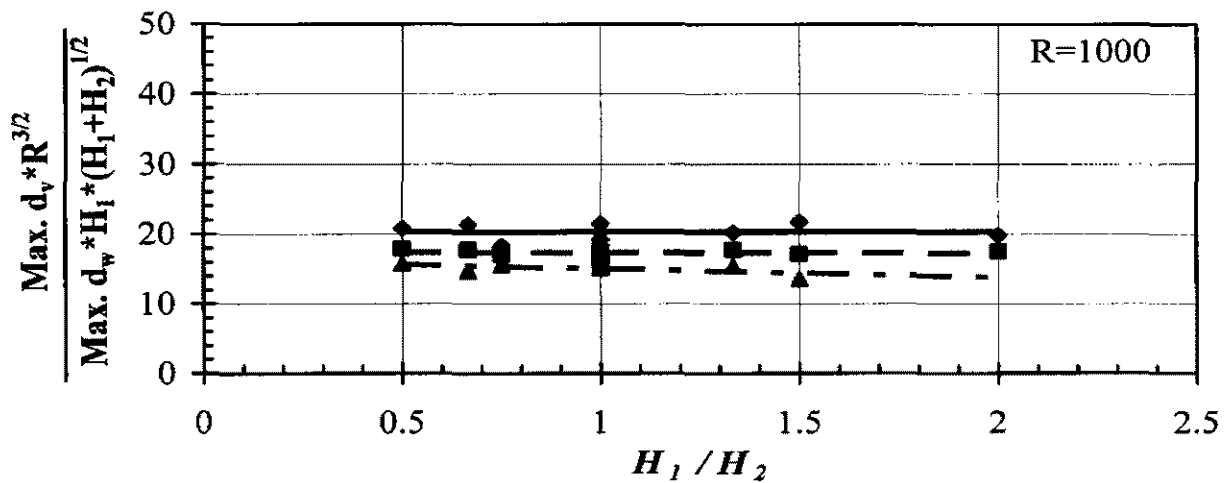


Figure 5.41: The relationship between the maximum vertical movement and the maximum water table drawdown with $r_p = 250\text{m}$, and $E_l = 5.0\text{MPa}$.

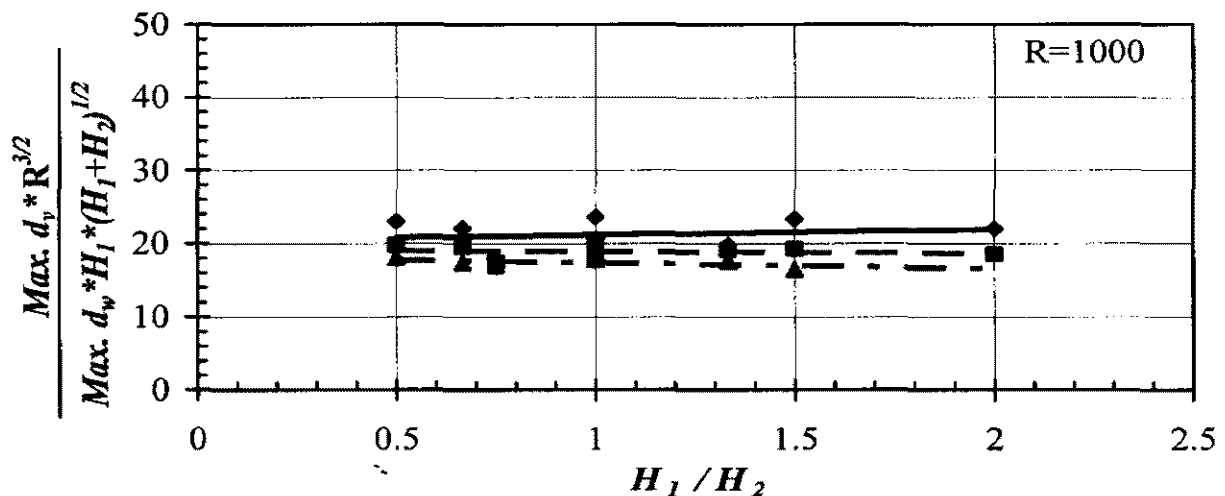
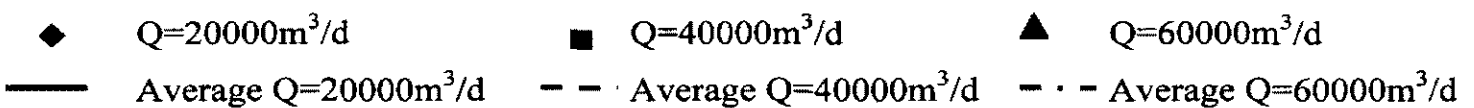


Figure 5.42: The relationship between the maximum vertical movement and the maximum water table drawdown with $r_p = 500\text{m}$, and $E_l = 5.0\text{MPa}$.



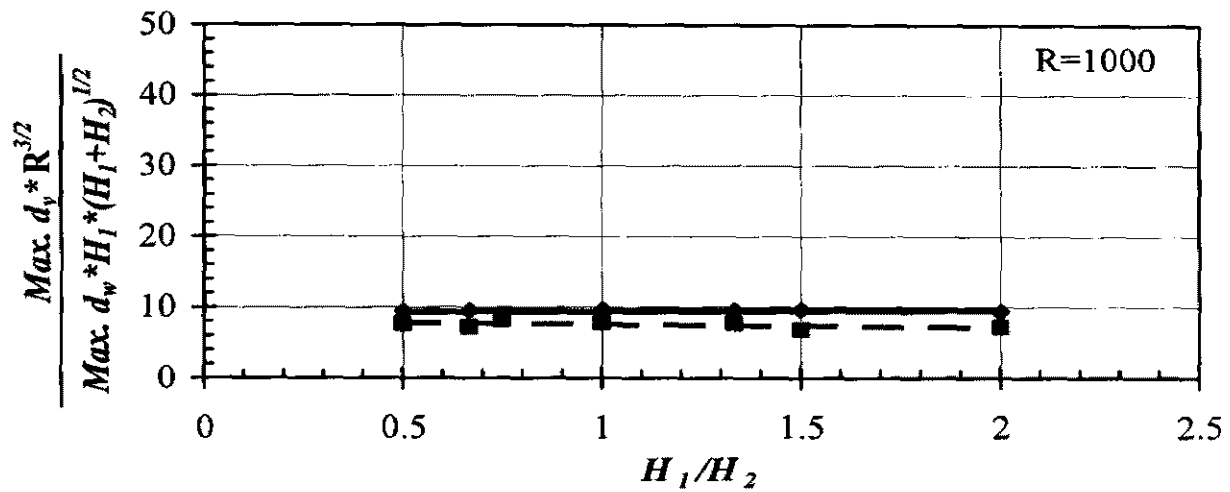


Figure 5.43: The relationship between the maximum vertical movement and the maximum water table drawdown with $r_p = 100\text{m}$, and $E_I = 10.0\text{MPa}$.

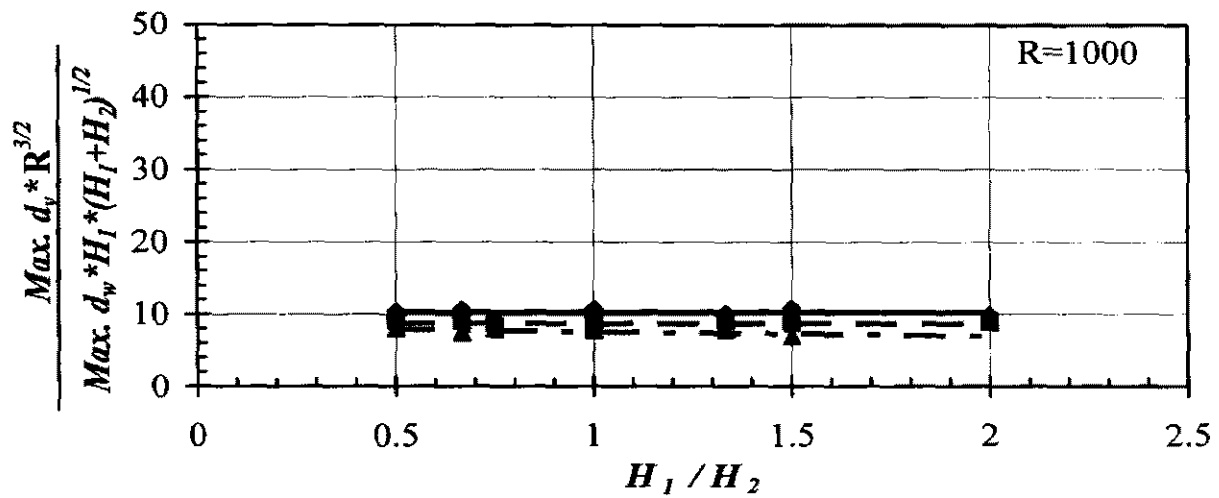


Figure 5.44: The relationship between the maximum vertical movement and the maximum water table drawdown with $r_p = 250\text{m}$, and $E_I = 10.0\text{MPa}$.

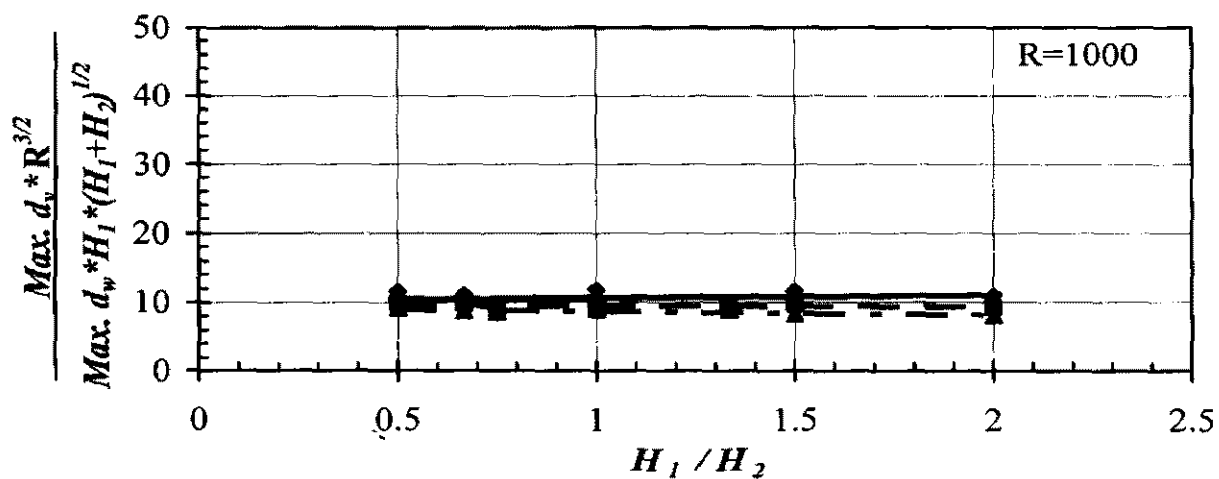


Figure 5.45: The relationship between the maximum vertical movement and the maximum water table drawdown with $r_p = 500\text{m}$, and $E_I = 10.0\text{MPa}$.

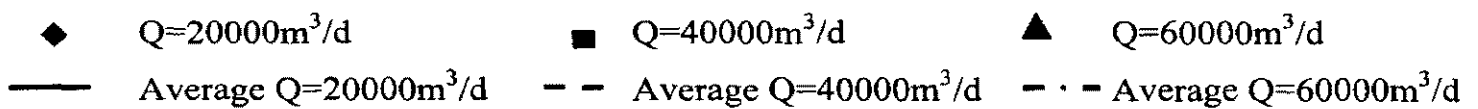


Figure 5 - 46: Maximum ground water table drawdown versus distance

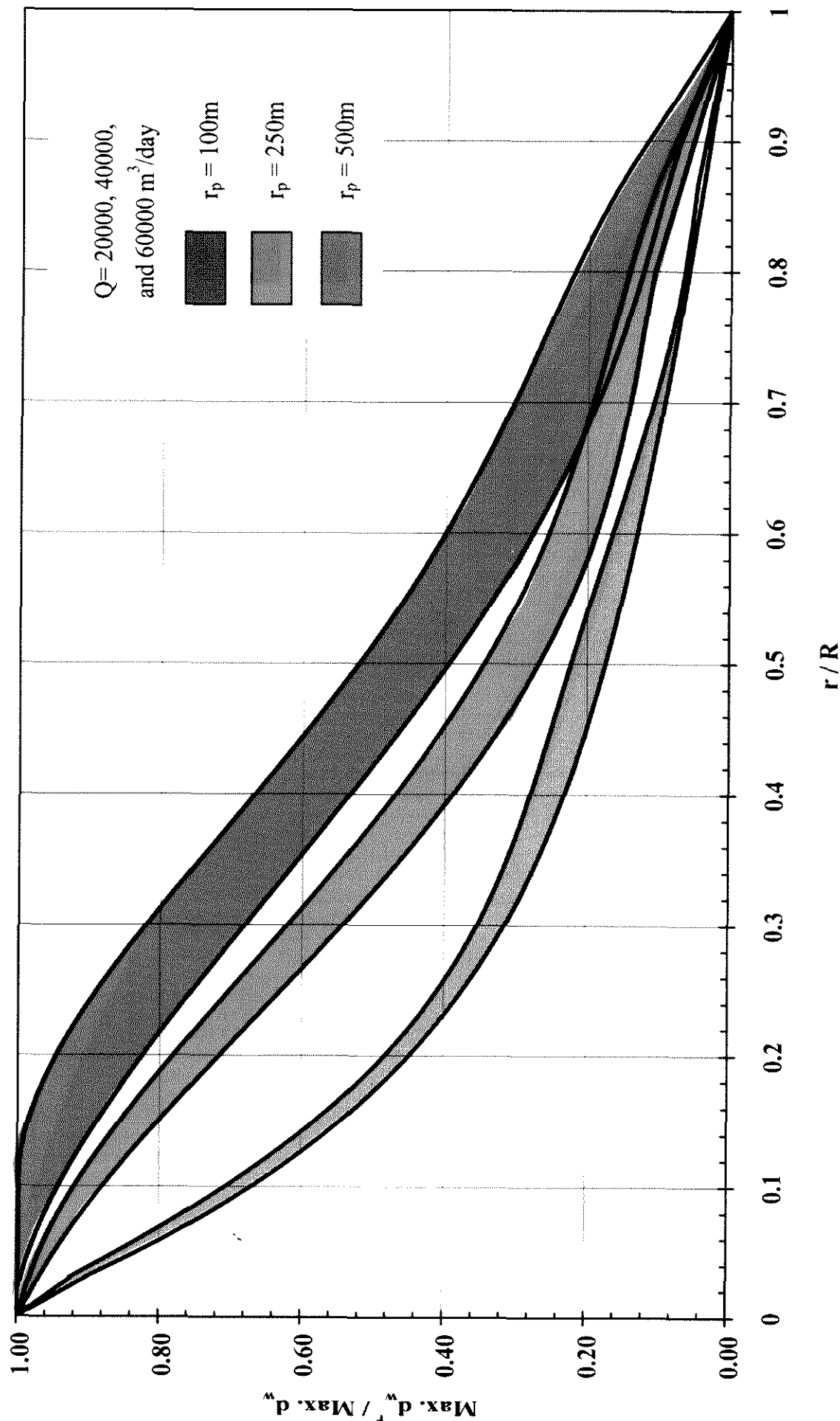
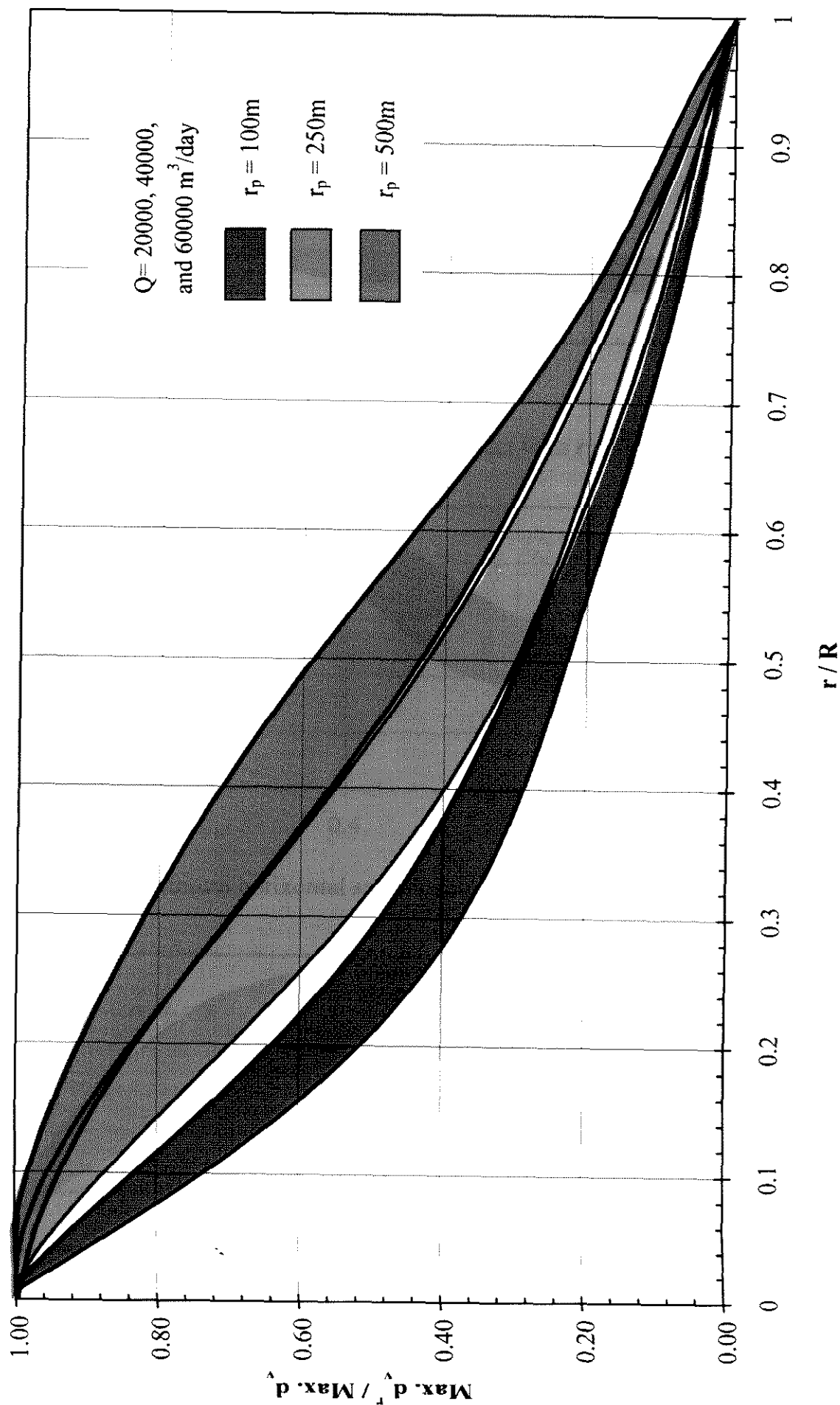


Figure 5 - 47: Maximum vertical soil movements versus distance.



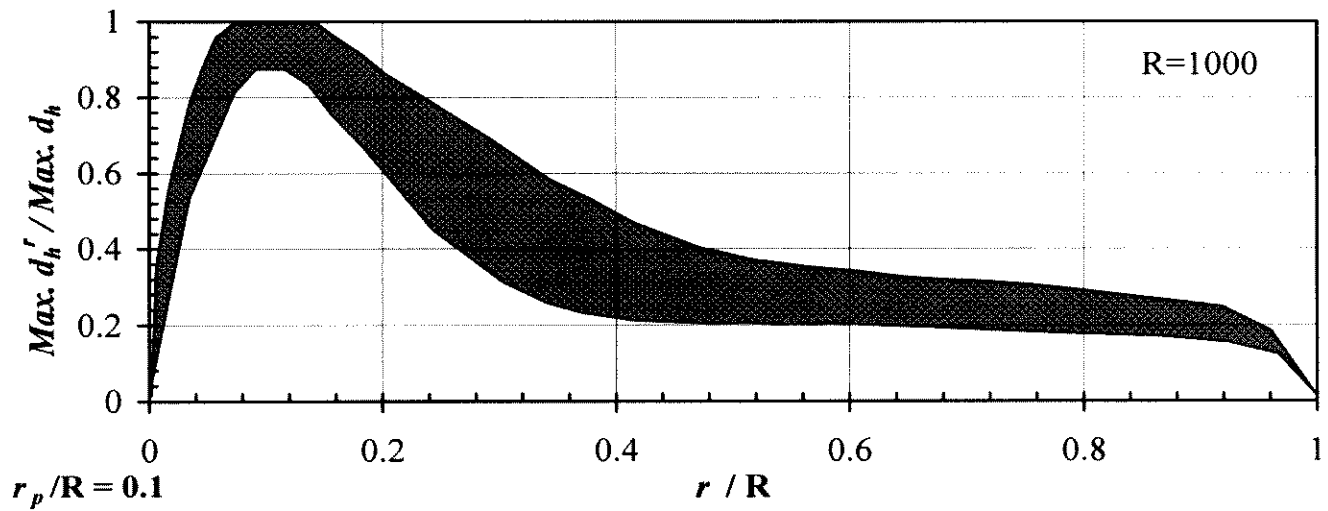


Figure 5.48: Range of maximum horizontal soil movement with $r_p = 100\text{m}$.

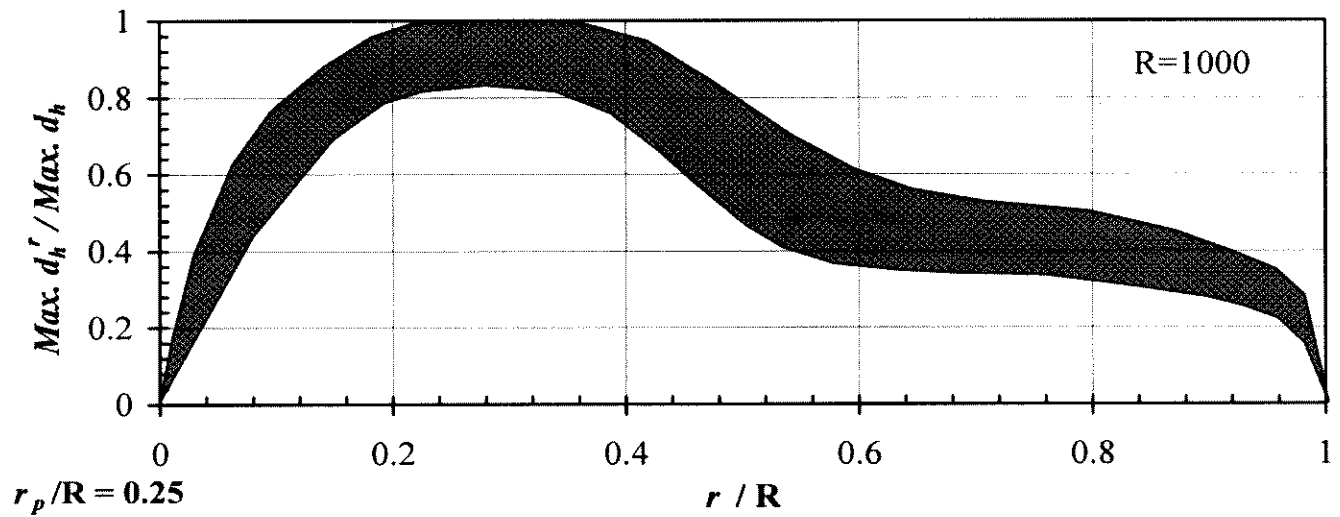


Figure 5.49: Range of maximum horizontal soil movement with $r_p = 250\text{m}$.

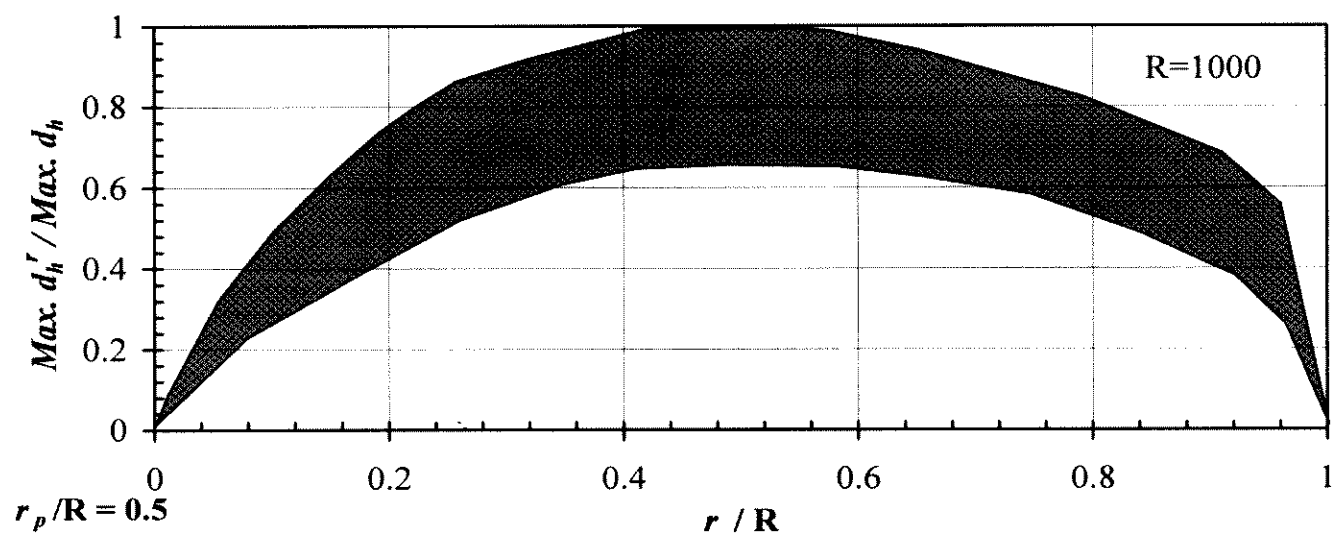


Figure 5.50: Range of maximum horizontal soil movement with $r_p = 500\text{m}$.

CHAPTER 6

NUMERICAL ANALYSIS OF PILED FOUNDATIONS

6.1 GENERAL INTRODUCTION

This chapter starts with a short critical review of pile behaviour due to induced soil movements caused by ground water pumping. Soil movements can be in the vertical and horizontal directions. Vertical soil movement can cause negative skin friction on the pile shaft and horizontal soil movement can induce bending moments in the pile. This critical review is subdivided into two parts: (1) pile response due to vertical soil movements, and (2) pile behaviour because of horizontal soil movements.

As can be seen from the previous chapter, extensive ground water pumping obviously causes vertical soil movements. Consequently negative skin friction on a pile can be inevitable when a pile foundation exists in the influence area of a ground water pumping station. From the finite element analysis, it can be seen that extensive ground water pumping can also cause horizontal soil movements. In some cases, the horizontal soil movements can be large enough to cause induced bending moments in a pile that exceed the allowable bending moments or cause failure of the pile.

In the small and crowded country of Vietnam, only one fourth of the country is covered by flat delta where most of the population live, and the rest is mountain and highland. One of the main cities, Hanoi, is in a flat delta area and accommodates up to 10% of the population (estimated at about 100 million in 2020). In Hanoi, therefore extensive water pumping stations have been established, and many of there are close to existing pile foundations. There are many cases where excessive ground water pumping has caused vertical soil movements over a large area, and caused significant horizontal soil movements in the area close to the pumping area, with evidence of damage to buildings and facilities on the ground surface. Also fissures that have formed due to pumping of ground water have been recorded.

The following part of this chapter deals with a hypothetical case where existing pile foundations are close to a ground water pumping station. Soil movement profiles are obtained from using the results of the previous chapter. A study will be made of the potential for damage of the foundation due to the induced ground movements.

6.2 CRITICAL REVIEW OF PILE BEHAVIOUR DUE TO SOIL MOVEMENTS

6.2.1 INTRODUCTION

Pile foundations have generally been used for supporting structures by transferring the applied load to deeper soil through the skin friction between the pile shaft and the surrounding soil and the pile base resistance.

Under some circumstances, the soil surrounding a pile may move and extra loads will be induced in the piles. As a consequence, over-stress or damage to a pile may occur. Some sources of ground movements are dewatering, installation of adjacent piles, consolidation, or expansive soil, construction of adjacent buildings, excavation, earthquake, or slope instability. The soil movements induced can be in the vertical or horizontal directions only or in both vertical and horizontal directions.

This chapter will consider both the vertical and horizontal movements of a soil. However, only downward soil movement will be considered, as upward soil movements are less likely to be of concern in ground water pumping situations.

6.2.2 PILE RESPONSE DUE TO VERTICAL SOIL MOVEMENTS

When the soil surrounding piles moves downward, it generates a down-drag force that is called negative skin friction on the pile shaft. Apart from the applied load that the pile supports it also has to carry an additional load due to the relative movement between the pile and the surrounding soil. As the axial load increases, the settlement of the pile will increase. Reduction of end bearing capacity may also be possible.

Analysis of pile foundations that takes into account the force of negative skin friction has been considered by several authors: for example, Broms (1977), Fellenius (1972), Bozozuk (1972), Johannessen and Bejerrum (1965), Meyerhof (1976), Sawaguchi (1971), Poulos and Davis (1980), Poulos and Mattes (1969), Kuwabara and Poulos (1989), and Van de Veen (1986).

Analysis of pile foundations including the effect of negative skin friction consists of determining the down-drag force and the pile tip settlement. However, according to Poulos (1990a), negative skin friction and the consequent downdrag force cannot lead

to plunging failure of the pile and settlement behaviour of a pile could be the key factor in design for negative skin friction. This is also agreed to by Van de Veen (1986).

For a simple analysis of downdrag forces, some assumptions have been made. These are that full slip occurs between the pile and soil along the whole length of the pile and that the maximum downdrag force occurs after completion of consolidation processes, for example under drained conditions.

Johannessen and Bjerrum (1965) suggested that when the relative movement of the surrounding soil and the pile exceeds 5 to 10mm, that negative skin friction c_a on a single pile can be calculated from the equation:

$$c_a = K \tan \varphi'_a \cdot \sigma'_v \quad (6.1)$$

where $K = 1 - \sin \varphi'_a$ and σ'_v is vertical effective stress; φ'_a is the interface friction angle which is about 0.75 times the effective friction angle of the soil.

The Canadian Building Code (National Research Council, Canada, 1975) suggested a value of 0.25 for $K \tan \varphi'_a$. Meyerhof (1976) also gave a simple equation for calculating the negative skin friction:

$$c_a = \beta \sigma'_v \quad (6.2)$$

where β is the skin friction factor which is between 0.2 and 0.3 for normally consolidated clays and it gradually reduces as the depth of penetration of a pile increases. σ'_v is vertical effective stress;

The rate of surface settlement also influences the negative skin friction. Based on field observations and measurements, Fellenius (1972) proposed that negative skin friction could be caused by extremely small settlement of the surrounding soil. In this case, the force in the 40m long pile was increasing by 15 ton/year when the rate of surface settlement was correspondingly increased 2-3mm/year. Broms (1977) recommended values of $K \tan \varphi'_a$ for computing the down-drag force when the rate of settlement is about 10mm/year, and values are shown in table 6.1 for different soil types.

Table 6.1: Values of $K \tan \phi'_a$ for different soil types, Broms (1977).

Material type	$K \tan \phi'_a$
Rock fill	0.4
Sand and gravel	0.35
Silt to normally consolidated clay ($I_p < 50$)	0.3
Normally consolidated clay ($I_p > 50$)	0.2

The negative skin friction relates mainly to the horizontal effective stress as pointed out by Bozozuk (1972). The equation is:

$$F_s = M_p \cdot K_o \cdot \sigma'_v \cdot \tan \phi' \quad (6.3)$$

where K_o is coefficient relating horizontal to vertical effective stress; σ'_v is vertical effective stress; ϕ' is effective friction angle of the soil; M_p is friction factor for the soil acting on the pile surface $0 \leq M \leq 1$, F_s is the unit skin friction.

For pile groups, Broms (1977) suggested an empirical method for calculating the negative skin friction on the pile group. If the spacing of the piles in the pile group is large enough, each pile in the group will carry the maximum skin friction as for a single pile. The same equation 6.1 may be used to calculate the down drag force. As the spacing is small, the pile group will carry a total load, which corresponds to the shear resistance of the soil on the perimeter of the group.

To estimate the settlement of piles, taking into consideration negative skin friction, a simple procedure can be applied. First, elastic compressions of a pile due to applied load on the pile head and due to down drag forces are calculated by employing simple column compression equations. Then the pile tip movement can be estimated by applying a simple elastic equation for a rigid circular footing resting on the bearing stratum with assumptions that the pile tip is rigid and the bearing stratum is deep.

More accurate analyses have also been developed by several authors, including Walker and Darwall (1970) who used the finite element method. Sawaguchi (1971) developed a simple analytical method where equations are given to calculate the depth of the inflection point, maximum compressive stress in the pile, compressive stress at

the pile tip, and settlement of the surrounding soil. In this analysis an elasto-plastic relationship between shear stress and displacements was used.

Poulos and Davis (1980) used elastic theory and employed a boundary element method, which is similar to the methods used to analyze pile settlement, to investigate negative skin friction. The analysis takes into account the partial pile-soil slip effects, the rate of development of down-drag force, effects of the bearing stratum, group effects, battered piles, and the delay of pile installation. Partial pile-soil slip is also allowed for rather than the assumption of full-slip between soil and pile. For short piles, full slip can occur for soil movement of only a few percent of pile diameter while a soil movement of up to 50% of pile diameter is required for long piles. The rate of development of downdrag force is generally similar to the rate of settlement of the soil layer, as pointed out by Poulos and Davis (1980).

Kuwabara and Poulos (1989) used the same method to investigate downdrag forces in pile groups. It was found that group effects led to a reduction in downdrag force in a pile as compared to a single isolated pile, and that inner piles in a group develop less downdrag force than the outer piles. Floating piles and end-bearing piles also have different behaviour under negative skin friction. For floating piles, there exists a neutral point below which positive skin friction is developed. Therefore the downdrag force is smaller than in end-bearing piles but the pile settlement is larger. In a design of pile foundations that takes into account the negative skin friction, some suggestions for the allowable load on the piles, P_a , are as follows.

Tomlinson (1975)

$$P_a = \frac{P_u}{F} - P_{s1} \quad (6.4)$$

CPI (1989)

$$P_a = \frac{P_u - P_{s1}}{F} - P_{s1} \quad (6.5)$$

Van der Veen (1986)

The lesser of

$$P_a = \frac{P_b + P_{s2}}{F_1} - \mu(P_{s1} + P_{s2}) \quad (6.6)$$

or

$$P_a = \frac{P_b + P_{s2}}{F_2} \quad (6.7)$$

where P_a is the applied axial load to pile, P_b is the maximum resting force due to end bearing in the stable layer, P_{s1} is the maximum downdrag force due to negative friction in the settling layers, P_{s2} is the maximum resting force due to positive friction in the stable layer, F is the overall safety factor, generally between 2 and 3, P_u is the ultimate axial load capacity of the pile, the value of F_1 is equal to 1.7, μ is equal to 0.7, and F_2 is equal to 2.5.

Negative skin friction on a pile can be reduced by using several methods such as surface coating with bitumen or bentonite or using electro-osmosis or a dummy casing outside the pile to protect the load bearing inner pile. However, reducing the negative skin friction means reducing the shaft resistance of piles and so this method is only applicable to end bearing piles.

6.2.3 PILE RESPONSE DUE TO HORIZONTAL SOIL MOVEMENTS

In general, piles undergoing horizontal soil movements are subjected to additional horizontal movements and bending moments. The structural integrity and serviceability are aspects that need to be calculated and designed for. Pile behaviour due to horizontal soil movements has been investigated by several researchers. Some methods have been developed, including empirical and numerical methods. Empirical methods are normally based on field observations and measurements. The collected data is then analyzed to give a general trend. Marche and Lacroix (1972) presented a series of collected data for measured pile movements near an embankment. Their results for relative displacement as a function of relative flexibility of a pile are shown in Figure 6.1. The authors concluded that horizontal movement of a pile is dependent on the relative flexibility of the pile K_c , and expressed it as shown in the following equation:

$$K_c = \frac{c_u L_p^4}{E_p I_p} \quad (6.8)$$

where c_u is the undrained shear strength of soil, L_p is the pile length, E_p and I_p are the Young's modulus and second moment of area of the pile respectively.

As horizontal pile deflection depends much on the flexibility of the pile itself then the more flexible the pile, the larger the horizontal movements.

Numerical analysis has also been employed as an effective tool to solve the problem of lateral pile behaviour when a pile is subjected to horizontal soil movement. Normally numerical analysis utilizes finite element methods such as in Carter (1982), Broms *et al.* (1987), Springman (1989), and Stewart *et al.* (1993). Stewart *et al.* (1993) used a finite element method to model a pile group subjected to lateral soil movements. In the analysis, the author modelled the pile group as rows of equivalent sheet-pile walls. A three-dimensional finite element analysis was used to model the problems by Springman (1989). In this analysis the soil is considered as elastic, however, non-linear analysis can be considered in this three-dimensional finite element analysis.

Boundary element methods have also been employed for calculating lateral behaviour of piles (Poulos and Davis, 1980). Actually this method is an extension of laterally loaded pile analysis in which horizontal soil movement applies stresses at the interface between pile and soil ("free-field" movements of soil). The interaction stresses between pile and soil can be modelled using an elastic continuum approach (Poulos and Davis, 1980) or the modulus of subgrade reaction method (Broms *et al.*, 1987). In the elastic continuum approach, the soil is modelled as an elastic mass and yielding of the soil is also taken into account by specifying a limiting pile-soil pressure of suitable value at various points along the pile. In the subgrade reaction method, the soil is modelled as series of independent springs that can be linear or non-linear. However interaction between piles is not considered as at a point on the soil-pile interface, deflection only depends on stress acting at the point. Each approach has its own advantages and disadvantages. The subgrade reaction approach is simple and can model soil layers but cannot be used for a pile group. On the contrary, elastic-based analysis can be used to consider pile groups, or the effect of cuts or slopes near a pile but it can only approximately consider layered soil profiles. According to Poulos and Davis (1980), pile behaviour is largely governed by the relative flexibility of pile to soil, K_p , which is defined as follows:

$$K_p = \frac{E_p I_p}{E_s L_p^4} \quad (6.9)$$

For a flexible pile, for which the relative flexibility is less than 10^{-5} the pile deforms with the soil and the bending moment in the pile can be approximately estimated from the curvature of soil movements. For stiffer piles, deflection decreases but bending moment increases significantly. This is also true when the pile head is restrained from rotating. The pile movement is decreased but bending moment is increased.

For non-linear behaviour of a pile, there are two main models of failure that need to be considered, to be precise, ultimate lateral resistance of piles and ultimate soil resistance. For ultimate lateral resistance of piles, there are two failure modes, short pile failure and long pile failure, which are failure of the supporting soil and yielding of the pile section respectively. For ultimate soil resistance, the theory presented by Broms *et al.* (1987) is widely applied. The analysis includes two typical cases, a pile installed in a cohesive soil (clay) and cohesionless soils (e.g. sand).

6.3 PILE ANALYSIS PROGRAMS

6.3.1 INTRODUCTION

Throughout this thesis, three boundary element programs have been adopted to analyze pile behaviour due to external soil movements (PALLAS, PIES and GEPAN), which can be used for analysis of vertical and horizontal soil movements respectively. This part introduces the three programs and discusses their ability to analyze pile behaviour. PALLAS and PIES are two separate programs for different types of soil movement, and therefore the analysis of pile behaviour, which is conducted by employing these two programs, is also divided into two parts, one for vertical soil movement and one for horizontal soil movement. GEPAN is a general program that can analyse both vertical and horizontal soil movements. However the current version of GEPAN does not allow analysis of the non-linear response of the interface between pile and soil. Hence, it will give very conservative analysis results.

6.3.2 AXIAL PILE BEHAVIOUR DUE TO VERTICAL SOIL MOVEMENT ANALYSIS. PROGRAM - PIES

The program PIES has been developed at the University of Sydney (Poulos, 1990b), and the User's Manual for the program can be found at the Centre for Geotechnical Research, the University of Sydney. The program computes the axial movement and load distribution within a single pile, or a pile within a group, subjected to axial load and/ or externally imposed soil movements. The soil can be represented either by a continuum or a series of springs. Non-linear response of the interface can be incorporated by assuming an elastic-plastic or hyperbolic relationship between soil stiffness and stress level; and allowing for the effects of pile-soil slip and end bearing failure by specifying limiting values of shaft resistance and end-bearing resistance.

The pile is assumed to be circular and behaves as a uni-axially loaded elastic column with a closed end. In the analyzed problem, the pile and soil are divided into three types of elements; cylindrical shaft elements, annular base elements, and annular discontinuity elements.

The response of the pile and soil elements to an increment of axial load, ΔP , are analyzed and the expression for the incremental vertical displacement of the pile elements is given below:

$$\{\Delta \rho_p\} = \Delta \rho_p \{1\} + [AD][FE]\{\Delta p\} \quad (6.10)$$

where $\{\Delta \rho_p\}$ is vector of incremental pile displacements

$\Delta \rho_p$ is incremental displacement of the pile tip

$\{1\}$ is a vector whose elements are unity

$[AD]$ is the summation matrix.

$[FE]$ is the pile compression matrix

$\{\Delta p\}$ is a vector of incremental pile-soil interaction stresses.

At any element i , the soil will be in one of three stages: the non-failure (NF) stage; the failure in compression stage (FC); or the failure in tension stage (FT).

In the non failure (NF) stage

$$f_{ti} \leq \Delta p_i + p_i \leq f_{ci} \quad (6.11)$$

where Δp_i is incremental pile-soil stress on element i ; p_i is existing pile-soil interaction stress on element i prior to a load increment; f_{ti} is limiting pile-soil interaction stress in tension at element i ; and f_{ci} is limiting pile-soil interaction stress in compression, at element i ;

In the compression failure (FC) stage:

$$\Delta p_i + p_{ti} \geq f_{ci} \quad (6.12)$$

where p_{ti} is total pile-soil stress at the previous load increment.

In the tension failure (FT) stage:

$$\Delta p_i + p_i \leq f_{ti} \quad (6.13)$$

where f_{ti} is limiting pile-soil stress at element i for tensile loading.

The analysis can be applied to both load-transfer analysis (t-z methods) and continuum analysis with the only difference occurring in the soil behaviour matrix [IF].

For modelling of soil behaviour, the program allows analysing pile-soil slip or non-linear response (in an approximate manner) by assuming that the soil Young's modulus varies with either stress or strain level, and allows post peak strain softening.

6.3.3 LATERAL PILE BEHAVIOUR DUE TO HORIZONTAL SOIL MOVEMENT ANALYSIS. PROGRAM - PALLAS

Program PALLAS was developed by Hull (1987). The User's Manual for the program can be found at the Centre for Geotechnical Research, the University of Sydney. In the PALLAS program, the pile is treated as a beam and it is discretized along its length so as to have two half elements, one at the head and one at the base, with full height elements for the remainder of the pile. Nodal points are chosen at the centre height of the full-length elements and at the top and bottom of the head and base elements respectively.

The governing equation to calculate pile deflection, u_p is given below:

$$\bar{u}_p = [K_p + K_s]^{-1} \{ \bar{W}_p + \bar{W}_s + [K_s] \Delta u \} \quad (6.14)$$

where $[K_p]$ is a pile stiffness matrix; $[K_s]$ is soil stiffness matrix; Δu is non-linear pile-soil interface behaviour; $\bar{W}_p$ is distributed load associated with the pile; and $\bar{W}_s$ is distributed load associated with the soil.

Based on finite difference approximations of the boundary conditions and the fourth order differential of beam bending, a pile stiffness matrix $[K_p]$ is defined by the following equation:

$$[K_p] \bar{u}_p = \bar{W}_p - \bar{W}_u \quad (6.15)$$

where $\bar{W}_u$ is the distributed load due to interaction between pile and soil.

The linear response of the soil is defined in terms of influence coefficient I_s . This influence coefficient relates the soil deflection, u_s , to the distributed load associated with the soil, $\bar{W}_s$, and the distributed load due to interaction between pile and soil, $\bar{W}_u$.

$$\bar{u}_s = [I_s] \{ \bar{W}_s + \bar{W}_u \} \quad (6.16)$$

If there are lateral soil movements u_{sm} at points in the soil, the relationship between the distributed loads associated with the soil and the soil movements is defined as

$$\bar{W}_s = [I_s]^{-1} \bar{u}_{sm} \quad (6.17)$$

Non-linear response of the soil causes a mismatch between the pile and soil deflection, Δu , and it can be defined as follows:

$$\Delta u = \bar{u}_p - \bar{u}_s \quad (6.18)$$

The cases of both hardening and softening pile-soil interface response can be modelled by using an incremental approach. In this approach the non-linear interaction load $\bar{W}_u$ is assumed to be a linear function of deflection mismatch.

6.3.4 GENERAL PILE BEHAVIOUR ANALYSIS PROGRAM-GE PAN

Program GEPAN was developed at the University of Sydney (Xu, 2000). In the GEPAN program, three-dimensional boundary element analysis of a pile is used to analyse single piles and pile groups that undergo general three-dimensional loading conditions or influence of the external soil movements. The difference from the PALLAS program, where only simplified of pile-to-pile interactions have been employed, the GEPAN program considers the pile element to pile element interactions to obtain more accurate load-deformation pile responses. Each three-dimensional pile element in GEPAN is subjected to a total of six components of load, hence, the deformation coupling effects due to three-dimensional loading is considered in the analyses of pile load-deformation behaviour. The governing equation for pile response is shown as follows:

$$\begin{bmatrix} A & B & \emptyset & D \\ \emptyset & \emptyset & G & H \\ O & P & Q & R \\ S & \emptyset & \emptyset & V \end{bmatrix} \begin{bmatrix} X_e \\ X_b \\ X_c \\ X_t \end{bmatrix} = \begin{bmatrix} Y_q \\ Y_c \\ \emptyset \\ Y_p \end{bmatrix} \quad (6.19)$$

where $[A]$ is the influence factor matrix relating element displacement and stress, $[B]$ is the influence factor matrix relating element displacement and pile tip displacement, $[D]$ is the influence factor matrix relating element displacement and pile head load, $[G]$ is the influence factor matrix relating cap displacement and cap-tie-cap beam force (to allow for pile caps joined by a tie beam), $[H]$ is the influence factor matrix relating cap load and pile head load, $[O]$ is the influence factor matrix relating pile head displacement and element stress, $[P]$ is the interaction factor matrix relating pile head displacement and pile tip displacement, $[Q]$ is the interaction factor matrix relating pile head displacement and pile cap displacement, $[R]$ is the interaction factor matrix relating pile head displacement and pile head load, $[S]$ is the interaction factor matrix relating pile head load and element stress, $[V]$ is the interaction factor matrix relating pile head load and pile head load, $\{X_e\}$, $\{X_b\}$, $\{X_c\}$, $\{X_t\}$ are vectors of pile soil stress, pile tip displacement, pile cap displacement, and capped pile head load respectively, $\{Y_q\}$, $\{Y_c\}$, $\{Y_p\}$ are vectors of element stress, cap load, and pile head load respectively.

Details of the governing equation as well as the influence factor matrixes are presented in Xu (2000). The effect of external soil movements are considered by either the known external soil movements that are imposed directly on the piles or the induced stresses due to the known external soil movements. In the governing equation (6.19) the vector of soil displacements is embedded into the vector of element stress $\{Y_q\}$ as follows:

$$\{Y_q\} = \{Y_{q,i}\} + \{Y_{q,e}\} \quad (6.20)$$

where $\{Y_{q,i}\}$ is vector of soil displacements imposed on element 'i' due to pile head load on individual piles, and $\{Y_{q,e}\}$ is soil displacement vector due to external sources.

Vector $\{Y_{q,e}\}$ is the directly imposed external soil movements at the pile soil interface in the case where the external soil movements are known. In the case where only the stresses due to the external soil movements are known then vector $\{Y_{q,e}\}$ is defined as follows:

$$\{Y_{q,e}\} = [SIF]\{\sigma_{soil}\} \quad (6.21)$$

where $[SIF]$ is the influence factor matrix relating element soil displacement and stress that is embedded inside matrix $[A]$, $\{\sigma_{soil}\}$ is the vector of induced soil stresses due to external soil movements.

The general three-dimension boundary element analysis program can model actual non-uniform stress distribution around the pile that is very useful in the case of piles undergoing lateral loading. The capability to analyse the coupling effect of general three-dimensional loading on piles enables the program to model load-deformation behaviour of piles more accurately. Though the program overcomes some of the limitations and simplifications that exist in other pile analysis programs, the current version of GEPAN only allows users to analyse purely elastic pile behaviour. Under some circumstances such as large external soil movement imposed directly on the pile-soil interface, a purely elastic solution may give analysis results that are too conservative. Hence, in this thesis, the program GEPAN is used in for a limited number of cases while PALLAS and PIES are used for most of the analyses.

6.4 SINGLE PILE ANALYSIS

6.4.1 INTRODUCTION

Intensive ground water pumping generally causes both vertical and horizontal soil movements. The vertical soil movement generates shear stresses along the pile shaft, which is called “negative skin friction”. The horizontal soil movement induces bending moment and lateral deflection into a pile.

For pile foundations near the areas of intensive ground water pumping, the soil movements may be large enough to influence the stability of the foundation. This part of the chapter deals with a single pile assumed to exist close to the ground water pumping area.

6.4.2 SINGLE PILE ANALYSIS

Figure 6.2(a) illustrates a general case where a single pile has been installed near a water station. The typical ground water pumping, which induces either the vertical and horizontal soil movements, is the Phap Van case study. In this study, which has already been presented in the previous chapter, the pumping area is about 500m to 600m in diameter, or 250m to 300m in radius from the assumed symmetrical axis. The area adjacent to this pumping area is strongly influenced by the ground water extraction. From the soil movement profiles (in Figures 5.14, 5.15, 5.16, 5.17, and 5.20, 5.21), the area in which a single pile is expected to be most influenced is within 300m away from the edge of the pumping area or about 300m to 600m away from the assumed symmetrical axis of the Phap Van water pumping station.

The soil movement profiles, soil properties, and pile geometry are as set out below:

- The end-bearing pile characteristics are as follows: the pile length was chosen to be 45m, and the bottom 5m of its length is installed into the hard layer of gravel, pebbles and coarse sand. The pile diameter is 1m and the pile Young's modulus is 30000 MPa. The floating pile is 30m long, 1m in diameter, and assumed to be installed into the medium sandy clay.

• The soil movement profiles for Phap Van are extracted and presented in Figures 6.3 to 6.5 as the source of the external soil movement input data for pile analyses. Three pile positions have been considered, which are located at 310m, 400m and 500m away from the axis of symmetry. Figures 6.3 to 6.5 show the calculated vertical and horizontal soil movements, which are then used as the soil movement input data for the pile analysis programs, PALLAS and PIES. At $R = 310\text{m}$ (Figure 6.3) after about 12 years (from 1988 to 2000) of continual pumping with a pumping rate equal to $30000\text{m}^3/\text{day}$, the maximum accumulated horizontal and vertical soil movements were about 28mm and 260mm respectively. If the ground water pumping rate is kept constant until 2010, the maximum horizontal soil movement increases by 38 mm; meanwhile the maximum vertical soil movement is still unchanged (see Figures 5.12 and 5.13). Provided that the ground water pumping rate in the Phap Van station is gradually increased ($45000\text{ m}^3/\text{day}$ from 2000 to 2010 then $60000\text{ m}^3/\text{day}$ from 2010 to 2020), the maximum horizontal soil movements are 56mm and 60mm, and the maximum vertical soil movements increase up to 387mm in 2010 and 485mm in 2020 correspondingly. Similar to Figure 6.3, Figures 6.4 and 6.5 show the vertical and horizontal soil movements at distances of 400m and 500m away from the assumed symmetrical axis of the Phap Van ground water pumping station.

The elastic soil properties are given in Table 6.1 for vertical pile analysis and Table 6.2 for horizontal pile analysis. It is recognised that there are increases in soil strength and stiffness due to soil compression however there is not enough data to take this effect into account. Hence, soil strength and stiffness properties are kept unchanged. Shaft resistance, f_s , is defined by adopting the equation suggested by Meyerhof (1976):

$$f_s = \beta \sigma'_v \quad (6.22)$$

where β is about 0.25 for clay and is from 0.25 to 0.35 for sand, σ'_v is the initial vertical effective stress in the soil.

Base resistance, f_b , is defined from correlation with SPT values. The plots from chapter 3 (Figures 3.20 to 3.25) provide some averaged values of SPT in Hanoi. At a depth of 40 to 50m from the ground surface, a soil layer of very dense sand with

gravel and cobbles has an average SPT value between 50 and 100, Figures 3.21, 3.22, and 3.25. Correlations between end bearing resistance, f_b , and SPT value can be estimated as follows:

$$f_b = K_{SPT} \cdot N_{SPT} \text{ (MN / m}^2\text{)} \quad (6.23)$$

where N_{SPT} is SPT value, K_{SPT} is equal to 0.1 for bored piles in sand (Shioi and Fukui, 1982) and 0.4 for driven piles in sand (Decourt, 1982).

An average value of $f_b = 8\text{MPa}$ is selected for a bored pile in soil layer number 7, (see Table 6.1).

For horizontal pile analysis, values of ultimate horizontal resistance of the pile is defined by applying the following equation:

$$W_y = 3K_p \sigma'_v \quad (6.24)$$

where $K_p = \frac{1 + \sin \varphi'}{1 - \sin \varphi'}$ for clay $\varphi' = 26^\circ$ to 28° , for sand $\varphi' = 28^\circ$ to 42° and σ'_v is effective stress.

According to Poulos and Davis (1980) for clays, Young's modulus is usually related to shear strength, c_u , as follows:

$$E_s = 150 - 400 c_u \quad (6.25)$$

where c_u is undrained shear strength; the low values are associated with normally consolidated clays and the high values with overconsolidated clays.

For sand, Kishida and Nakai (1977) relate E_s to SPT value N_{SPT} as follows:

$$E_s = 1.6 N_{SPT} \text{ (MPa)} \quad (6.26)$$

Tables 6.1 and 6.2 give soil properties that are used for vertical and horizontal pile analysis.

Table 6.1: Soil properties used for vertical pile analysis.

No.	Material type	H m	E'_s MPa	ν'	RFS	RFB	f_s kPa	f_b MPa	γ_s kN/m ³
1	Man made soil	3.3	6.5	0.3	0.5	0.9	4.08	-	19.70
2	Medium brown clay	3.7	6.5	0.3	0.5	0.9	7.22	-	16.60
3	Silt, Silty clay	7.0	2.5	0.3	0.5	0.9	15.79	-	19.60
4	Medium sandy clay	19.0	7.0	0.3	0.5	0.9	38.80	-	19.50
5	Medium to stiff brown clay	4.0	8.0	0.3	0.5	0.9	43.35	-	18.90
6	Fine to medium sand	3.0	15.0	0.3	0.5	0.9	48.43	-	19.50
7	Gravel and pebbles	5.0	150.0	0.3	0.5	0.9	116.2	8.0	20.10

Note: RFS and RFB are hyperbolic ratios; f_s and f_b are shaft resistance and base resistance of the pile; H is soil layer thickness.

Table 6.2: Soil properties for horizontal pile analysis.

No.	Material type	H m	c_u kPa	N	E'_s MPa	ν'	σ'_v kPa	ϕ'	W_y MPa	γ_s kN/m ³
1	Man made soil	3.3	30	-	6.0	0.3	32.64	26	0.25	19.70
2	Medium brown clay	3.7	30	-	6.0	0.3	57.76	26	0.44	16.60
3	Silt, Silty clay	7.0	15	-	3.0	0.3	126.29	26	0.97	19.60
4	Medium sandy clay	19.0	35	-	7.0	0.3	310.40	28	2.58	19.50
5	Medium to stiff brown clay	4.0	40	-	8.0	0.3	346.76	28	2.88	18.90
6	Fine to medium sand	3.0	-	15	24.0	0.3	375.83	32	3.67	19.50
7	Gravel and pebbles	5.0	-	80	130.0	0.3	427.28	35	4.73	20.10

Note: W_y is ultimate horizontal lateral resistance of pile-soil interaction and H is soil layer thickness.

The predicted axial behaviour of a single pile, which is assumed to be installed near the ground water pumping area, is shown in Figures 6.6, 6.7 and 6.8. These Figures illustrate the axial deflections and axial forces in a single pile installed at distances of 310m (near the edge of the pumping area), 400m, and 500m away from the assumed symmetrical axis of the Phap Van water pumping station, in the period of 1988 to 2000, 2000 to 2010, and 2010 to 2020. The conclusions that can be drawn from these figures are as follows:

- The increase in pumping rate of the ground water results in an increase in soil movements. Consequently downward movements of the pile are also increased. From 2000 to 2020, a further downward movement of about 16mm is added to the pile head at a position 310m away from the symmetrical axis (from 28mm in 2000 to 44.5mm in 2020) as a result of an increase in downward movement of the surrounding soil. As the downward soil movement increases from 1988 to 2020, the maximum induced axial forces in the pile at different positions (310, 400m, and 500m) are seen to slightly change. This can be explained by the fact that the vertical soil movements in 2000, 2010, and 2020 are relatively large and cause full slip at the interface between the pile and soil in the soft soil layers. Hence the axial forces are insignificantly changed. The slight changes of the axial forces here are accounted for by the partial slip of the soil-pile interface at the hard soil layer such as the medium stiff brown clay and the fine medium sand (layers 5 and 6), which are at a depth of between 30m and 40m from the ground surface. The downward movements of the single pile are much less than the maximum downward movement of the soil, and the movement of the pile tip is just slightly less than the movement of the pile head. Accordingly the relative movement between the pile and the soil is very large which in turn can cause a gap between the pile cap above the single pile. If the pile is connected to a raft and the whole system is designed to work together, then a considerable additional load that was assumed to be taken by the raft can transfer to the pile. Thus, the pile head movement and axial forces in the pile can be larger. In this case, as the pile is relatively stiff compared to the soil then the elastic deformation of the pile itself is rather small.

- In Figure 6.22a, settlement of a pile group is calculated without taking into consideration of pile-soil slip effect. Hence, the reduction in downdrag force due to the group effect is taken into account and this results in less settlement than a single

pile. When a full pile-soil slip occurs there is no reduction in downdrag force due to the group effect (Poulos and Davis, 1980). This results in more settlement in the pile group, Figure 6.20.

- The accumulated bending moments of the end-bearing single pile are calculated at three positions (310m, 400m, and 500m away from the symmetrical axis) and at three different times. Obviously, the further the distance between the pile and the ground water pumping station, the less is the horizontal soil movement. Therefore the maximum bending moment induced in the pile is reduced with an increase in distance between the pile and the symmetrical axis. The induced bending moment is also increased with an increase in pumping rate. In the free head end-bearing single pile, there are two peaks in the bending moment plots, one at a depth of 8m and the other at a depth of about 40m where the pile penetrates into the hard gravel and pebbles layer. With the condition of a fixed head pile, the maximum bending moments normally occur at the ground surface where the pile head is assumed to be restrained from rotation but free to translate.

A floating single pile is assumed to exist at the positions that are similar to the first two positions of the end-bearing pile mentioned above (310m and 400m). Analyses of the floating pile have been carried out with PIES and PALLAS to investigate the vertical and horizontal behaviour of the pile. The results of analyses are presented in Figures 6.9 to 6.18. Some conclusions are drawn from these figures and are presented below:

- The floating pile undergoes a downward vertical soil movement and settles much more than the end-bearing pile, which suffers the same downward soil movements. As a result, the relative movements between the soil and the floating pile can be much less than the relative movements of the soil and the end-bearing pile. However, a relative difference in movements between the floating pile head and the soil still exists, hence a gap between the pile head and cap or a raft above the pile head can potentially occur. In the floating pile, there is a natural line at a depth of about 20m where the maximum axial force occurs.
- The maximum induced bending moments in the floating pile for the fixed head condition are presented in Figures 6.17 and 6.18. Actually, the maximum bending

moments are equal to the maximum bending moments in the end-bearing pile with the same condition of a fixed head. This can be explained by the fact that the curvature of the horizontal soil movements in the two cases is identical.

6.5 PILE GROUP ANALYSIS

6.5.1 INTRODUCTION

A group of piles is assumed to exist in the area close to the Phap Van ground water pumping station, and an analysis of an end-bearing pile group is carried out. The soil properties that are utilized are the same as the soil properties presented for the single pile analysis. The GEPAN program is also employed to analyse this case for the purpose of comparison between elastic and elasto-plastic analysis.

6.5.2 PILE GROUP ANALYSIS

Figure 6.19 shows the 3x3 pile group analysed. Figures 6.20 to 6.21 present the axial deflections and axial forces induced in the pile group allowing pile-soil slip. As the piles in the pile group are spaced at 3 times pile diameter, then all the single piles in the group have similar settlement magnitude due to downdrag. However, a pile group settles more than a single pile. This can be explained by the group effect that influences the settlements of the pile group. For a single end-bearing pile, the influenced soil area below the pile tip is much smaller and less deep than under a pile group. The axial force calculated by PIES is similar for every pile in the group and also similar to the axial force induced in a single pile. This is because there is a large relative soil movement between the pile and soil and hence every pile in the pile group is given an equal additional force due to downdrag.

Figures 6.22 and 6.23 show the comparison of axial and lateral pile behaviours between an elastic analysis (GEPAN) and an elasto-plastic analysis (PIES). For axial pile behaviour, elastic analysis gives slightly smaller values of pile settlements compared with an elasto-plastic analysis of a single pile. The axial forces are also different. In elastic analysis, there are differences of axial forces between the corner piles, edge piles and centre pile, because the piles inside the pile group undergo less

soil movements than the piles at the edge and the corner. For the lateral deflections and bending moments, results of both analysis methods are in good agreement. This can be explained by the fact that horizontal pile-soil pressures induced by the horizontal soil movements are still less than the ultimate pile-soil pressures, hence the soil still behaves as an elastic material. However, as the elastic analysis can not take into account the pile soil slip, then it can not be used for the analysis in this case.

Figures 6.24 and 6.25 illustrate the lateral behaviour of piles in a pile group that have been analysed by using elasto-plastic methods (PALLAS program) where the group is at a distance of 310m away from the assumed symmetrical axis of the Phap Van ground water pumping station. The bending moments induced over the period of time are also presented for years 2000, 2010, and 2020. In the pile group, the corner pile, pile 1, always has a bending moment at the pile head that is larger than the bending moment at the head of the side and the middle pile, piles 2 and 5 respectively.

6.6 PARAMETRIC STUDY

6.6.1 SOIL MOVEMENT PROFILES

A parametric study of soil movements due to excessive ground water pumping have been conducted and presented in the last part of Chapter 5. In this analysis, a series of relationships between the horizontal and vertical soil movements, the drawdown of the water table, the ground water pumping rate, the soil layer Young's modulus, and the area of pumping were investigated. This analysis is utilized here to provide input of the soil movements for pile analysis.

In Figures 5.28 to 5.36 the relationship of horizontal and vertical soil movement is presented as the ratio $Max d_h * R / Max d_v * H_2$ plotted against H_1/H_2 . The definitions of these parameters are presented in Figure 5.22. If R is fixed at 1000m, then the ratio $Max d_h * R / Max d_v * H_2$ can be written as $1000 * Max d_h / Max d_v * H_2$. As H_1 and H_2 vary between 20m and 40m then the ratio can be presented as shown in Tables 6.3 and 6.4:

Table 6. 3: Average ratio $1000 \cdot \text{Max. } d_h / \text{Max. } d_v \cdot H_2$.

H_2 (m)	H_1 (m)		
	20	30	40
20	2-3 / 3	3.6	4-6 / 5
30	1.6	2-3 / 3	3.5-4 / 4
40	1.5	1.8	2-3 / 3

Note: 2-3 / 3 is the range of value is from 2 to 3 and the averaged value is taken as 3.

Table 6. 4: Average value of ratio $\text{Max. } d_h / \text{Max. } d_v \%$.

H_2 (m)	H_1 (m)		
	20	30	40
20	4-6 / 6	7.2	8-12 / 10
30	4.8	6-9 / 9	10.5-12 / 12
40	6	7.2	8-12 / 12

From the tables above, $\text{Max. } d_h$ can be approximately of 5% to 6%, 7% to 9%, and 10% to 12% of $\text{Max. } d_v$ with the thickness of the soft soil layer being 20m, 30, and 40m respectively.

From Figures 5.37 to 5.45 the relationship of $\text{Max. } d_v$ and $\text{Max. } d_w$ can be presented by ratios $\text{Max } d_v \cdot R^{3/2} / \text{Max } d_w \cdot H_1 \cdot (H_1 + H_2)^{1/2}$ and H_1/H_2 . As R is fixed at 1000m then the ratio $\text{Max } d_v \cdot R^{3/2} / \text{Max } d_w \cdot H_1 \cdot (H_1 + H_2)^{1/2}$ can be expressed as $31622.77 \cdot \text{Max } d_v / \text{Max } d_w \cdot H_1 \cdot (H_1 + H_2)^{1/2}$. This ratio depends on the soil Young's modulus, (it equals 20 and 10 with $E_s = 5.0\text{MPa}$ and 10MPa), as presented in Table 6.5.

Table 6. 5: Average ratio $Max. d_v / Max. d_w$ %.

E'_1 (MPa)	H_2 (m)	H_1 (m)		
		20	30	40
5	20	8	13.4	19.6
	30	8.9	14.7	21.17
	40	9.8	15.87	22.6
10	20	4	6.7	9.8
	30	4.5	7.35	10.6
	40	4.9	7.95	11.3
20	20	2	3.35	4.9
	30	2.25	3.7	5.3
	40	2.5	3.95	5.65

The maximum drawdown of the water table $Max. d_w$ depends on the pumping area, r_p , pumping rate, Q , the soil layer thickness, H_1 and H_2 , and soil permeability, $k = k_x = k_y$. Figures 5.23 to 5.27 present the relationships of the maximum drawdown of water table and these parameters. Assuming that the ratio $Max. d_w / H_1$ equals 40%, 80% and 100% then vertical soil movements due to ground water pumping are as given in Tables 6.6, 6.7, 6.8, 6.9, and 6.10. The maximum horizontal soil movements are given in Table 6.11.

Table 6.5 shows that for the case of ground water pumping, the vertical soil movement is a result of the drawdown of the water table. The maximum vertical soil movement that is supposed to be at the assumed centre of the pumping area relates to the maximum drawdown of water table at the centre. In Table 6.6, the average ratio $Max d_v / H_1$.

Table 6. 6: Average ratio $Max. d_v / H_1$ %.

E'_1 (MPa)	H_2 (m)	H_1 (m)								
		20			30			40		
		$Max. d_w / H_1$ (%)			$Max. d_w / H_1$ (%)			$Max. d_w / H_1$ (%)		
		40	80	100	40	80	100	40	80	100
5	20	3.2	6.4	8	5.36	10.72	13.4	7.84	15.68	19.6
	30	3.56	7.12	8.9	5.88	11.76	14.7	8.2	16.4	21.17
	40	3.92	7.84	9.8	6.35	12.7	15.87	9.04	18.08	22.6
10	20	1.6	3.2	4	2.68	5.36	6.7	3.92	7.84	9.8
	30	1.8	3.6	4.5	2.94	5.88	7.35	4.24	8.48	10.6
	40	1.96	3.92	4.9	8.18	6.36	7.95	4.52	9.04	11.3
20	20	0.8	1.6	2	1.34	2.68	3.35	2.0	3.92	4.9
	30	0.9	1.8	2.25	1.5	2.94	3.68	2.12	4.24	5.3
	40	1.0	1.95	2.5	4.9	3.18	3.98	2.26	4.52	5.65

Table 6. 7: Average value of $Max. d_v$ (m).

E'_1 (MPa)	H_2 (m)	H_1 (m)								
		20			30			40		
		$Max. d_w / H_1$ (%)			$Max. d_w / H_1$ (%)			$Max. d_w / H_1$ (%)		
		40	80	100	40	80	100	40	80	100
5	20	0.64	1.28	1.6	1.608	3.216	4.02	3.136	6.272	7.84
	30	0.712	1.424	1.78	1.764	3.528	4.41	3.28	6.56	8.468
	40	0.784	1.568	1.96	1.905	3.81	4.761	3.616	7.232	9.04
10	20	0.32	0.64	0.8	0.804	1.608	2.01	1.568	3.136	3.92
	30	0.36	0.72	0.9	0.882	1.764	2.205	1.696	3.392	4.24
	40	0.392	0.784	0.98	2.454	1.908	2.385	1.808	3.616	4.52
20	20	0.16	0.32	0.4	0.4	0.8	1	0.78	1.57	1.96
	30	0.18	0.36	0.45	0.44	0.88	1.1	0.85	1.7	2.12
	40	0.2	0.4	0.49	1.23	0.95	1.19	0.9	1.8	1.26

Table 6. 8: Average value of $Max. d_v$ (m) at a distance of r_p from the centre with $r_p = 100\text{m}$.

E'_1 (MPa)	H_2 (m)	H_1 (m)								
		20			30			40		
		$Max. d_w / H_1$ (%)			$Max. d_w / H_1$ (%)			$Max. d_w / H_1$ (%)		
		40	80	100	40	80	100	40	80	100
5	20	0.512	1.024	1.280	1.286	2.573	3.216	2.509	5.018	6.272
	30	0.570	1.139	1.424	1.411	2.822	3.528	2.624	5.248	6.774
	40	0.627	1.254	1.568	1.524	3.048	3.809	2.893	5.786	7.232
10	20	0.256	0.512	0.640	0.643	1.286	1.608	1.254	2.509	3.136
	30	0.288	0.576	0.720	0.706	1.411	1.764	1.357	2.714	3.392
	40	0.314	0.627	0.784	1.963	1.526	1.908	1.446	2.893	3.616
20	20	0.128	0.256	0.320	0.320	0.640	0.800	0.624	1.256	1.568
	30	0.144	0.288	0.360	0.352	0.704	0.880	0.680	1.360	1.696
	40	0.160	0.320	0.392	0.984	0.760	0.952	0.720	1.440	1.008

Table 6. 9: Average value of $Max. d_v$ (m) at a distance of r_p from the centre with $r_p = 250\text{m}$.

E'_1 (MPa)	H_2 (m)	H_1 (m)								
		20			30			40		
		$Max. d_w / H_1$ (%)			$Max. d_w / H_1$ (%)			$Max. d_w / H_1$ (%)		
		40	80	100	40	80	100	40	80	100
5	20	0.384	0.768	0.960	0.965	1.930	2.412	1.882	3.763	4.704
	30	0.427	0.854	1.068	1.058	2.117	2.646	1.968	3.936	5.081
	40	0.470	0.941	1.176	1.143	2.286	2.857	2.170	4.339	5.424
10	20	0.192	0.384	0.480	0.482	0.965	1.206	0.941	1.882	2.352
	30	0.216	0.432	0.540	0.529	1.058	1.323	1.018	2.035	2.544
	40	0.235	0.470	0.588	1.472	1.145	1.431	1.085	2.170	2.712
20	20	0.096	0.192	0.240	0.240	0.480	0.600	0.468	0.942	1.176
	30	0.108	0.216	0.270	0.264	0.528	0.660	0.510	1.020	1.272
	40	0.120	0.240	0.294	0.738	0.570	0.714	0.540	1.080	0.756

Table 6. 10: Average value of *Max. d_v* (m) at a distance of r_p from the centre with $r_p = 500\text{m}$.

E'_1 (MPa)	H_2 (m)	H_1 (m)								
		20			30			40		
		<i>Max. d_w / H_1</i> (%)			<i>Max. d_w / H_1</i> (%)			<i>Max. d_w / H_1</i> (%)		
		40	80	100	40	80	100	40	80	100
5	20	0.320	0.640	0.800	0.804	1.608	2.010	1.568	3.136	3.920
	30	0.356	0.712	0.890	0.882	1.764	2.205	1.640	3.280	4.234
	40	0.392	0.784	0.980	0.953	1.905	2.381	1.808	3.616	4.520
10	20	0.160	0.320	0.400	0.402	0.804	1.005	0.784	1.568	1.960
	30	0.180	0.360	0.450	0.441	0.882	1.103	0.848	1.696	2.120
	40	0.196	0.392	0.490	1.227	0.954	1.193	0.904	1.808	2.260
20	20	0.080	0.160	0.200	0.200	0.400	0.500	0.390	0.785	0.980
	30	0.090	0.180	0.225	0.220	0.440	0.550	0.425	0.850	1.060
	40	0.100	0.200	0.245	0.615	0.475	0.595	0.450	0.900	0.630

Table 6. 11: Average value of *Max. d_h* (m).

E'_1 (MPa)	H_2 (m)	H_1 (m)								
		20			30			40		
		<i>Max. d_w / H_1</i> (%)			<i>Max. d_w / H_1</i> (%)			<i>Max. d_w / H_1</i> (%)		
		40	80	100	40	80	100	40	80	100
5	20	0.0384	0.0768	0.0960	0.1158	0.2316	0.2894	0.3136	0.6272	0.7840
	30	0.0342	0.0684	0.0854	0.1372	0.2743	0.3428	0.3936	0.7872	1.0162
	40	0.0470	0.0941	0.1176	0.1588	0.3175	0.3969	0.4339	0.8678	1.0848
10	20	0.0192	0.0384	0.0480	0.0579	0.1158	0.1447	0.1568	0.3136	0.3920
	30	0.0173	0.0346	0.0432	0.0767	0.1374	0.1717	0.2035	0.4070	0.5088
	40	0.0235	0.0470	0.0588	0.0794	0.1588	0.1985	0.2170	0.4339	0.5424
20	20	0.0096	0.0192	0.024	0.0289	0.0579	0.0723	0.0784	0.1568	0.196
	30	0.0086	0.0173	0.0216	0.0383	0.0687	0.0858	0.1017	0.203	0.2544
	40	0.0117	0.0235	0.0294	0.0397	0.0794	0.0992	0.1085	0.2169	0.2712

Tables 6.7, 6.8, 6.9, and 6.10 are the values of $Max\ d_v$ at the centre of the assumed ground water pumping area and at the edges of the pumping area with r_p equal to 100, 250, and 500m respectively.

Table 6.11 presents average values of the maximum horizontal soil movements that occur at the edges of the ground water pumping area. Figures 5.31 to 5.36 show values of ratio $R*Max\ d_h / H_2*Max\ d_v$. It seems that the ratio $R*Max\ d_h / H_2*Max\ d_v$ depends much on the ratio H_1 / H_2 rather than the values of r_p . Hence, the maximum horizontal soil movement, which occurs at the edge of the pumping area in most of the cases, are presented in only one table, Table 6.11. The relationship of the maximum horizontal soil movement and r_p is neglected here.

6.6.2 HORIZONTAL PILE BEHAVIOUR

Table 6.11 is used as the input data for the maximum horizontal soil movements to the horizontal pile behaviour program, PALLAS. The profiles of soil movements (for both vertical and horizontal) are shown as a triangular shape in Figure 6.26. The movements of the hard soil layer are conservatively considered as being zero.

Tables 6.12 and 6.13 present maximum bending moment of a free head end bearing single pile and a fixed head floating single pile respectively that is assumed to exist close to a ground water pumping station for a base case analysis. The grey shaded area shows that the induced maximum bending moment is over the bending moment limit that a standard pile with diameter of 0.5m, 30MPa concrete, and 2% of reinforcement can sustain, which is about 200kNm. Hence, an approximate vertical soil movement value of 1.6m can cause horizontal soil movement large enough to damage an existing end bearing free head pile with a diameter of 0.5m, which is close to the pumping area. An equation is provided here to show the influence of each factor on the maximum bending moment:

$$Max. M^s = Max. M_b^s \times k_d^s \times k_r^s \quad (6.27)$$

where $Max. M_b^s$ is maximum bending moment (kNm) given in Tables 6.12 and 6.13, k_d^s is correction factor for pile diameter, and k_r^s is correction factor for pile distance from the edge of the pumped region.

Figure 6.27 shows maximum bending moment correction factor for a free head end-bearing single pile. Figure 6.28 shows maximum bending moment correction factor for the distance from the pile to the symmetrical axis. The following conclusions can be drawn from the analysis results:

- The maximum induced bending moments are larger for a soft upper clay layer (5MPa in this case) because a soft soil layer causes a large movement as it will consolidate more due to pumping. Hence the induced bending moment is increased.
- Maximum bending moment is also influenced by the soft soil layer thickness. With the same amount of drawdown of water level the induced maximum bending moment is larger where the soft soil layer is thicker.
- Maximum bending moment increases when the hard soil layer thickness increases.
- Maximum bending moment reduces as the distance from the pile to the symmetrical axis of the pumping area increases, and it depends on radius of the pumping area, r_p .

Table 6.12: Maximum bending moments (kNm) for free head end-bearing single pile; the base case analysis with pile diameter $d = 0.5\text{m}$.

E'_1 (MPa)	H_2 (m)	H_1 (m)								
		20			30			40		
		$Max. d_w / H_1$ (%)			$Max. d_w / H_1$ (%)			$Max. d_w / H_1$ (%)		
		40	80	100	40	80	100	40	80	100
5	20	82.0	164.1	202.5	165.2	330.2	412.4	335.4	671.0	838.7
	30	73.0	146.2	182.6	195.5	391.0	488.9	421.0	842.1	1087.0
	40	100.8	200.9	251.4	226.7	452.9	565.9	463.9	928.3	1160.0
10	20	42.5	84.1	106.2	85.4	171.0	213.3	173.5	347.1	433.9
	30	38.1	77.0	95.6	113.3	202.9	253.2	225.6	450.8	563.1
	40	52.3	104.5	130.1	116.9	234.7	293.0	240.0	480.0	600.5
20	20	21.5	42.9	53.6	43.2	86.0	107.3	87.5	174.9	218.7
	30	19.4	38.7	48.3	57.2	102.6	127.7	113.4	226.8	284.0
	40	26.3	52.1	65.6	58.9	117.9	147.3	120.7	241.9	302.7

Table 6. 13: Maximum bending moments (kNm) for fixed head single pile; the base case analysis with pile diameter $d = 0.5\text{m}$.

E'_1 (MPa)	H_2 (m)	H_1 (m)								
		20			30			40		
		Max. d_w / H_1 (%)			Max. d_w / H_1 (%)			Max. d_w / H_1 (%)		
		40	80	100	40	80	100	40	80	100
5	20	82.69	168.8	210.7	169.1	311.2	381.5	344.4	628.5	752.3
	30	75.1	147.1	183.2	200.5	359.3	441.7	432.2	765.8	960.3
	40	103.3	201.9	257.8	227.9	413.2	502.9	476.2	833.9	1029
10	20	47.69	94.88	119.9	96.51	193.7	230.6	196.1	381.4	466.4
	30	42.69	87.63	107.6	130.8	220.7	260	253.7	470.9	577.9
	40	58.32	117.6	151.6	132.6	248.3	293.9	279	496	599.6
20	20	27.1	54.5	69.6	45.43	78.23	94.84	112	234.8	287.9
	30	25.07	49.35	61.6	75.84	136.7	161.5	152.8	285.7	349.8
	40	33.34	66.63	83.89	76.61	152.1	174.9	156.1	310.6	366.9

Comparison between the maximum induced pile bending moment in a single free head end bearing pile that is calculated by using the design charts and the result from a full analysis is carried out to check whether the correction factors can be used with reasonable accuracy. The soil properties, the pumping area, the pile diameter and pile distance to the pumping area are provided as follows:

- The drained Young's soil modulus of the soft soil layer is $E'_1 = 5\text{MPa}$. Soil layer thicknesses are $H_1 = 30\text{m}$ and $H_2 = 20\text{m}$.
- The ground water pumping radius is $r_p = 100\text{m}$. The maximum water table drawdown due to ground water pumping is assumed as 80% thickness of the soft soil layer, H_1 .
- The pile diameter, d_p , is 1.0m . The distance between the pile and the assumed symmetrical axis of the nearby ground water pumping area is 500m .

From Table 6.12, Maximum bending moment for the base case is $\text{Max. } M_b^s = 330\text{ kNm}$. From Figures 6.27 and 6.28, the correction factors are as follows: for $d_p = 1\text{m}$,

$k_d^s = 9$, for a distance from the pile to the assumed symmetrical axis of 500m, $k_r^s = 0.26$. From Equation 6.27 $Max. M^s = 330 \times 9 \times 0.26 = 772 \text{ kNm}$.

From Table 6.11 $Max. d_h = 0.2316\text{m}$. From Figure 5.48, at a distance of $r = 500\text{m}$, horizontal soil movement is about 0.2 to 0.39 $Max. d_h$. Average of the maximum horizontal soil movement at the distance of 500m from the symmetrical axis is about $0.3 \times Max. d_h = 0.069\text{m}$. A horizontal soil movement of triangular shape with a maximum horizontal soil movement at zero depth of 0.069m was put into the PALLAS program to calculate the induced bending moment. PALLAS gives a maximum bending moment of $Max. M^s = 770 \text{ kNm}$ at a depth of 30m. For this particular case, the result for the maximum bending moment is much the same as the result archived from Equation 6.27.

6.6.3 AXIAL PILE BEHAVIOUR

In the parametric analysis of a single end bearing pile subjected to vertical soil movements the soil properties, pile geometry and soil movements are as set out below:

- A single pile has diameter of 0.5, 0.8, 1.0, 1.2 and 1.5m; the pile length varies from 25m, 35m, and 45m with the bottom 5m of pile length penetrating into the hard soil layer.
- Total soil unit weight for both soil layers are taken as 19.6kN/m^3 . Values of f_s and f_b are given in Tables 6.14, 6.15, and 6.16. The pile shaft resistance, f_s , is estimated by applying equation 6.22. The value of pile base resistance, f_b , for the hard soil layer can be estimated as follows:

$$f_b = N_q \cdot \sigma'_v \quad (6.28)$$

where N_q is equal to 8 to 20 for a bored pile in uncemented calcareous sand (Poulos, 1989) and 40 for driven pile in silica sand (API, 1984), σ'_v is vertical effective stress.

At a depth of 25m from the ground surface, the average value of SPT is about 25 to 30 (Figures 3.21 and 3.24). From equation 6.23, the end bearing resistance value, f_b , for a bored pile can be calculated as 3.0MPa. This means that the value of $N_q = 12$ falls in the range of 8 to 20, which is suggested by Poulos (1989).

In Figure 3.24, at a depth of 35m, the average value of SPT is about 30 to 50. Then for bored pile installed in this very dense gravely sand layer, f_b would be approximately equal to 3 to 5MPa, from equation 6.23. The end bearing resistance for a bored pile at the depth of 35m, in the dense gravely sand, is therefore taken as 5.0 MPa, (see Table 6.15).

A value of N_q is selected that is equal to 18 for a bored pile (Poulos, 1989). Then in the case where a very dense gravely sand layer lies at a depth of 40m and a bored pile is installed to a depth of 45m, the value of the end bearing resistance, f_b , approximately equals $18 \times 440.55 = 7900$ kPa. The value of f_b at a depth of 45m that approximately equals 8.0MPa for a bored pile is estimated via SPT value in Table 6.1. Hence, the end bearing resistance for a bored pile at a depth of 45m, in the very dense gravely sand, is taken as 8.0MPa, (see Table 6.16).

- Maximum vertical soil movements are given in Tables 6.8, 6.9 and 6.10.

Figures 6.29(a) to (c) show maximum pile head displacement in a single end bearing pile subjected to a triangular shape of vertical soil movements. As can be expected, increase in vertical soil movement causes increase in vertical pile head displacement. However, increase in vertical pile head displacement has a limit. There is no further development of vertical pile head displacement over the limit value even if vertical soil movement still increases. The limit value of pile head displacement varies differently and depends on the thickness of the soft soil layer, where most of the vertical soil movement occurs, and pile diameter. The longer the pile the larger is the pile head displacement.

Figure 6.30 shows a case where a pile behaves as an end bearing pile at the beginning, when the maximum value of soil movement is still small. As the vertical soil movement increases the pile head displacement also proportionally increases until the vertical soil movement reaches to the value of about 1.0m. When the vertical soil movement is over 1.0m then the pile head displacement develops more rapidly. It seems that there is no limit of the vertical pile head deflection as the vertical soil movement increases further. In this case, the pile behaves as an end bearing pile at the beginning, but when the base load reaches a maximum and there is an end bearing failure it behaves as a floating pile. The load deflection behaviour of the pile is similar

to a floating pile undergoing an external soil movement, which means a natural line exists at a depth where there is the maximum axial force. In this case the natural line is at the interface between the soft and the hard soil layers.

Table 6.14 Soil properties used for vertical bored pile analysis (pile length = 25m).

Depth (m)	σ'_v kPa	β	f_s (kPa)	f_b (MPa)
0	0.00	0.25	0.00	-
5	48.95	0.25	12.24	-
10	97.90	0.25	24.48	-
15	146.85	0.25	36.71	-
20	195.80	0.25	48.95	-
21	205.59	0.35	71.96	-
25	244.75	0.35	85.66	3.0

Table 6.15 Soil properties used for vertical pile analysis (pile length = 35m).

Depth (m)	σ'_v kPa	β	f_s (kPa)	f_b (MPa)
0	0.00	0.25	0.00	-
5	48.95	0.25	12.24	-
10	97.90	0.25	24.48	-
15	146.85	0.25	36.71	-
20	195.80	0.25	48.95	-
25	244.75	0.25	61.19	-
30	293.70	0.25	73.43	-
31	303.49	0.35	106.22	-
35	342.65	0.35	119.93	5.0

Table 6.16 Soil properties used for vertical pile analysis (pile length = 45m).

Depth (m)	σ'_v kPa	β	f_s (kPa)	f_b (MPa)
0	0.00	0.25	0.00	-
5	48.95	0.25	12.24	-
10	97.90	0.25	24.48	-
15	146.85	0.25	36.71	-
20	195.80	0.25	48.95	-
25	244.75	0.25	61.19	-
30	293.70	0.25	73.43	-
35	342.65	0.25	85.66	-
40	391.60	0.25	97.90	-
41	401.39	0.35	140.49	-
45	440.55	0.35	154.19	8.0

6.7 SUMMARY AND CONCLUSIONS

In this chapter, a study of pile behaviour due to ground water pumping has been conducted. In the first section a short critical review of pile behaviour due to induced soil movement caused by ground water pumping has been carried out. Both pile responses due to vertical and horizontal soil movements are of concern. Introduction of existing pile analysis programs that are utilized to analyse pile behaviour in this chapter are discussed. PALLAS and PIES are two pile analysis programs that have been used often in this chapter. GEPAN is also a powerful method for pile analysis. It is a general three-dimensional pile analysis program that employs the boundary element method. It can analyse influences of both vertical and horizontal external soil movements on piled foundations and also consider the coupling effect of the three-dimensional loading on piled foundations. However, as the current version of GEPAN only allows the user to analyse purely elastic pile behaviour, it is only used for some of the analyses presented here.

In the following section of the chapter, pile behaviour analyses of theoretical single piles and pile groups, which are assumed to exist at distances from the Phap Van ground water station, have been conducted. The soil movements, in both vertical and horizontal directions, estimated from the previous chapter were utilized as the source of induced external movement imposed at the pile-soil interface for the pile behaviour. The pile behaviour analysis results due to the external soil movements near the Phap Van ground water pumping station are presented and discussed. As can be expected, the further away from the pumping area the smaller the vertical soil movement, and this results in less vertical pile settlement.

For the same vertical soil movement, a pile group was predicted to settle more than a single pile when there is full slip along the piles. This is due to interaction among the piles in the pile group. It was also shown that the horizontal soil movement at Phap Van may not cause any severe damage to a nearby piled foundation.

The final part of this chapter presents a parametric study of single pile behaviour due to ground water pumping. The results of the parametric study of soil movements due to pumping ground water that were obtained in the previous chapter are used as the external soil movement input in this parametric study. Results and conclusions are then presented and discussed. This study shows that under severe ground water pumping, damage to an existing pile can occur because of a significant bending moment induced.

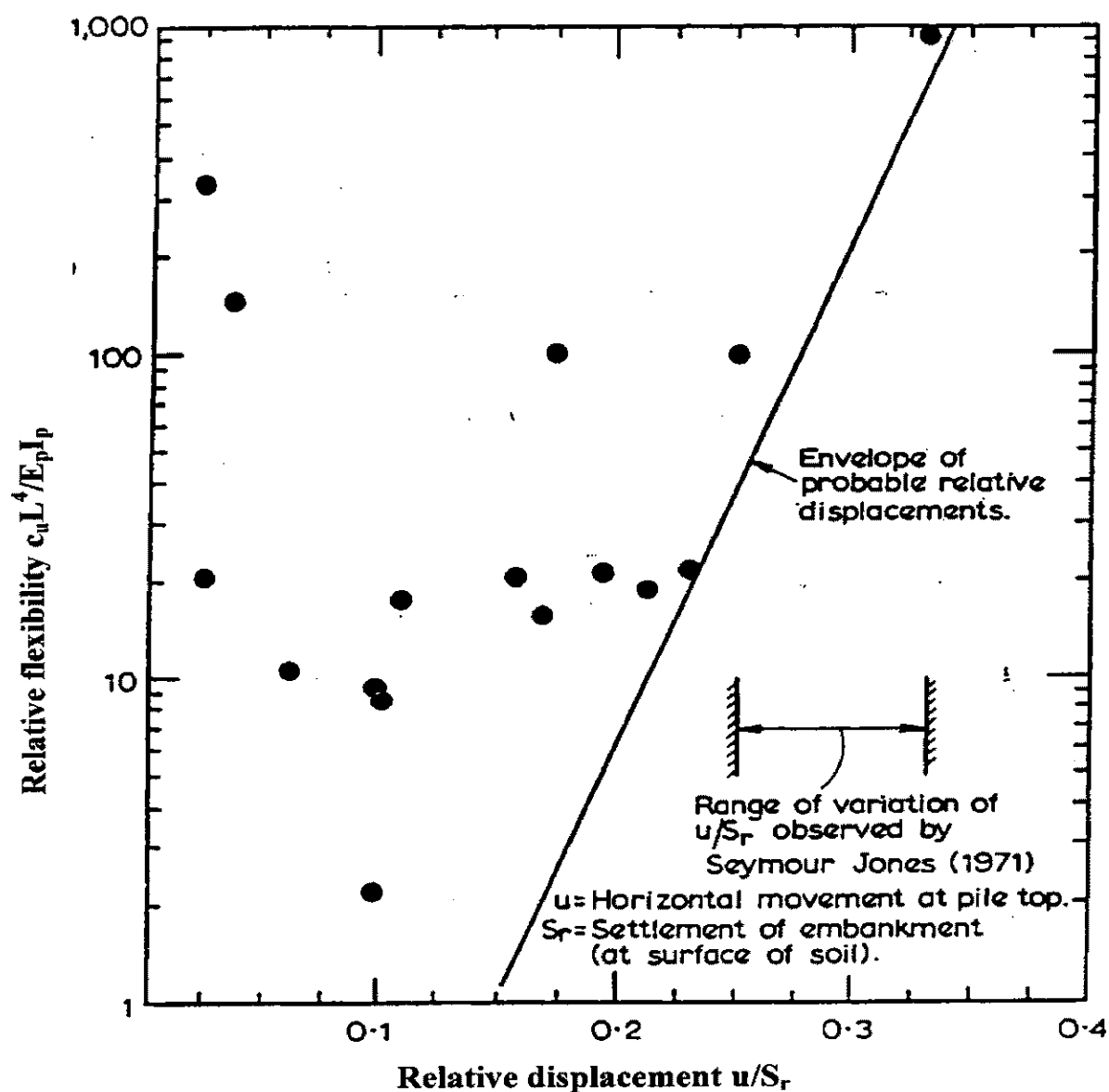


Figure 6.1: Relative displacement as a function of relative flexibility (after Marche and Larcroix, 1972)

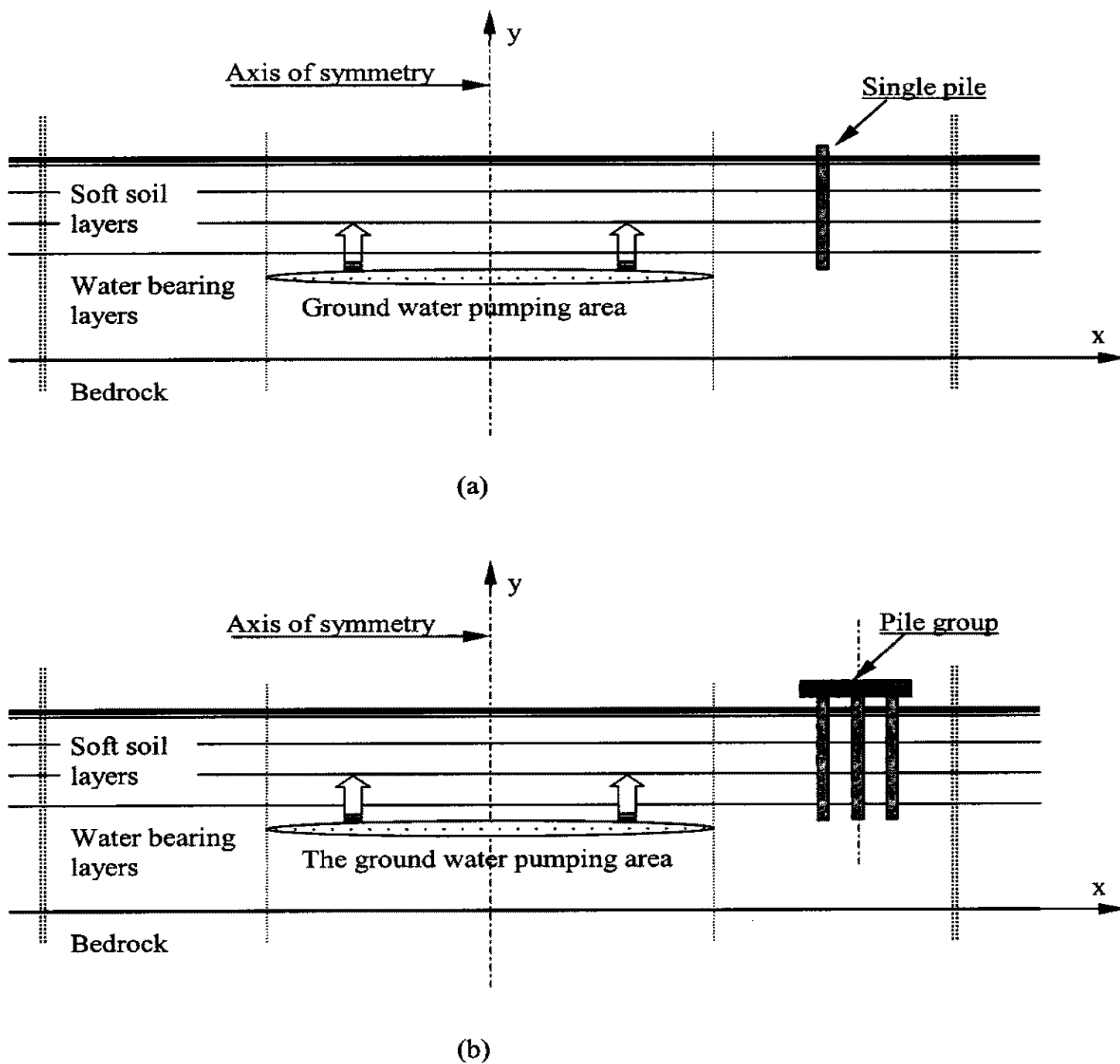


Figure 6.2: A single pile (a) or a pile group (b) near an extensive ground water pumping area.

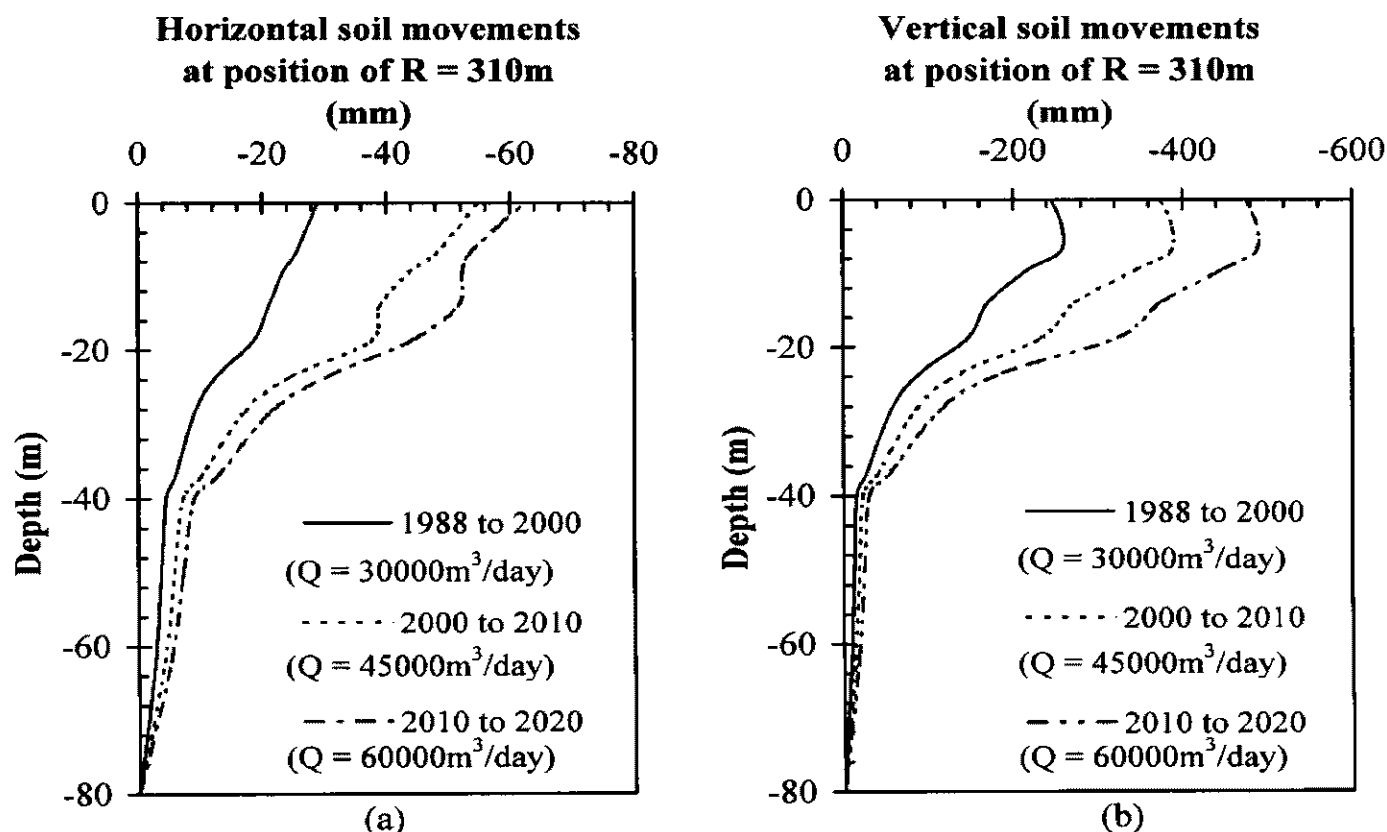


Figure 6.3: Typical accumulated soil movements at R = 310 m away from the assumed symmetrical axis at the Phap Van ground water pumping station.

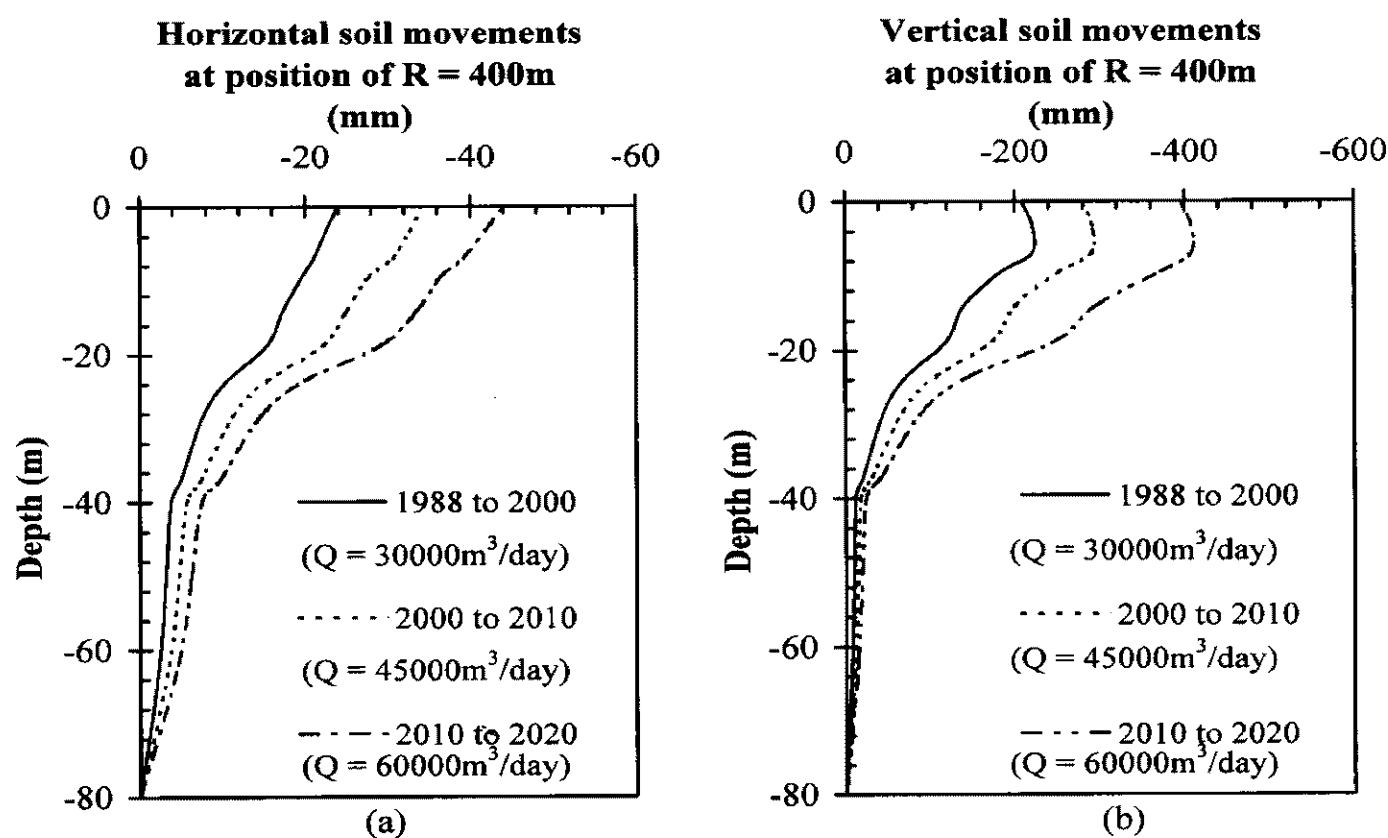


Figure 6.4: Typical accumulated soil movements at R = 400 m away from the assumed symmetrical axis at the Phap Van ground water pumping station.

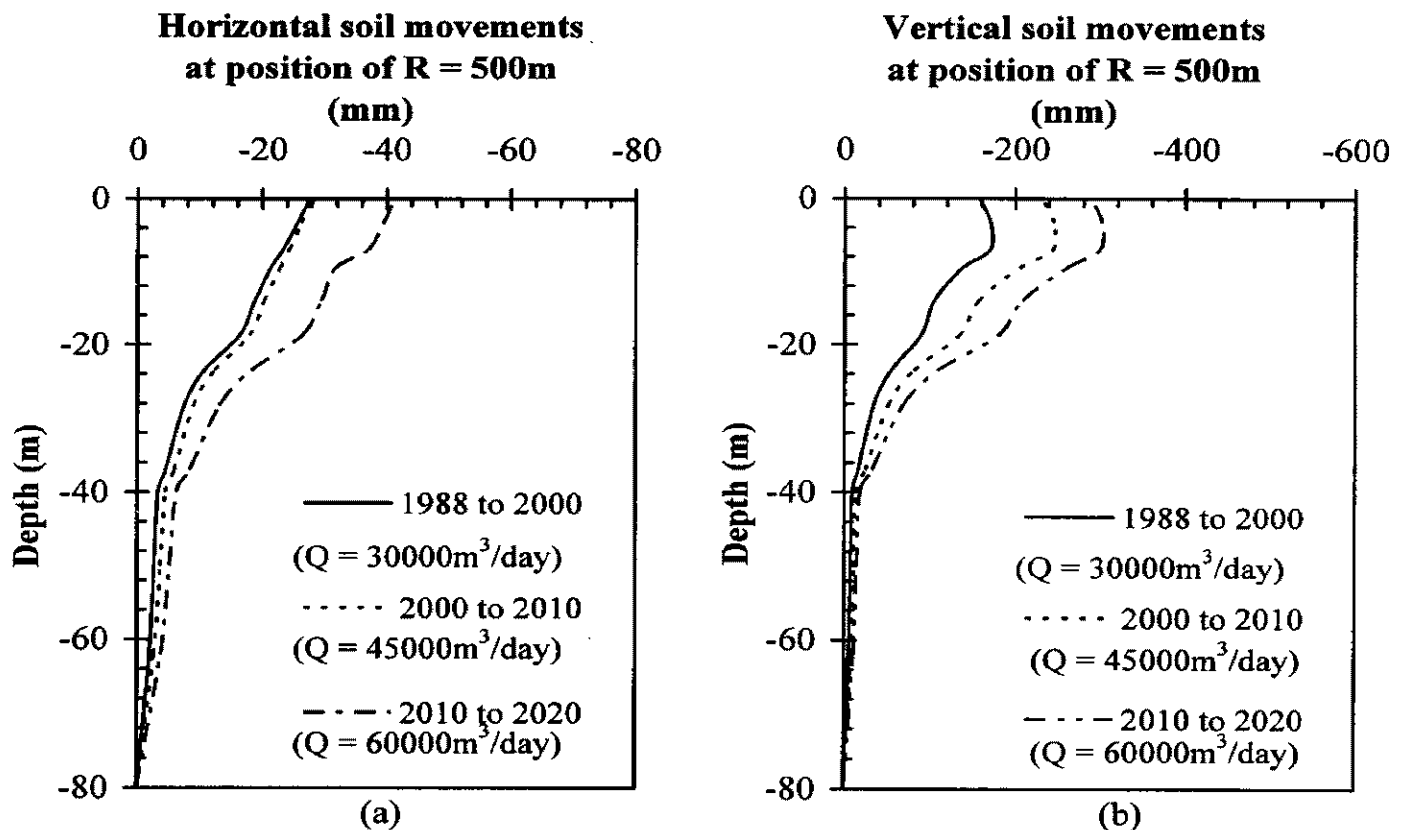


Figure 6.5: Typical accumulated soil movements at R = 500 m away from the assumed symmetrical axis at the Phap Van ground water pumping station.

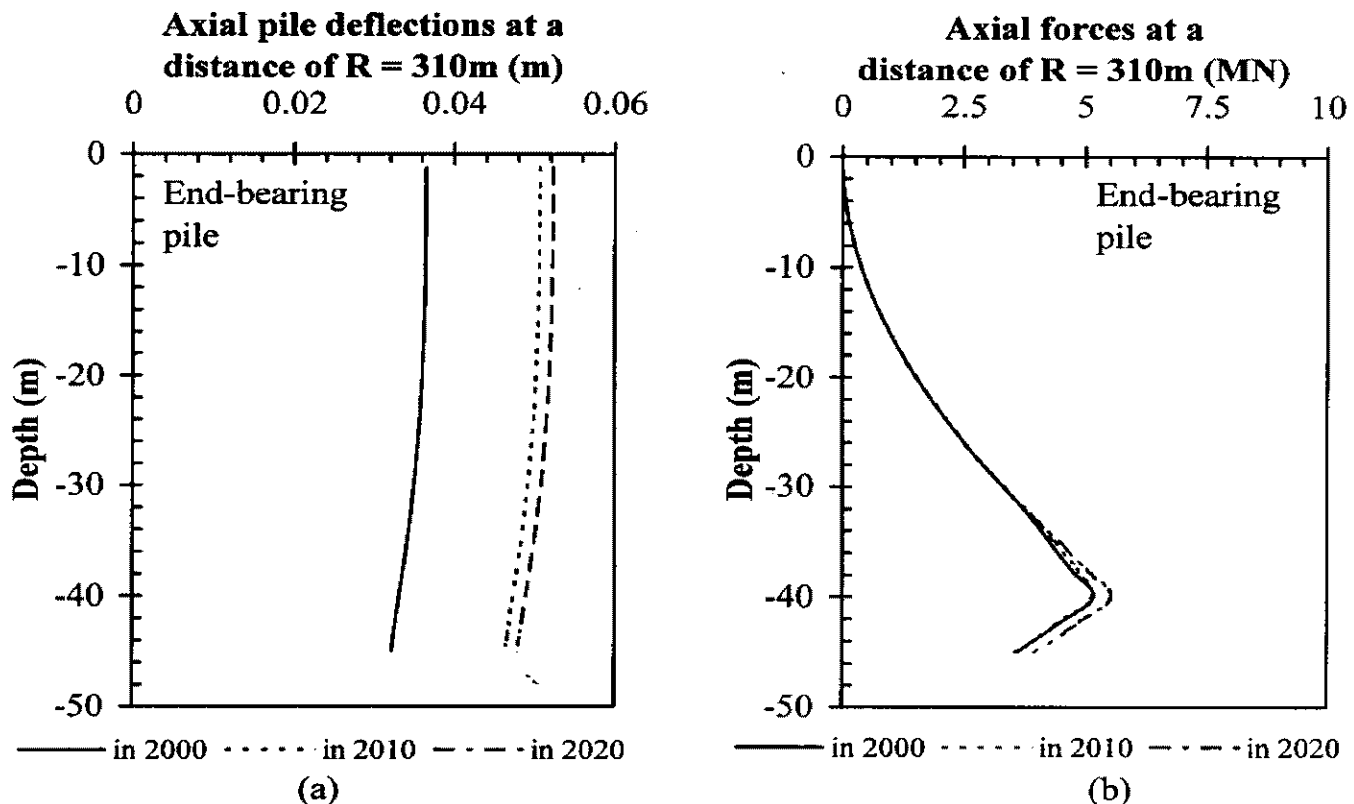


Figure 6.6: Axial deflection and axial force in a single pile at a distance of 310m from the assumed symmetrical axis of the Phap Van ground water pumping station.

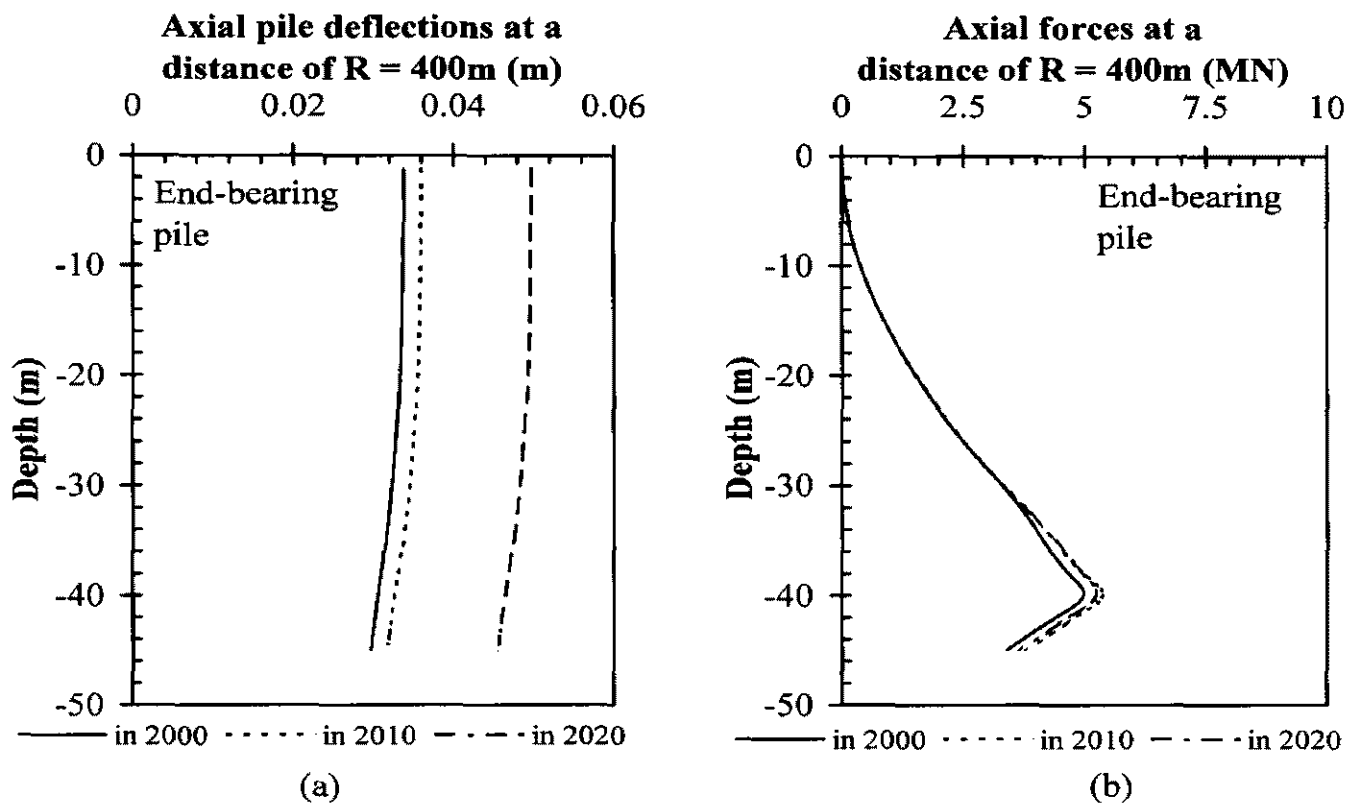


Figure 6.7: Axial deflection and axial force in a single pile at a distance of 400m from the assumed symmetrical axis of the Phap Van ground water pumping station.

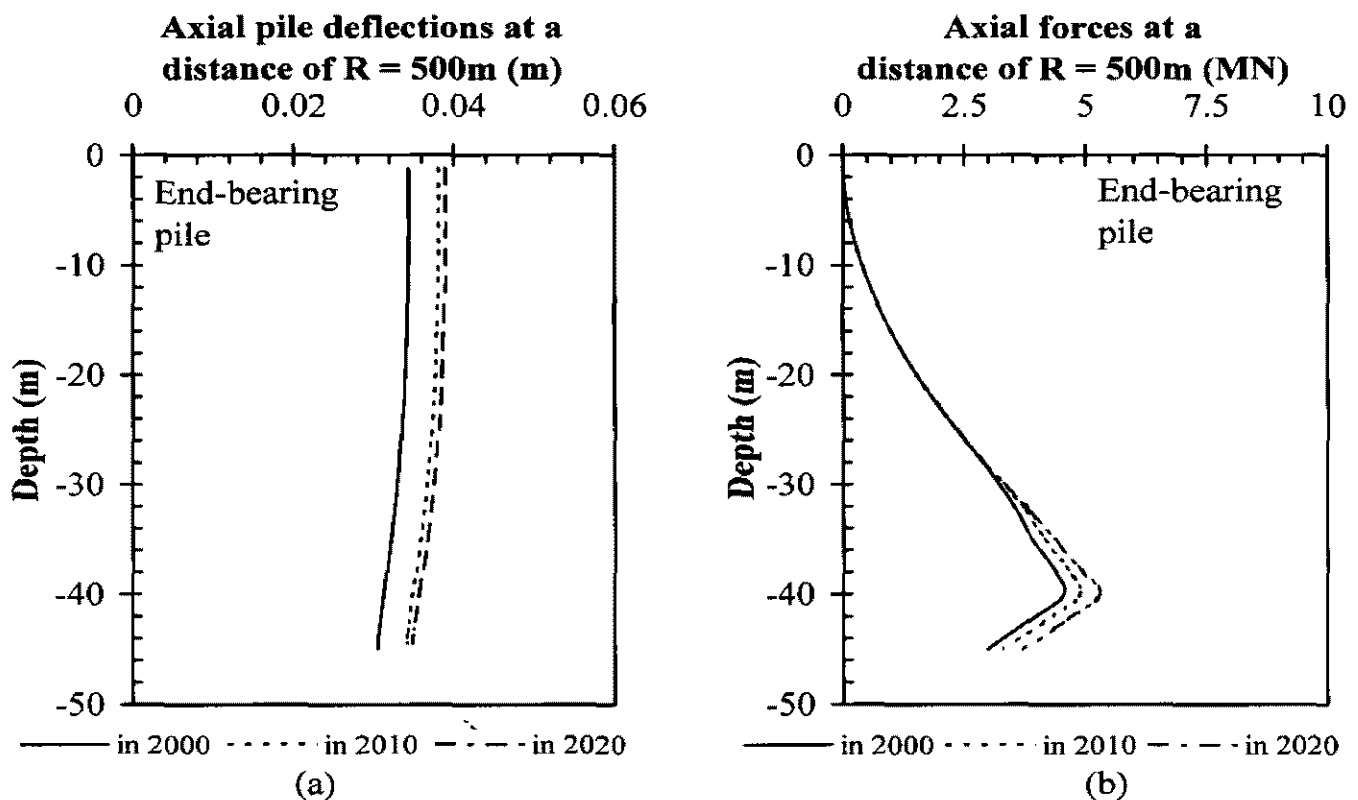


Figure 6.8: Axial deflection and axial force in a single pile at a distance of 500m from the assumed symmetrical axis of the Phap Van ground water pumping station.

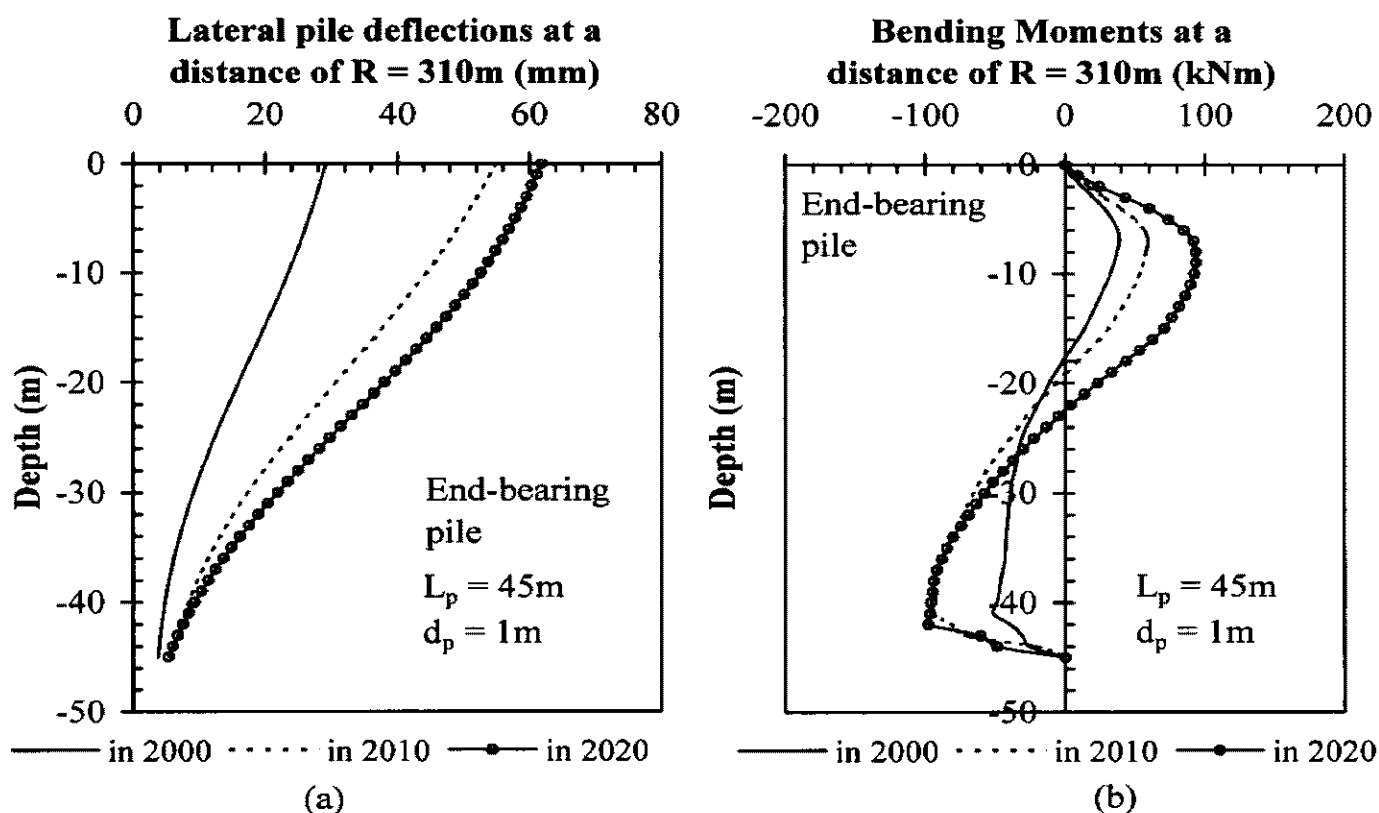


Figure 6.9: Lateral deflection and induced bending moments in a single free head pile at a distance of 310m from the assumed symmetrical axis.

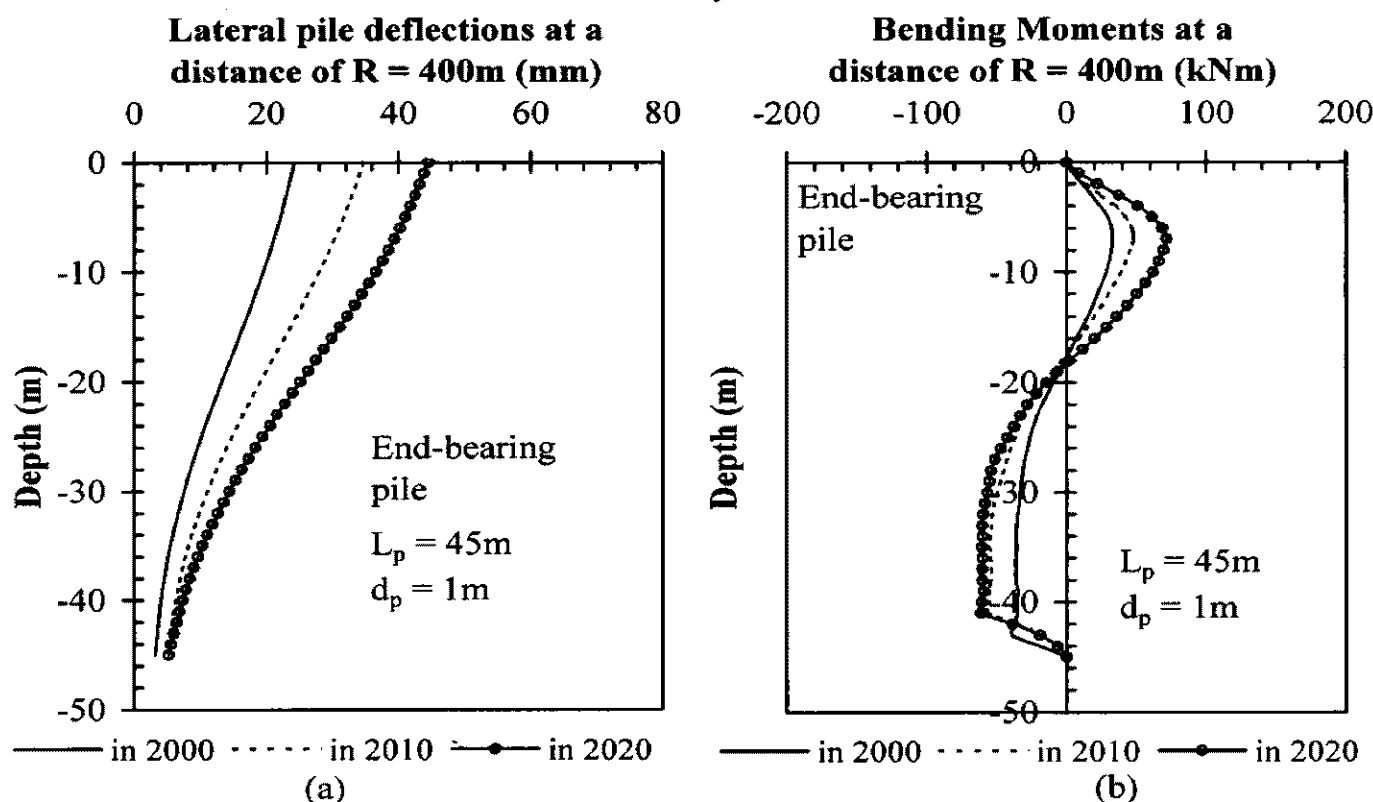


Figure 6.10: Lateral deflection and induced bending moments in a single free head pile at a distance of 400m from the assumed symmetrical axis.

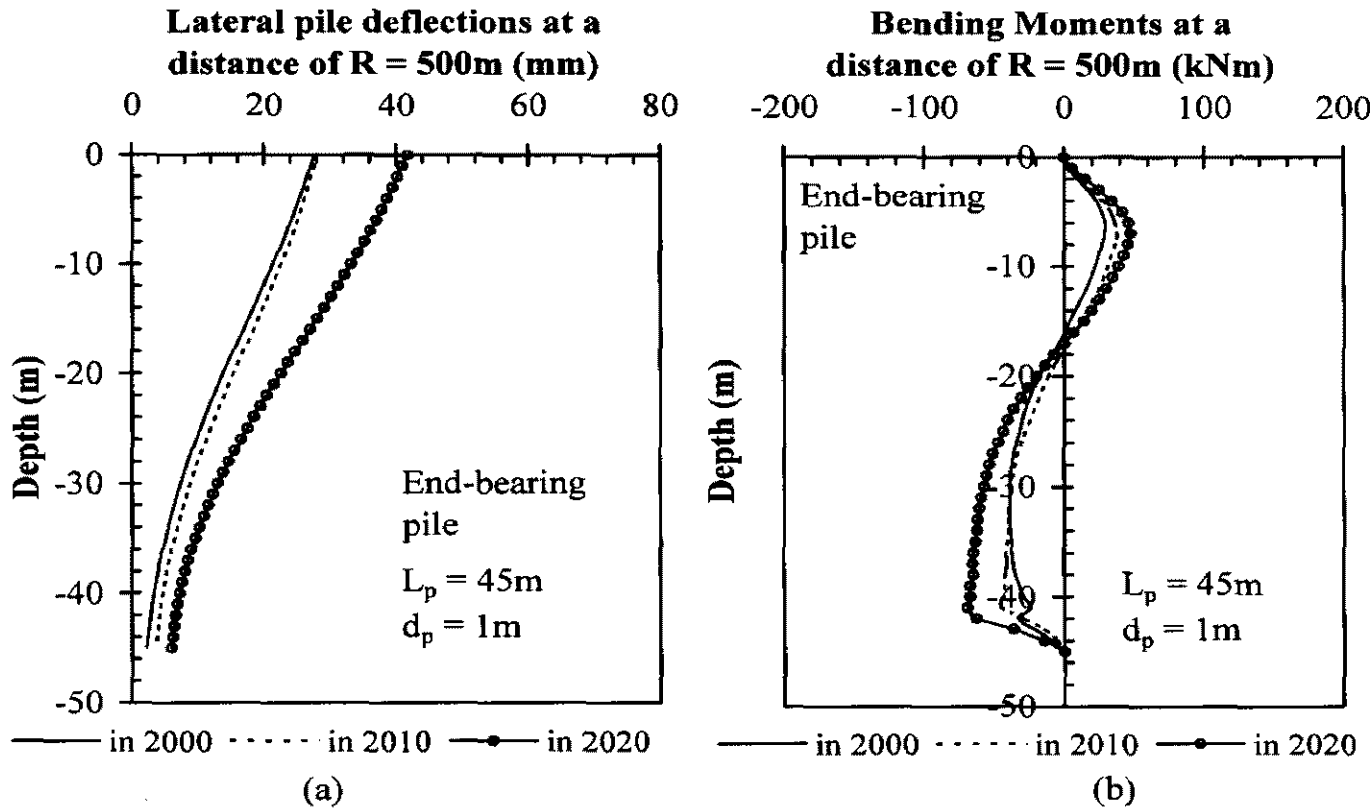


Figure 6.11: Lateral deflection and induced bending moments in a single free head pile at a distance of 500m from the assumed symmetrical axis.

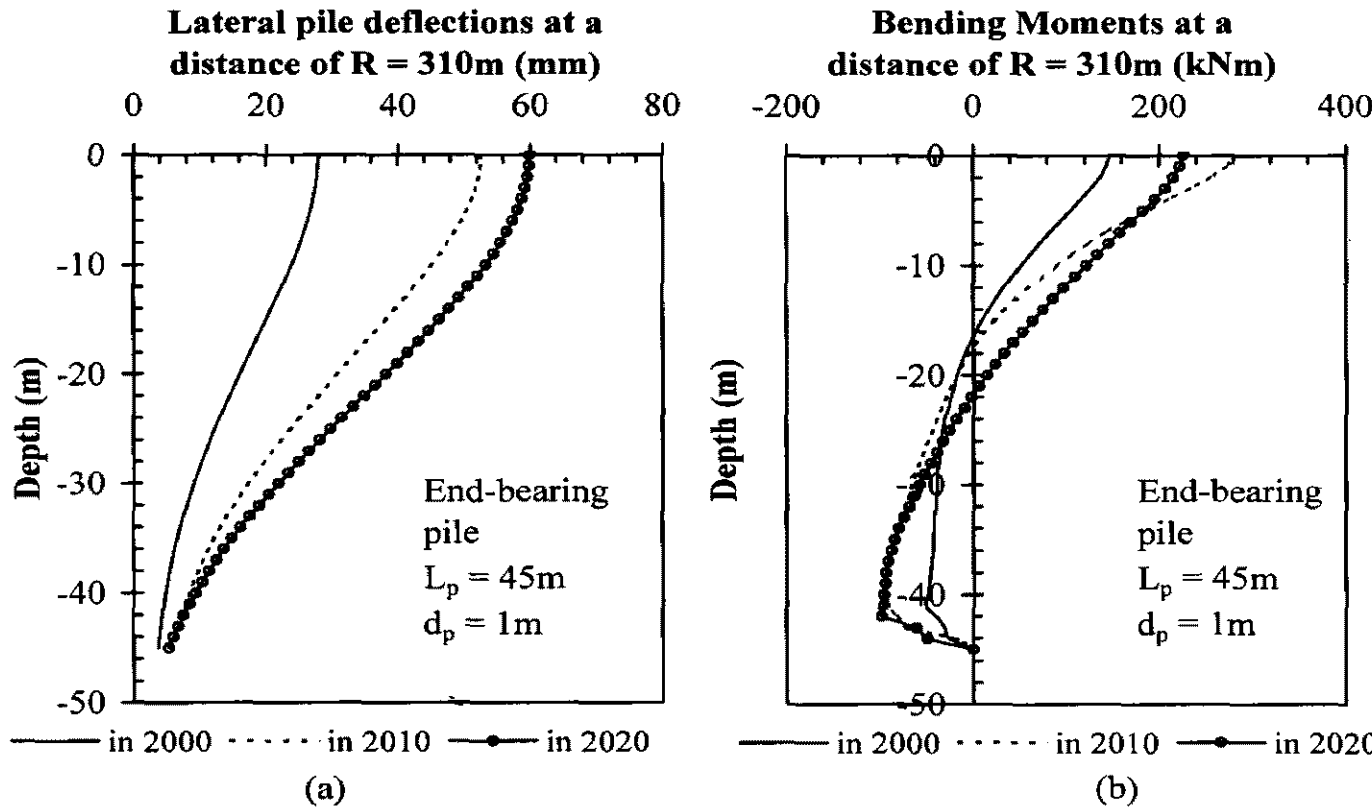


Figure 6.12: Lateral deflection and induced bending moments in a single fixed head pile at a distance of 310m from the assumed symmetrical axis.

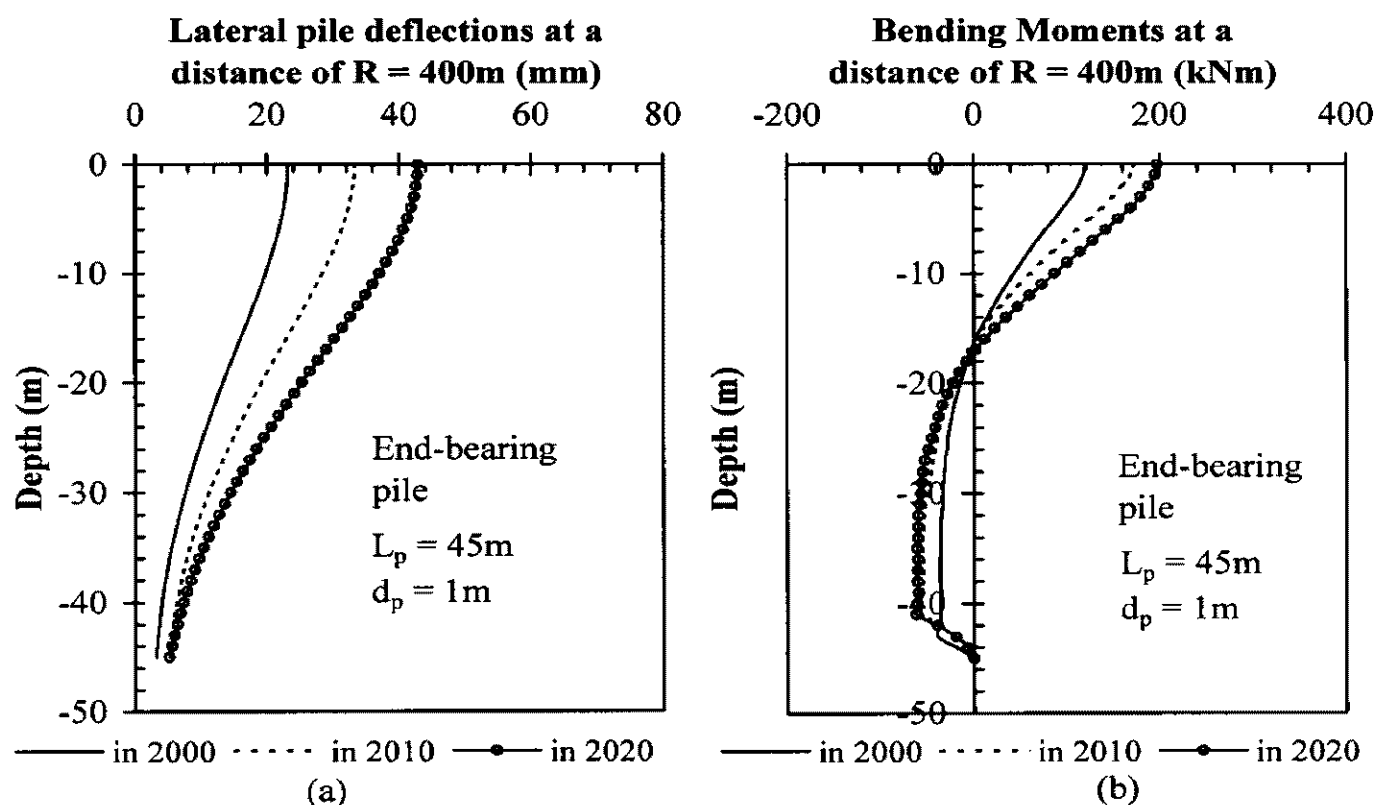


Figure 6.13: Lateral deflection and induced bending moments in a single fixed head pile at a distance of 400m from the assumed symmetrical axis.

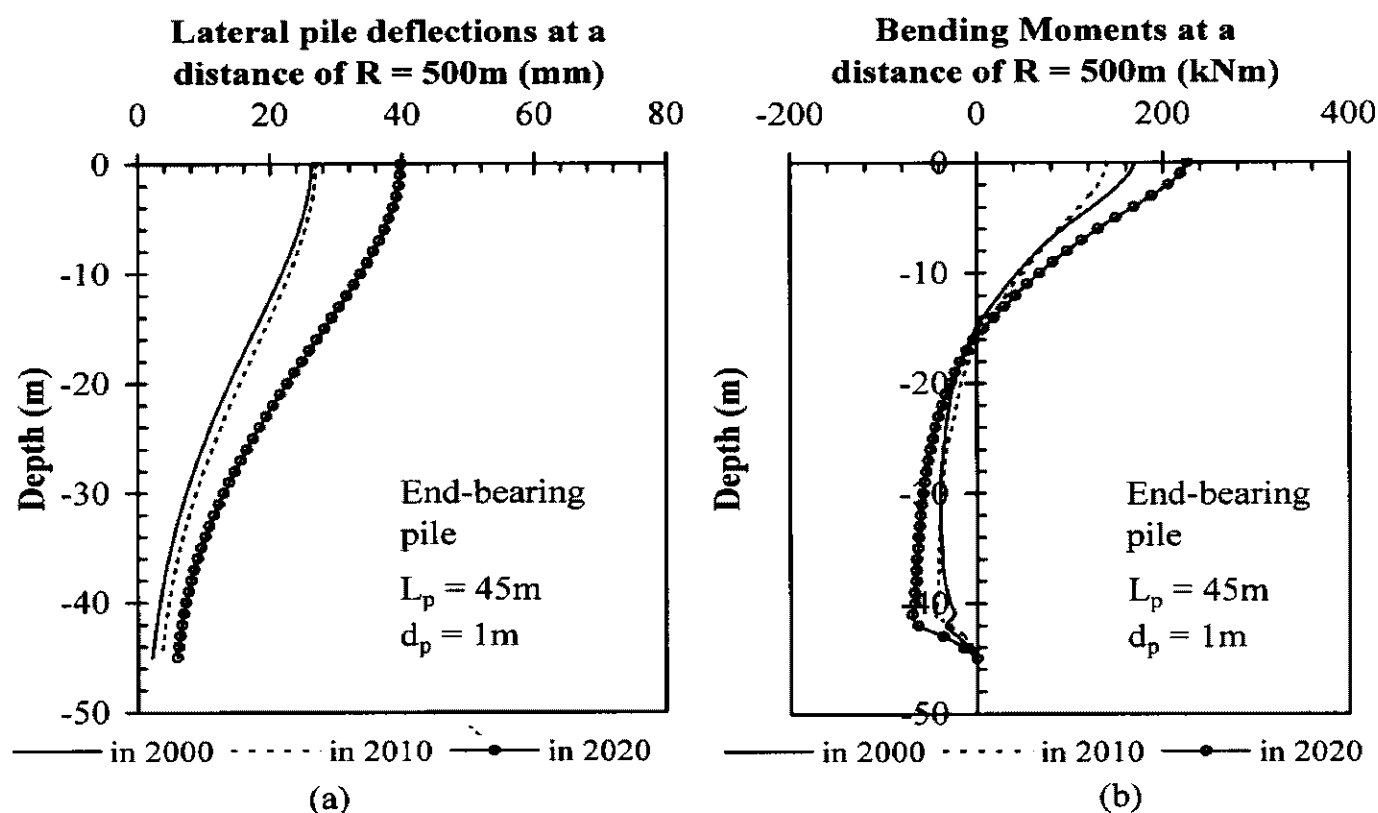


Figure 6.14: Lateral deflection and induced bending moments in a single fixed head pile at a distance of 500m from the assumed symmetrical axis.

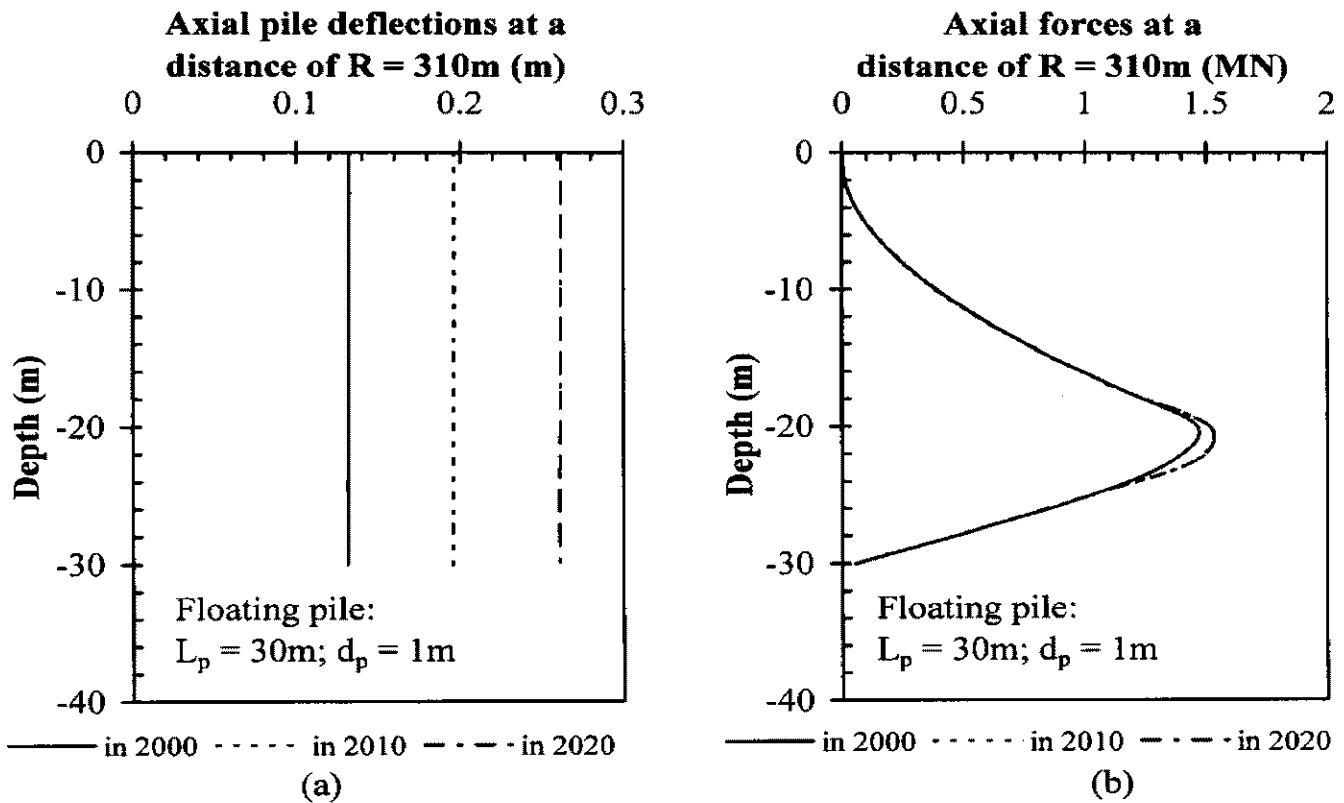


Figure 6.15: Axial deflection and axial force in a single floating pile at a distance of 310m from the assumed symmetrical axis of the Phap Van ground water pumping station.

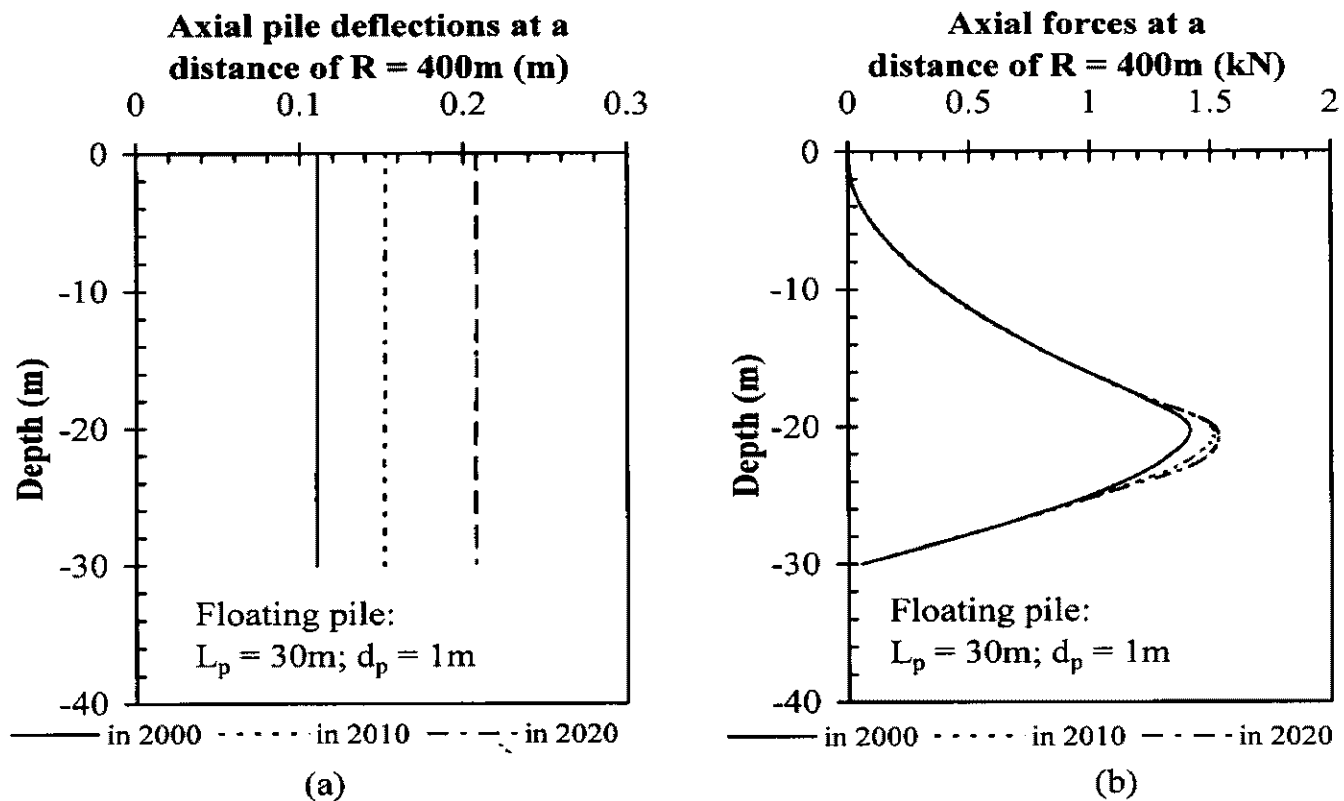


Figure 6.16: Axial deflection and axial force in a single floating pile at a distance of 400m from the assumed symmetrical axis of the Phap Van ground water pumping station.

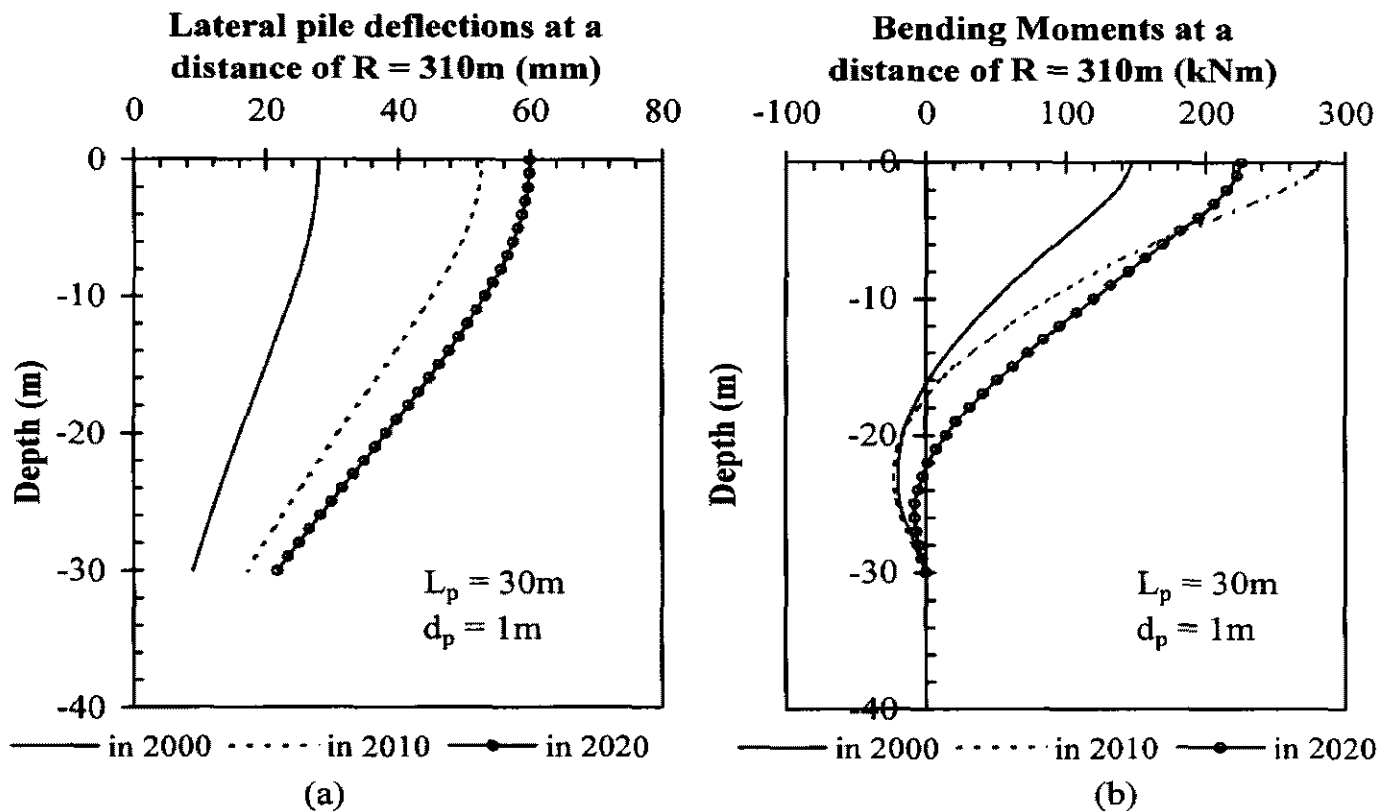


Figure 6.17: Lateral deflection and induced bending moments in a single floating fixed head pile at a distance of 310m from the assumed symmetrical axis.

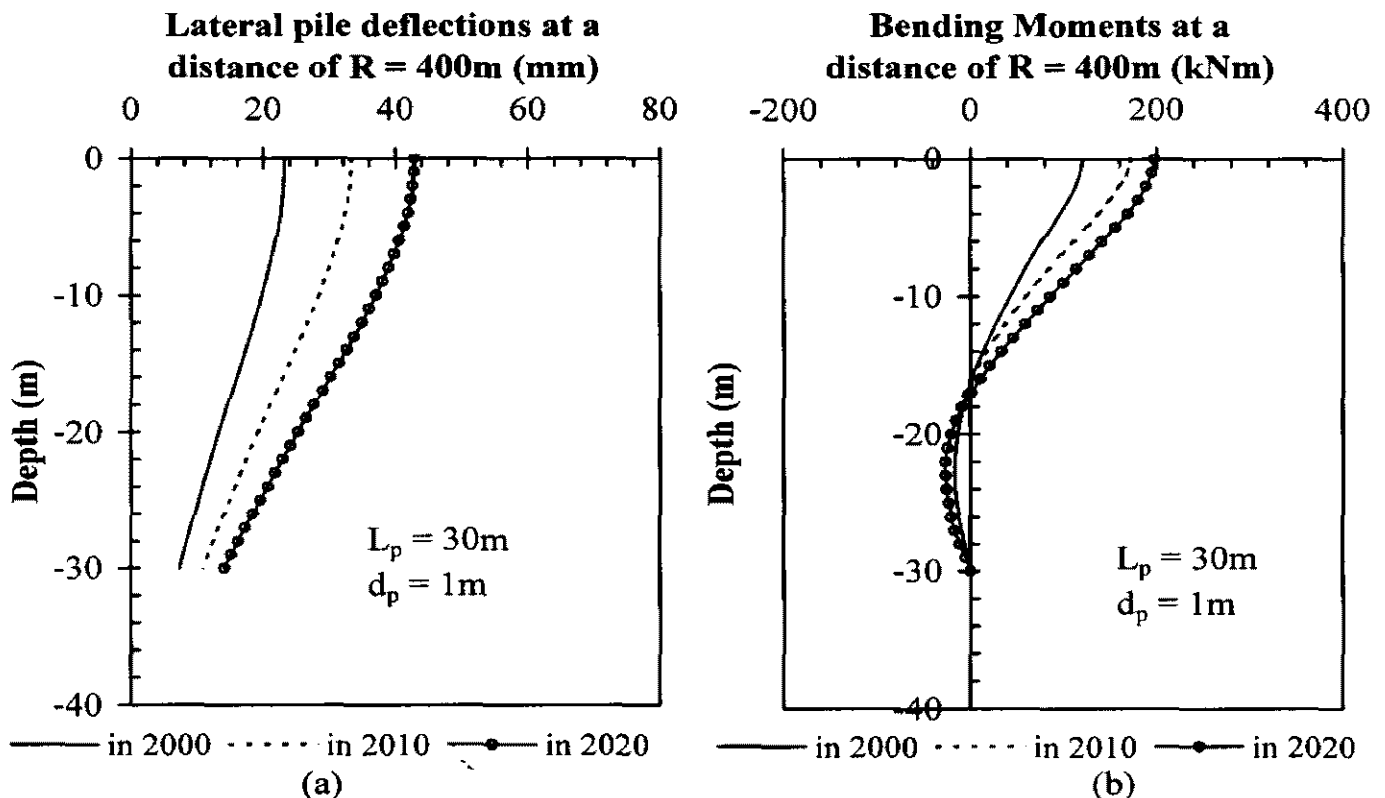


Figure 6.18: Lateral deflection and induced bending moments in a single floating fixed head pile at a distance of 400m from the assumed symmetrical axis.

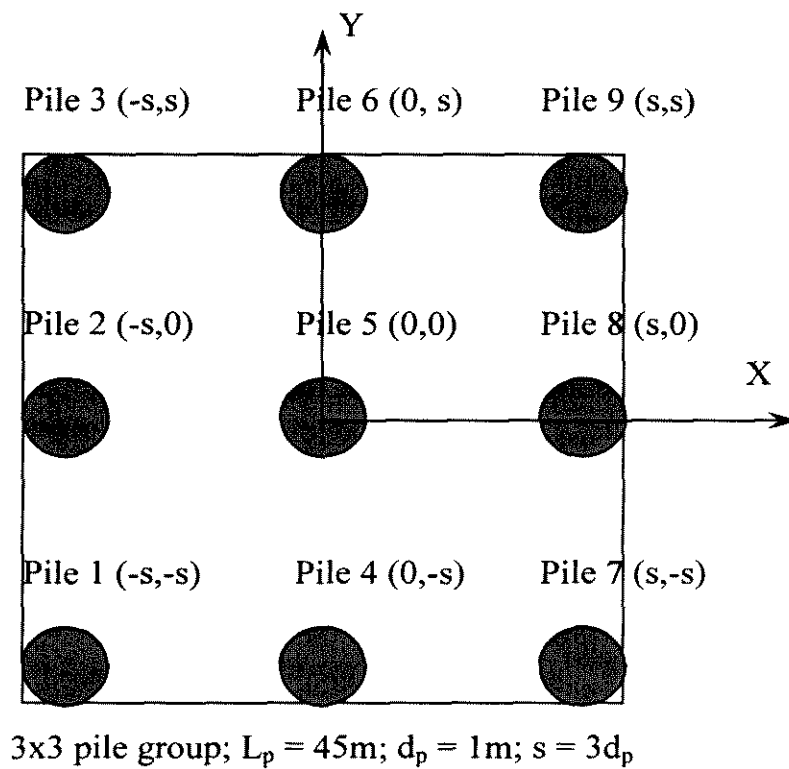


Figure 6.19: 3x3 pile group with a spacing of 3 times pile diameter.

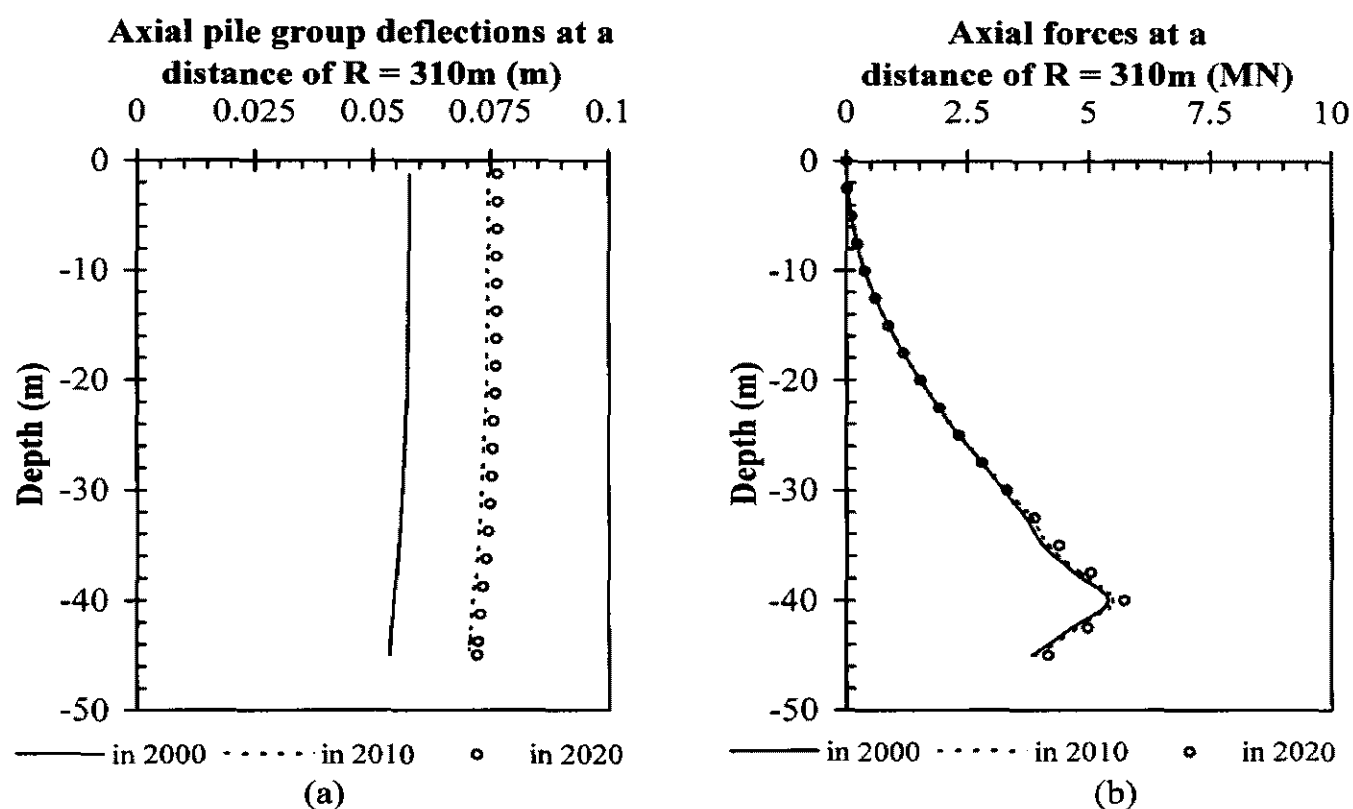


Figure 6.20: Axial deflections and axial forces in a 3x3 pile group at a distance of 310m from the assumed symmetrical axis of the Phap Van ground water pumping station.

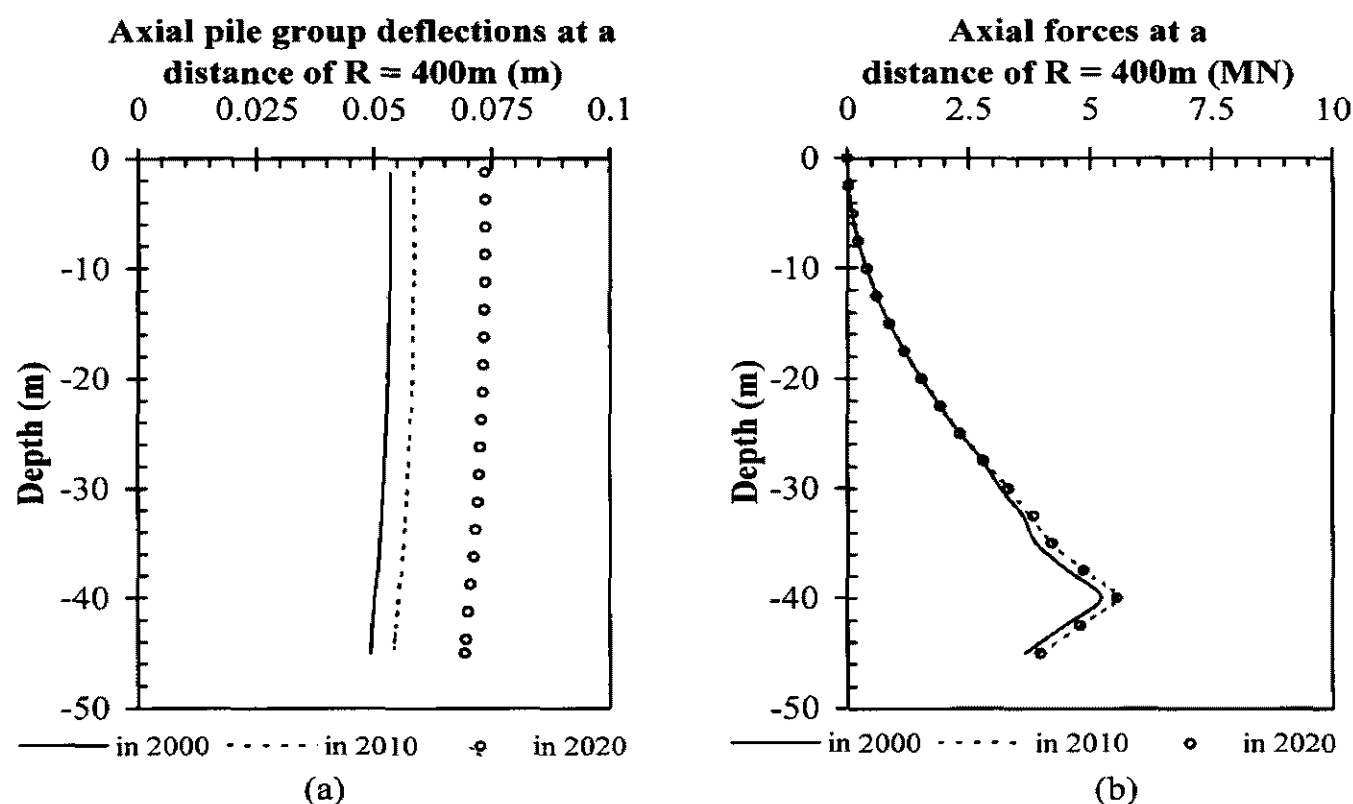


Figure 6.21: Axial deflections and axial forces in a 3x3 pile group at a distance of 400m from the assumed symmetrical axis of the Phap Van ground water pumping station.

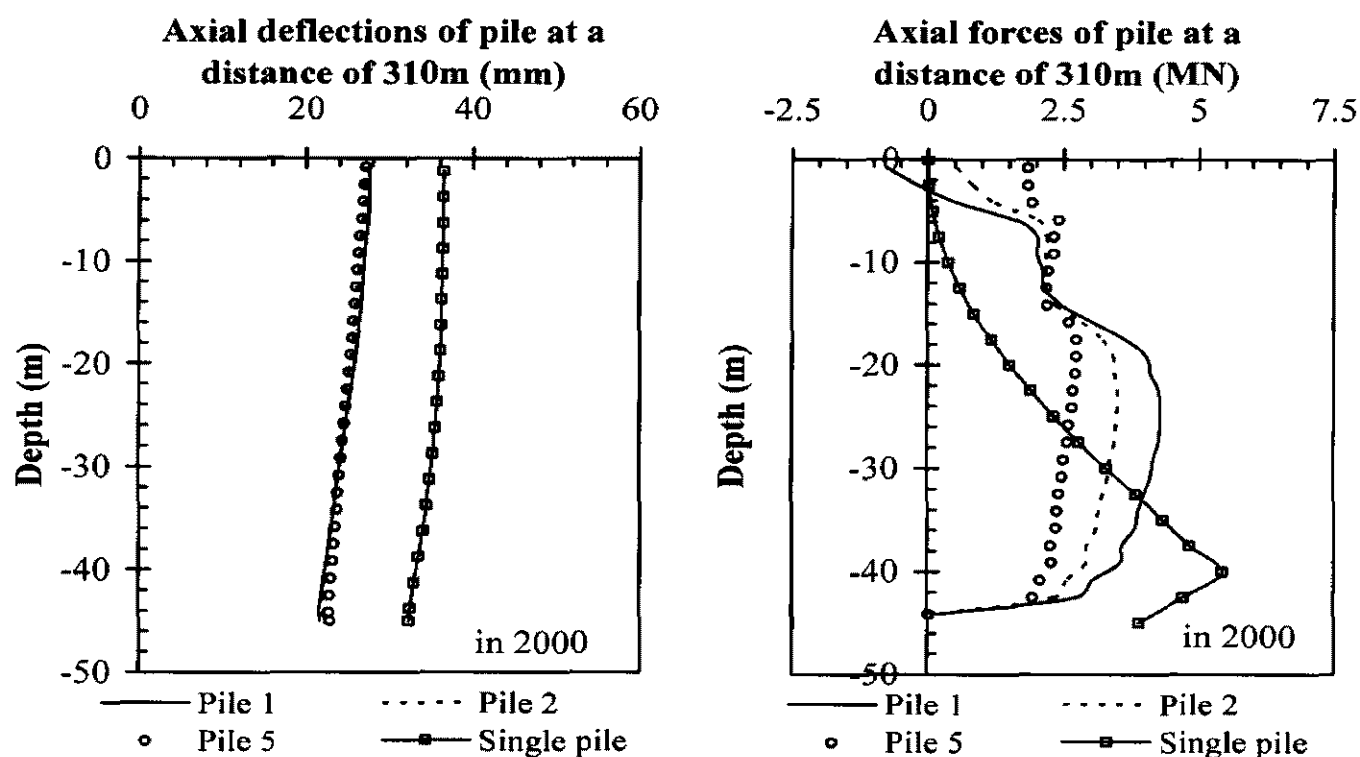


Figure 6.22: Axial deflections and axial forces in 3x3 pile group at a distance of 310m away from the assumed symmetrical axis (in 2000, Phap Van case).

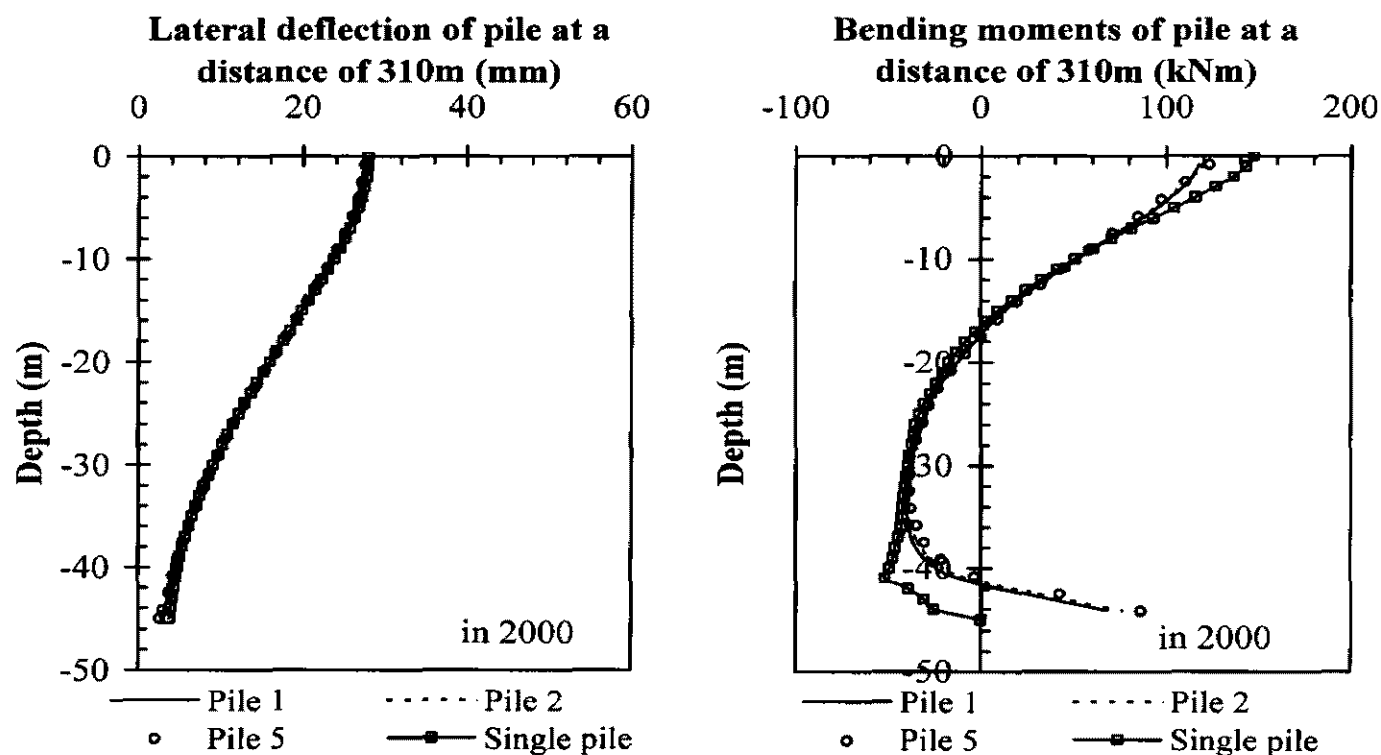


Figure 6.23: Lateral deflections and bending moments in 3x3 pile group at a distance of 310m away from the assumed symmetrical axis (in 2000, Phap Van case).

Note: the pile group was analysed by employing the GEPAN program; the single pile results were obtained from PIES and PALLAS programs in Figures 6.22 and 6.23.

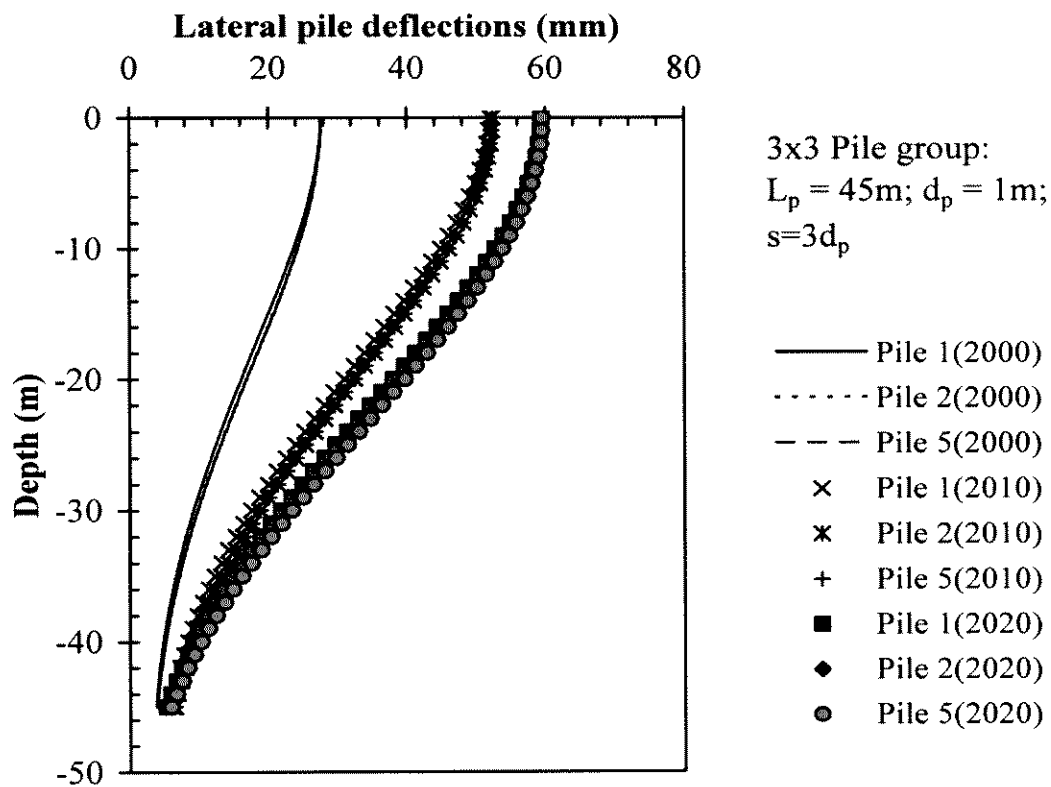


Figure 6.24: Lateral deflections of the corner, edge, and centre piles in a 3x3 pile group at a distance of 310m from the assumed symmetrical axis.

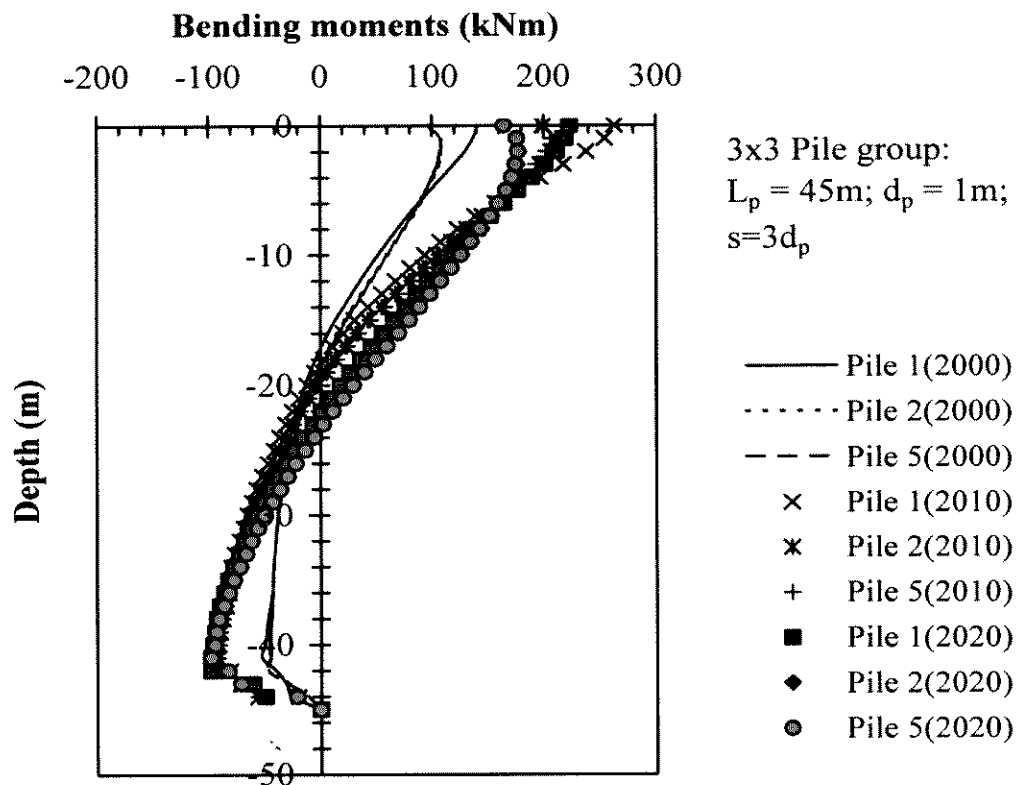


Figure 6.25: Bending moments in the corner, edge, and centre piles in a 3x3 pile group at a distance of 310m from the assumed symmetrical axis.

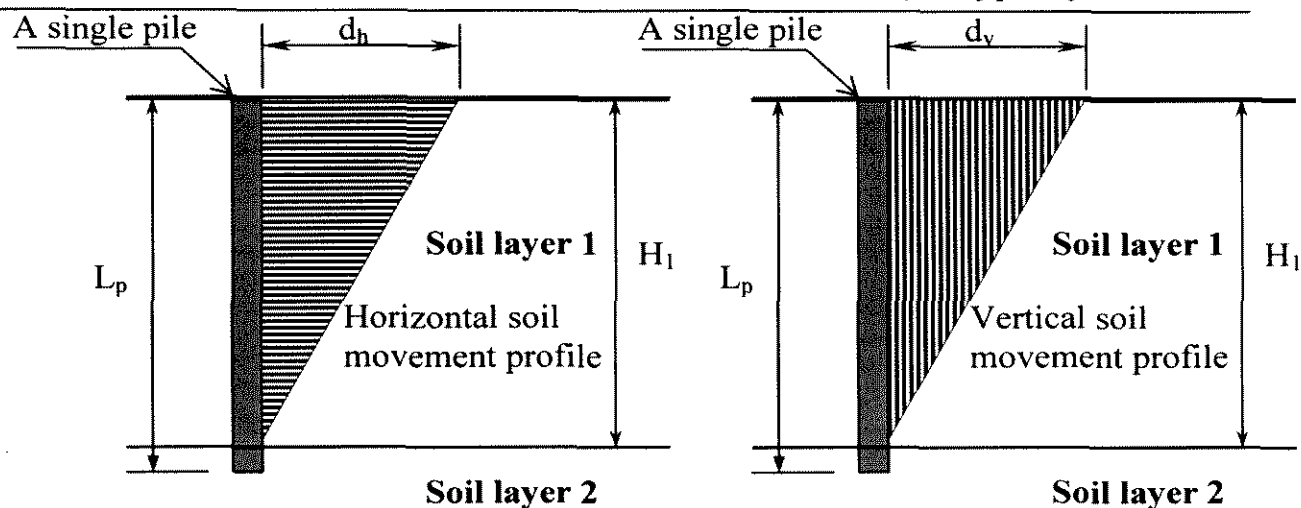


Figure 6.26: Soil movement profiles.

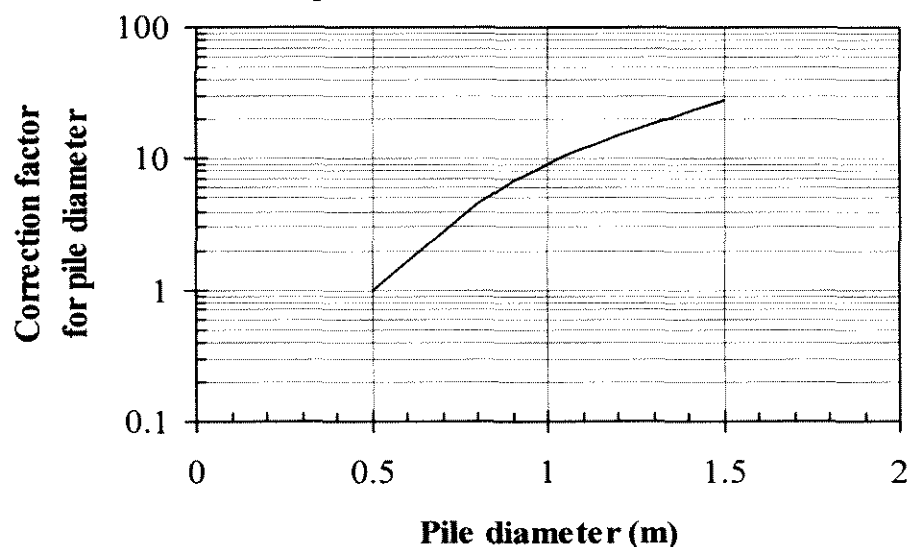


Figure 6.27: Bending moment correction factor for pile diameter of a single free head end-bearing pile.

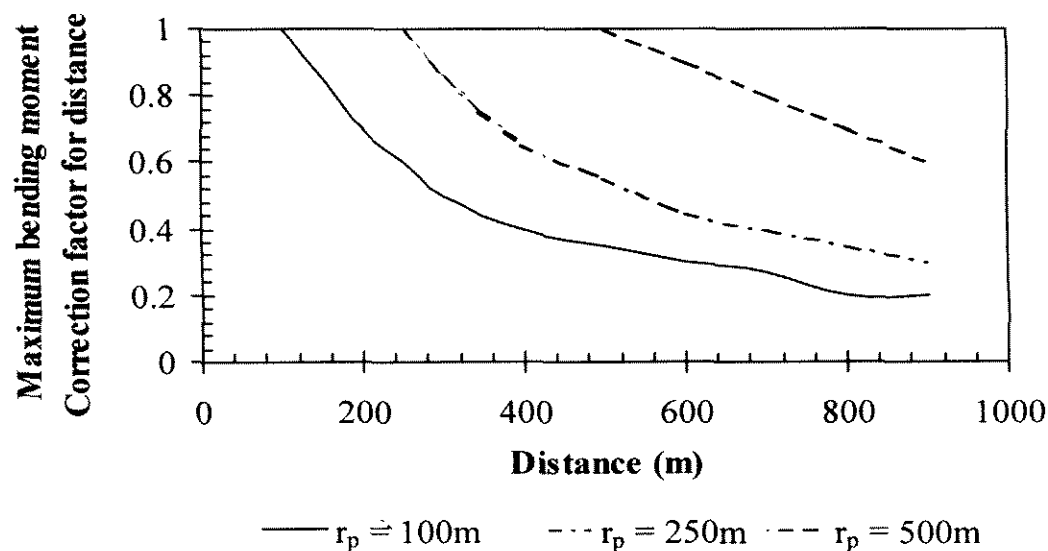


Figure 6.28: Bending moment correction factor for distance of a single free head end bearing pile from the assumed symmetrical axis of the nearby groundwater pumping area.

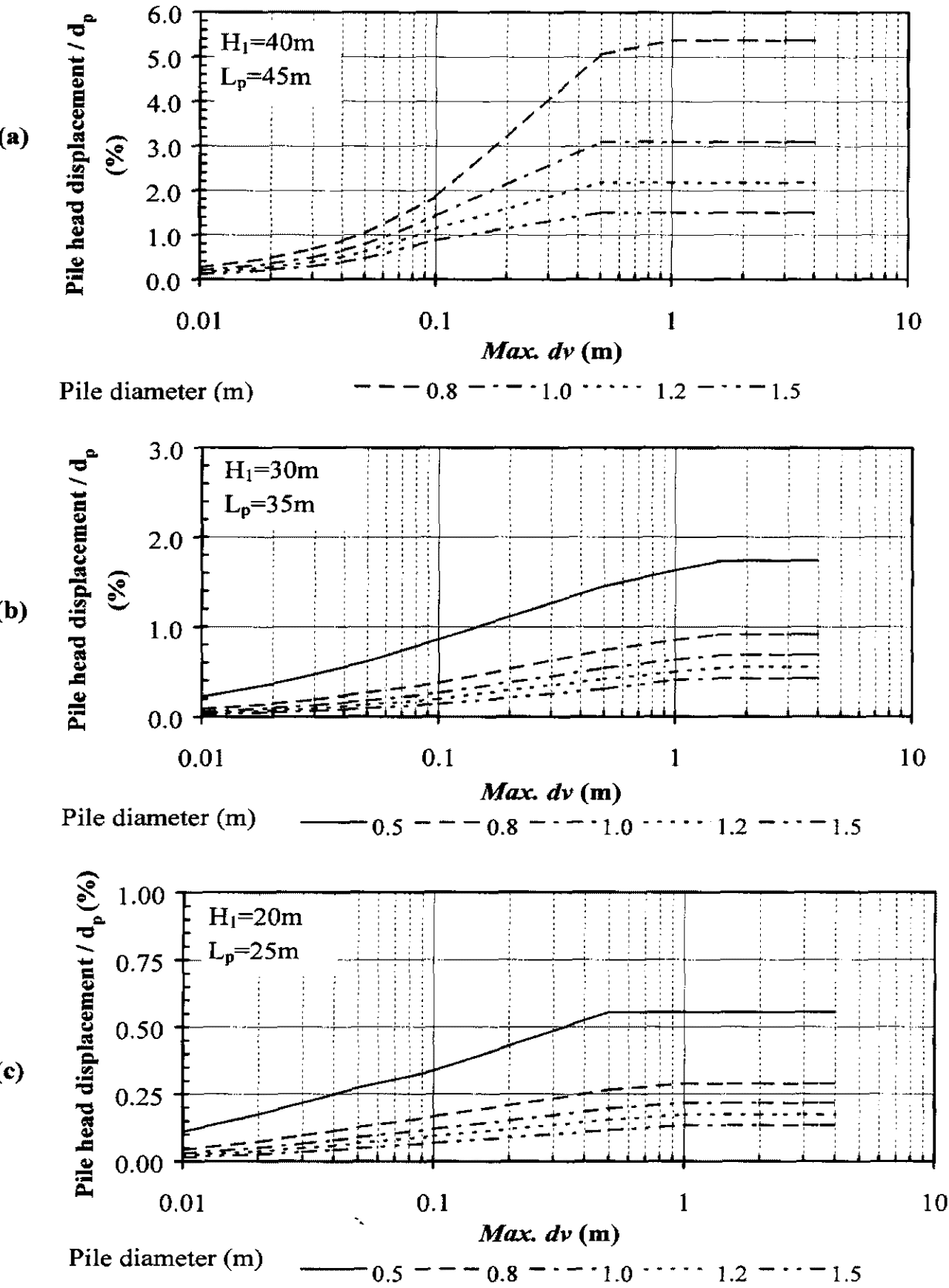


Figure 6.29: Pile head displacement of a single end-bearing pile subjected to a triangular shaped vertical soil movement.

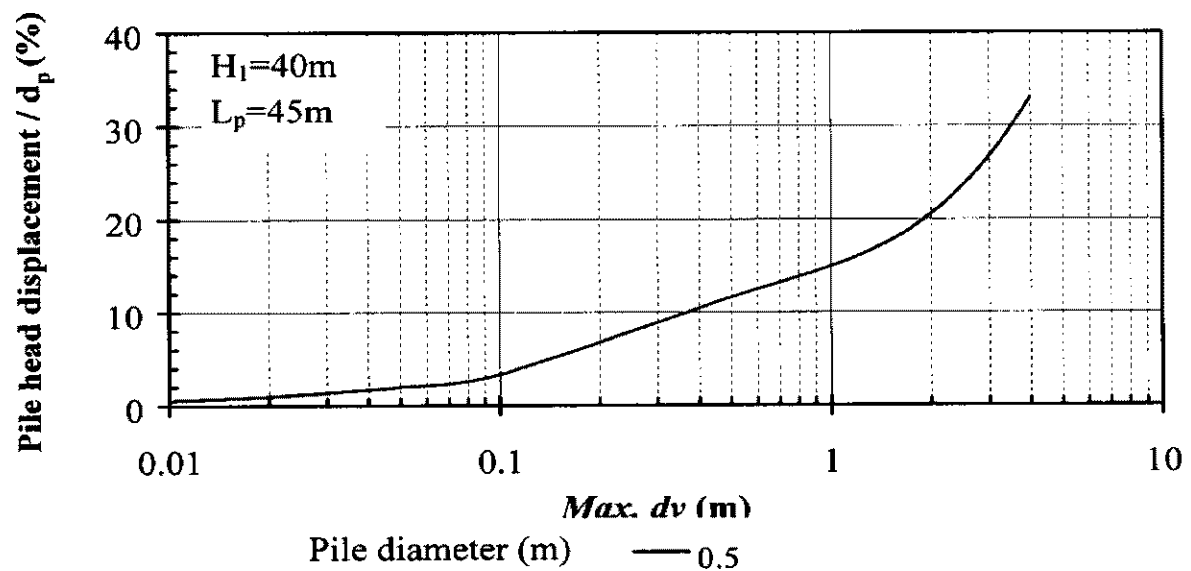


Figure 6.30: Pile head displacement of a single pile subjected to a triangular shaped vertical soil movement.

CHAPTER 7

NUMERICAL ANALYSIS OF EXCAVATION INDUCED SOIL MOVEMENTS

7.1 INTRODUCTION

The purpose of this chapter is to verify the capability of finite element program EF4 so that it can be used to predict excavation induced soil movement. In the first part of this chapter, a brief review of available approaches for excavation induced soil movement prediction is presented. The available approaches are classified in three categories that are empirical, total stress, and effective stress approaches.

In the latter part of this chapter, the finite element program EF4 is employed to analyse soil movement due to excavation. Some typical excavations have been chosen which are an underground garage at Post Office Square, Boston, the excavations along Davison Avenue and Rankin Street, San Francisco, and a large cavitation channel excavation, in Memphis Tennessee. The three excavations presented here were made in different soil conditions and also different excavation procedures were used.

7.2 EXCAVATION INDUCED SOIL MOVEMENT REVIEW

7.2.1 EMPIRICAL RULES

The most commonly used practical approach for analysing excavations was presented by Peck (1943). Based on studies of the Chicago Subway Excavation, he presented a rule for computing earth loads on bracing in clay, which was called the trapezoidal rule. In the state-of-the-art paper presented by Peck (1969) and another paper by Peck (1973), the empirical rule was again used but was improved due to additional data on excavation becoming available. In this study, lateral deflections, surrounding ground settlements, basal heaves and lateral earth pressures were considered. Two types of walls (sheet pile walls and soldier pile walls) and different soil conditions (sand, soft to medium clays, and stiff clays) were also discussed. Figure 7.1 shows results (based on data from measurement) for the surrounding ground settlement, divided by excavation depth, H , plotted against the distance from the wall also divided by excavation depth. Three categories of settlements corresponding to three soil conditions (sand, and soft to hard clay, very soft to soft clay with or without appearance of basal heave) were presented. Figure 7.2 shows Peck's classification of excavation behaviour in clay. In this figure, the author presented a relationship for the

behaviour of excavations with the undrained shear strength of the soil at the base of the excavation and the factor of safety. The behaviour of excavations was classified into different zones that varied from elastic responses to large soil movements due to excavation. The problem of stability of strutted excavations in clay was discussed by Terzaghi (1943) and Tschebotarioff (1951). However these were only limited to shallow excavations. The paper presented by Bjerrum and Eide (1956) for the stability of strutted excavations in clay was based on the bearing calculations for foundations on clay. A formulation of the critical depth of an excavation was given and the reality of this formula was then investigated by comparisons with 14 excavations. Though it gave a general solution for basal heave analysis of deep excavations, it could not indicate how much the danger of a basal failure was reduced as sheet piles were driven below the bottom of the excavation.

The classical earth pressure theories such as those of Rankine and Coulomb may be used for analysing braced excavations in soft clay. This is because these methods allow other factors to be taken into consideration such as layered soil conditions, surcharge load near the excavation and unlevel ground outside the bracing. The classical methods usually gave total earth loads, which are smaller than the results given by Peck's methods.

In recent years, the database of the measured soil movements due to excavation has been enlarged. More and more effort has been given to collecting measured data and to analysing these to find a general trend for soil movements due to excavation. These contributions broaden the knowledge of engineers and allow a better understanding of excavation behaviour. Wong *et al.* (1997) collected data for soil movements of excavations for a depressed expressway in Singapore. These authors found that for the case where soft soil with thickness less than 0.9 times excavation depth was lying above a hard soil layer, vertical and horizontal soil movements are about 0.35% and 0.5% of excavation depth respectively. However, if the soft soil layer thickness is less than 0.6 the excavation depth, then the soil movements are about 0.2% and 0.35% correspondingly. They also discovered that using an anchored excavation rather than a propped one can reduce soil movements due to excavation and the first prop can be used effectively to reduce soil movements when it is placed close to the ground surface. Based on 10 high quality case histories of excavations in soft Taipei soil, Ou *et al.* (1993) suggested general graphs to predict ground surface settlements due to

excavations in soft soil, especially for the Taipei soft soils. In his prediction, settlement of the ground surface due to excavation extends to a considerable distance from the excavation wall and those buildings and public facilities beyond that area are not influenced by the settlements. He called this area the apparent influence range (AIR). AIR also depends on excavation depth and is defined by the equation

$$AIR = (H_0 + D_w) \tan \left(45 - \frac{\phi}{2} \right) \quad (7.1)$$

where ϕ represents the internal friction angle, H_0 is the final excavation depth and D_w is the penetration depth of the retaining wall.

Figure 7.3 shows the relationship between the apparent influence range and height of the retaining wall. The maximum horizontal wall deflection was found to be between 0.2% and 0.5% of the final excavation depth with the location of the maximum horizontal wall close to the excavation depth. These values are larger than Clough and O'Rourke's (1990) results (0.2% of excavation depth) and smaller than Peck's (1969) data, which is 0.1% of excavation depth. Hsien and Ou (1998) proposed a rule to predict ground surface settlement profiles caused by excavation. The authors identified two general types of settlement profiles induced by excavation that are the spandrel type and the concave type. Figure 7.4 shows the proposed method for predicting spandrel and concave settlement profiles.

A paper written by Long (2001) presented over 300 case histories of ground movements due to deep excavation world wide to build up a database. The purpose of this database is, from a large volume of collected data, to examine general trends and patterns of soil movements due to excavation. Though some of the data will be influenced by ancillary construction activities, as recognised by the author, this database is a useful general tool for engineers who want to understand soil behaviour due to excavation. The database is divided into four main sets with different types of soil as shown on Figure 7.5. Figure 7.6 shows the reasons for excessive deflection determined from the data. In about 55% of the considered cases (36 cases) excessive deflection is due to the initial cantilever stage and the flexibility of the retaining wall, however, in up to 25% of the considered cases reasons could not be assigned.

7.2.2 NUMERICAL ANALYSIS – TOTAL STRESS APPROACHES

With the development of numerical methods, especially the finite element method, gradually, more and more factors that influence the results of excavation analysis and the performance of excavations can be taken into account. Bjerrum *et al.* (1972) used both theoretical methods and finite element methods to investigate the earth pressures behind a wall. It was found that the earth pressure and its distribution were greatly influenced by the flexibility of the retaining structure. In addition it was found, combined with the observed data, that the total earth pressures on excavation systems had a relation to the shear strength of the soil. Since many excavations were carried out between existing buildings, the movements of the supporting system and the adjacent ground were more critical and gradually became more important issues than stability. A simplified method for predicting movements of braced excavations in soft to medium clay is presented by Mana and Clough (1981). The finite element technique provides a method by which effects of variations in parameters such as soil shear strength, soil stiffness, wall and strut properties and excavation geometry can be assessed. Figure 7.7 shows the relationship between factor of safety against basal heave and non-dimensionalized maximum lateral wall movements. The results of analyses were compared to the measured data. To some degree, most natural clay is anisotropic, and Clough and Hansen (1981) used the finite element method to investigate the effects of anisotropy on basal heave potential, and movements. The research indicated that the basal heave factor of safety may be as much as 10-30% or even 50% above the actual one for clays that were strongly and moderately anisotropic. Consequently, lateral wall movements and ground surface settlements can be larger than expected, which may explain the unexpectedly large movements and strut loadings noted in the past. Stability of excavations was once again considered in a paper by Britto and Kusakabe (1984). In this paper, the wall failure mechanism for unsupported and supported walls for both plan strain and axisymmetric cases were presented. Base failure modes for plane strain and axisymmetric cases were also considered. Bound theorems of limit analysis were used for producing stability charts for both wall and base failure. It was found that slurry support is more effective than rigid lateral support for axisymmetric excavations while the possibility of base failure was greatly increased for excavations with rigid lateral support.

Commonly the theories of linear elasticity have been employed in predicting soil deformation and stresses. Nevertheless, field and laboratory studies show that even at a very small strain, many soils exhibit non-linear stress-strain behaviour.

Jardine *et al.* (1986) conducted a detailed study of the influence of non-linear stress-strain characteristics in soil-structure interaction, which can be used to investigate the interaction between the braced wall and soil of a strutted excavation. The results of the analysis were compared with the results from a linear elastic soil model. The phenomenon of the small strain characteristics was first noted in the field by Burland and Hancock (1977). Simpson (1992) introduced a soil model to describe this behaviour. In a state-of-the-art paper by Simpson (1992), the behaviour of retaining walls on ground movements for many deep excavations were studied by applying both conventional and the new small strain models of soil behaviour. The calculated results then were compared with field measurements and these showed good agreement. The new method took into account that the small strain characteristics can be used for both normally consolidated and over consolidated clays. Potts and Fourie (1984) used a finite element method that employed an elastic-plastic constitutive law to model the soil behaviour. Those distinctions such as the type of construction or of the stress state in the soil prior to construction that had not been discussed before were investigated in this paper. The results of this study had serious practical implications for diaphragm or secant pile walls installed in stiff over-consolidated clays with high K_0 values. Wong and Broms (1989) examined the lateral deflections of strutted or anchored sheet-pile walls in clay using finite element methods. They proposed a simple procedure to eliminate wall deflection and compared the obtained results to the FEM results that compared favourably with field records and published results.

Borja *et al.* (1989) proposed a method, which produced a unique solution for any number of excavation stages. Actually, the uniqueness principle for the simple elastic cases was presented before by Brown and Booker (1986). In this method, the elasto-plastic finite element equations are derived from a variational formulation which accounts for a time-varying problem domain and boundaries. Borja (1990) also included finite element equations for the modified Cam-Clay model called a stress-point integration algorithm that was both accurate and stable.

7.2.3 NUMERICAL ANALYSIS – EFFECTIVE STRESS APPROACHES

Duncan and Clough (1971), Clough *et al.* (1972) and Christian and Wong (1973) were the first to apply finite element methods to braced excavation analysis using effective stress models. In these analyses the significant achievements were considering the pore pressure changes during excavation and the non-linear, stress-strain-strength properties of saturated soils. The simulation of the steady-seepage conditions during excavation were first determined by drawing a flow net based on the observed data of Duncan and Clough (1971). Then the corresponding water pressure and effective stress in the soil were established. Instead of drawing a flow net to calculate the pore pressure and the effective stress, Osaimi and Clough (1979) developed a finite element program for analysing one-dimensional excavation in homogeneous clay, in which, for the first time, the coupling between consolidation theory and the soil deformations was used. From this study, it was observed that zero consolidation occurs when the soil has a permeability of 1.0×10^{-10} m/s even if the slowest soil removal rate of 0.03 m/day is applied. Figure 7.8 presents a relationship between soil deformation and soil permeability. By employing a fully coupled finite element analysis, Karlsrud (1986) showed the effect of dewatering on earth pressures and the required toe depth of a wall. He also proposed a method to allow estimation of ground settlements from predicted lateral wall movements. The proposed method is presented in Figure 7.9. Eisenstein and Medeiros (1983) studied a deep retaining structure in till and sand, while the stress-strain behaviour of glacial till and dense preglacial sand has been studied by Medeiros and Eisenstein (1983). The stress-strain behaviour was then used as the soil model for a finite element program, which was developed by Medeiros (1979). Based on their results, they found that the calculated lateral pressure distribution and magnitude were different from the pressure distribution diagram normally used in design practice. Bolton *et al.* (1989) carried out a centrifuge test which modelled cantilever retaining walls embedded in over-consolidated clay. The drainage of a heavy fluid in flight was employed to simulate the effects of excavation, following the establishment of a high initial ground water level. The test results were compared with finite element analysis results for the excess pore pressure, soil displacement and bending moments in the wall and prop faces.

Finno *et al.* (1991) presented parametric studies based on a coupled finite element formulation. In these studies, effects of constitutive soil models (3 models which include an anisotropic soil model were used), boundary conditions, and details of the construction process were considered. The results of an analysis of boundary displacement and stress-controlled simulation of construction were compared with observed performance of a 40 ft deep cut through saturated clay.

Borja (1992) introduced a fully undrained analysis of excavation, which could investigate the influence of fluid flow on the strength and deformation behaviour of unsupported and braced excavations. The study showed that the influence of fluid flow on the deformation behaviour of an excavation is significant and should be incorporated into the analysis.

Hsi and Small (1992a, and b) also presented a fully coupled finite element analyses of excavation and produced a comparison between computed and observed results of excavations. Hashash and Whittle (1996) investigated the effects of wall embedment depth, support conditions, and stress history profile on the undrained deformations of a braced diaphragm wall in a deep clay deposit. The MIT-E3 soil model, which can describe small-strain nonlinearity and anisotropic stress-strain-strength were employed. The study gave a better understanding of the ground movements caused by deep excavations, where the diaphragm wall is embedded in a very deep layer of soft clay.

7.3 CASE STUDIES

7.3.1 CASE 1: UNDERGROUND PARKING GARAGE AT POST OFFICE SQUARE, BOSTON.

The underground parking garage at Post Office Square (POS), in the heart of the financial district of Boston was built in an area of 6880 m² (approximately 116x61m). More than 10 buildings with foundations consisting mostly of shallow footings surrounded the seven-storey underground parking station. The garage excavation was carried out to an elevation of -16.8m relative to the Boston Official Base Datum that meant the excavation depth was in total about 22.9m. The excavation-supported system was a cast-in-situ concrete diaphragm wall (0.9m thick) extending down to the bedrock (-21.0m elevation) of Boston City. In the finite element analysis the

permeability of the concrete diaphragm wall was assumed to be zero. The interior steel columns (H-section) were used for supporting the structure of the seven-story garage. Both the interior steel columns and the cast-in-situ concrete diaphragm wall had been installed before the excavation was carried out. The details of this underground garage structure and the excavation support system can be found in Whittle *et al.* (1993). Figure 7.10 (a) shows the garage structure while Figure 7.10(b) shows the initial pore water pressure used as input data for the Finite Element program.

Figure 7.11 presents the finite element mesh used to model the excavation at POS. To simplify the analysis, the excavation simulation is divided into 9 stages that correspond to the number of excavation stages presented by Whittle *et al.* (1993). The number of stages and the period of time for each excavation stage are provided in Figure 7.12. The numbers of these stages are similar to the original stage numbers presented by Whittle *et al.* (1993).

The soil conditions are presented in Tables 7.1(a) and (b). The subsurface profile at the site consisted of six individual strata, which are an average of conditions given in the site investigation report. Because of the availability of the soil parameters two soil models are utilized in the analysis. The first and second soil layers, fill and Boston blue clay, are modelled by a modified Cam clay model. The other soil layers are modelled as elastic material.

Table 7.1(a): Soil properties of the site at POS used for finite element analysis.

Stratum	K_0	OCR	Specific yield ratio	Permeability ($k_x=k_y$)	Total unit weight
			s_y	(m/day)	(kN/m ³)
Fill	0.5	1.5	0.002	8.64E-01	19.6
Boston Blue Clay	1.22	4.0	0.002	4.32E-04	19.6
Sand	0.4	-	0.002	8.64E-02	19.6
Till	1.0	-	0.002	8.64E-03	20.4
Weathered argillite	1.0	-	0.002	8.64E-03	22.0
Intact argillite	1.0	-	0.002	8.64E-03	23.6

Table 7.1(b): Soil properties of the site at POS used for finite element analysis.

Stratum	Soil Model	Young's modulus	Modified Cam Clay parameters			In Situ Void ratio	Poisson's ratio
		(MPa)	λ	κ	M	e_o	ν
Fill	MCAM	-	0.25	0.05	1.5	1.5	0.3
Boston Blue Clay	MCAM	-	0.184	0.20	1.35	1.12	0.3
Sand	ELAS	1.0E+05	-	-	-	-	0.3
Till	ELAS	7.5E+05	-	-	-	-	0.3
Weathered argillite	ELAS	5.0E+05	-	-	-	-	0.3
Intact argillite	ELAS	2.5E+06	-	-	-	-	0.3

Figures 7.13 to Figure 7.20 show the finite element analysis results for the lateral wall deflections, the lateral and vertical soil movements, the surface settlements, and the piezometric elevation. The computed results have been compared with the measured data and Whittle *et al.*'s (1993) results as well.

The lateral wall deformations are presented in Figures 7.13 (a) to (c) for the excavation stage 10 (relative level +1m), stage 19 (relative level -7.52m), and stage 28 (relative level -15.33m). At the first stages of the excavation (e.g. stage 10) the wall deflected in a cantilever mode. This could be due to the fact that at each excavation stage, soil excavation was carried out first and followed by the installation of the concrete rooftop or floor. This delay could cause the cantilever deflection mode of the wall at the initial stages. At later excavation stages the maximum lateral deformation of the wall moved downward as the excavation deepened. Figure 7.13 (b) shows the maximum lateral wall deflection at stage 19 of the excavation (the excavation base elevation was at -7.52m) to be at an elevation of about -5m. As the excavation approached the final depth (at stage 28 of the total 32, Figure 7.13(c)), the measured lateral wall movement seemed to be similar to the measured values of the wall deflections at stage 19, Figure 7.13(b). However, the maximum measured lateral wall deflections at stage 28 moved downward from an elevation of -5m at stage 19 to

an elevation of -7 m to -8 m. The finite element results for the lateral wall deformations of about 30mm were stable at the elevation of -5 m through stage 19 to stage 28. The lateral soil movements were measured at distances of 4m, 9m, and 14m away from the centre line of the wall, and so there are figures for each of these locations. There is good agreement between the calculated results and the measured ones, however at a distance of 14m (stage 28), Figure 7.14(h), from the excavation the predicted lateral soil movements are well over predicted.

Figures 7.14 (a) to (h) present a comparison of the calculated lateral soil movement results with the measured data at distances of 4m, 9m and 14m away from the wall at different stages of excavation (stages 10, 19 and 28). Figures 7.14 (a) to (c) show a comparison of the lateral soil movements at stage 10. The lateral soil movement comparisons at distances of 4m and 14m away from the wall at stage 19 of the excavation are given in Figures 7.14 (d) and (e). At stage 28 of the excavation, the comparison of the calculated and measured results at three distances (4m, 9m, 14m away from the wall) can be seen in Figures 7.14 (f), (g), and (h). At a distance of 4m and 9m away from the wall the finite element results and the measured data as well as the results of Whittle *et al.* (1993) for the lateral soil movements show good agreement. However, at a distance of 14m away from the wall, only at stage 10 are the calculated and the measured data close together in terms of both the trend and the magnitude of the maximum lateral movements. At stages 19 and 28, the calculated lateral soil movements are over-predicted compared with the measured data. Not only are the calculated results for the lateral soil movements over-predicted but the vertical soil movements and the pore pressure (at stage 28) also show differences between the calculated and the measured data. The locations of the inclinometers that were used for measuring horizontal soil movements at POS are provided in Figure 7.15.

The measured vertical movements of the soil layers are illustrated in Figure 7.17. Figure 7.17(d) illustrates the extensometer plan at the excavation site at POS. The vertical soil movements were also measured at the same distances as the lateral soil movement measurements. The results obtained from the finite element analysis slightly over-predict the measured results except at stage 28. The predicted vertical soil movements at this stage are much larger than the results at all the measured positions. The explanation may be that because the predicted pore pressures are different, then the vertical effective stresses will be different and hence the vertical

settlement will not be the same as that observed. The stiffer soils caused by loading under the shallow foundations of existing buildings, that have not been taken into consideration in the finite element analysis, may also be a reason for poor predictions in soil movements.

Figure 7.16 shows a plan of the surface settlement measurement points around the excavation site at POS. The surface settlements at stages 10, 19 and 28 are presented in Figures 7.18 (a) to (c). Reasonable estimates of the surface settlements and the measured ones can be seen at stages 10 and 19 (Figures 7.18(a) and (b)), though the surface settlements are under-predicted in the area close to the wall (over a range of 10m from the wall). At stage 28, as the estimated surface settlement in the area 10m from the wall is close to the measured values, the settlements are over-estimated in the zone that is more than 10m away from the wall.

Figures 7.19(a) to (h) show the measured data at a distance of 14m, 48m, and 95m away from the wall and the calculated results for the piezometric head at stages 10, 19, and 28. At stage 19, the measured data for the piezometric head are only available at distances of 14m and 95m. Figure 7.19(f) shows the piezometric head at a distance of 14m from the wall (stage 28) is lower than the measured values. The result of this difference in the piezometric head causes the soil in the finite element analysis at this position (14m from the wall) to predict greater deformation than the obtained data. The explanation could be that, in the finite element analysis, as the excavation went through the clay layer and reached the high permeability layers of sand, till and argillite, the water ran into the excavation faster. However, in reality, the ground water was drawn down constantly by using deep pumping well systems. The piezometer location plan at the excavation site at POS is shown in Figure 7.20.

7.3.2 CASE 2: DAVIDSON AVENUE AND RANKIN STREET, SAN FRANCISCO

The construction of the sewer culvert system was conducted during 1977-1978 in the Islais Creek Bay, 5km south of a Centre District of the City of San Francisco, with the purpose of temporarily storing storm water runoff. The culvert excavation was about 8m wide and 9m deep, and ran alongside Rankin Street and Davidson Avenue. Figure

7.22 shows the plan view of the culvert alignment and locations of borings and instrumentation along Davison Avenue and Rankin Street.

Generally, the soil conditions underneath the excavation area (Davidson avenue and Rankin street) include a rubble fill layer of 1.5 to 3.0m in thickness lying near the surface. The fill was placed during 1915-1940 to bring the ground surface above high tide level. Below the rubble fill is a layer of the New Bay Mud. Its thickness is about 12m-18m along Rankin Street and increases up to 30m along Davidson Avenue. The soft New Bay Mud is the weakest layer of clay in the area. The idealized soil profile given by Clough and Reed (1984) shows the New Bay Mud as a clay with a high water content, a high sensitivity, a low consolidation pressure, a low unit weight and a low undrained shear strength. The Old Bay Mud lying right beneath the soft clay layer of New Bay Mud is the thickest layer in the excavation area. There are sand lenses present between the New and Old Bay Mud in several areas. Because the culvert excavation was conducted at shallow depth and the Old Bay Mud is also rather stiff, the characteristics of the Old Bay Mud have not been presented here. Furthermore, in the finite element analysis, the Old Bay Mud has been treated as the bedrock with no movements. Clough and Reed (1984) presented details of the soil conditions at Rankin and Davidson. A Modified Cam Clay model of the soil behaviour is employed in the Finite Element analysis. Tables 7.2 and 7.3 show the soil parameters used for both the Rankin Street and the Davidson avenue case studies.

Figure 7.23 shows the soil profile along the culvert axis from west to east.

Table 7.2: The Rankin Street and the Davidson Avenue soil parameters.

Stratum (Rankin/Davidson)	Total unit weight	K_0	OCR	Poisson's ratio	Permeability ($k_x=k_y$)
	(kN/m ³)	-	-	-	(m/day)
Fill	19.6	0.5	1.5	0.3	1.0E-02
New Bay Mud	15.7/20.7*	0.5	1.5/1.0*	0.3	1.0E-04

Stratum (Rankin/Davidson)	Soil model	Modified Cam Clay parameters			In situ void ratio	Specific yield ratio
		Lamda	Kappa	M	e_o	S_y
Fill	MCAM	0.025	0.005	1.5	1.50	0.002
New bay Mud	MCAM	0.37	0.054	1.4	2.57	0.002

Note: * Soil parameter at Rankin street / soil parameter at Davidson avenue.

The excavation that was 8m wide and 9m deep was supported by a 13.7m interlocking sheet-pile wall and braced at four levels. The top and the bottom level struts were wooden with a square cross-section of 304.8x304.8mm. Two intermediate struts were made from steel (designated HP 12x74). Table 7.3 gives the strength values of the excavation-support system, including the wooden and steel struts and the sheet-pile wall. There are four stages of excavation that are explained in Figures 7.24 and 7.25.

Table 7.3: The Rankin Street and Davidson Avenue excavation support system parameters

Strut/wall	Cross sectional area (m ²)	Length (m)	Young's modulus (kPa)	Poisson's ratio	Yield strength (kPa)
Wooden strut	0.0929	8.0	1.4×10^6	0.15	-
Steel strut	0.0029	8.0	2.0×10^8	0.15	-
Sheet-pile wall (HP12x74)	-	13.7	2.0×10^8	0.20	3.1×10^4

Figure 7.21 shows the finite element mesh that has been used for both cases. Only the thickness of the New Bay mud needs to be changed for Davidson Avenue (21.57m) and for Rankin Street (17.07m) cases. The plan view of Davidson Avenue and Rankin Street, and location of bore holes and instrumentation is provided in Figure 7.22.

Figure 7.23 shows the cross section through Davidson Avenue and Rankin Street in the west-east direction. It illustrates soil profiles underneath the excavation site. Figures 7.24 and 7.25 present the number of stages of the excavation at Davidson Avenue and Rankin Street and the dates to finish excavation stages correspondingly.

The analysis results are shown in Figures 7.26 to 7.31. At Davidson Avenue, the analysis result of the horizontal wall movements compared with the measured data are shown in Figures 7.26 and 7.27. Figure 7.26 shows the comparison throughout every step of excavation for a typical section (the section at inclinometer 7). In this figure, the comparisons have been carried out at three stages (stage 2 at the 15th of March, stage 3 at the 23rd of March, and stage 4 when the excavation finished at the 9th of April). At stage 2, the measured wall deflection is less than the calculated result, however at stage 3 the numerical prediction is an underestimate. At both stages, the maximum analysis results occur at depths that are in good agreement with the measured data. The maximum deflection of the wall at the final stage is close to the measured data though the finite element calculation gives the maximum deflection at a point slightly lower than the measured one. The measured maximum horizontal soil movements at inclinometers 2, 5, 6, 7 along Davidson Avenue and the maximum finite element result are shown in Figure 7.27. The predicted results fall between the ranges of the measured data. Below the foot of the wall down to the bedrock, the finite element analysis gives a large horizontal soil movement while the measured data shows the horizontal movements of the soil to be much smaller than the numerical analysis results. The settlements of the surface in this case cannot be considered, as there is no measured data available. Clough and Reed (1984) also gave measurements of the wall settlements at a short distance from the west end of the Davidson Avenue excavation. However, this settlement data did not show the real settlement due to the excavation because it was influenced by heavy traffic along Davidson Avenue next to the excavation.

A comparison of the pore pressure behind the wall is presented in Figure 7.28. For the first two stages (stage 1 and 2) the computed pore pressures are in agreement with the measured pore pressure but at stage 3 of the excavation at a depth of 9.1m the pore pressure is significantly less than the measured data. The initial effective stress used in both finite element models (Rankin Street and Davidson Avenue) is presented in

Figures 7.29 and 7.30. Figure 7.30 also gives a comparison of the maximum horizontal wall deflection at Rankin Street. Again it can be seen that there is an agreement of the depth where the maximum deflection of the wall occurs but the soil below the excavation bottom in the finite element prediction seems to move much more than the measured data shows. As there was no measured pore pressure at the Rankin Street excavation, Figure 7.31 only presents the calculated pore pressures at each excavation stage.

7.3.3 CASE 3: LARGE CAVITATION CHANNEL EXCAVATION, PRESIDENT'S ISLAND, MEMPHIS, TENNESSEE.

Finno *et al.* (1991) described a Large Cavitation Channel excavation project in Memphis, Tennessee. The deep excavation was carried out next to the Chicago Bridge and Iron's (CBI) Nuclear building. The CBI building that is supported by 21-22m long step-tapered pile groups (4 by 7 or 4 by 8 piles per group) was modified to fit the reinforced concrete vault for the channel.

The excavation of approximately 15.25m depth was supported by a temporary tied back sheet pile wall. The tied back sheet pile wall was installed at a distance of about 0.61m from the main column of the existing framed structure of the CBI Nuclear Building that was supported by step tapered pile groups. As the concrete piles were only designed to carry vertical load and were un-reinforced or slightly reinforced, the horizontal movements of soil due to the excavation could cause a potential failure to the piles. However Poulos and Chen (1997) concluded, based on finite element analysis, that the excavation induced moment in the piles was not large enough to cause the piles to fail. This case is still considered in this thesis to look at the soil movement profiles as well as the maximum soil movements due the excavation and the effect of dewatering during excavation, which has not been considered before. The EF4 program is used in order to investigate the induced horizontal deflection of the concrete piles.

According to Finno *et al.* (1991) the subsoil condition at the excavation site generally consists of three layers of soil that are medium dense hydraulic fill sand of average thickness 9-10m near the surface, 2.5-6m of stiff clay and silt and finally, a layer of dense, alluvial sand of average thickness 24m lying below the clay and silty layer.

The excavation was constructed in the area above the two soil layers (the hydraulic sand and the clay and silty layer). The water level, which existed in the hydraulic sand layer prior to the excavation, was maintained at the bottom of the excavation (in the alluvial sand) by a deep-dewatering well system. The soil parameters that were used for the finite element analysis input are given in Table 7.4, following the parameters given by Poulos and Chen (1997). The details of soil conditions can be found in the paper by Finno *et al.* (1991). As the information about initial stresses in the soil mass is not available, the soil has been chosen to be an elastic material. This approximation can be accepted because the three soil layers presented at the site are stiff and over consolidated clay or very dense sand.

Table 7.4: Soil parameters used in the finite element analysis.

Stratum	Young's modulus (kPa)	Specific yield	Poisson's ratio	Permeability (m/day)	Total unit weight (kN/m ³)
Sand fill	1.0E+04	0.01	0.3	1.0E-01	17.0
Clay and silt	1.8E+04	0.002	0.3	1.0E-04	18.0
Alluvial sand	6.0E+04	0.01	0.3	1.0E-01	18.0

The initial water level is set to be at the surface. As the permeability of the sand fill layer at the surface is rather high the water will drain quickly and dewater the upper sand fill after the first level of tieback has been installed as reported by Finno *et al.* (1991). The sheet pile wall RZ10 has a stiffness of $EI_w = 5.92 \times 10^4$ kN.m² as reported by the manufacturer. The tieback has one end fixed to the wall, while the other end is assumed to be fixed so as not to move. The tieback can be modelled as two springs (in the horizontal and vertical directions) in order to model the inclined tieback, which is assumed to be tilted at about 30°. The horizontal spring stiffness is 1.0×10^4 kN/m/m and the vertical spring stiffness parameter is 0.58×10^4 kN/m/m.

The excavation sequence includes six stages. The first four stages involve excavating the soil elements. The fifth stage involves constructing the vault that can be approximately modelled as a loading on the bottom of the excavation with the load

equal to the weight of the concrete vault. The final stage is to release the tieback. The extraction of the sheet pile wall cannot be modelled here. The Finite element mesh used for the soil in the analysis and the excavation sequences are presented in Figures 7.23 and 7.33.

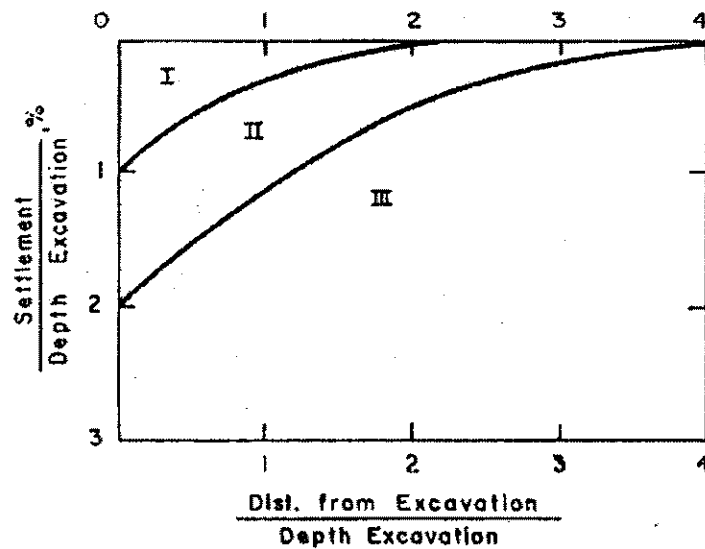
The comparison of the finite element results for the horizontal wall deflections with the results of other researchers such as Finno *et al.* (1991) and Poulos and Chen (1997) are shown in Figures 7.34 to 7.37. Figures 7.34 to 7.36 show the horizontal wall deflection at stages 1, 4, and 6. At stage 1 the finite element result for the horizontal wall movement is 30% less than for the Other's methods (Figure 7.34). However at stage 4 and 6 (Figure 7.35 and 7.36) the finite element results are similar to the Finno *et al.* (1991) prediction and to the analytical result given by Poulos and Chen (1996). Figure 7.37 shows the measured data for the incremental deformations of the wall between certain periods of time (between stage 4 and 1 or stage 4 and 3). The finite element analysis result for the incremental horizontal wall deflection is also presented in Figure 7.37. The finite element result falls well inside the range of the measured data.

7.4 SUMMARY AND CONCLUSIONS

In this chapter there are two main parts. In the first part, a short critical review of the available methods that are used to analyse soil movements due to excavation has been introduced. In general, there are three main approaches, namely, empirical rules, total stress methods, and effective stress methods. Empirical rules are normally based on the collected measured data from the excavations that have been constructed. Then the measured data is gathered into groups in order to indicate a general trend. This trend then is used as preliminary analysis of excavation or for comparison with numerical analysis. Both total stress methods and effective stress methods are numerical analyses but effective stress methods take into account the change in pore pressure while the total stress methods do not.

The second part of the chapter determines the use of an effective stress method to back analyse three typical excavations with different soil conditions. The effect of not

considering drawdown of the water table during excavation can be very large as shown by Hsi and Small (1992a). A fully coupled finite element program EF4, that can take into consideration lowering of the ground water table has been employed. Different soil conditions have been considered such as soft clay in the Davidson Avenue and Rankin Street excavation, or excavation in medium saturated sand with lowering of the ground water table in the case study of a large cavitation channel excavation, at President's Island, Memphis, Tennessee, and an underground parking garage at Post Office Square, Boston. The study shows that the finite element program EF4 can be used to carry out excavation analysis with lowering of the ground water table.



I- Sand and Soft to Hard Clay, Avg. Workmanship

II- Very Soft to Soft Clay

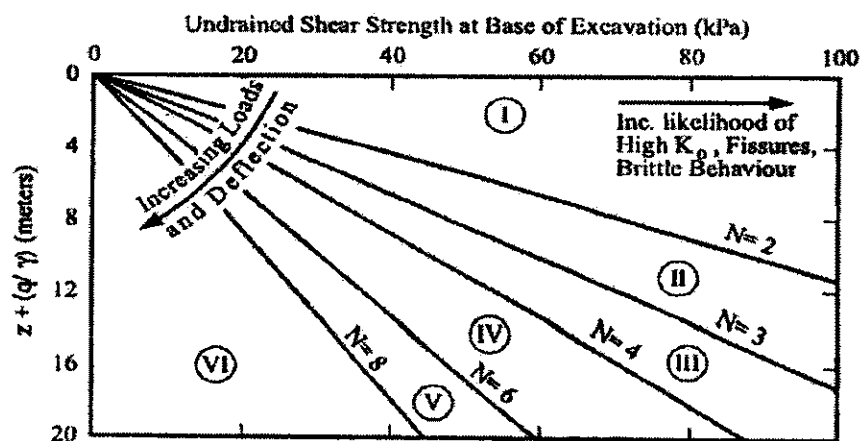
1. Limited Depth of Clay Below Bott. Exc.
2. Significant Depth of Clay Below Bott. Exc., But $N_b < N_{cb}$ *

III- Very Soft to Soft Clay to a Significant Depth Below Exc. Bott. and $N_b > N_{cb}$

$$N_b = \text{Stability No. Using } C \text{ "Below Base Level"} = \frac{\gamma H}{C_0}$$

$$N_{cb} = \text{Critical Stab No. for Basal Heave}$$

Figure 7.1: Summary of soil settlements behind insitu walls (Peck, 1969).



Note: q = surcharge pressure

Zone	Characteristic behaviour
I	Elastic response, small movements (Assuming no fissures)
II	Problems with unsupported slopes in brittle clays
III	Problems with cantilever walls
IV	Moderate movements and loads on supported excavations
V	Basal heave problems where no nearby underlying stiff layer
VI	Large movements, high loads even with nearby underlying stiff layer

Figure 7.2: Classification of behaviour of excavations in clay (Peck, 1969).

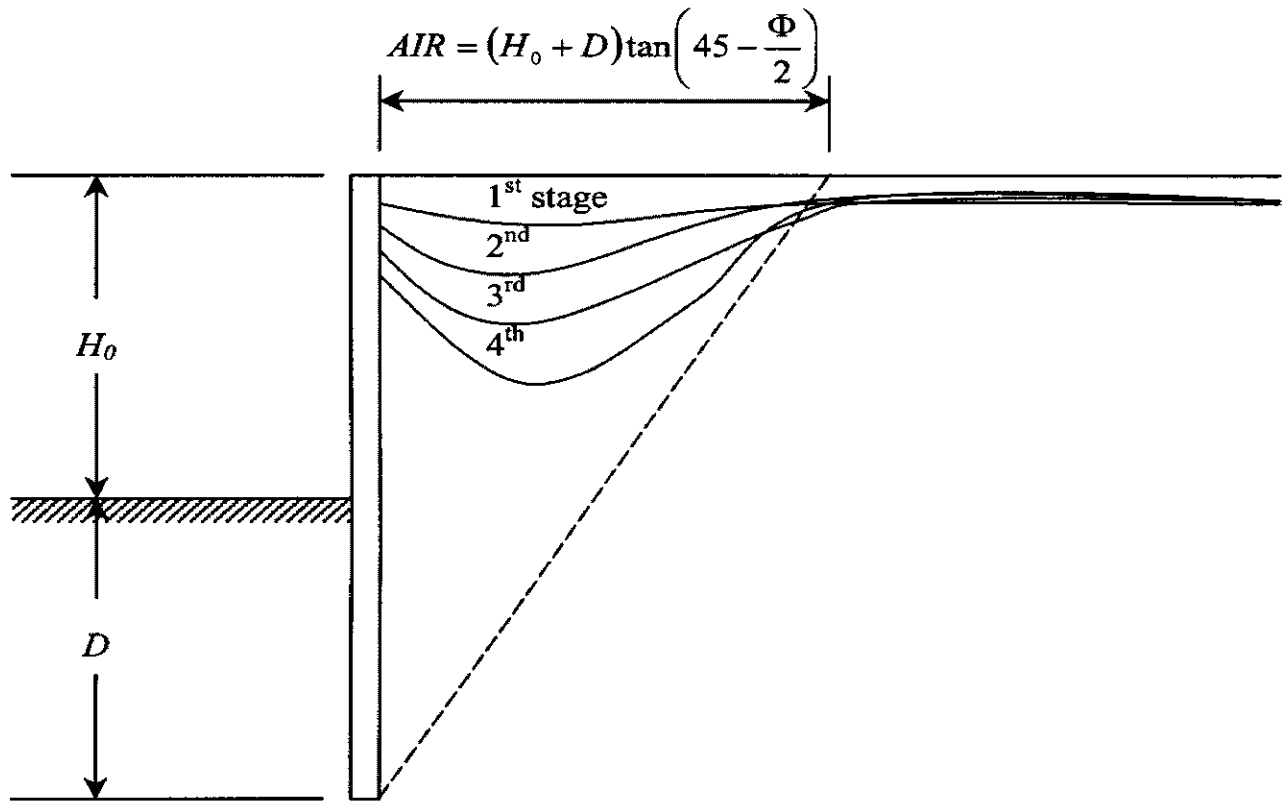


Figure 7.3: Relationship between apparent influence range and length of the retaining wall.

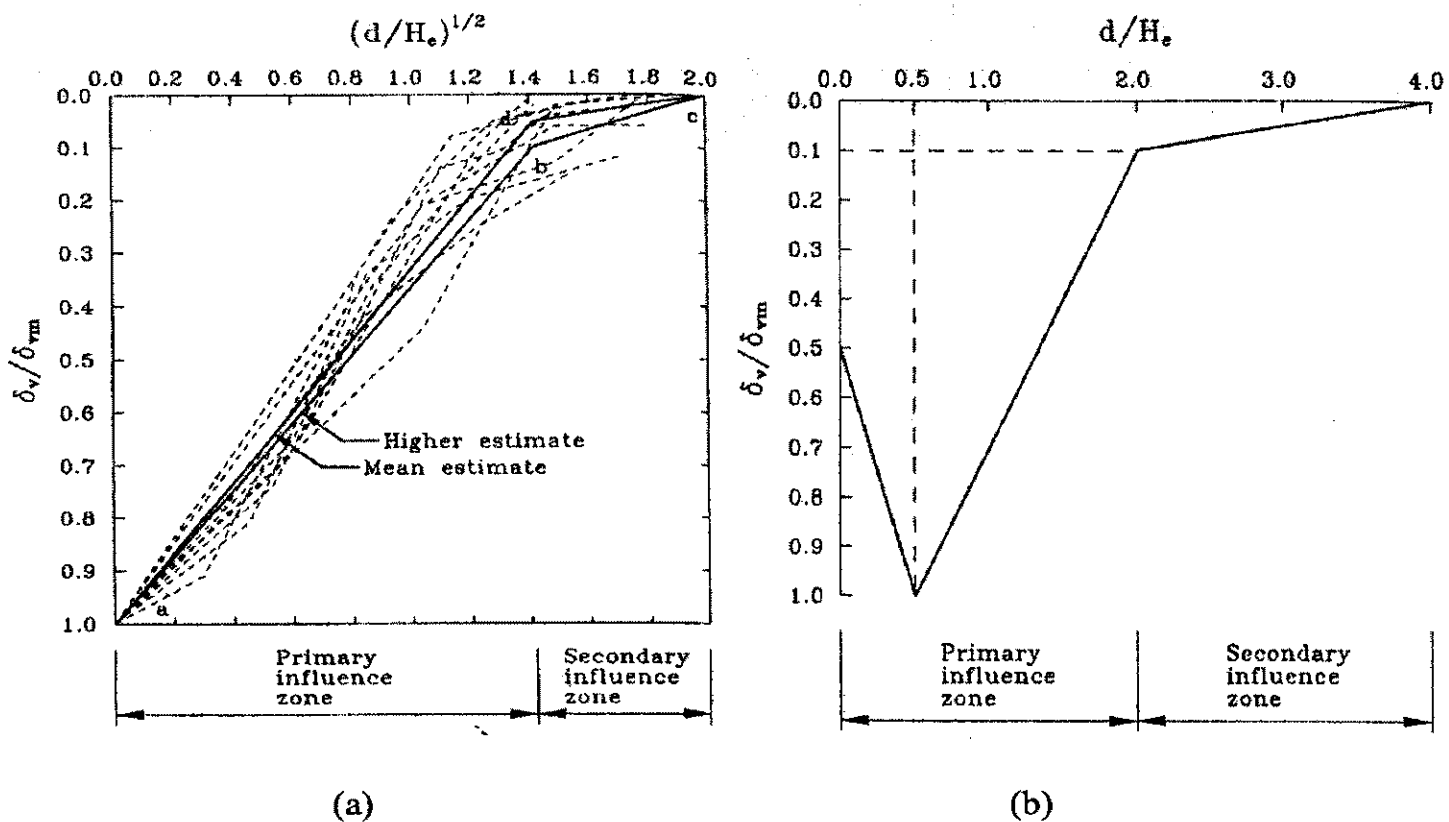


Figure 7.4: Proposed methods for predicting spandrel (a) and concave (b) settlement profiles.

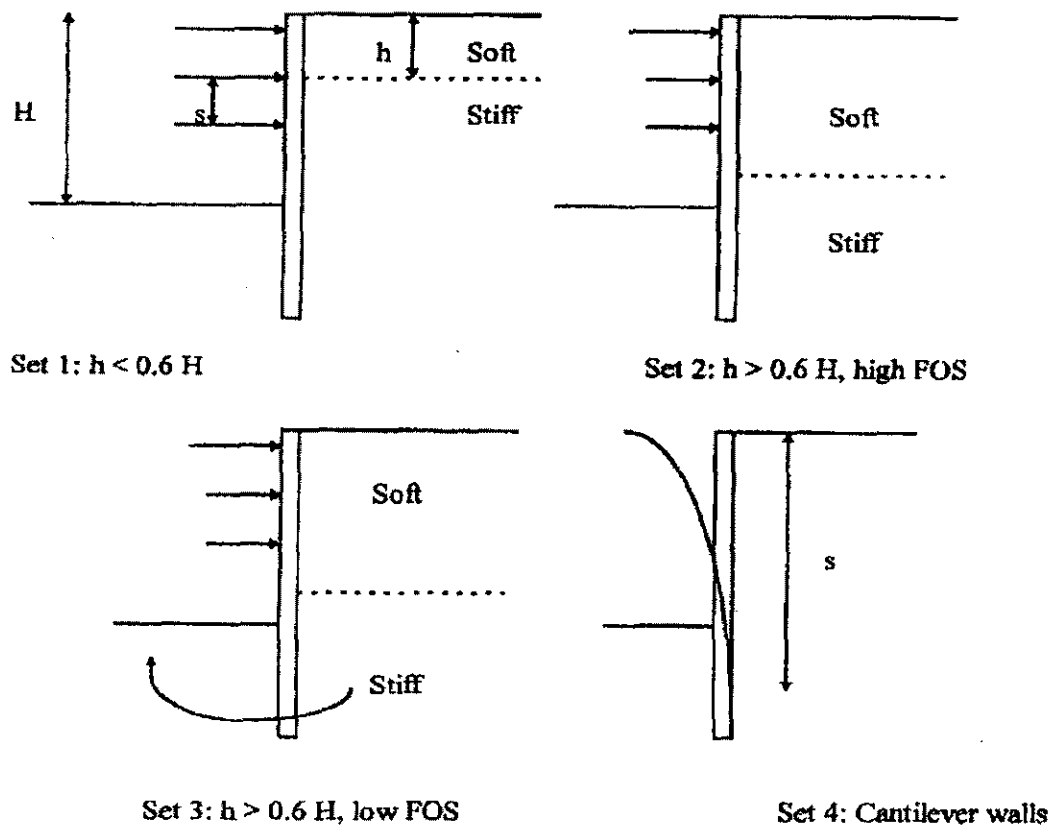
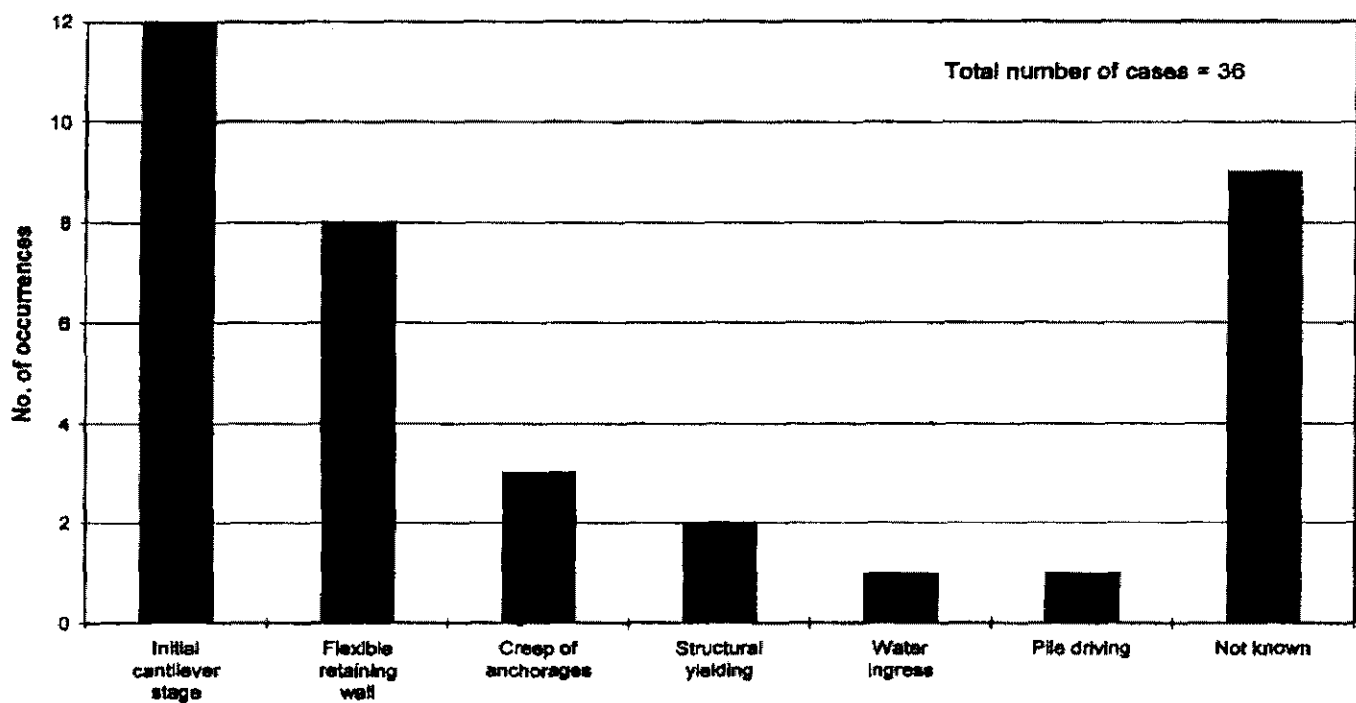


Figure 7.5: Subdivision of data.

Figure 7.6: Reason for excessive deflection ($\delta h_{\max}/H > 0.3\%$).

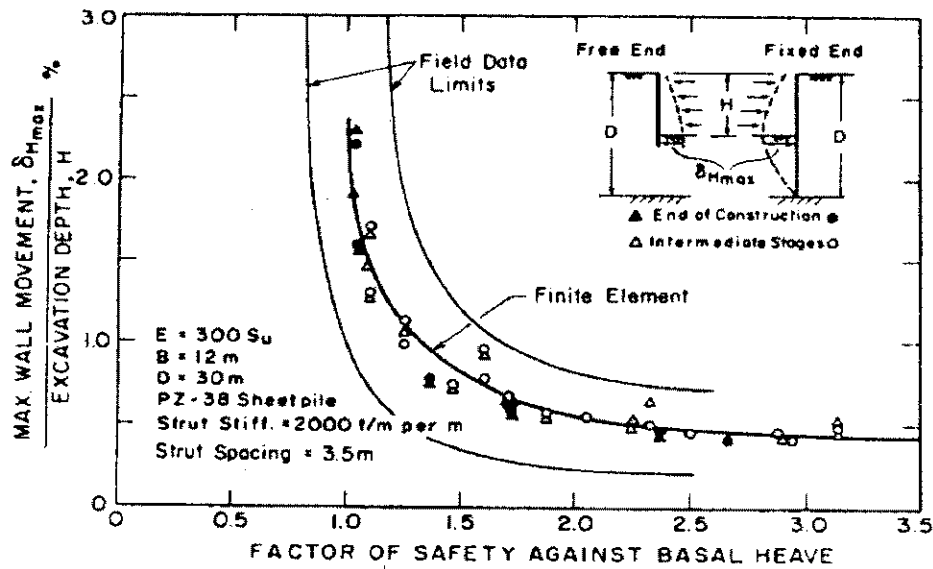


Figure 7.7: Analytically defined relationship between factor of safety against basal heave and non-dimensionalized maximum lateral wall movement (Mana and Clough, 1981).

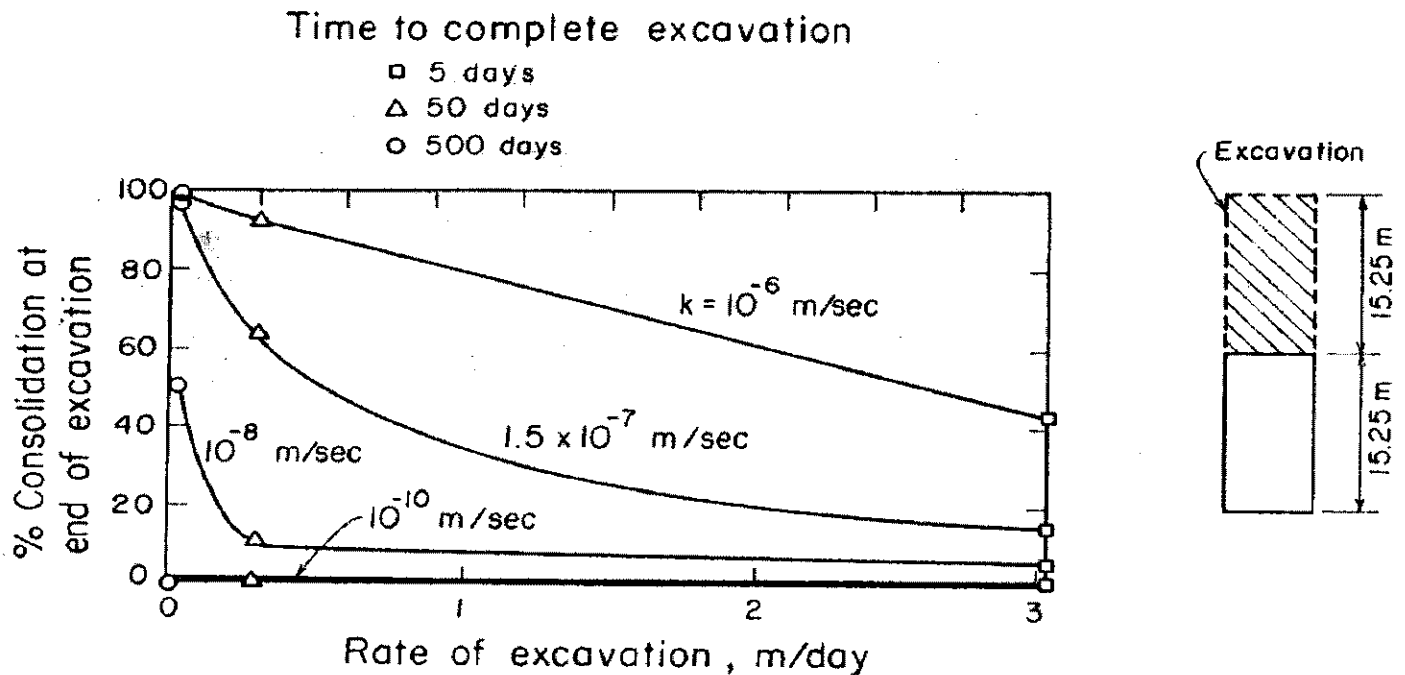


Figure 7.8: Degree of consolidation at the end of one-dimensional excavation into homogeneous clay as a function of excavation rate and permeability (Osaimi and Clough, 1979).

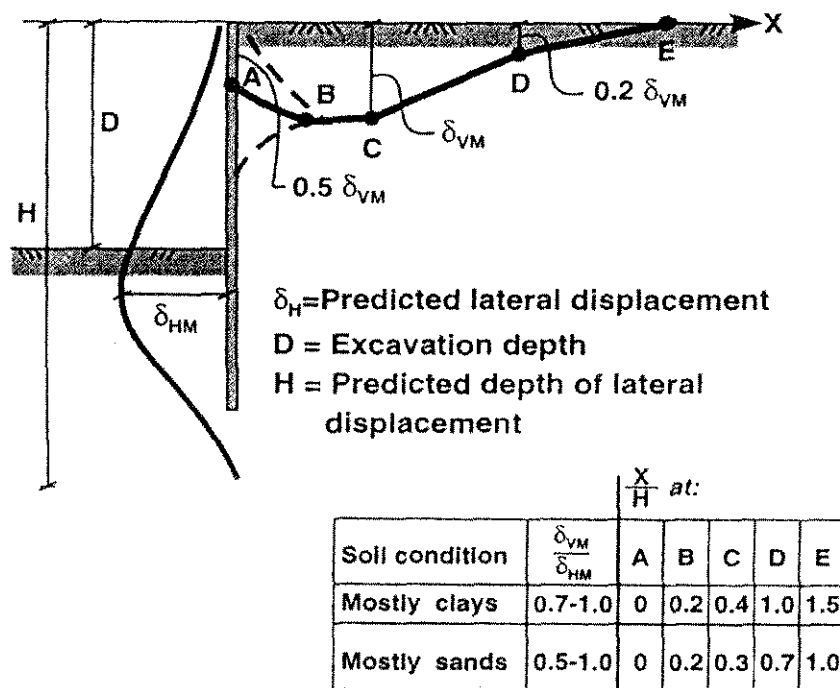


Figure 7.9: Proposed method for estimating ground settlements from predicted lateral wall movements (Karlsrud, 1986).

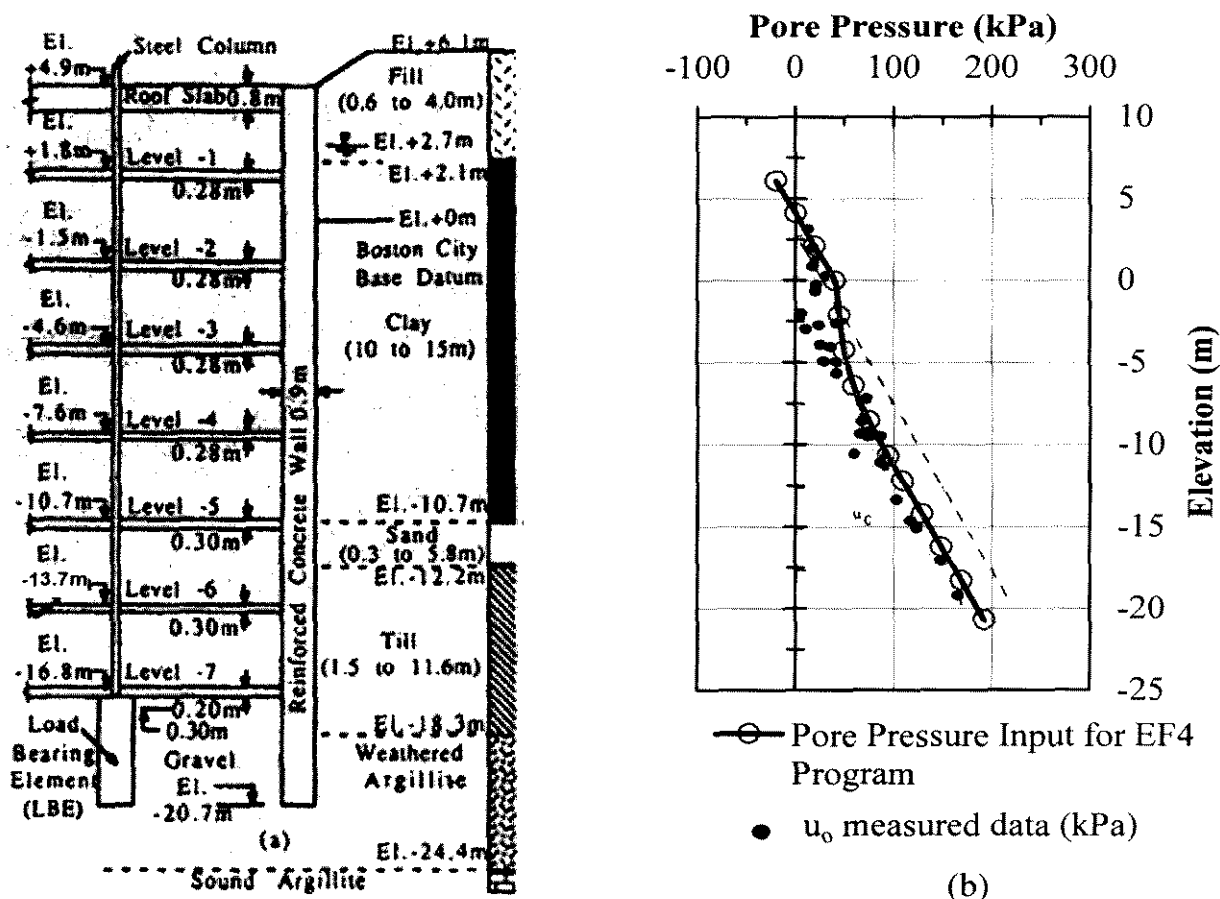


Figure 7.10: Garage structure and initial soil conditions at Post Office Square (Whittle *et al.*, 1993).

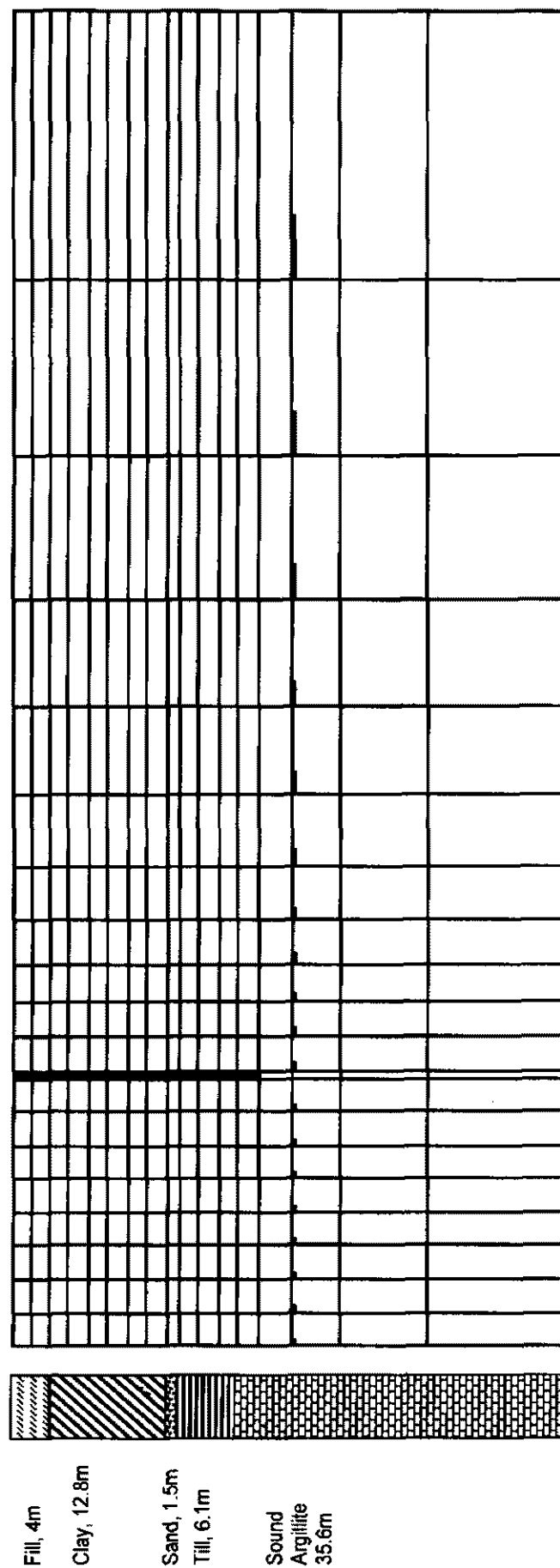
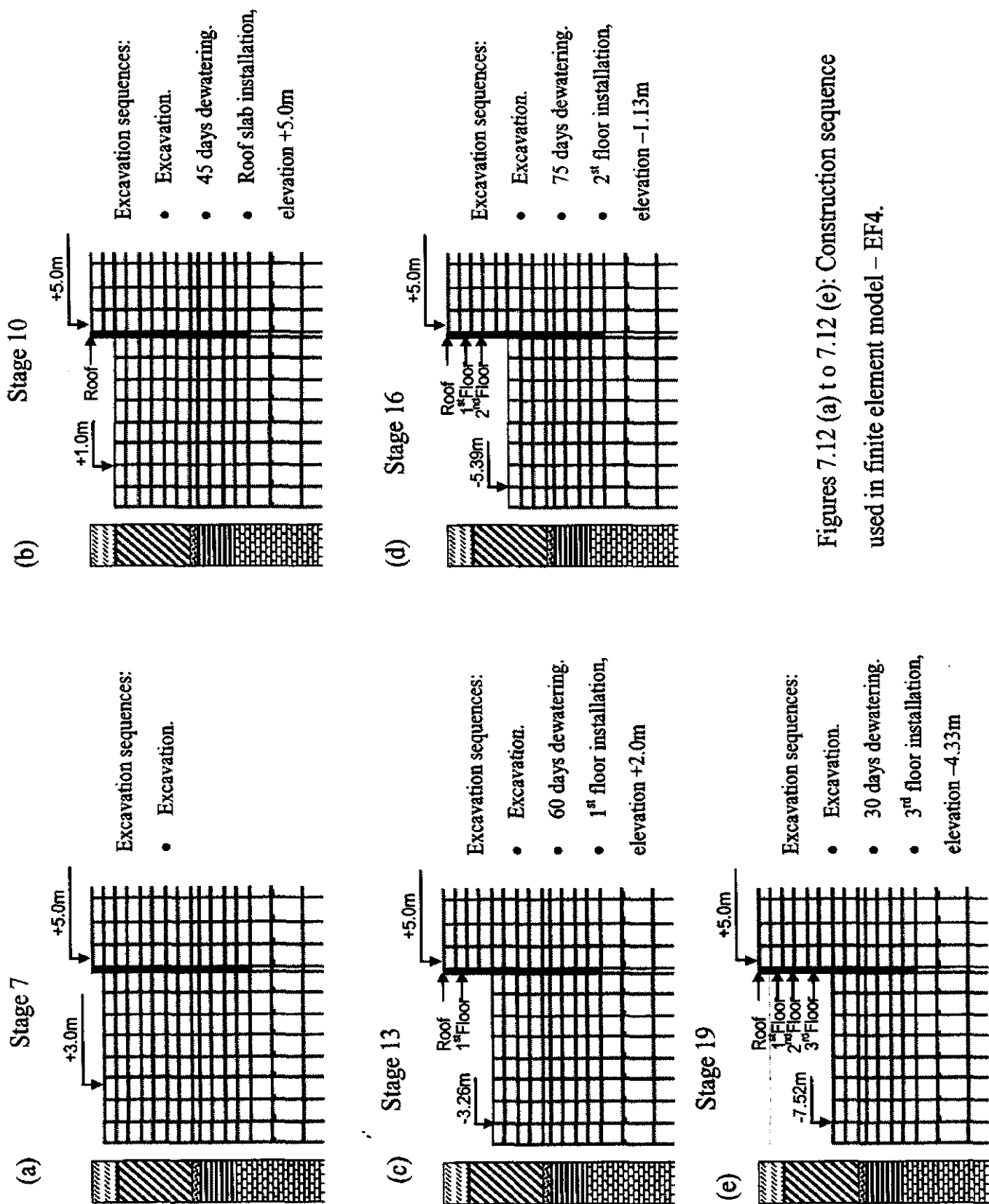
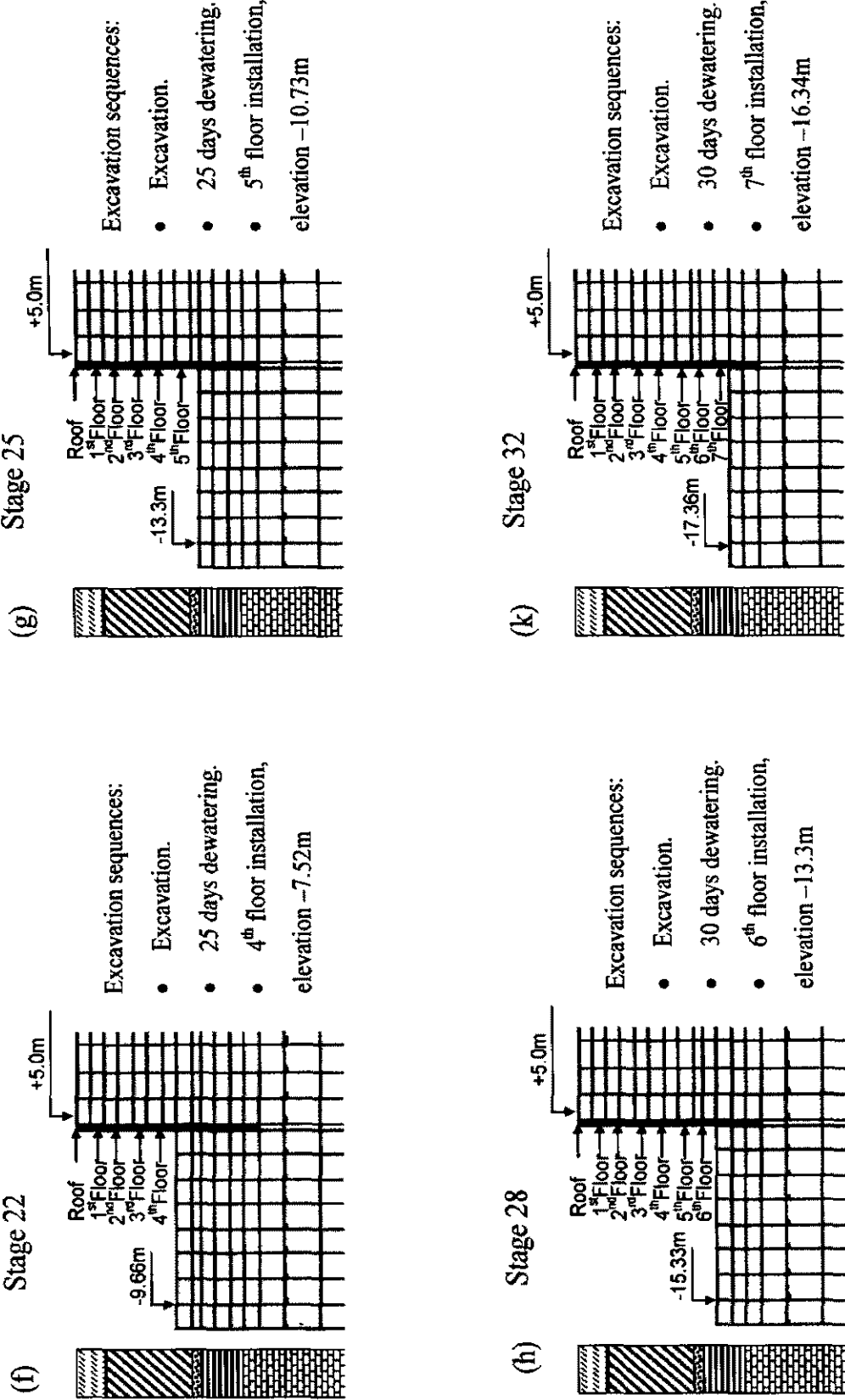


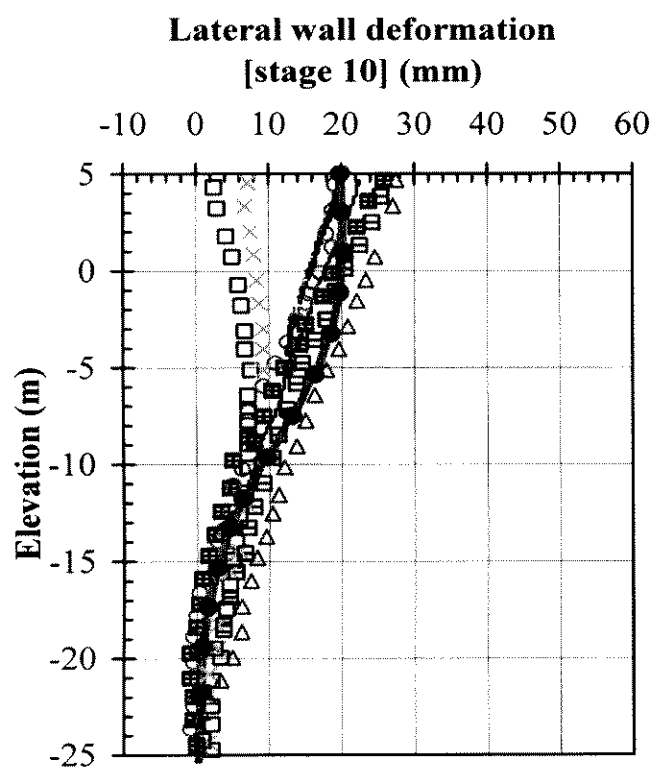
Figure 7.11: Finite element model – EF4.



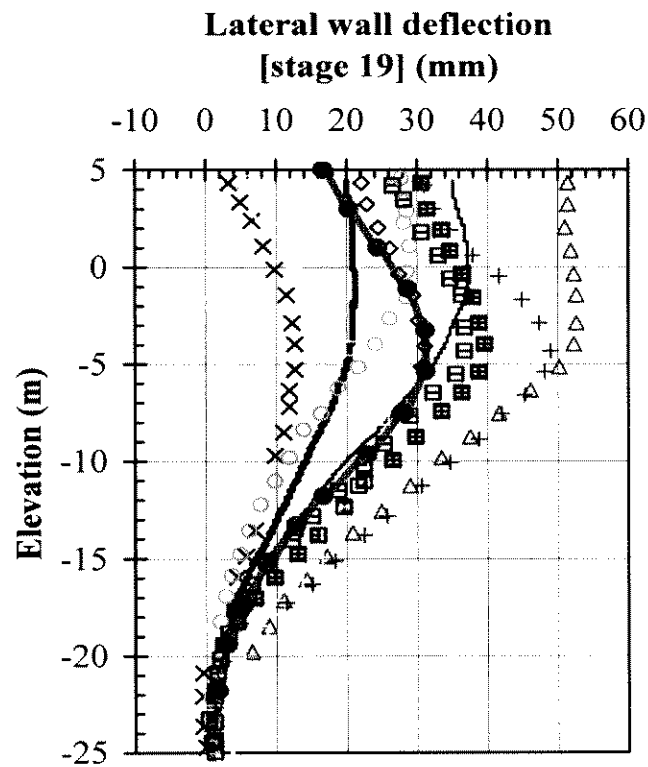
Figures 7.12 (a) to 7.12 (e): Construction sequence used in finite element model – EF4.



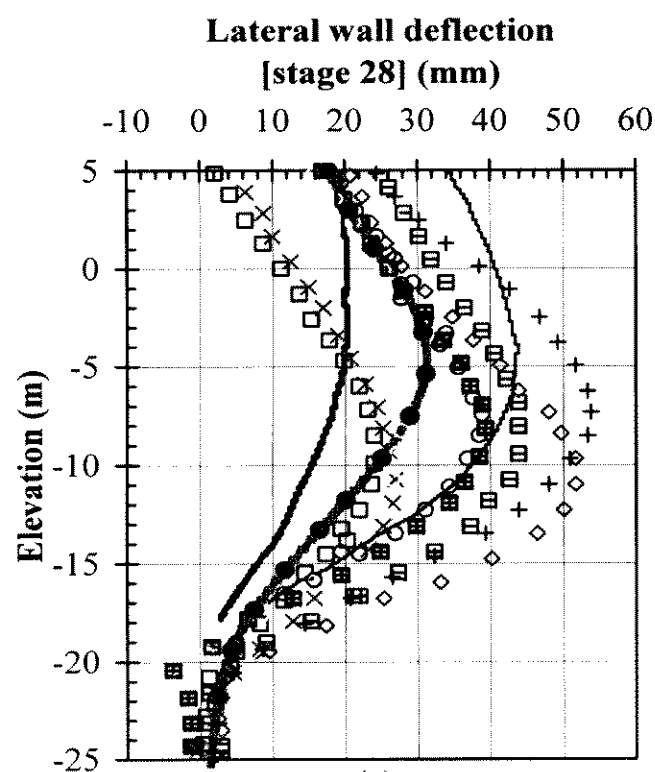
Figures 7.12(f) to 7.12(k): Construction sequence used in finite element model – EF4 (continued).



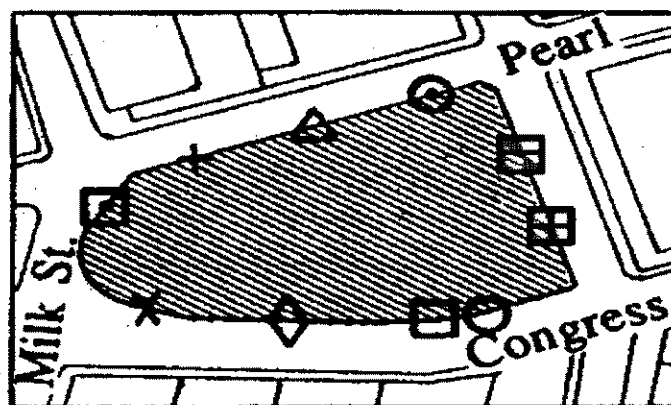
(a)



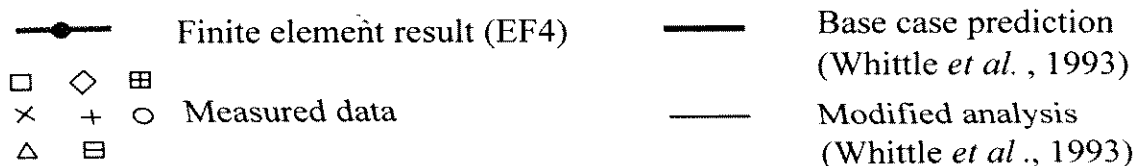
(b)



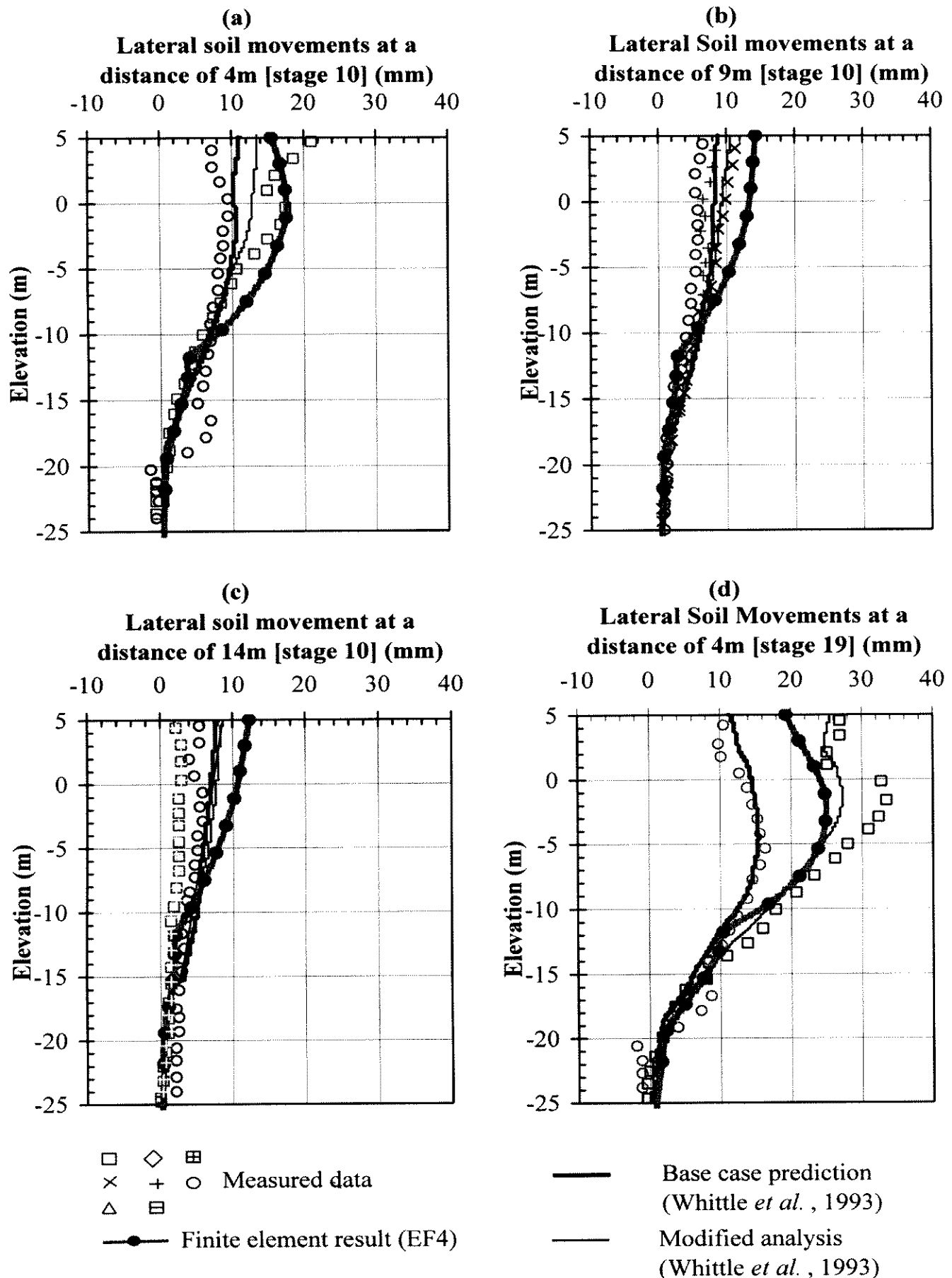
(c)



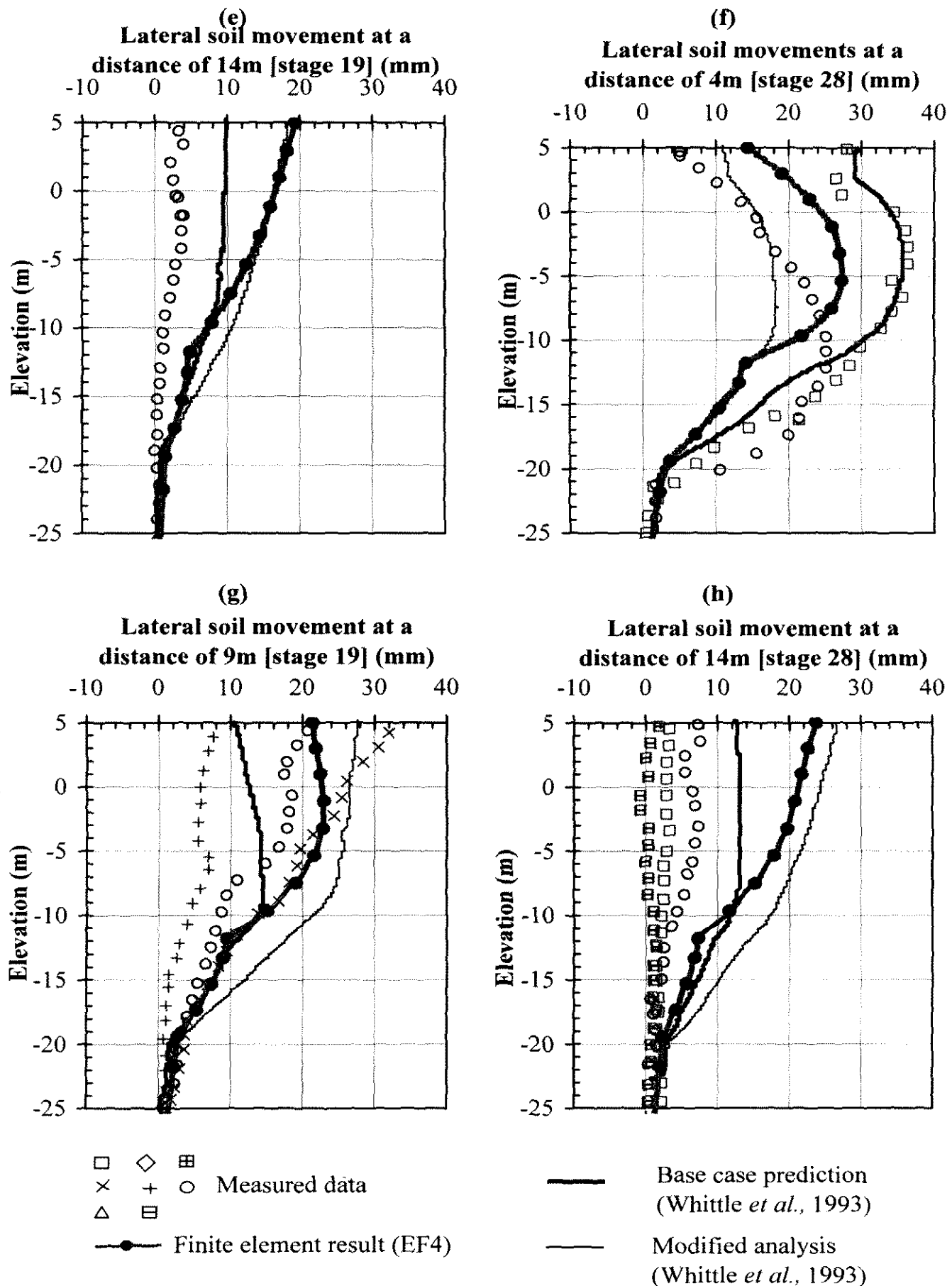
(d) Inclinometer location plan for lateral wall deflection measurements.



Figures 7.13a - 7.13d: Comparison of the calculated lateral wall deflections with the finite element results of Whittle *et al.* (1993) and the measured data.



Figures 7.14a - 7.14d: Comparison of the calculated lateral soil movements with the finite element results of Whittle *et al.* (1993) and the measured data.



Figures 7.14e - 7.14h: Comparison of the calculated lateral soil movements with the finite element results of Whittle *et al.* (1993) and the measured data.

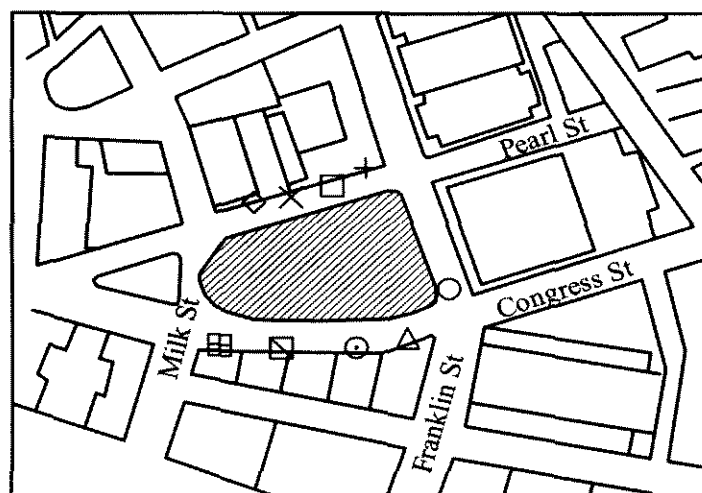


Figure 7.15: Inclinometer location plan for lateral soil deflection measurements (Whittle *et al.*, 1993).

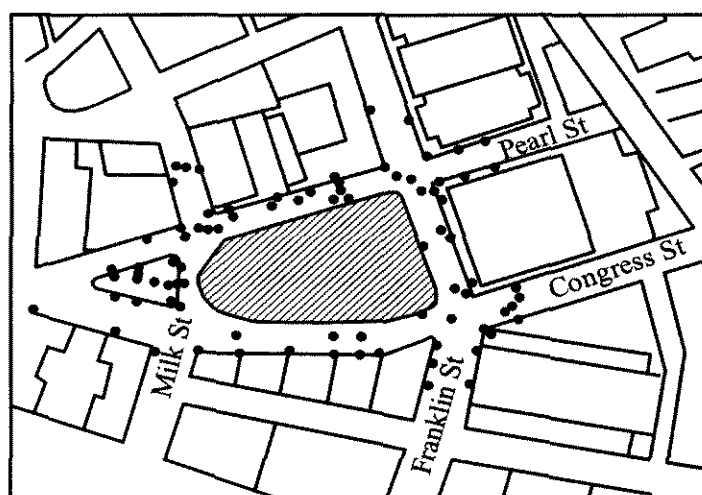
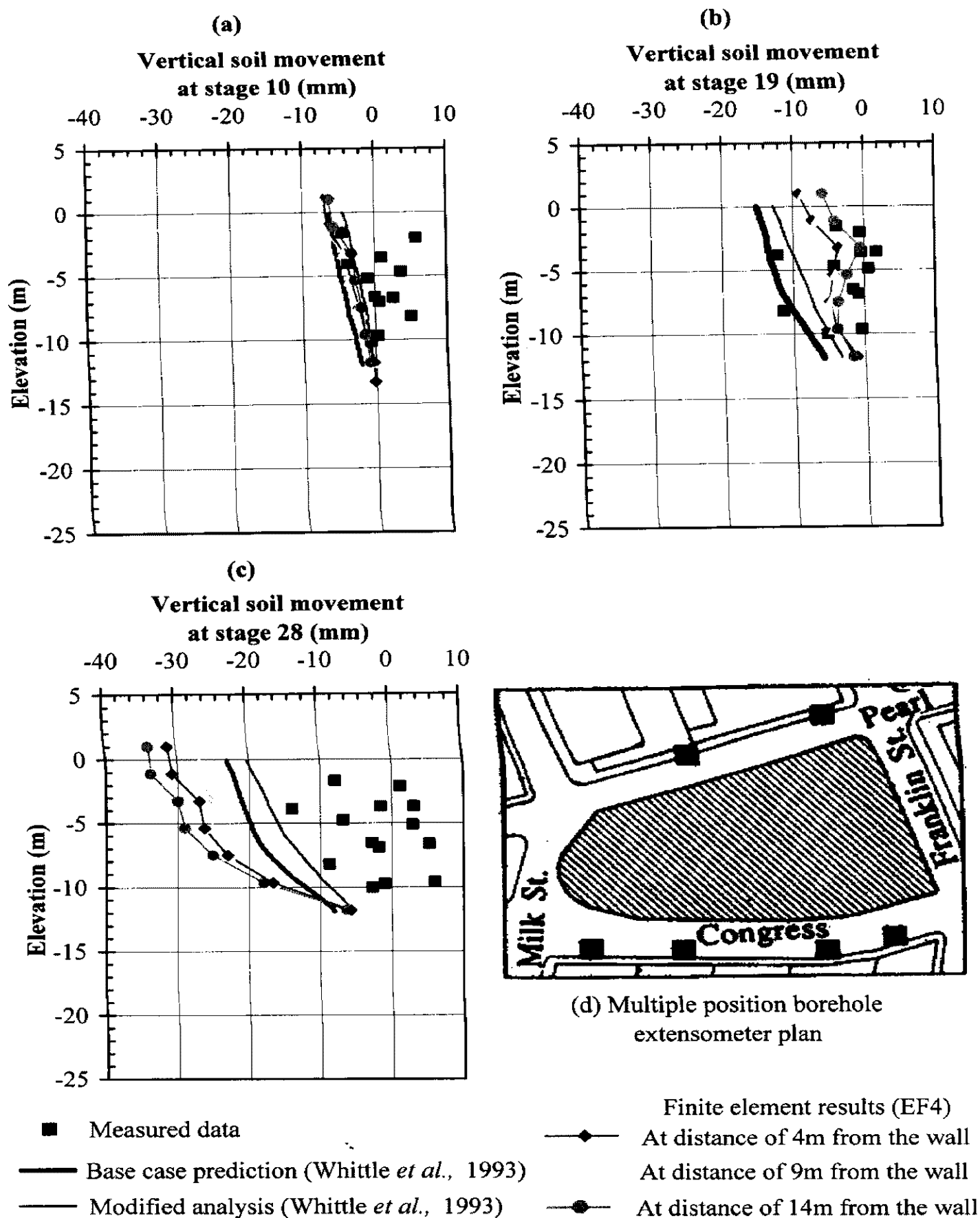
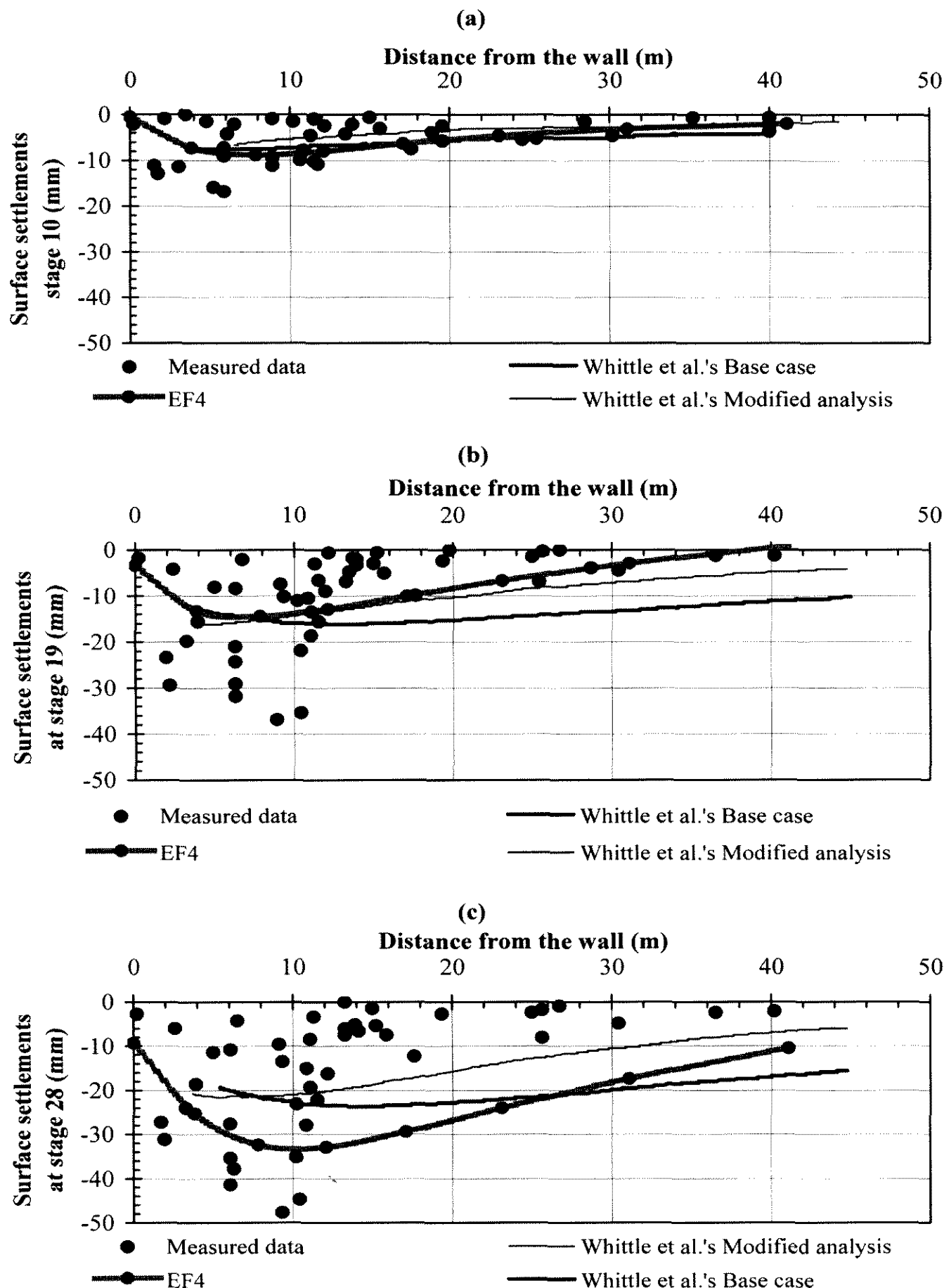


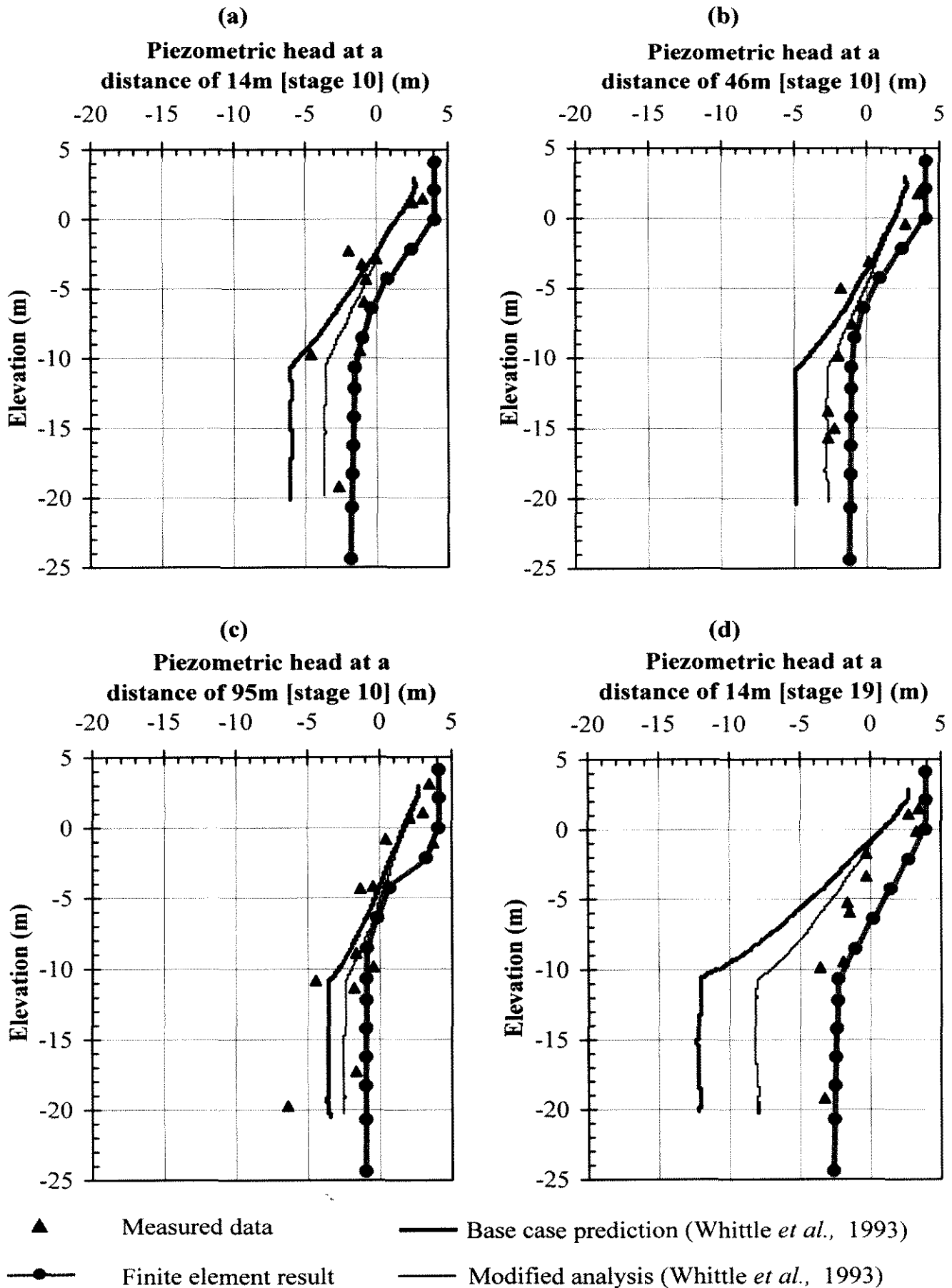
Figure 7.16: Surface settlement point plan (Whittle *et al.*, 1993).



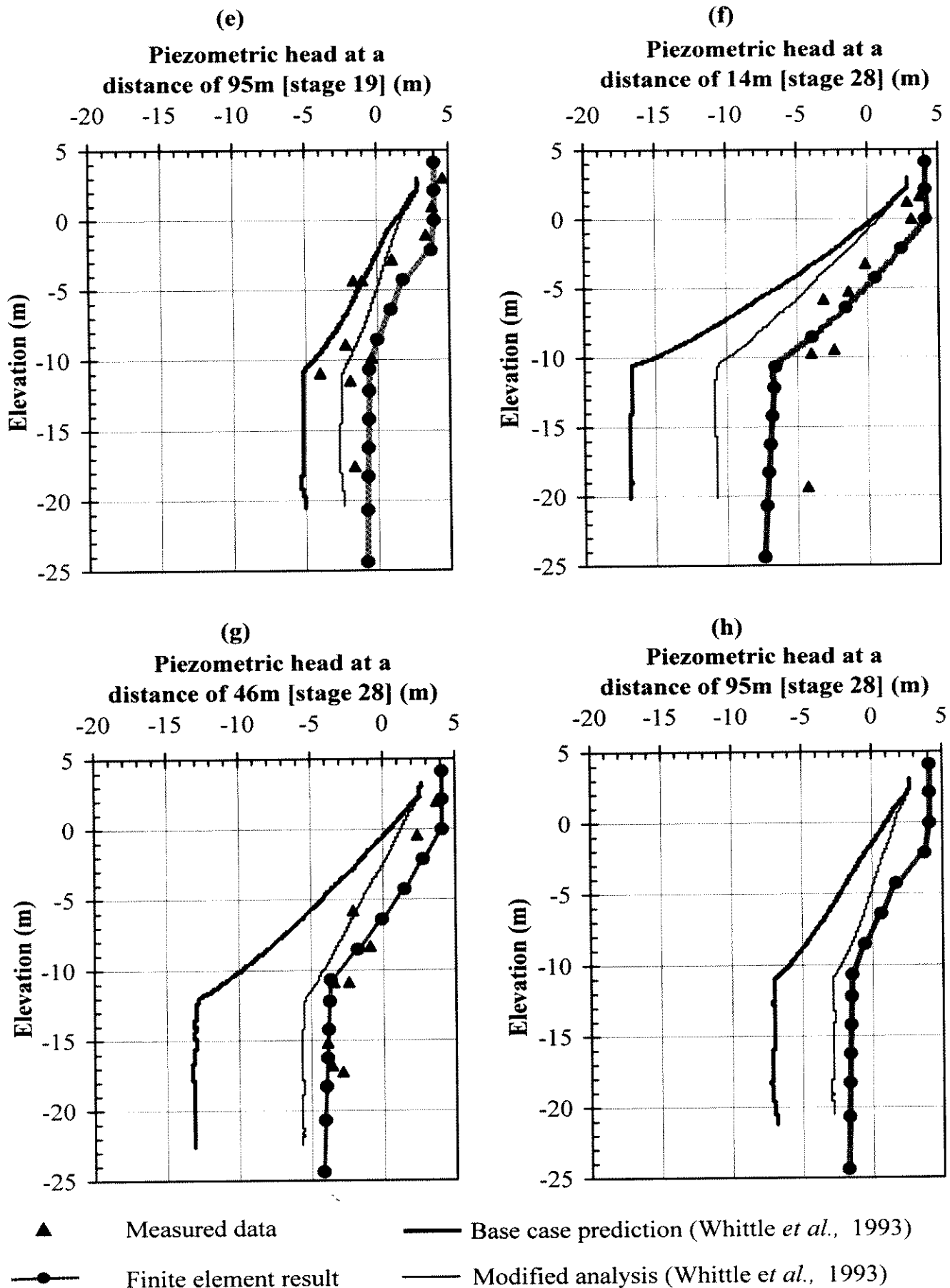
Figures 7.17a - 7.17d: Comparison of the calculated vertical soil movements with the finite element results of Whittle *et al.* (1993) and the measured data, and the extensometer plan.



Figures 7.18a - 7.18c: Comparison of the calculated surface settlements with the finite element results of Whittle *et al.* (1993) and the measured data.



Figures 7.19a - 7.19d: Comparison of the calculated piezometric head with the finite element results of Whittle *et al.* (1993) and the measured data.



Figures 7.19e - 7.19h: Comparison of the calculated piezometric head with the finite element results of Whittle *et al.* (1993) and the measured data.

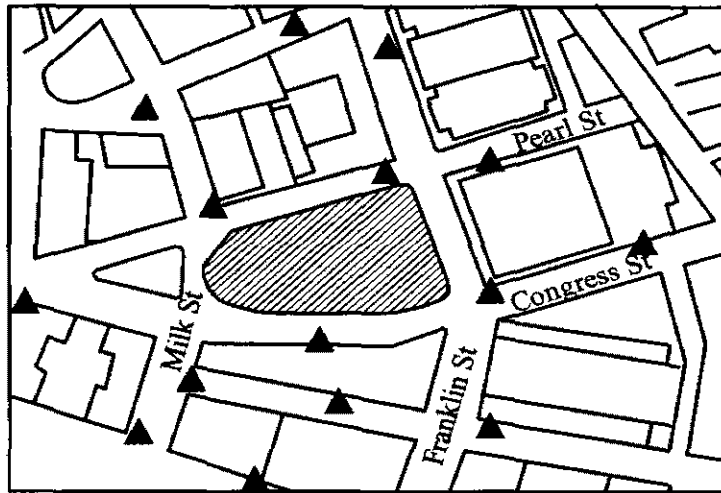


Figure 7.20: Piezometer location plan (Whittle *et al.*, 1993).

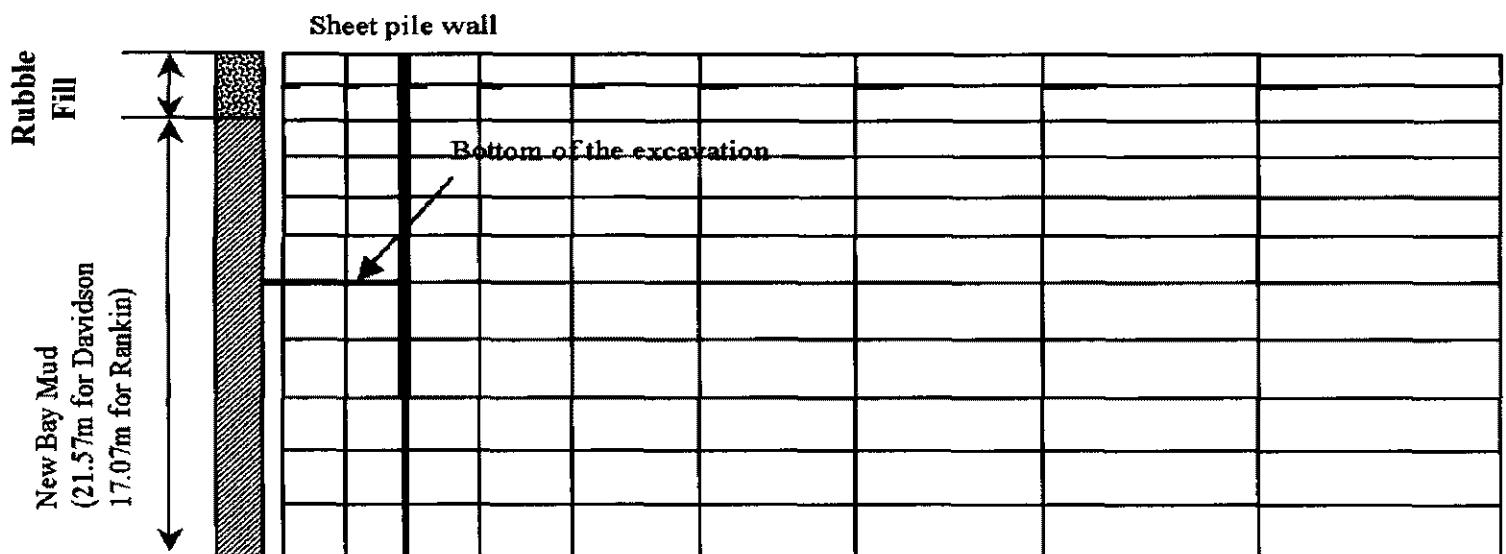


Figure 7.21: The mesh used for finite element analysis in both the Davidson Avenue and the Rankin Street cases.

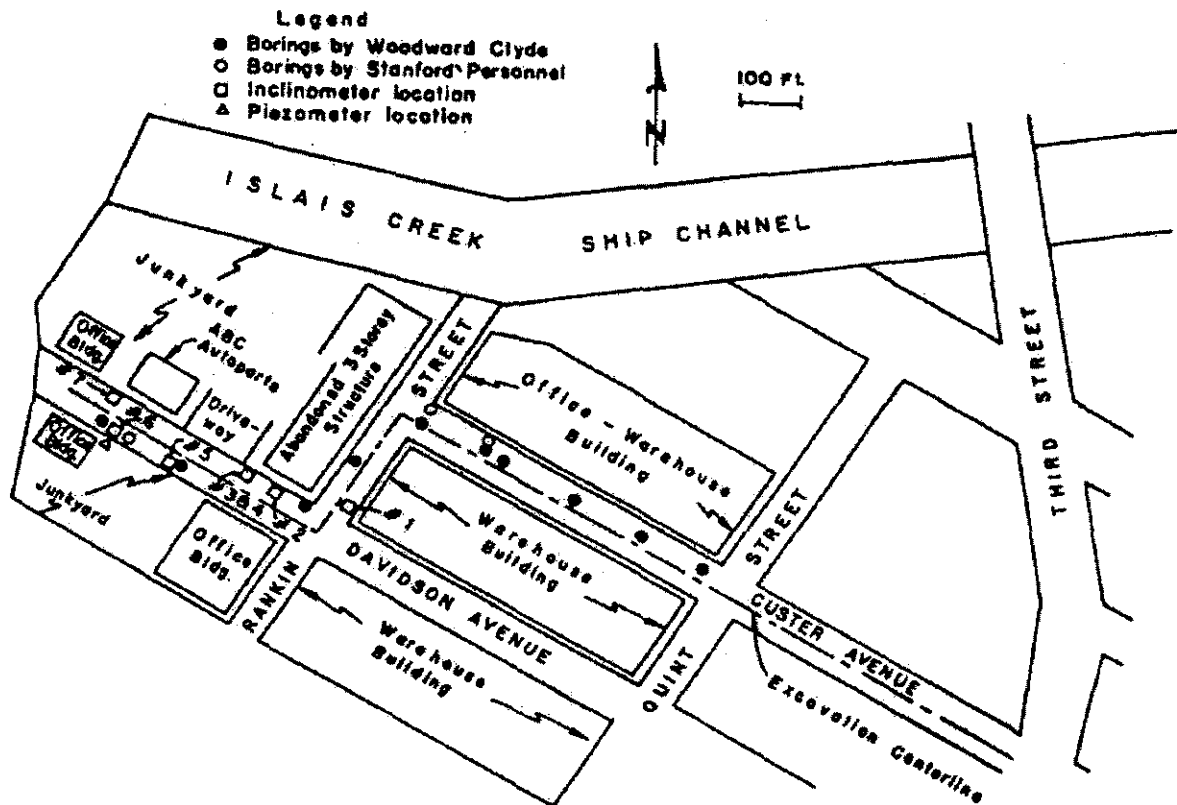


Figure 7.22: Plan view showing culvert alignment and location of borings and instrumentation (after Clough and Reed, 1984), (1ft = 0.305m).

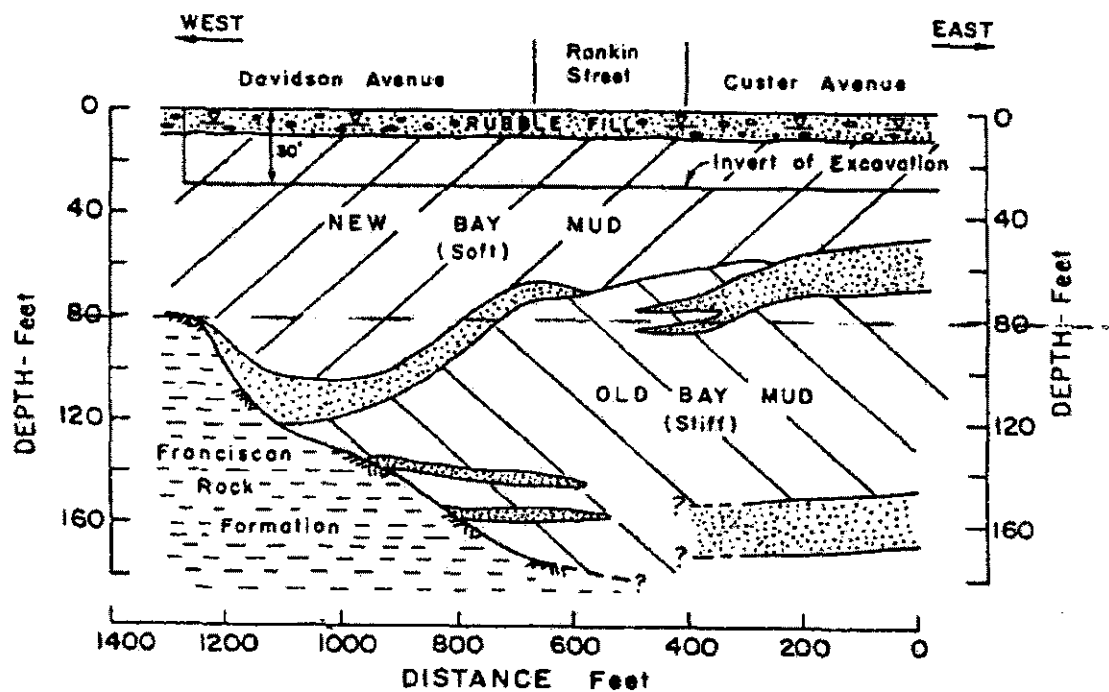


Figure 7.23: Soil profile along culvert axis (after Clough and Reed, 1984), (1ft = 0.305m).

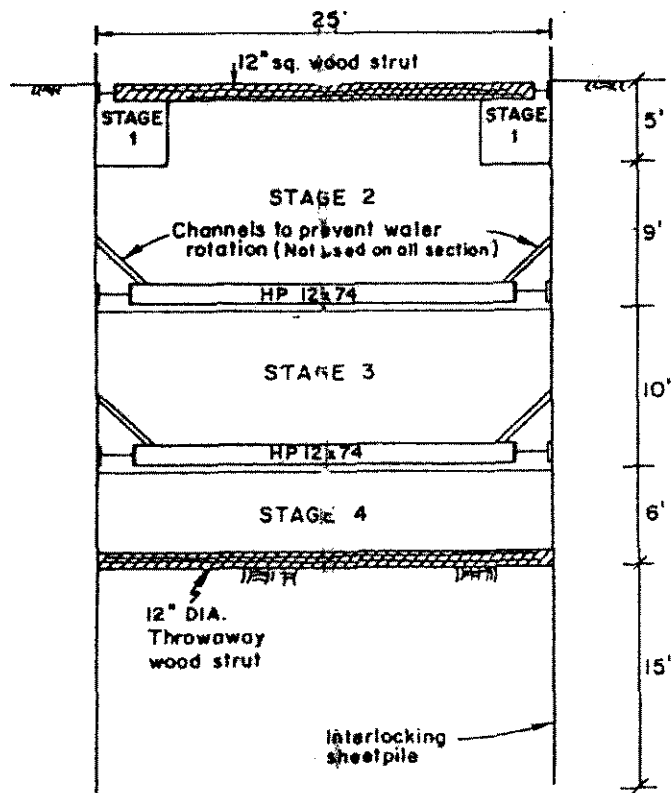


Figure 7.24: Structural support and excavation sequence (after Clough and Reed, 1984), (1ft = 0.305m).

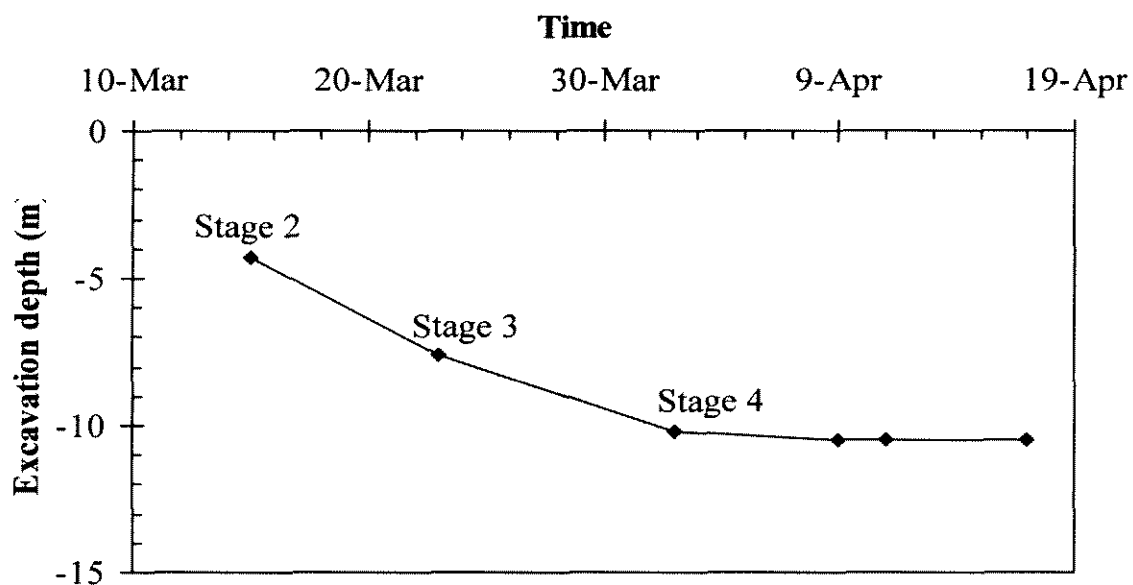


Figure 7.25: Excavation time scale and sequence at Davidson Avenue (after Clough and Reed, 1984).

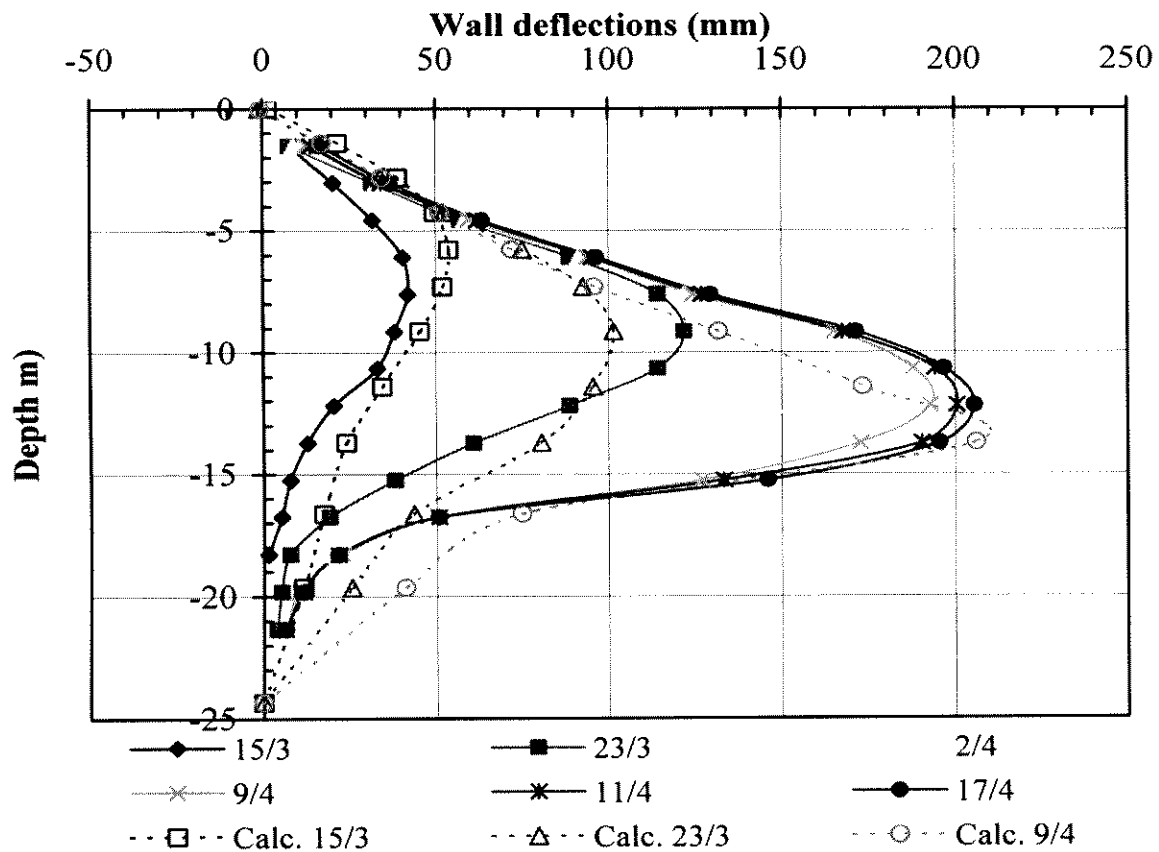


Figure 7.26: Comparison of the finite element results and the measured data from inclinometer number 7 (Davidson avenue) for horizontal wall deflections.

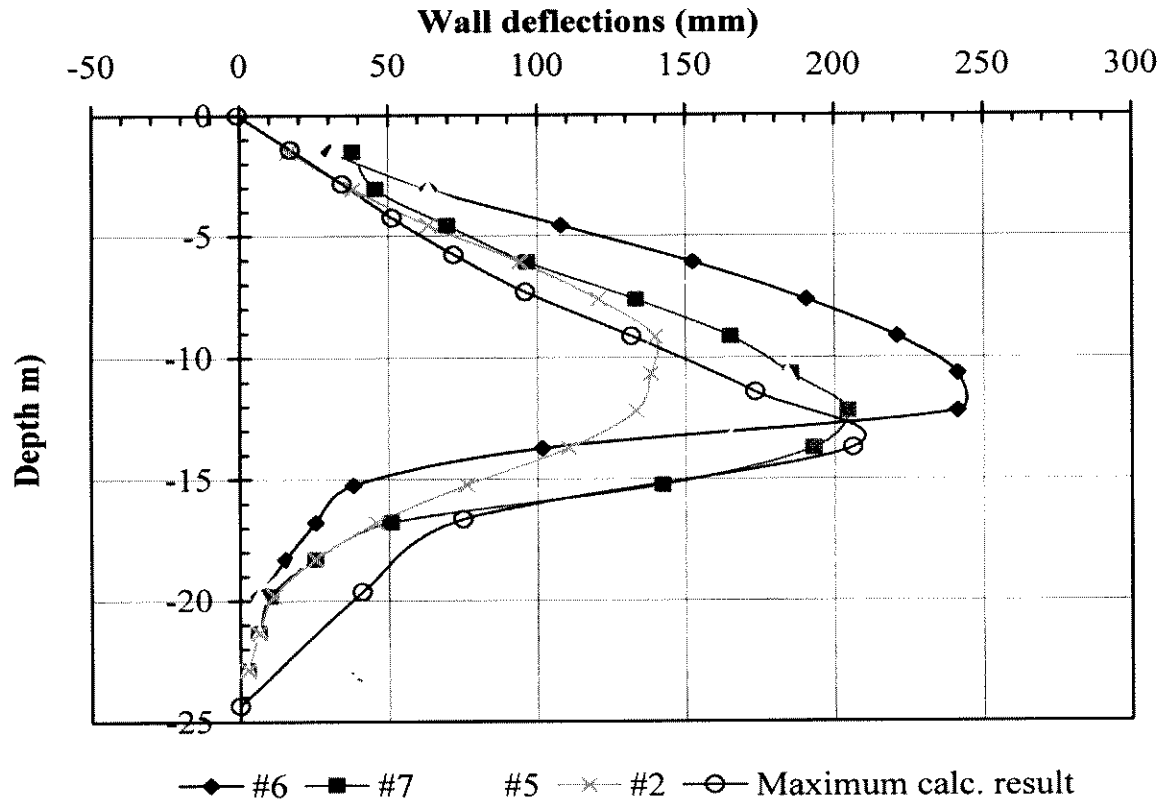


Figure 7.27: Comparison of the finite element results and the measured data for inclinometer numbers 2,5,6,7 (Davidson avenue) for maximum horizontal wall deflections.

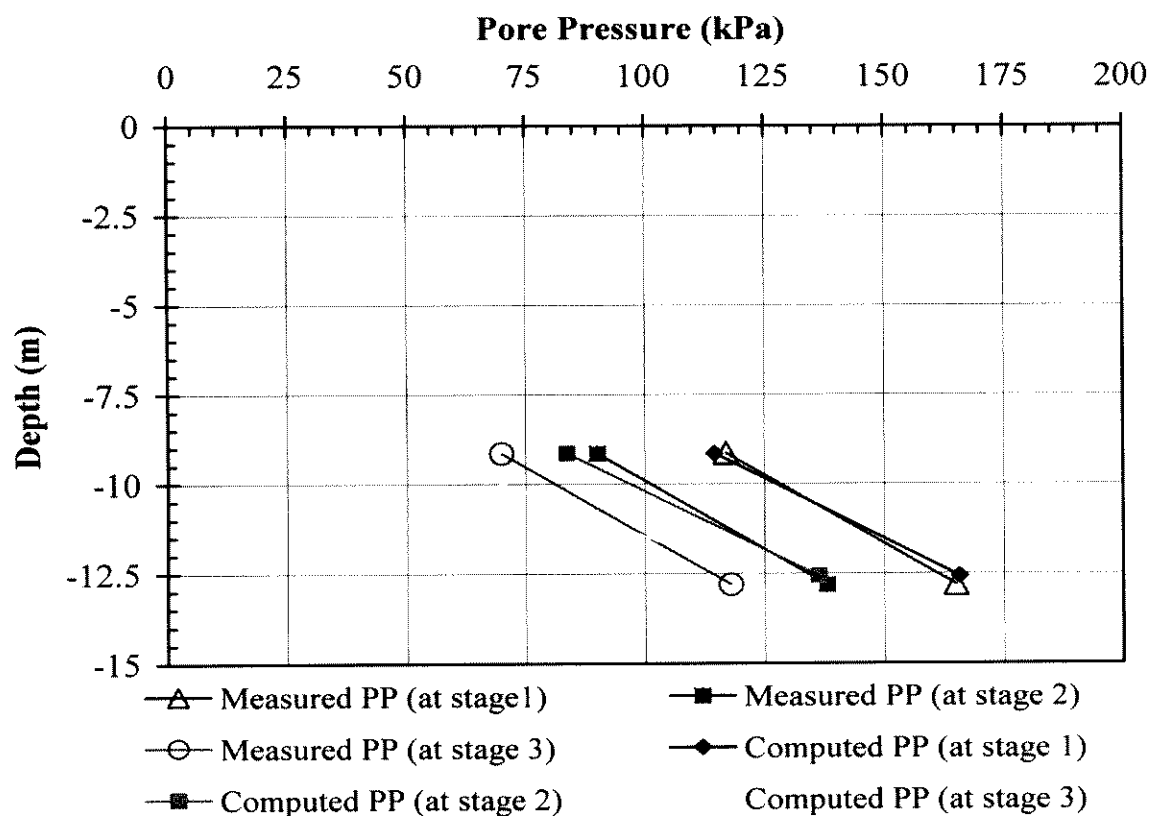


Figure 7.28: Comparison of the finite element results and the measured data at piezometer close to inclinometer number 6 (Davidson avenue).

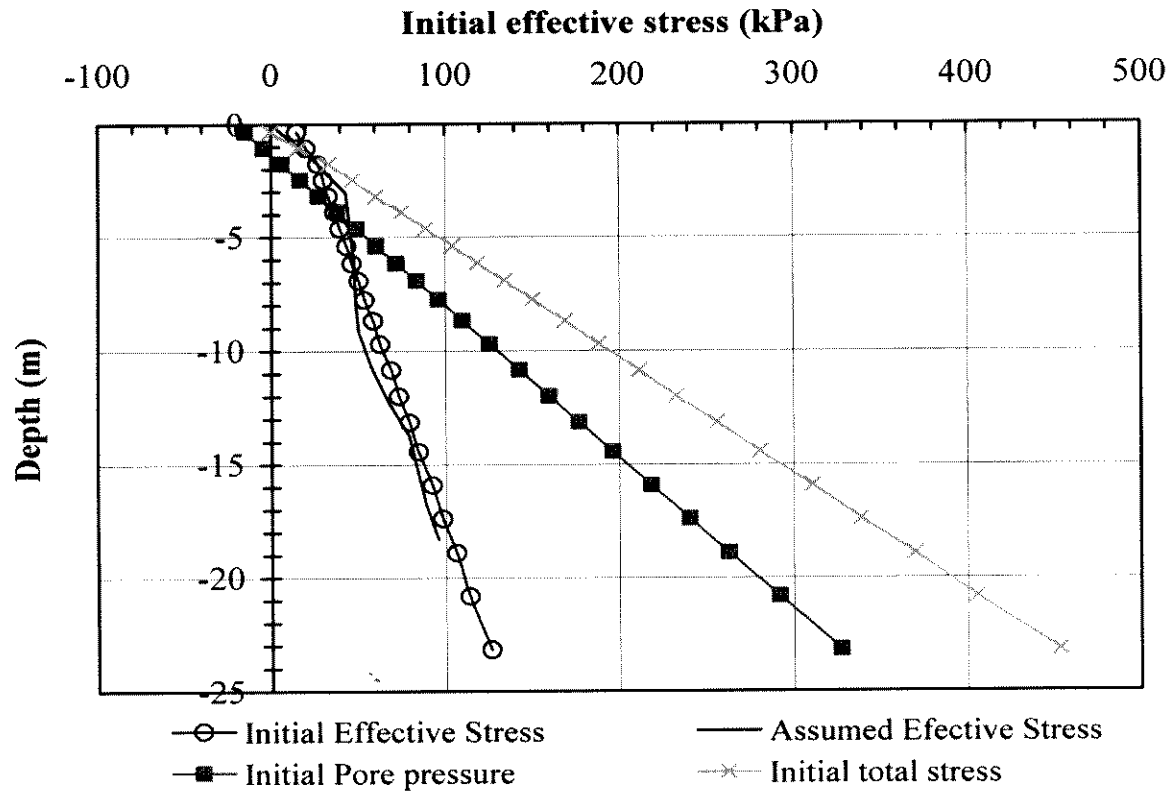


Figure 7.29: Comparison of the input and the measured data (Davidson avenue) for initial effective stresses.

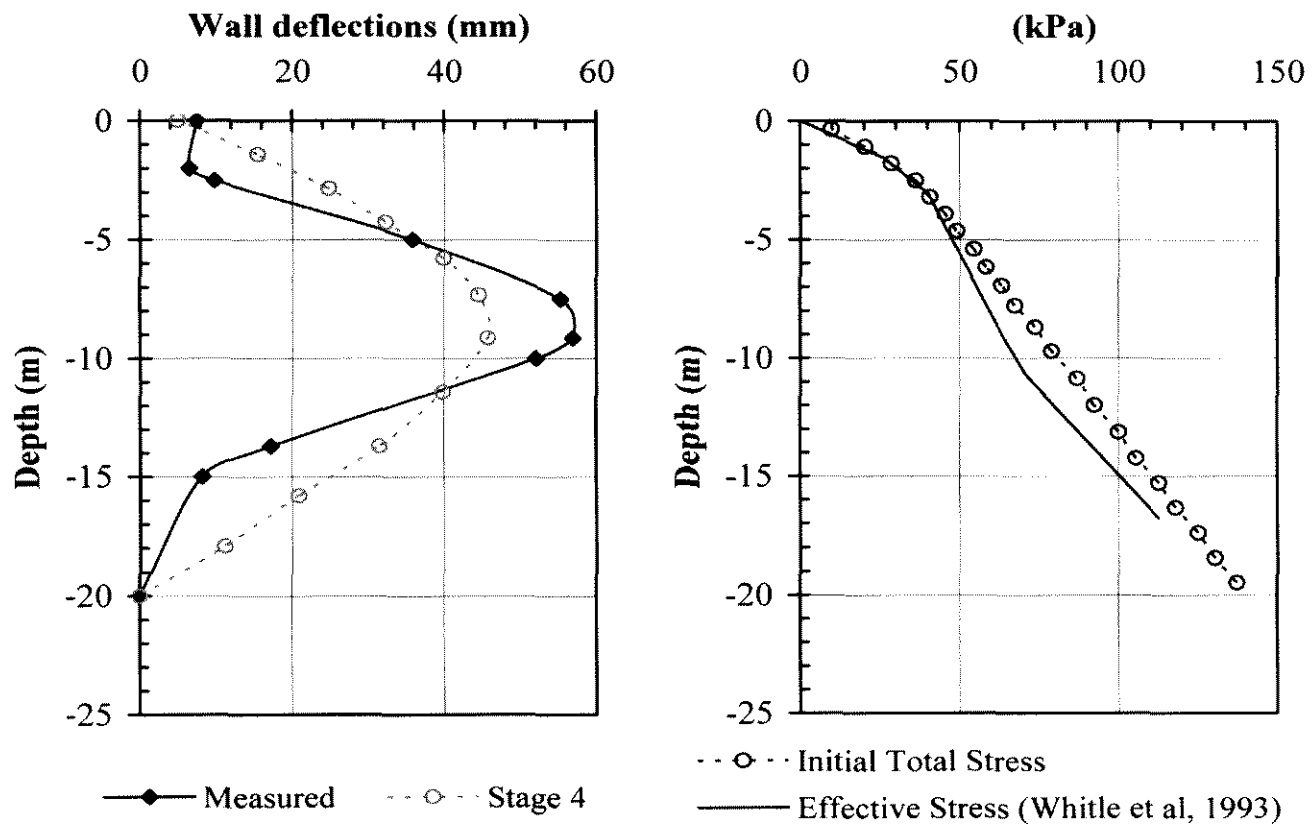


Figure 7.30: Maximum horizontal wall deflections and initial effective stress used in analysis. Comparisons of the finite element results and the measured data (Rankin street).

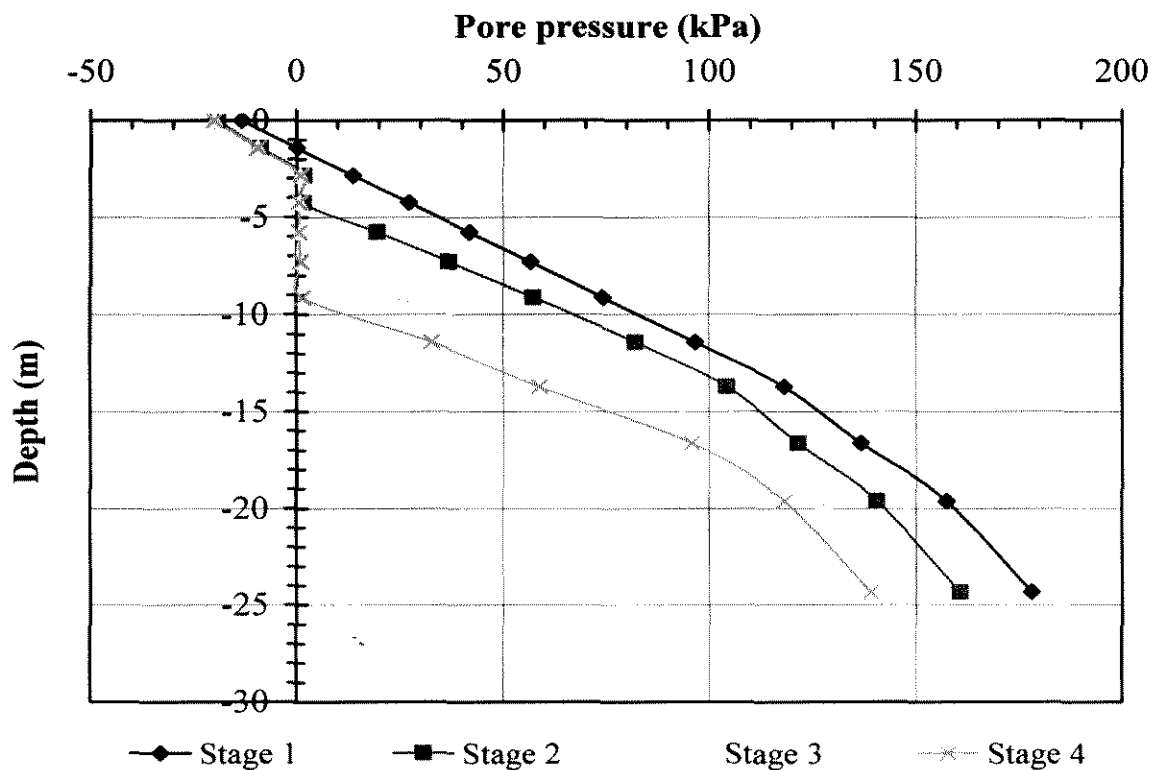


Figure 7.31: Pore pressure behind the wall taken from finite element analysis (Rankin street).

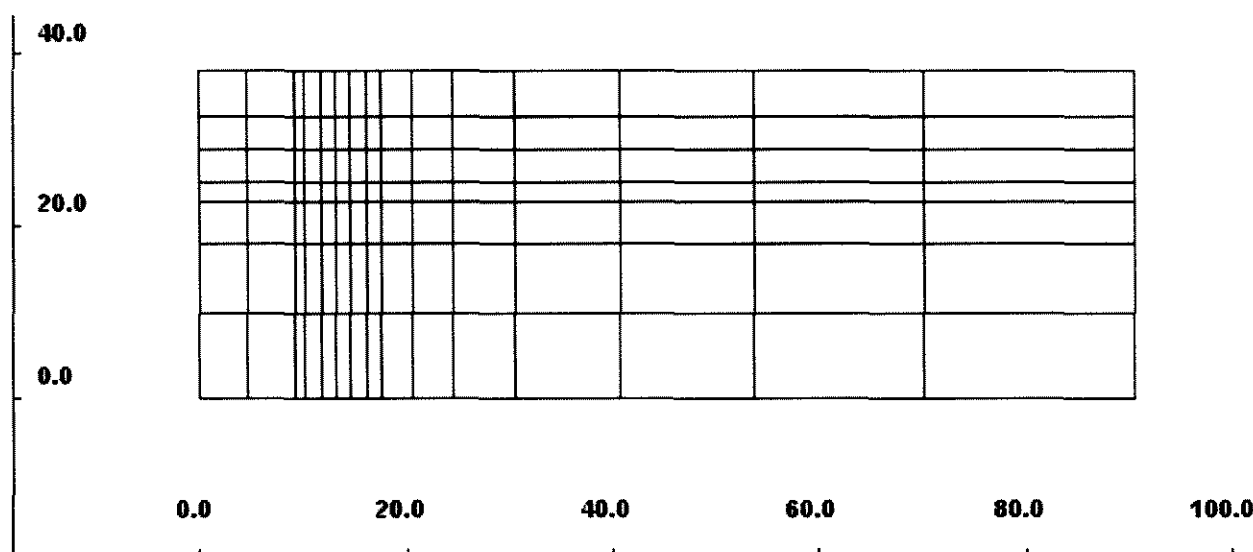
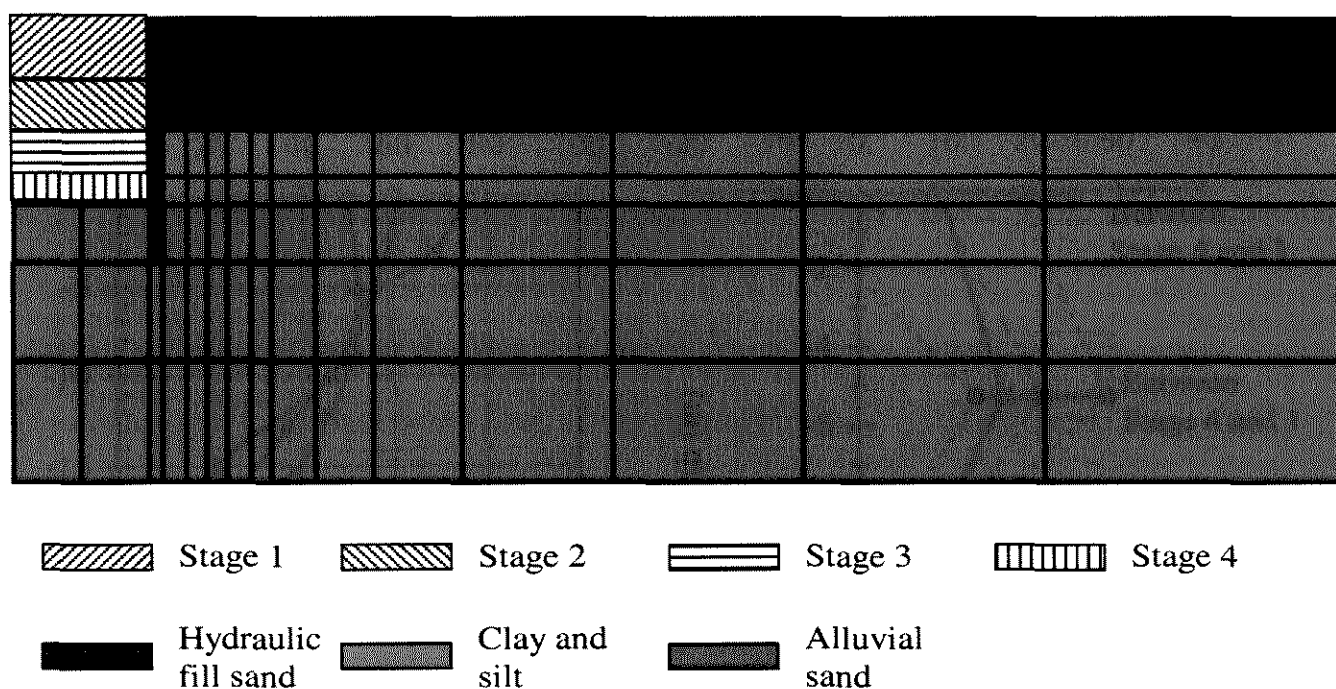


Figure 7.32: Soil mesh used in finite element analysis of large cavitation channel excavation, Memphis, Tennessee.



Note: Stages 5 and 6 are waiting time periods to complete construction of a concrete vault inside the excavation and to release the tiebacks.

Figure 7.33: Excavation sequences.

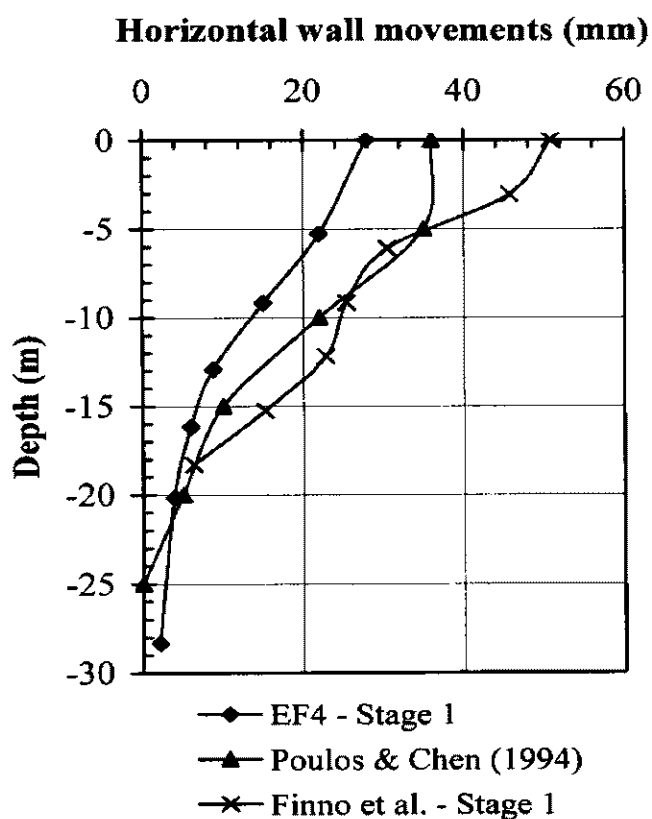


Figure 7.34: Horizontal wall deformation (Stage 1).

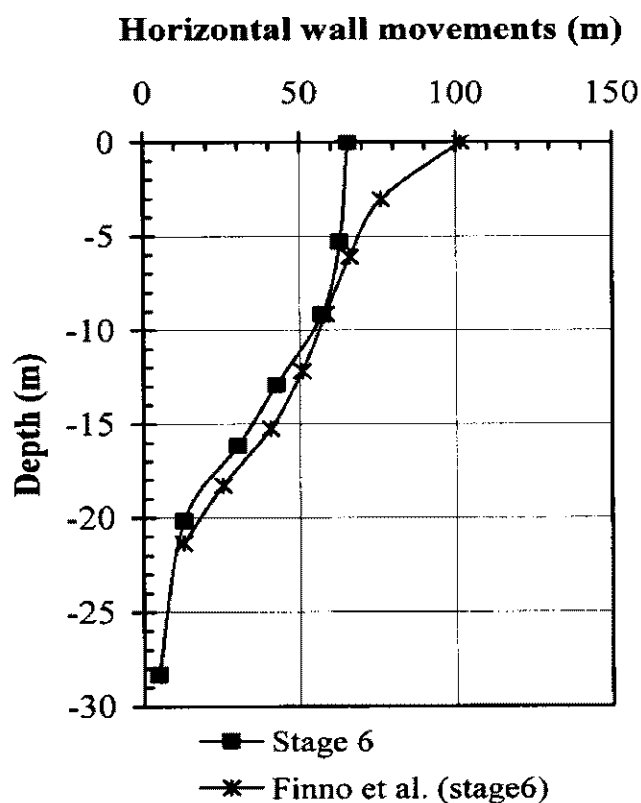


Figure 7.35: Horizontal wall deformation (Stage 6).

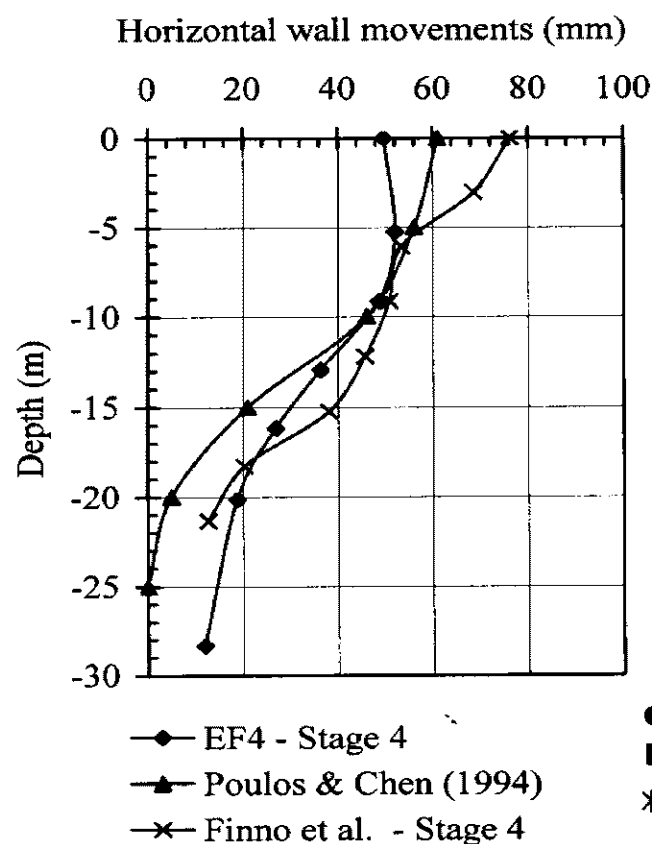


Figure 7.36: Horizontal wall deformation (Stage 4).

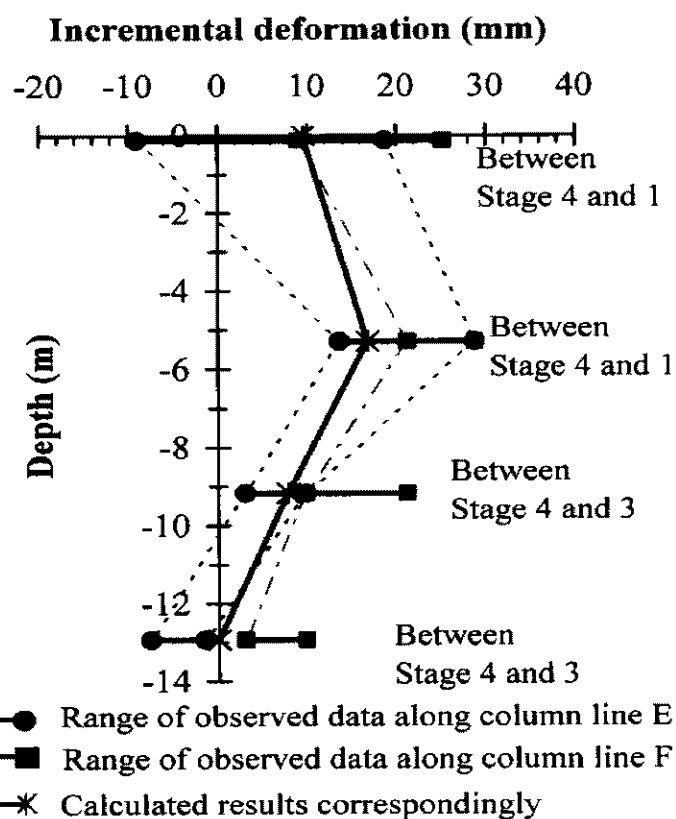


Figure 7.37: Incremental horizontal wall deformation between stages (4 and 1 or 3).

CHAPTER 8

PILE BEHAVIOUR DUE TO EXCAVATION-INDUCED SOIL MOVEMENTS

8.1 INTRODUCTION

In urban areas where buildings are normally close together, any construction activities that involve excavation such as basement construction, tunnelling and underground structures may require digging next to existing buildings. Hence, cases, where excavation is carried out next to an existing pile foundation, are likely to occur, it seems in congested areas high-rise buildings with pile foundations are common. In this chapter, the influence of excavation induced soil movements on piles is investigated. Moreover during the excavation it is often necessary to prevent water flooding the excavation site to allow easy construction by using methods to lower the ground water around the excavation site before or during the process of excavation. As may be expected, the analysis of these processes will involve drained analysis of both the excavation and the behaviour of the pile foundation nearby.

The first part of this chapter will present some case studies that include both actual full-scale excavations and centrifuge cases studies. This is to show that the theory presented can reproduce actual field behaviour. The second part of the chapter will present results for a hypothetical case of behaviour for a pile foundation next to drained and undrained excavations. The last part of the chapter will be a parametric study of the behaviour of existing pile foundations in the vicinity of a drained excavation. The parametric study can be used for cases where the excavation process involves pumping of the ground water at the site.

8.2 CASE STUDY

8.2.1 LEUNG (2001) CENTRIFUGE MODELLING OF EXCAVATION IN SOFT KAOLIN

Leung et al (2001) presented the results of a centrifuge modelling test on pile behaviour due to excavation-induced soil movements in Kaolin. The finite element analysis to model the centrifuge tests was carried out using the finite element program EF4. The excavation was approximately modelled as a plane strain excavation. As the centrifuge model of the excavation had dimensions of 540mm long x 200mm wide and 250mm high and the tests were conducted under 50g, the prototype excavation that was modelled by using finite elements was 27m long x 10m wide and 12.5m

high. For each excavation step 0.5m of soil depth was excavated to reach the maximum excavation depth, which was 4.0-4.5m. In the finite element analysis 3.0m was the maximum excavation depth because after that the wall collapsed and the analysis could not be continued. Figure 8.1 shows the finite element mesh that was used for the excavation analysis. The retaining wall was 160mm thick and was installed to 8m depth. The bending stiffness of the wall is equivalent to 24×10^3 kNm²/m. As very soft, Kaolin clay was used in the centrifuge tests, the soil behaviour model in the finite element analysis was chosen to be Modified Cam Clay as that often to be the best model to describe the behaviour of this type of clay soil. The soil properties, which are provided by professor Y K Chow, the National University of Singapore, are presented in Table 8.1.

Table 8. 1: Over consolidated Malaysian Kaolin properties.

Symbol	Over consolidated Kaolin
K_o	0.792
λ	0.4
κ	0.05
M	1.0
ν	0.3
$\gamma_s(\text{kN/m}^3)$	16.5
e_o	2.0
$k_x = k_y (\text{m/day})$	1.0E-03

As the excavation was carried out, the soil movements were induced. According to the information provided in the paper by Leung *et al.* (2001), the time to excavate was rather short hence the excavation can be considered as an undrained process. The behaviour of the pile installed at 1m, 3m, and 4m behind the wall can also be considered as undrained. The prototype flexural rigidity of the pile was determined as $2.2\text{E}+05$ kNm².

Consequently, the limit pressures that are used to calculate pile-bending moments induced by horizontal soil movements was taken to be about $3s_u$ for a distance of $3d$ close to the ground surface and $9s_u$ all along the pile length below a depth of $3d$. The undrained shear strength of Kaolin is approximately 10kPa and Young's soil modulus given for Kaolin is $E_s = 200s_u = 2000$ kPa.

The comparisons between the pile bending moments from the analysis and the measured results are given in Figures 8.2 (a) to (c). Good agreement may be seen between the numerical and the measured results except for the pile bending moments induced when the excavation is at 1m depth. The differences may be that the numerical analysis not only under-predicts the soil movements in the first period of the excavation process but also that it must predict the wrong shape of the deflection profile. The error in the moments at low excavation depth (e.g. 1m) could be due to the calculated cantilever action of the wall that is not observed in the centrifuge model. No comparisons between the bending moments from the analysis and the centrifuge tests results were possible when the excavation was over 2.5m depth, as in the analysis, the soil fails and the movements become very large. As documented in Leung (2001), in two out of three centrifuge tests, the retaining wall collapsed at an excavation depth of approximately 2.5m. The last centrifuge excavation test was the only one carried out to the maximum depth of 3m.

8.2.2 GOH *et al.* (2003) FULL-SCALE EXCAVATION NEXT TO A SINGLE PILE

The measured results of actual full-scale behaviour of an existing pile due to nearby deep excavation activities was presented by Goh *et al.* (2003). In that paper, the authors described the construction of a cut and cover tunnel as part of a new Mass rapid Transit Line in Singapore. This cut and cover tunnel was 127.5m long and 20m wide, with an excavation depth of 16m. The excavation was supported by 31m long and 0.8m thick diaphragm walls with six levels of struts. The strut arrangement is shown in Figure 8.3. Figure 8.4 shows the plan layout of piles and inclinometers. The inclinometers were installed in pile 17A and in the soil at the same distance from the wall, which was 3m. The distance between pile 17A and the inclinometer that was installed in the soil was 6m.

Soil properties for the finite element analysis input data have been provided in Table 8.2. The soil properties values are chosen based on the report of Saharma *et al.*, (1999) and the correlation between SPT values and soil deformation characteristics is based on Terzaghi (1955). Loose sand, medium sand and dense sand have SPT values respectively less than 10 blows/30cm, between 10 and 30 blows/30cm, and between 30 and 50 blows/30cm.

Table 8.2: Soil properties.

Stratum	E'_s kPa	s_y	ν	k_x m/day	k_y m/day	SPT N	s_u kPa	γ_s kN/m ³
Sand fill	1.0+04	0.001	0.3	1E-01	1E-01	4	-	17
Loose sand	1.5E+04	0.001	0.3	1E-04	1E-04	7	-	17
Marine clay	3.0E+03	0.001	0.3	8.64E-5	8.64E-5	-	10	16
Old alluvium I	2.E+04 -6.E+04	0.001	0.3	1.62E-2	0.54E-2	9 - 41	-	19
Old alluvium II	15E+04	0.001	0.3	0.55E-2	0.18E-2	>100	-	19

The diaphragm wall has a thickness of 0.8m and its drained Young's modulus has been taken as approximately 3.0E+07 kN/m². The excavation was also supported by six strut levels however the information on strut stiffness is not available. In this analysis strut stiffness is assumed to be 1.0E+05 kN/m/m. This value is taken from the data of other supported excavations that have a similar depth.

This problem was firstly analysed treating the piles as a continuous wall. Figure 8.5 shows the comparison of measured and calculated horizontal soil movements at a distance of 3m away from the retaining wall. There is good agreement between measured and calculated values of the maximum horizontal soil movements. The peak of the maximum horizontal soil movements occur at the same depth for both measured and calculated results. The finite element analysis results seem to

underestimate the horizontal soil movements of the soil layers above the soft marine clay layer. This may be because the elastic soil model used in the finite element analysis for the loose sand layers is not appropriate, and the behaviour of 3m of fill material near the ground surface is very difficult to predict. The finite element analysis also overestimates the horizontal soil displacement at depths below the peak position.

In this case study, a row of piles was installed along the retaining wall at a distance of 3m from the excavation, as shown in Figure 8.4. The finite element analysis was used to model the row of piles at 3m away from the retaining wall. However, the finite element analysis used here assumed plane strain conditions so that an equivalent value of the pile Young's modulus was employed instead of the real value of the pile Young's modulus to represent the pile row as a wall. The equivalent Young's modulus of the smeared pile-wall is calculated using the following equation:

$$E_p I_p + E_s I_s = E_{eq} I_{eq} \quad (8.1)$$

where: $E_p I_p$ is pile flexural rigidity; $E_s I_s$ is the flexural rigidity of the soil between two piles; and $E_{eq} I_{eq}$ is the equivalent flexural rigidity of the equivalent smeared wall.

The thickness of the smeared wall was assumed to be 1m. The soil Young's modulus was taken as the averaged value of Young's modulus of the soil layer within 30m depth where the horizontal soil movements mostly occur. Hence, soil Young's modulus, E_s , is equal to 20 MPa. The equivalent value of the pile wall Young's modulus is therefore 5.9E+06 kN/m². The computed and measured horizontal pile deflections are given in Figure 8.6. As can be seen, the finite element analysis gives larger horizontal movements of the pile than the actual measured values. This indicates that the equivalent wall in the finite element analysis may have been too flexible.

An approximate value of Young's modulus of concrete piles was applied to the equivalent wall, i.e. 3.0E+07 kN/m². Pile deflection results from the finite element analysis are close to the measured data though deflections are over predicted (Figure 8.7). However most of the overestimated results of the lateral pile deflections occur at

a depth below the peak of horizontal soil movements and the finite element gives the same overestimated answer for horizontal soil movements. Hence an approximate method to model a row of piles as an equivalent wall is inaccurate because in this case, the lateral soil movement is larger than the lateral pile movement. Modelling the pile row as an equivalent wall simply treats the piles and soil between them as a single material with the same movements and flexible stiffness. By doing this the piles become more flexible than they are and the soil between the piles is stiffer than the real soil. In the case where the piles are so flexible that their deflections are similar to the soil movement profiles, or where the lateral movement is so small that soil still behaves as an elastic material, then this approximate method can be applied. A more accurate analysis would be a 3D analysis with a proper model for soil behaviour.

Figure 8.8 presents a comparison between measured and calculated horizontal pile deflection due to horizontal soil movements with an equivalent pile row Young's modulus of $5.0\text{E}+07 \text{ kN/m}^2$. In this case there is better agreement between the measured and calculated behaviour.

The bending moments and lateral pile deflection analysis results are presented in Figures 8.9 to 8.12. Figures 8.9 and 8.10 are comparisons of the analysed bending moments and pile deflections (using PALLAS) with the measured values correspondingly. The pile Young's modulus value is taken as $3.0\text{E}+07 \text{ kN/m}^2$. The input lateral soil movements in PALLAS are directly from measured data (Figures 8.9 and 8.10) and from the finite element analysis (Figure 8.11 and 8.12). With the measured input lateral soil movements, PALLAS gives very close agreement between the measured and calculated results of the pile bending moments and lateral deflections. At excavation stage A, the pile bending moment given by PALLAS is very close to the measured one while the pile deflection comparison is not as good as the bending moment comparison (Figures 8.9a and 8.10a). At excavation stage B, the analysed pile deflection curve agreed well with the calculated results. However the numerical analysis underestimates the pile bending moments.

Rather good comparison of the pile bending moment and the lateral pile deflection are obtained, when the PALLAS input is lateral soil movement from a finite element

analysis, as presented in Figures 8.11 and 8.12. Hence, PALLAS appears to be a reliable tool for analysing lateral pile behaviour, especially when the input lateral soil movement is from measured data. In cases where the measured horizontal soil movement is not available and the soil movement is taken from finite element analysis for PALLAS, the calculated horizontal pile deflection and bending moments are also quite close to the measured data. In the case where the pile flexural stiffness is rather high compared to the horizontal soil stiffness, the method of treating a row of piles as a smeared wall with an equivalent Young's modulus is not accurate. In this case, a 3D analysis is required or else a separate analysis of lateral soil movements and pile behaviour is needed.

8.2.3 LEUNG (2000) CENTRIFUGE MODELLING OF EXCAVATION IN DENSE SAND NEXT TO A SINGLE PILE

Leung et al (2000) presented a series of centrifuge tests to investigate the behaviour of a single pile subjected to excavation-induced lateral soil movements in dense sand. Recently, there was a series of centrifuge tests that had been carried out by Leung *et al.* (2003) to investigate the behaviour of a pile group due to excavation-induced soil movements in dense sand. However, due to the lack of soil properties and also the measured soil movement profiles, only pile behaviour analysis has been carried out. The finite element analysis that is supposed to be used for soil movement prediction is not applied in this case.

The PALLAS program was used to investigate the pile behaviour in this case study. The soil properties and pile geometry are given below:
The soil Young's modulus that was given by Leung *et al.* (2000) is proportion to the depth z as follow:

$$E_s = 6z \text{ (MPa)} \quad (8.2)$$

This was based on cone penetration tests.

- The ultimate soil pressure acting on a pile, W_y , can be determined by using Broms's (1964a, b) equation for a pile installed in sand.

- The single pile diameter is 0.63m and it is 12.5m long. It was installed at a distance of 3m away from the retaining wall in the prototype model. The flexural stiffness of the pile $E_p I_p = 2.2\text{E}+05 \text{ kNm}^2$.
- The simplified horizontal soil movement profile is given in Figure 8.13(a). The lateral soil movements profiles were measured at the final excavation step of 4.5m depth.

Figures 8.13(b) and (c) present a comparison of horizontal pile deflections and bending moments between the measured values and the numerical analysis results. As can be seen from Figure 8.13, the numerical results lateral behaviour of the single pile at a distance of 1m from the retaining wall are over-predicted. The lateral pile deflection and the maximum bending moment are about 35% and 30% over-predicted respectively. However, good agreement of the numerical results and the measured data for lateral pile deflection and bending moments of the single pile at a distance of 3m and 5m from the retaining wall can be seen. The calculated distribution of bending moments with depth and the position of the maximum bending moments agree well with the measured data.

8.3 HYPOTHETICAL ANALYSIS OF PILE BEHAVIOUR DUE TO INDUCED SOIL MOVEMENTS BEHIND A BRACED EXCAVATION

8.3.1 INTRODUCTION

In this part, hypothetical analysis of a specific braced excavation is presented. There are two analyses carried out; undrained and drained analysis. The undrained analysis is performed by using the finite element program AFENA. In this calculation, the elasto-plastic Mohr-Coulomb model is employed to model the soil behaviour. The drained analysis is carried out by using the finite element program EF4 with an elastic soil model to describe the soil behaviour. The purpose of this analysis is to provide comparisons of soil movements induced by braced excavation under two conditions the undrained and drained conditions. The soil movements induced by both undrained and drained excavation will be used with the pile analysis programs PALLAS to analyse pile behaviour due to horizontal soil movements and PIES to analyse pile behaviour due to vertical soil movement. This study will give an understanding of the

pile behaviour near an excavation under two conditions (1) undrained excavation (2) drained excavation with water table drawdown.

8.3.2 EXCAVATION ANALYSIS

Program AFENA is used to analyse the undrained excavation. The finite element program AFENA was developed from computer code written by R. L. Taylor and published in Chapter 24 of “The Finite Element Method” (3rd edition) by O.C. Zienkiewicz (McGraw-Hill, 1977). The program has been enlarged and enhanced by Carter and Balaam (2000) much and now has many additional features but it has retained the very versatile structure of Taylor’s original code. Features of AFENA are particularly related to Geotechnical engineering, which includes the Mohr-Coulomb or Tresca failure criteria so that soils may deform plastically and flow according to an associated or non-associated flow rule. It also has features that allows removal or addition of material with self-weight that may be used for excavation or fill problems. The program is maintained by the Centre for Geotechnical Research, the University of Sydney. The reason for using both the finite element programs is that: (1) the finite element program AFENA can analyse both undrained analysis and totally drained analysis but without the lowering of ground water table (2) the finite element program EF4 can handle excavation with lowering of the ground water table. Hence, it is necessary to use these two programs to carry out analysis of undrained excavation and drained excavation with lowering of the ground water table are necessary.

The geometry of the specific excavation analysis is presented below.

- The maximum excavation depth is 10m and the half width of the excavation is 10m. The excavation is assumed to be in a homogeneous soil layer of thickness 30m. The first strut is assumed to be installed prior to the excavation to avoid large movement of the flexible sheet pile wall. The analysis is carried under plane strain conditions with half of the excavation discretized because of symmetry. The axis of symmetry is on the left and the far right boundary is chosen to be 60m away from the excavation wall, as can be seen in Figure 8.14.
- Two types of retaining walls are considered here is a sheet pile wall and a diaphragm wall. The depth to which the walls are installed is assumed to be 15m from

the ground surface. The sheet pile wall is considered to allow water to pass through in the drained analysis. The flexural stiffness of the sheet pile wall was chosen to be $EI_{s-wall} = 7.3E+04 \text{ kNm}^2/\text{m}$. The diaphragm wall is a 1.0m thick concrete wall and the flexural stiffness is $EI_{d-wall} = 2.5E+05 \text{ kNm}^2/\text{m}$. The diaphragm wall is assumed to be impermeable. The axial stiffness of the struts is $k_{st} = 5.8E+04 \text{ (kN/m/m)}$. The spacing of the struts is 2.5m. The strut systems are similar in both drained and undrained cases.

- The 30m thick homogeneous soil layer thick has the properties given in Table 8.2 for both undrained and drained analysis.

Table 8.2 Soil properties used for analysis

Symbol	Undrained values	Drained values
Young's modulus (kN/m^2)	11550	10000
Poisson's ratio	0.49	0.3
Unit weight (kN/m^3)	19.6	19.6
Permeability $k_x = k_y$ (m/day)	-	0.001
Coefficient of horizontal earth pressure at rest K_o	0.8	0.5

The Young's modulus value used for drained analysis E'_s is equal to 10MPa, while the Young's modulus for the undrained analysis is obtained from the drained Young's modulus by applying the following equation:

$$E_s = \frac{1.5 E'_s}{1 + \nu'} \quad (8.3)$$

In undrained analysis, the value of Poisson's ratio is assumed as 0.49 to get an approximate incompressible material. The analysis results are presented in Figures 8.15 to 8.18.

From Figures 8.15 to 8.18 some conclusions can be drawn as follows:

In the undrained and drained analysis, the maximum magnitudes of horizontal soil movements in the undrained analysis is always higher than in the drained analysis in the area close to the wall. In the area further away from the wall (20m away from the wall for the diaphragm wall and 10m away from the wall for the sheet pile wall, the maximum magnitudes of horizontal soil movements in the drained condition are calculated larger than the results for the undrained analysis.

The profile of horizontal soil movement reaches a maximum and as the distance to the wall increases over about 20m, twice the excavation depth, the horizontal soil movement reduces significantly.

- In both cases undrained and drained analysis, the maximum horizontal soil movement magnitude at all distances from the wall experience less than 5% difference between different walls (diaphragm and sheet pile walls). This conclusion is also recognized by Long (2001) for cases where retaining walls were installed in stiff soil with a large value of FOS against excavation base heave. The data indicate that less stiff walls may perform adequately in many instances and worldwide design practice may be somewhat conservative.

However the positions of the maximum horizontal soil movements are different for the diaphragm wall and the sheet pile wall. While the position of the maximum horizontal soil movement in the diaphragm wall case is more or less at the wall tip, its position in the case of the sheet pile wall is above the tip regardless of the pile tip position. This can be agreed by Richards and Powrie (1998), who used centrifuge tests to model a braced excavation in over consolidated clay with differences of wall stiffness. The authors concluded that the maximum magnitudes of horizontal soil movements are independent of the depth to which the wall is installed.

8.3.3 PILE BEHAVIOUR ANALYSIS

In the analysis it is assumed that there is a single pile placed next to the excavation at a distance prior to the excavation construction carried out. The soil movements induced by the excavation processes obviously influence the behaviour of the single pile. The soil next to the wall deforms as the excavation depth increases. Figure 8.19 shows a schematic diagram of the case where a single pile exists at a distance from the wall.

The analysis of horizontal pile behaviour is carried out by employing boundary element methods, PALLAS for horizontal pile behaviour and PIES for vertical pile behaviour.

The assumptions for pile geometry are presented in the following:

- Pile diameter is $d = 0.5\text{m}$, pile length is $L_p = 30\text{m}$, so that the pile reaches the bottom of the homogeneous soil layer described above, and Young's modulus is 30000MPa .

The pile is assumed to be at distances of 1m, 5m, 10m, 20m, and 50m away from the retaining wall.

- Soil properties that are used to calculate the horizontal pile behaviour are given in the following:

For undrained analysis, shear strength is defined by applying a relationship between Young's soil modulus and undrained shear strength $E_s = 400s_u$. Undrained Young's soil modulus is $E_s = 11550(\text{kN/m}^2)$ then $s_u = 11550/400 = 28.87$ (kPa). The limiting pile-soil pressure is defined by $W_y = 3s_u$ at the surface and $W_y = 9s_u$ at depths greater than about 3 times the pile diameter. Then the limit pile-soil pressures will be $86.61(\text{kN/m}^2)$ and $259.61(\text{kN/m}^2)$ respectively.

For drained analysis the limiting pile-soil pressure depends on the effective stress level that is defined by using the following equation

$$W_y = 3K_p\sigma'_v \quad (8.4)$$

where $K_p = \frac{1 + \sin \varphi'}{1 - \sin \varphi'}$, for clay $\varphi' = 26^\circ$ to 28° , for sand $\varphi' = 28^\circ$ to 42° and σ'_v is effective stress at the depth considered.

This means that W_y equals $0.0(\text{kPa})$ at the ground surface and approximately $750(\text{kPa})$ at the bottom of the 30m thick homogeneous soil layer. The ratio of peak pile-soil pressure over the ultimate pile-soil pressure is taken as unity and the yielding displacement is considered to be 20% of the pile diameter for undrained analysis (Matlock, 1970) and 3.75% of the pile diameter for drained analysis (Reese et al, 1974).

Figure 8.20 shows the bending moments in a single pile at different distances from the retaining walls caused by horizontal soil movements, and Figures 8.21 and 8.22 show

the behaviour of a 3x3 pile group due to horizontal soil movements with spacing between piles in the group equal to 3 times the pile diameter. For the pile group of 3x3 piles with a spacing between piles of 1.5m, the soil movements are not much different over a distance of 1.5m therefore each single pile in the pile group is assumed to sustain the same soil movement profile and that it is similar to the soil movement at the pile which is closest to the retaining wall. All piles in the pile group have the fixed pile head condition preventing it from rotation.

From these figures, the following points can be observed:

Bending moments induced in a pile due to horizontal soil movements under undrained analysis are normally much larger than under drained analysis. This conclusion is also made by Leung (2001) from a study of centrifuge tests for an excavation next to an existing single pile.

- Due to the fixed head pile condition, the induced bending moments in piles in a pile group at the pile head level are always higher than a free head single pile at the same distance from the retaining wall.
- While the induced bending moments in a pile group under drained conditions gradually reduce as the distance from the retaining wall increases, the induced bending moments in a pile group under undrained conditions decrease rapidly. The single pile only experiences a gradual reduction in bending moments as the distance to the retaining wall increases in both the drained and undrained analysis.
- Assuming that the pile is concreted and contains 2.5% of the axial section area of steel reinforcement all along the pile length, the allowable bending moment of the pile is about 250kNm. As can be seen, a pile in a group that was installed at a distance less than 15m from the retaining wall could have an induced moment that exceeds this allowable moment and could be damaged if that the excavation is carried out under undrained conditions.
- Undrained analysis of excavation can produce more severe maximum bending moment on nearby piles than drained analysis because of the fact that the undrained pile-soil limit pressure is more than the drained pile soil limit pressure.

8.4 PARAMETRIC STUDY OF EXCAVATION WITH WATER TABLE DRAWDOWN

8.4.1 INTRODUCTION

As introduced above, excavation-induced soil movements can cause an impact on existing pile foundations nearby. Some methods have been developed in the past to predict the behaviour of a pile foundation that is close to an excavation. Chen and Poulos (1996, 1997) used a finite element analysis to obtain soil movements due to excavation and then the results of soil movements from the finite element analysis became the input data for pile behaviour analysis that employed a boundary element technique. A series of design charts presenting undrained behaviour of a pile were produced. Chow and Young (1996) also introduced a method that allows analysis of pile behaviour due to horizontal soil movements that can be employed to examine the case where a pile foundation is next to an excavation. Other researchers have carried out centrifuge tests to investigate pile behaviour due to a nearby excavation, for example, Leung *et al.* (2000, 2001, and 2003).

In reality, pumping water to lower the ground water table prior to or even during excavation is inevitable in any pervious soils and if there is an operating pile foundation in the neighbourhood of the excavation, soil movements will cause the pile to move. To understand this situation, a parametric study has been carried out that employs the same methods that Chen and Poulos (1996, 1997) used. Firstly, this parametric study will focus on investigating soil movements due to excavation with lowering of the ground water table. A fully coupled finite element program, EF4, is employed to analyse the soil movements. Then the pile foundation behaviour will be evaluated by applying boundary element methods.

8.4.2 EXCAVATION PARAMETRIC STUDY

A basic problem is considered for a typical excavation that is shown in Figure 8.14. The assumptions are as follows:

- The soil is a deep homogeneous layer in the drained condition. The drained Young's soil modulus $E_s = 10\text{MPa}$, and total unit weight is 19.6 kN/m^3 . The vertical

permeability of the soil is equal to the horizontal permeability and equal to 1.0×10^{-3} m/day.

- The width and maximum depth of the excavation are similar and equal to 10m.
- The flexural stiffness of the sheet pile wall is chosen to be as $EI_{s-wall} = 7.3E+04$ kNm²/m. The sheet pile wall is considered to be permeable. The axial stiffness of the struts is $k_{st} = 5.8E+04$ (kN/m/m). The vertical spacing of the struts is 2.5m.
- The water table is considered to be drawdown to the bottom of the excavation, and the water recharge condition is constant at the far boundary of the finite element mesh.

Soil movements due to a drained excavation with drawdown of water table for the basic case are presented in Figures 8.23 and 8.24.

To explore the effect of the different parameters on the induced soil movements due to excavation, a parametric study has been conducted with variation of the parameters as discussed in the following:

The geometry of excavation, the width of excavation B and the maximum depth of an excavation h ; the depth of the homogeneous soil layer H ; the stiffness of the retaining wall, EI_w ; the strut stiffness and vertical layout; k_s and s respectively are the parameters allowed to vary. As the maximum values of soil movements are the significant items to consider and the water table is allowed to be drawn down to the bottom of the excavation then the variation of the permeability of the soil is not included in the parametric study. However the dissipation of pore pressure is still coupled in the analysis of excavation.

Maximum horizontal soil movements $Max \delta_h$ and maximum vertical soil movements $Max \delta_v$ are estimated by using the following relationships:

Maximum horizontal soil movements:

$$Max \delta_h = Max \delta_{hb} \times k_E^h \times k_s^h \times k_h^h \times k_B^h \times k_H^h \quad (8.5)$$

Maximum vertical soil movements:

$$Max \delta_v = Max \delta_{vb} \times k_E^v \times k_s^v \times k_h^v \times k_B^v \times k_H^v \quad (8.6)$$

where $Max \delta_{vb}$ and $Max \delta_{hb}$ are maximum vertical and horizontal soil movements of the typical analysis (the base case analysis, Figures 8.23 and 8.24); k_E^h, k_E^v are correction factors for drained soil Young's modulus; k_s^h, k_s^v are correction factors for strut stiffness; k_h^h, k_h^v and k_B^h, k_B^v are correction factors for excavation depth and excavation width respectively; k_H^h, k_H^v are correction factors for the thickness of the homogeneous soil layer.

Figures 8.25 and 8.26 present the correlation factors of wall stiffness and strut stiffness for horizontal soil movements. As the wall stiffness and the strut stiffness increase the horizontal soil movements do not seem to vary much. Figures 8.27 to 8.31 and from 8.32 to 8.36 show the correction factors for maximum horizontal soil movements and maximum vertical soil surface movements at different distances from the retaining wall.

Obviously, soil deformation depends much on the deformation characteristics of the soil, which is represented here by Young's modulus. The stiffer the soil, the less soil deformation there is.

Other factors that influence the soil movements due to excavation are the excavation geometry itself, and both the excavation width and depth have an effect on the soil deformation. However, while the depth of excavation influences both horizontal and vertical soil movements, the width of excavation seems to affect only the horizontal soil movement. The vertical soil movement is less influenced by the width of excavation. This outcome can be explained by the fact that the drawdown of the water table contributes to the major part of the vertical soil movements. Hence, only the depth of excavation, and the depth to which the water table is assumed to be drawn down to, plays an important role in the vertical soil movements due to excavation with drawdown of the water table.

In addition, the depth of the homogeneous soil layer where the excavation is carried out also has an influence on the soil movements though the effect of the variation of the soil layer thickness is minor compared to other factors such as the excavation geometry factors. The increase in vertical strut spacing causes a rise in the horizontal

soil movement, but only a slight increase in vertical soil movements. It has been noted before (Figures 8.25, and 8.26) that soil movements due to excavation seem to be hardly affected by the retaining wall stiffnesses that were used in this study.

Comparison between the maximum induced soil movements that are calculated by using the design charts and the result from a particular theoretical case was carried out to prove that the correction factors used in this study could give acceptably accurate results. The soil properties, the excavation geometry, the strut and sheet pile wall stiffness, and the strut spacing are provided as follows:

- The soil is a deep homogeneous layer with a thickness of 30m and is in the drained condition. The drained Young's soil modulus $E_s = 20\text{MPa}$, and total unit weight is 19.6 kN/m^3 . The vertical permeability of the soil is equal to the horizontal permeability and equal to $1.0 \times 10^{-3} \text{ m/day}$.
- The width and maximum depth of the excavation are equal to 15m and 7.5m respectively.
- The flexural stiffness of the sheet pile wall was chosen to be $EI_{s\text{-wall}} = 2.0\text{E}+04 \text{ kNm}^2/\text{m}$. The sheet pile wall is considered to be permeable. The axial stiffness of the struts is $k_{st} = 2.9\text{E}+04 \text{ (kN/m/m)}$. The vertical spacing of the struts is 5.0m.
- The water table was considered to be drawn down to the bottom of the excavation, and the water level was assumed constant at the far boundary of the finite element mesh.

From Figure 8.23c, the maximum horizontal soil movement for the base case is $Max. \delta_{hb} = 0.06\text{m}$ at a distance of 10m from the wall. From Figures 8.25 to 8.31, the correlation factors are as follows: for drained Young's soil modulus $E'_s = 20\text{MPa}$, $k^h_E = 0.5$; for strut stiffness $k_{st} = 2.9\text{E}+04 \text{ (kN/m/m)}$, $k^h_s = 1.0$; for an excavation depth of 7.5m, $k^h_h = 0.85$; for an excavation with of 15m, $k^h_B = 1.35$; for a soil layer thickness of 30m, $k^h_H = 1$. From equation 8.5, $Max. \delta_{hb} = 0.06 \times 0.5 \times 1 \times 0.85 \times 1.35 \times 1 = 0.035\text{m}$. Figure 8.32a shows the horizontal soil movement at different distances from the wall calculated by using EF4. At a distance of 10m away from the wall, maximum horizontal soil movement is 0.031m. This result is very close to the results obtained by using the design charts.

From Figure 8.24c, the maximum vertical soil movement for the base case is $Max. \delta_{vb} = 0.1\text{m}$ at a distance of 10m from the wall. From Figures 8.33 to 8.37, the correlation factors are as follows: for drained Young's soil modulus $E'_s = 20\text{MPa}$, $k_E^h = 0.5$; for strut stiffness $k_{st} = 2.9\text{E}+04$ (kN/m/m), $k'_s = 1.0$; for an excavation depth of 7.5m, $k_h^h = 0.8$; for excavation with of 15m, $k_B^h = 1.0$; for a soil layer thickness of 30m, $k_H^h = 1$. From equation 8.6, $Max. \delta_{hb} = 0.1 \times 0.5 \times 1 \times 0.8 \times 1.0 \times 1 = 0.04\text{m}$. Figure 8.32b shows the vertical soil movement at different distances from the wall calculated by using EF4. At a distance of 10m away from the wall, maximum vertical soil movement is 0.04m at the ground surface. The result obtained from the design charts is similar to the result that is calculated by using the finite element program.

8.5 PARAMETRIC STUDY OF PILE BEHAVIOUR

8.5.1 INTRODUCTION

A parametric study of pile behaviour next to an excavation with drawdown of the water table is presented. Chen and Poulos (1996, 1997) have studied pile behaviour next to an excavation in an undrained soil. In the previous part of this chapter, pile behaviour analysis of a hypothetical case where a single pile and a pile group of 3x3 piles exist next to an excavation have been conducted. Comparison between undrained and drained analysis of pile behaviour has been carried out. It was found that undrained pile behaviour analysis always gives a higher value of pile bending moment than drained analysis. However, in reality where excavation is conducted in company with drawdown of the water table to the bottom of the excavation (due to construction requirements) then undrained analysis overestimates horizontal pile behaviour but underestimates vertical pile behaviour. Hence, this parametric study is performed to investigate pile behaviour where there is a single pile existing next to an excavation in drained condition.

8.5.2 PILE ANALYSIS

A basic analysis of a single pile existing next to an excavation is shown in Figure 8.19b. As can be seen from the previous results of excavation analysis, the horizontal soil movements due to excavation with drawdown of the water table depend much on

the excavation geometry, the depth and the deformation characteristics of the homogeneous soil layer that the excavation is carried out in, and also the strut spacing. The retaining system in this case insignificantly influences the horizontal soil movements. Hence, the horizontal behaviour of a pile foundation involves only the geometry of excavation, the depth and the deformation characteristics of the homogeneous soil layer and the strut spacing. An equation is provided to show the influence of each factor on the horizontal pile behaviour.

$$\text{Max. } M^s = \text{Max. } M_b^s \times k_E^s \times k_d^s \times k_h^s \times k_B^s \times k_H^s \times k_s^s \quad (8.7)$$

$$\text{Max. } \rho^s = \text{Max. } \rho_b^s \times k_E'^s \times k_d'^s \times k_h'^s \times k_B'^s \times k_H'^s \times k_s'^s \quad (8.8)$$

where $\text{Max. } M^s$, and $\text{Max. } \rho^s$ are maximum bending moment, kNm, and maximum pile deflection, m, respectively for a single pile; $\text{Max. } M_b^s$ and $\text{Max. } \rho_b^s$ are maximum bending moments, kNm, and maximum horizontal pile deflection, m, respectively of a single pile for the base case; k_E^s and $k_E'^s$ are correction factors for drained Young's modulus; k_d^s and $k_d'^s$ are correction factors for pile diameter; k_h^s and $k_h'^s$ are correction factors for excavation depth; k_B^s and $k_B'^s$ are correction factors for excavation width; k_H^s and $k_H'^s$ are correction factors for the thickness of the homogeneous soil layer; k_s^s and $k_s'^s$ are correction factors for strut spacing.

Results for the maximum induced pile bending moment of the base case are provided in Figure 8.38. Figures 8.39 to 8.44 show the correction factors for maximum pile bending moment at different distances from the retaining wall. Results of lateral pile deflection are provided in Figure 8.45. Figures 8.46 to 8.51 show the correction factors for maximum horizontal pile deflection at different distances from the retaining wall.

Comparison between the maximum induced pile bending moment in this study, which is a drained analysis, and the undrained study results of Chen and Poulos (1997) is now carried out. The soil properties, the wall and strut system stiffness, the excavation geometry, and the pile diameter and pile distance to the excavation face are provided as follows:

- The drained Young's soil modulus $E'_s = 10\text{MPa}$, and total unit weight is 19.6 kN/m^3 . The vertical permeability of the soil is equal to the horizontal permeability and equal to $1.0\text{E-}03\text{ m/day}$.

The maximum depth of the excavation is equal to 10m , the width of excavation is 4.0m

- The flexural stiffness of the sheet pile wall is chosen to be as $EI_{s\text{-wall}} = 7.3\text{E+}04\text{ kNm}^2/\text{m}$. The sheet pile wall is considered to be permeable. The axial stiffness of the struts is $k_{st} = 5.8\text{E+}04\text{ (kN/m/m)}$. The vertical spacing of the struts is 2.5m .
- The pile diameter, d_p , is 0.5m , and the distance between the pile and the excavation face is 1m .

From Figure 8.38, $Max. M_b^s = 112\text{ kNm}$ for a pile at a distance of 1m from the excavation face. From Figures 8.39 to 8.44 the correction factors are as follows: for $E'_s = 10\text{MPa}$, $k_E^s = 1$, for $d_p = 0.5\text{m}$, $k_d^s = 1$, for $B = 4\text{m}$, $k_B^s \approx 0.98$, for $h = 10\text{m}$, $k_h^s = 1.0$, for $H = 22\text{m}$, $k_H^s \approx 1.25$, for $s = 2.5\text{m}$, $k_s^s = 1$, From equation 8.7 $Max. M^s = 112 \times 1 \times 1 \times 0.98 \times 1 \times 1.25 \times 1 = 140\text{ kNm}$.

From Chen and Poulos (1997), $Max. M^u = 314.49\text{ kNm}$ for the undrained case. This comparison shows that analysis of pile behaviour subjected to undrained excavation can produce the induced bending moment in pile that is larger than the drained analysis.

8.6 SUMMARY AND CONCLUSIONS

The analyses of three case studies have been carried out in the first part of this chapter to show that the theory presented can reproduce actual field behaviour. Of three case studies, one case study is analysis of an actual full-scale excavation with a row of piles existing next to the excavation. Other case studies are centrifuge tests. The finite element analyses have been conducted to calculate the soil movements due to excavation. Then the finite element analysis results for the soil movements are used as input data for the pile behaviour analysis programs. Comparisons between the calculated results of excavation induced soil movements and pile behaviour with the measured data of both centrifuge and full-scale tests indicate that the combination of the finite element analysis and the boundary element analysis can be use to predict

pile behaviour near an excavation with reasonable accuracy. The approximate method to model a row of piles as an equivalent wall near excavation needs to be used carefully.

The hypothetical analyses of pile behaviour subjected to excavation induced soil movements shows that the undrained excavation can produce more severe bending moments in piles than undrained excavation. Undrained excavation produces much smaller vertical soil movements than drained analysis.

A parametric study of drained excavation with ground water table drawdown has been carried out. The soil deformation depends much on the deformation characteristics of the soil, the excavation geometry and the depth of the homogeneous soil layer where excavation is carried out. The depth of excavation influences both the vertical and horizontal soil movements. The width of excavation only affects the horizontal soil movement. The depth of the homogeneous soil layer where the excavation is carried out also has an influence on the soil movements. Soil movements due to excavation seem to be hardly affected by the retaining wall stiffness that is used in this study. A parametric study of pile behaviour near an excavation with drawdown of the water table has also been conducted. Comparison between this study and the study of Chen and Poulos (1997) has been carried out to show that the undrained analysis can produce a greater induced pile bending moment than the drained analysis.

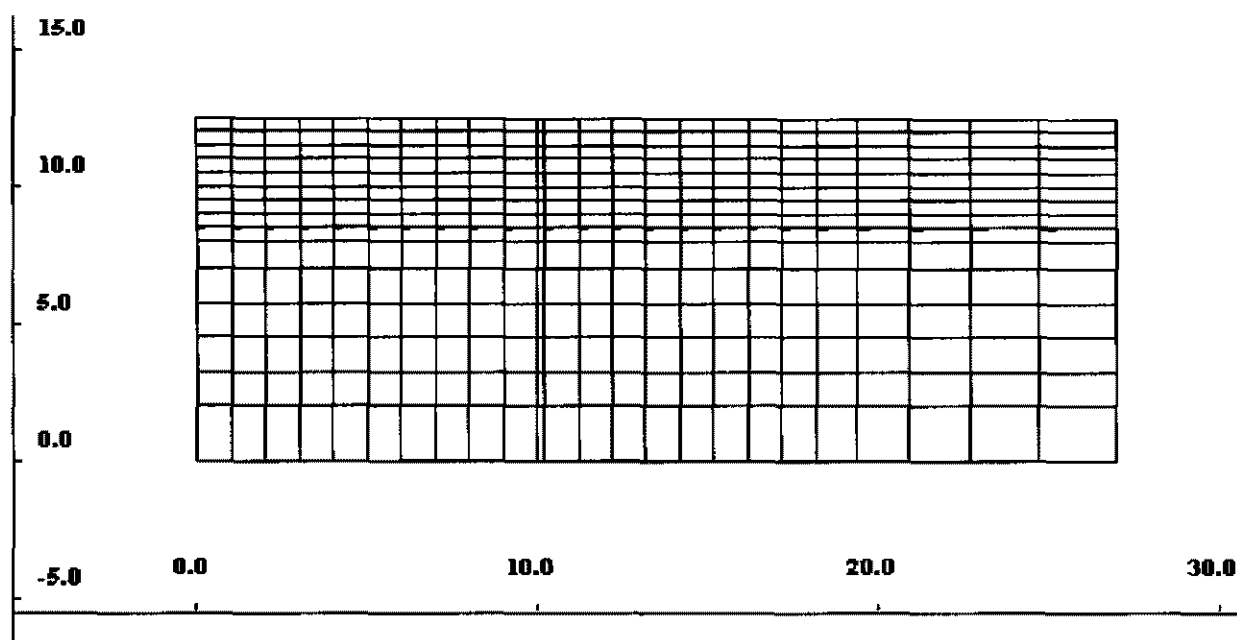
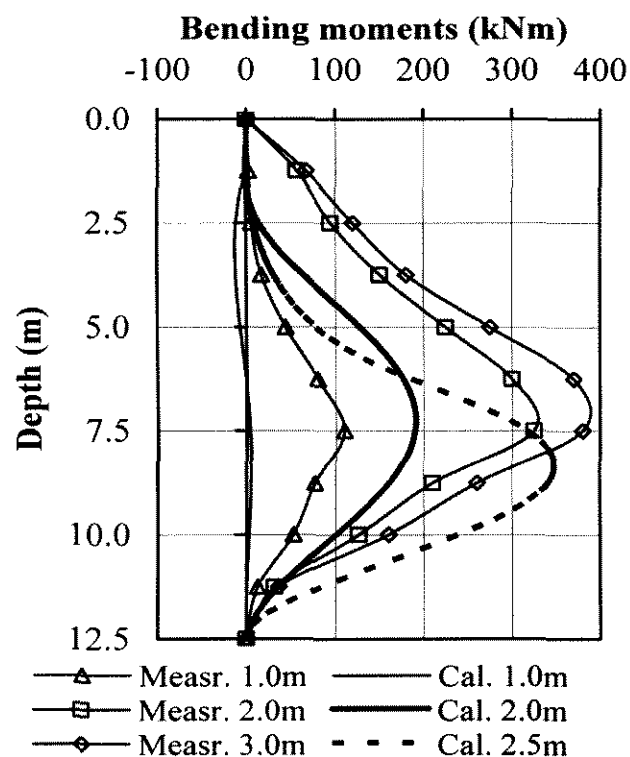
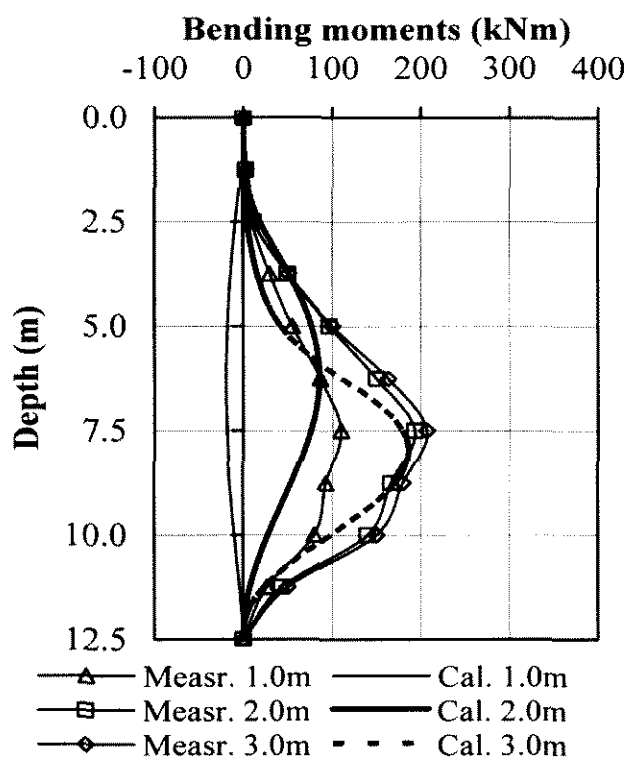


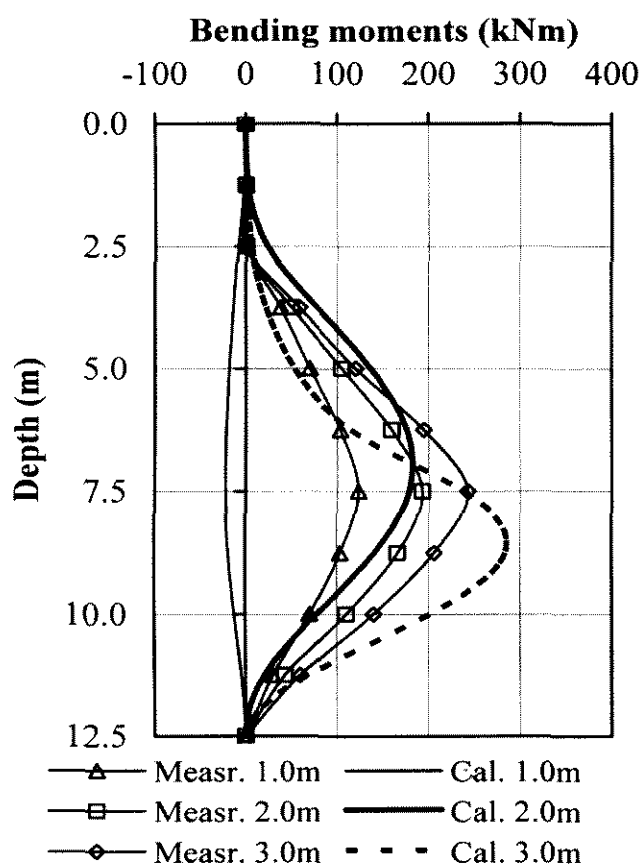
Figure 8.1: Finite element mesh used in the analysis (Leung, 2001).



(a) Pile at 1m away from the wall.



(b) Pile at 4m away from the wall.



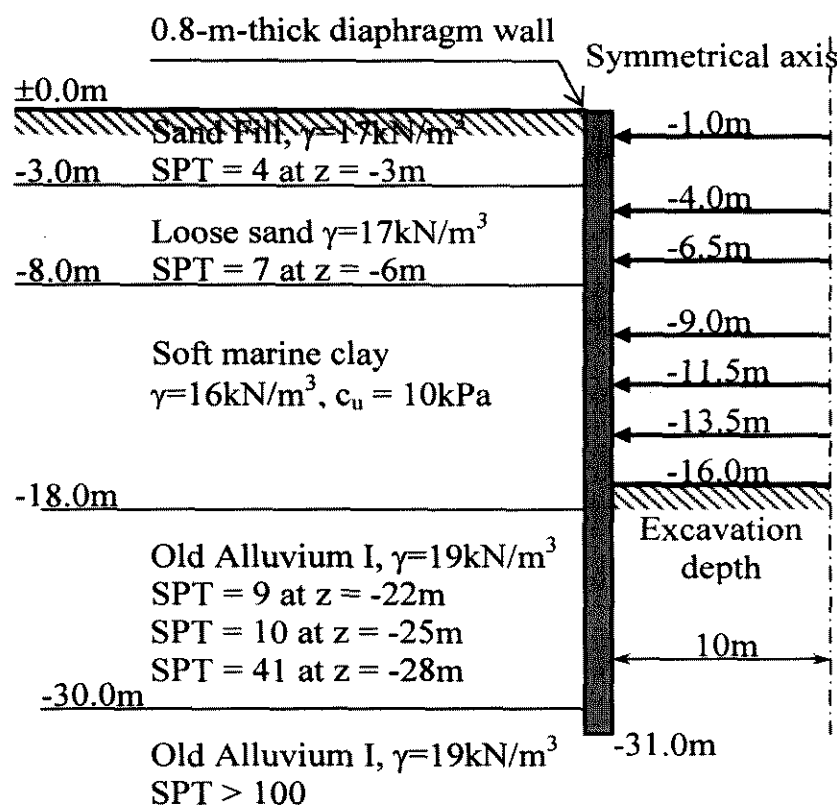
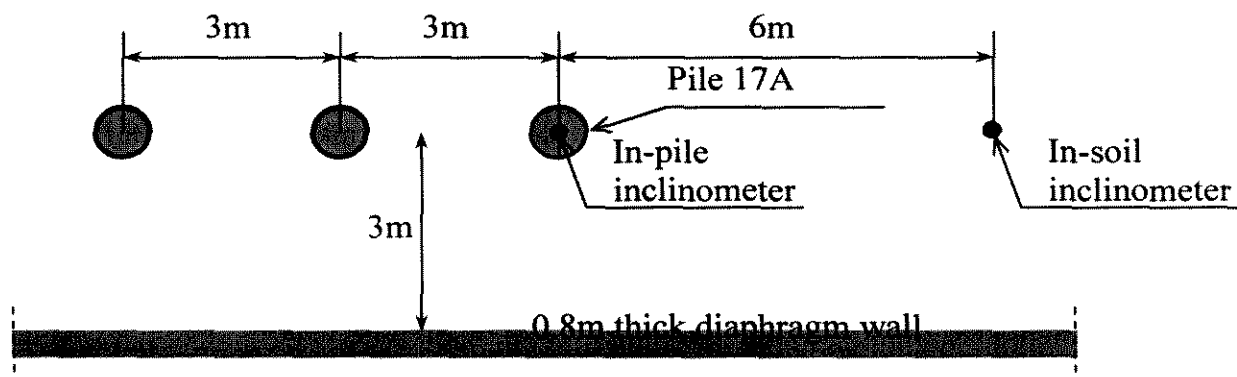
(c) Pile at 3m away from the wall.

Figure 8.2: Comparisons of analysis results for bending moments in a pile due to excavation induced soil movements (Leung, 2001).

Note:

Measr. 1.0m = Measured result at an excavation depth of 1.0m.

Cal. 1.0m = Calculated result at an excavation depth of 1.0m.

Figure 8.3: Cross-section of the excavation (after Goh *et al.*, 2003).Figure 8.4: Plan layout of piles and inclinometers (after Goh *et al.*, 2003).

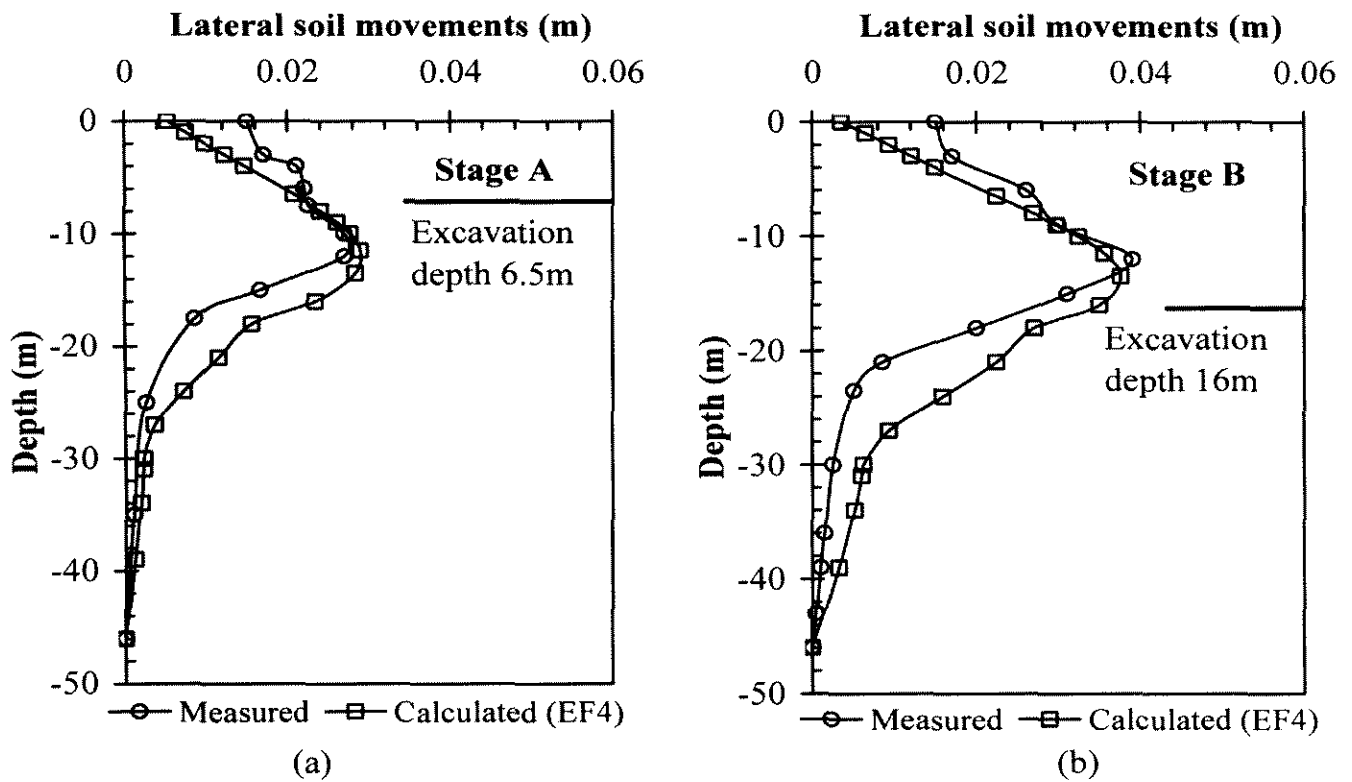


Figure 8.5: Comparison of horizontal soil movements at 3m away from the retaining wall.

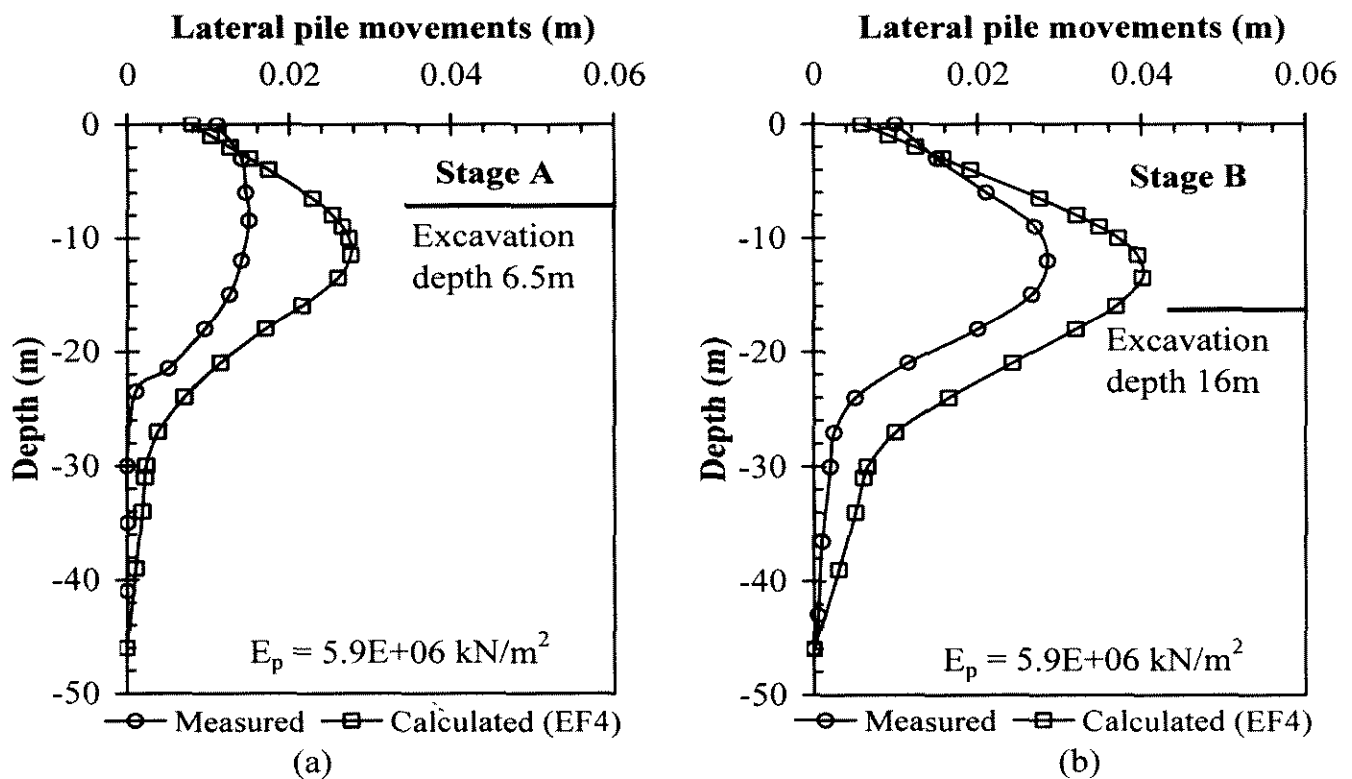


Figure 8.6: Comparison of horizontal pile deflections at 3m away from the retaining wall.

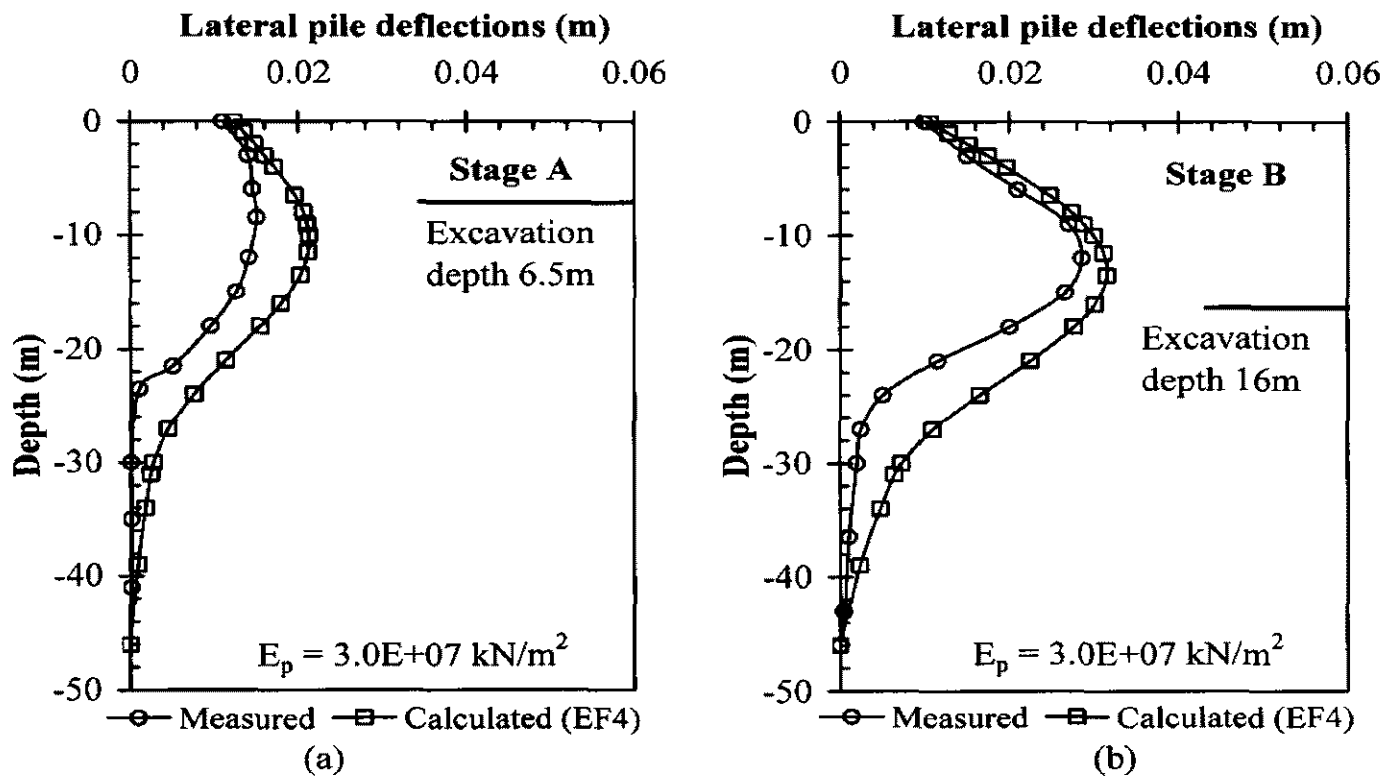
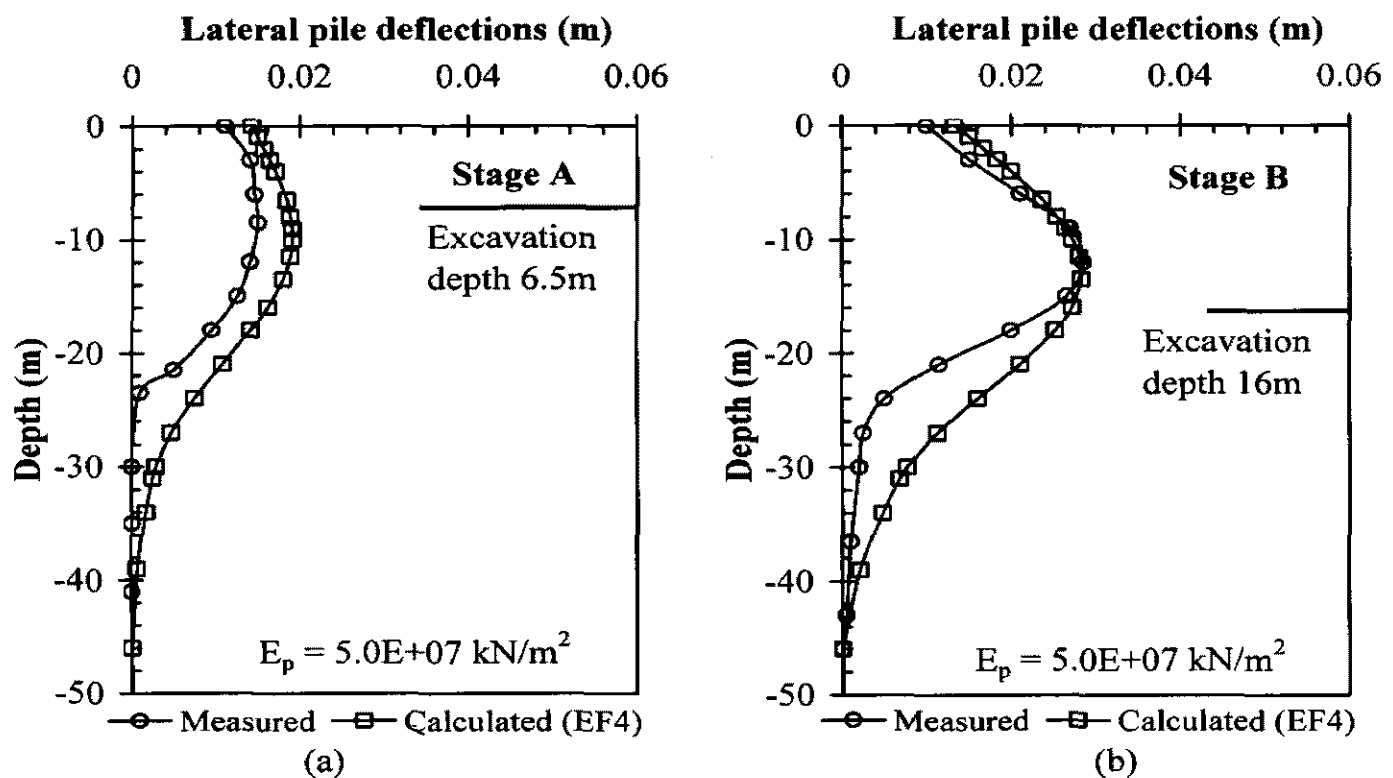


Figure 8.7: Comparison of horizontal pile deflections at 3m away from the retaining wall.

Figure 8.8: Comparison of horizontal pile deflections at 3m away from the retaining wall ($E_p = 5.0E+07 \text{ kN/m}^2$).

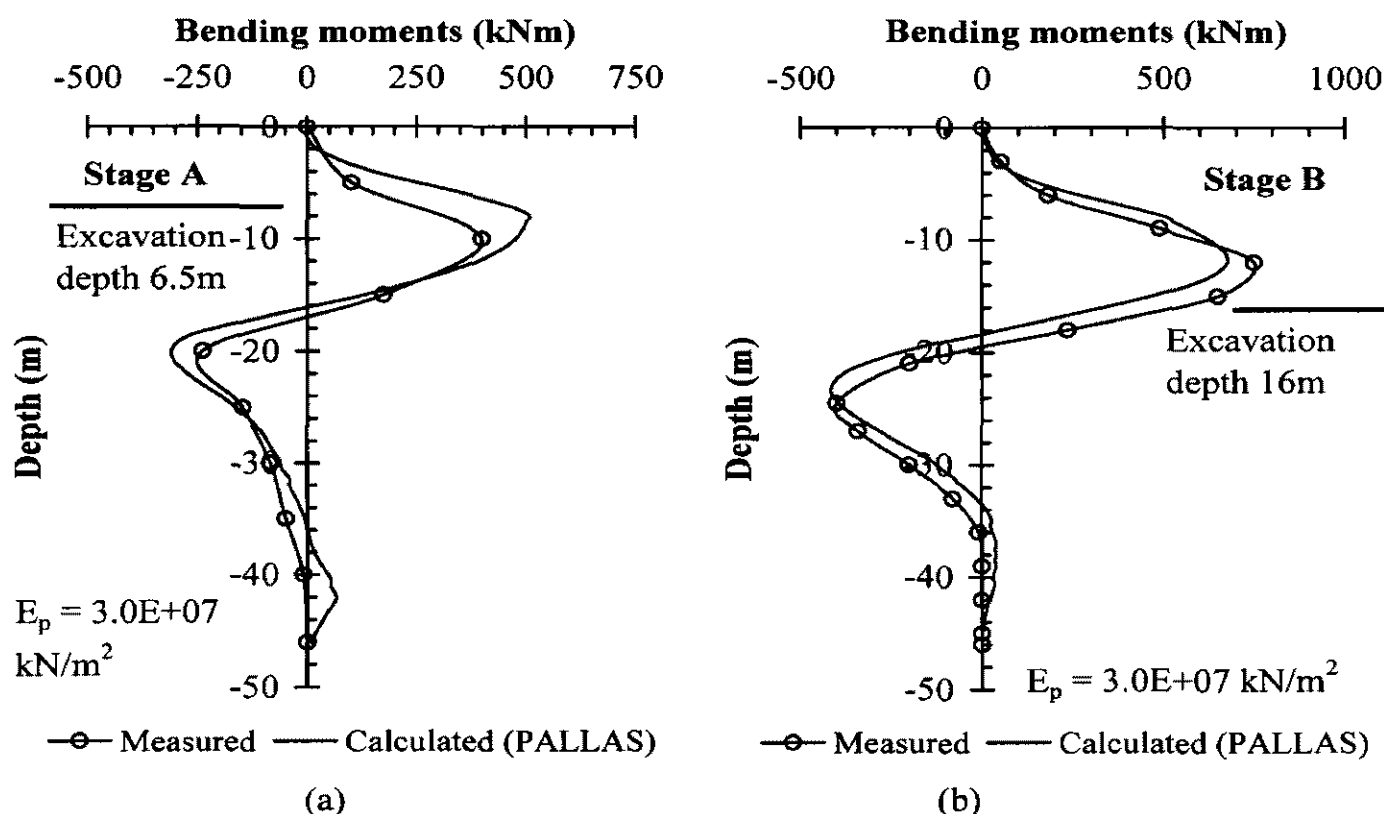


Figure 8.9: Comparison of pile bending moments at 3m away from the retaining wall
(Lateral soil movements input from measured data, $E_p = 3.0E+07$ kN/m^2).

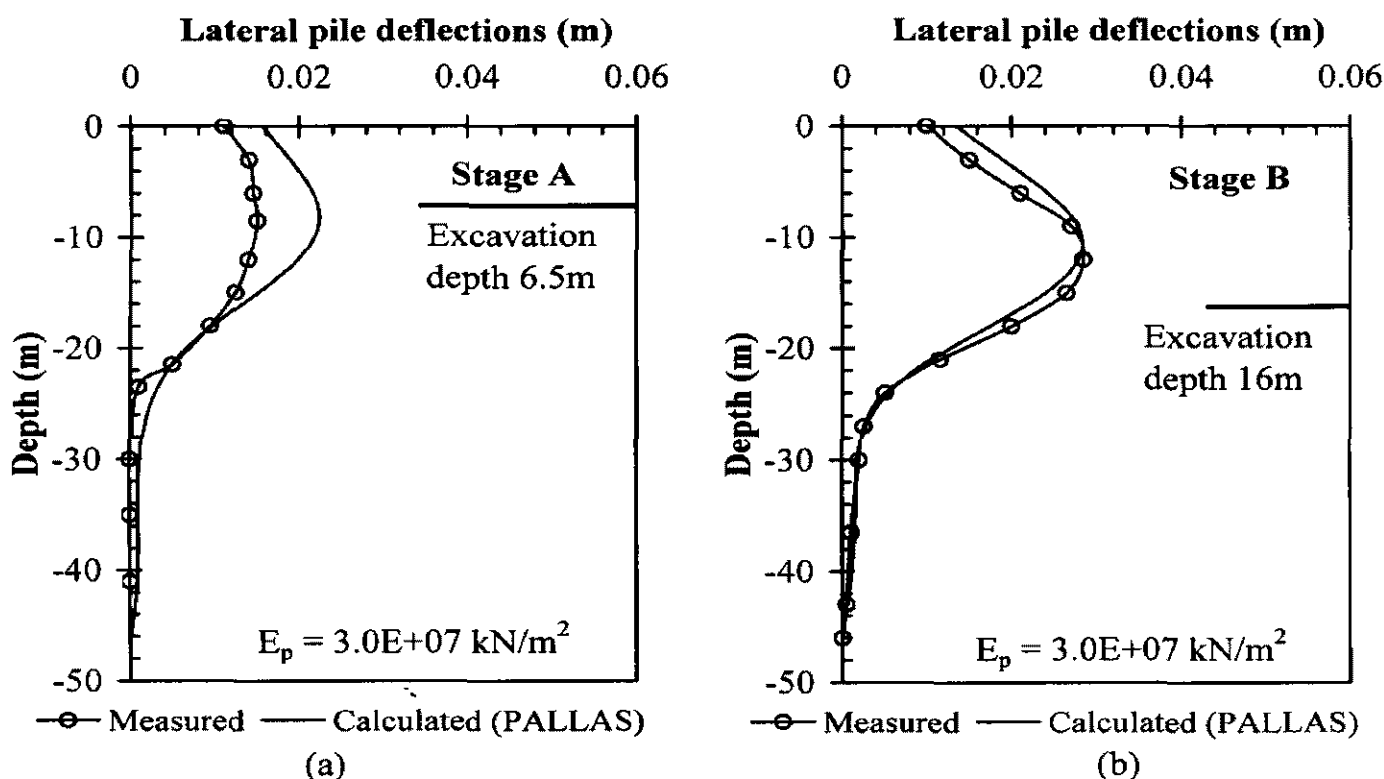


Figure 8.10: Comparison of pile deflections at 3m away from the retaining wall
(Lateral soil movements input from measured data, $E_p = 3.0E+07$ kN/m^2).

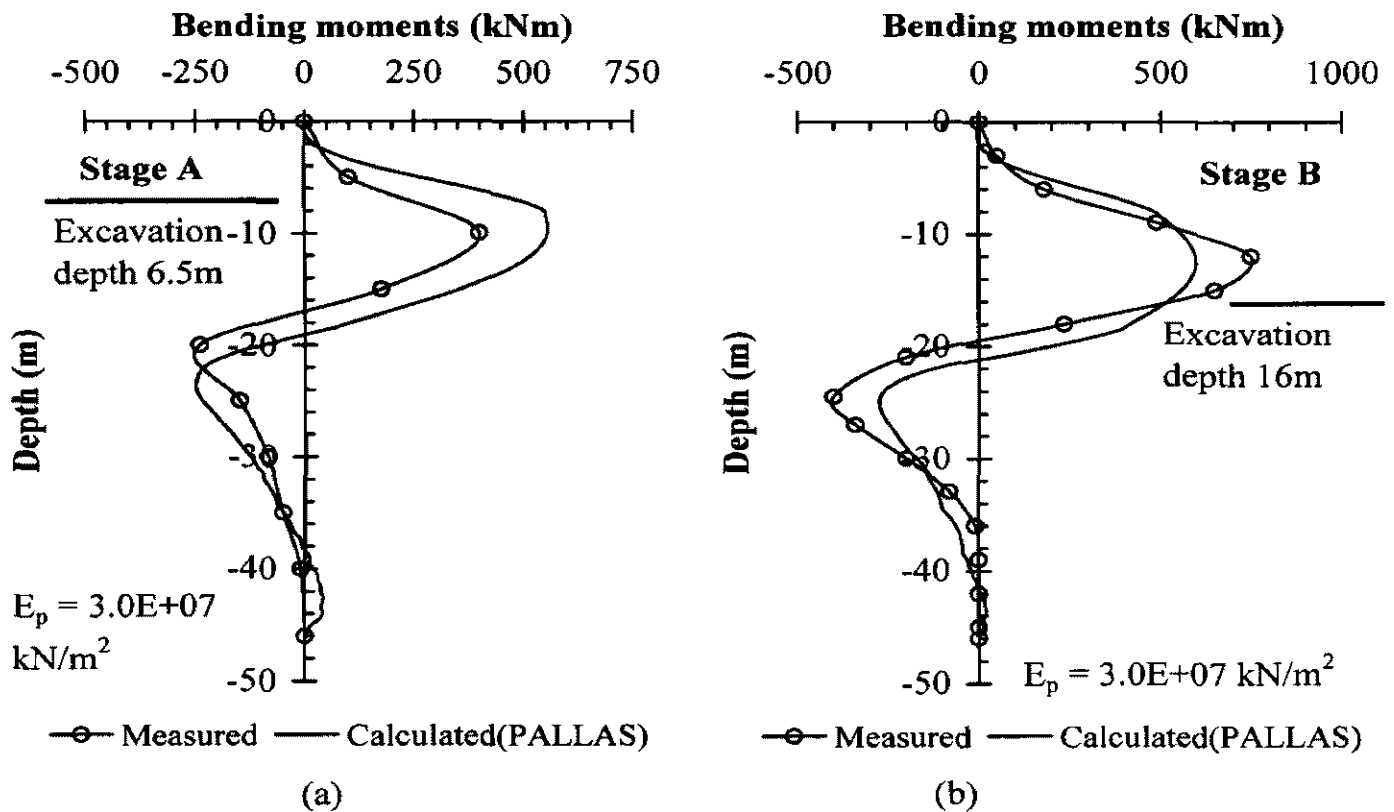


Figure 8.11: Comparison of pile bending moments at 3m away from the retaining wall (Lateral soil movements input from EF4, $E_p = 3.0E+07$ kN/m²).

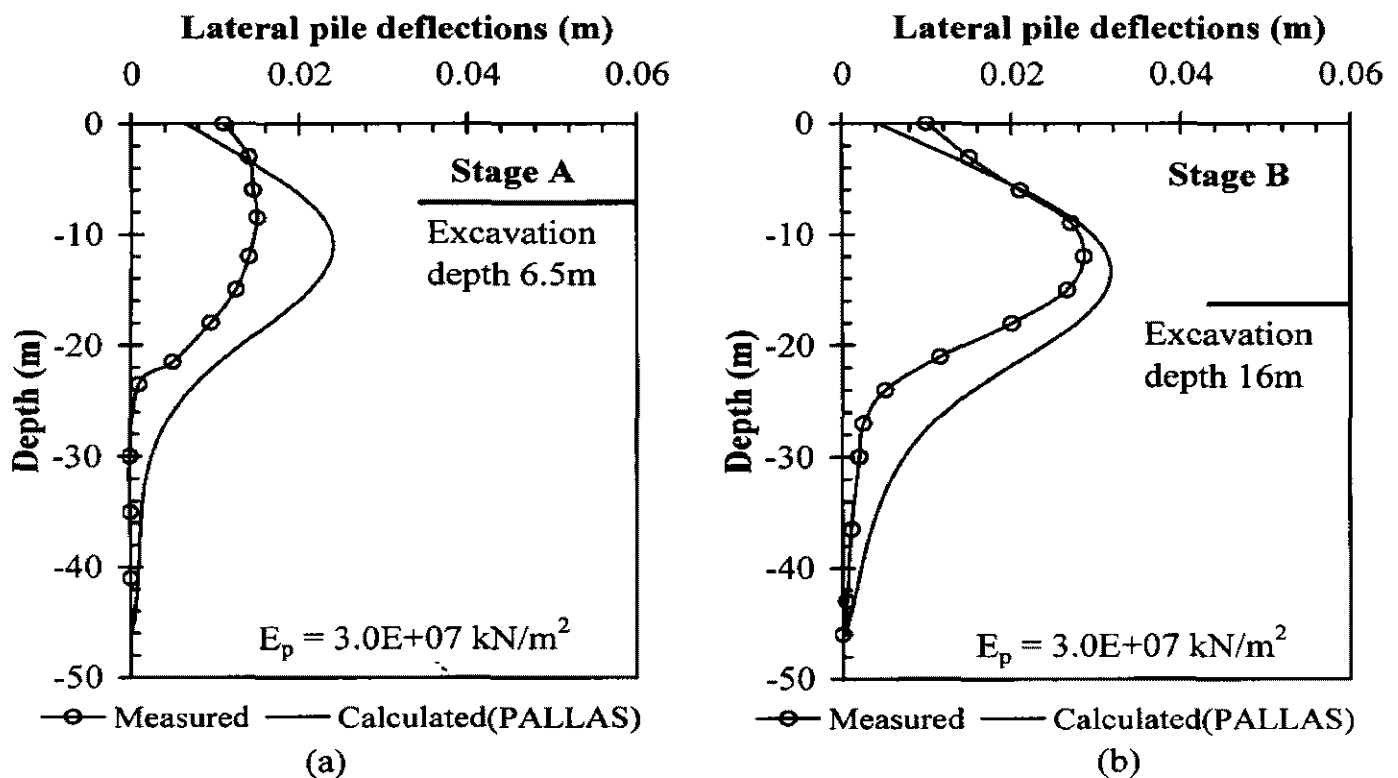
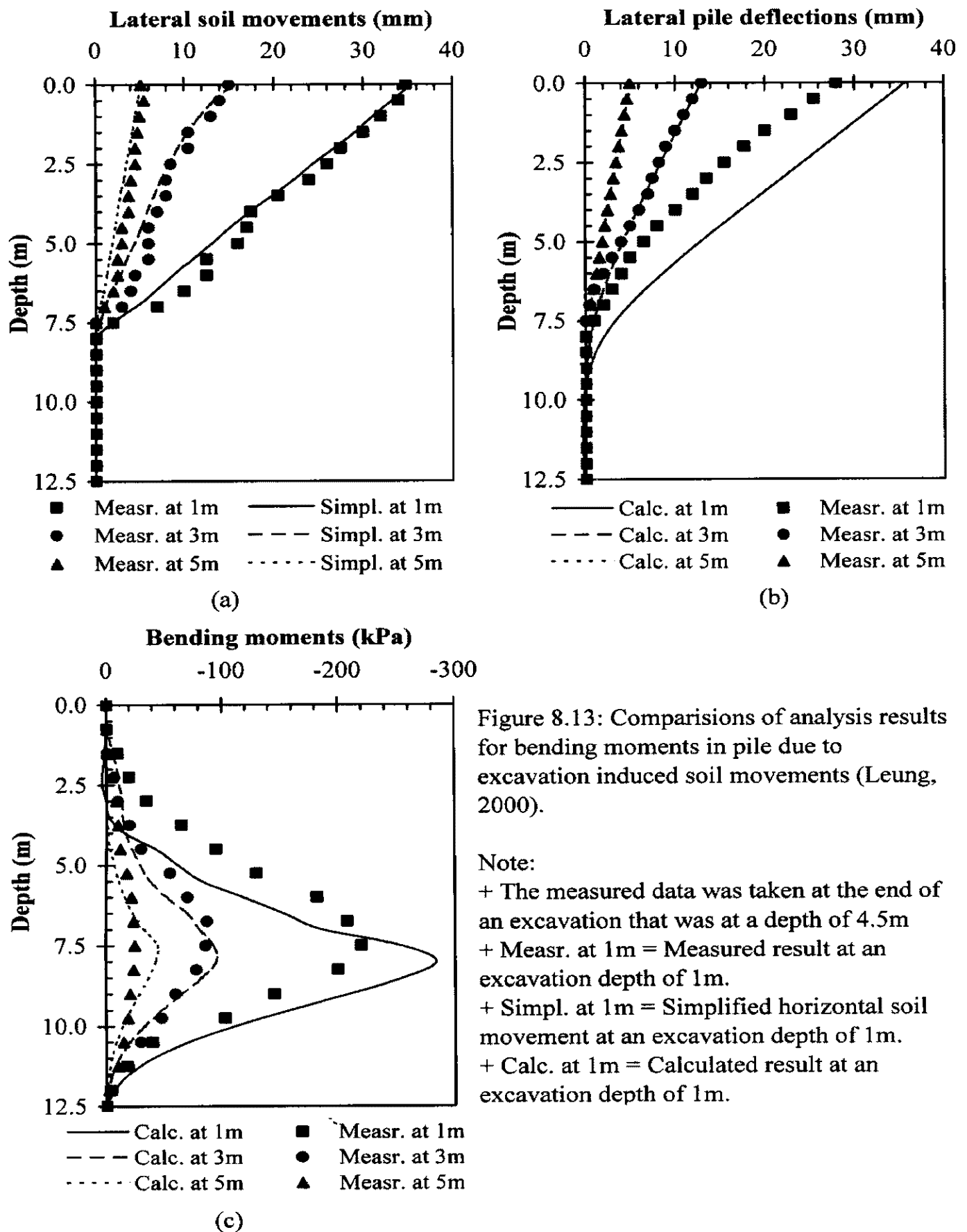


Figure 8.12: Comparison of pile deflections at 3m away from the retaining wall (Lateral soil movements input from EF4, $E_p = 3.0E+07$ kN/m²).



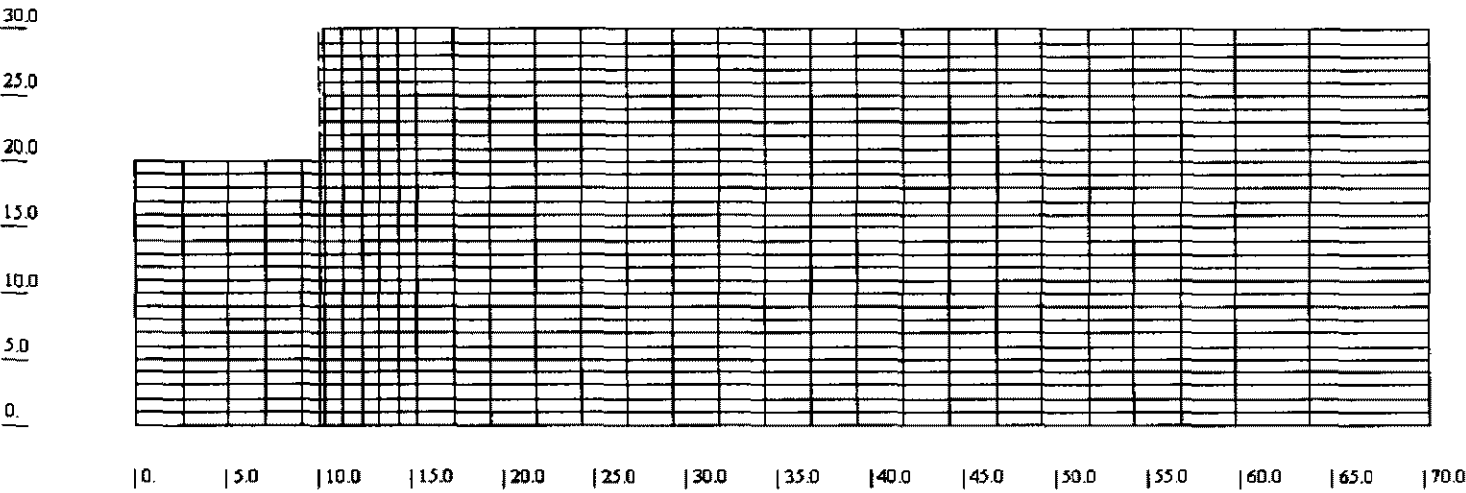
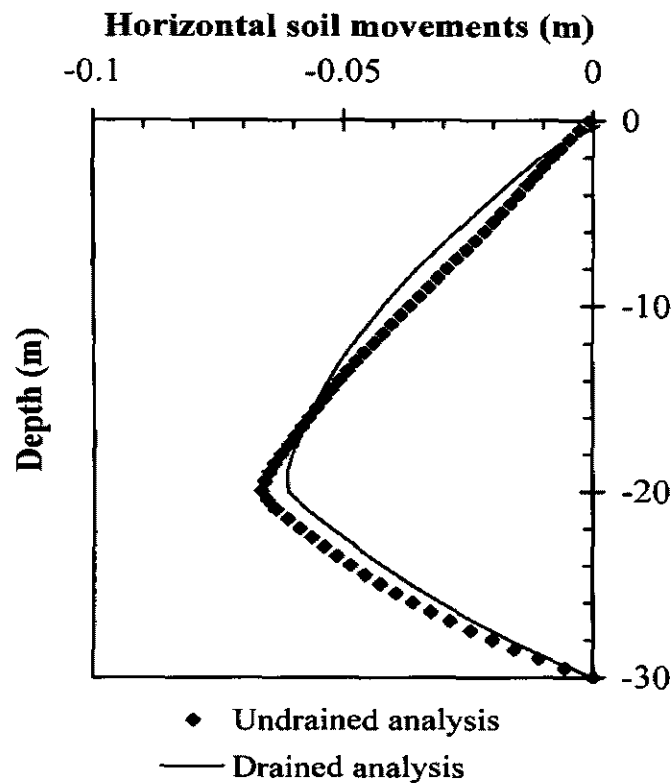
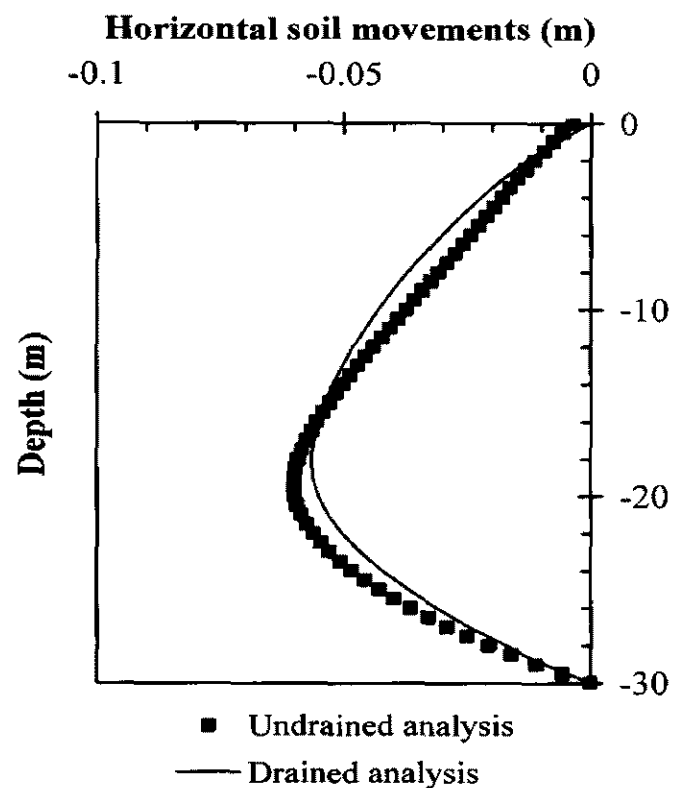


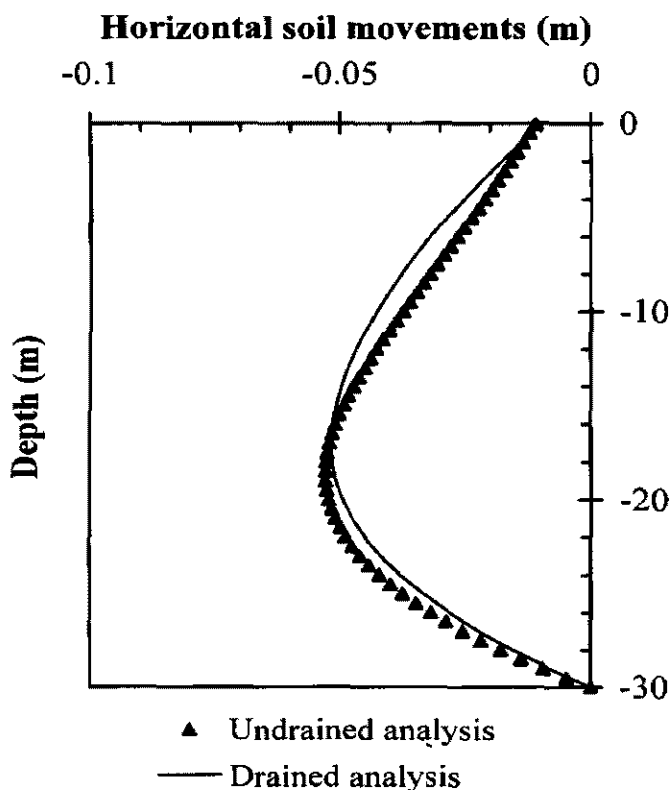
Figure 8.14: Finite element mesh used in the analysis.



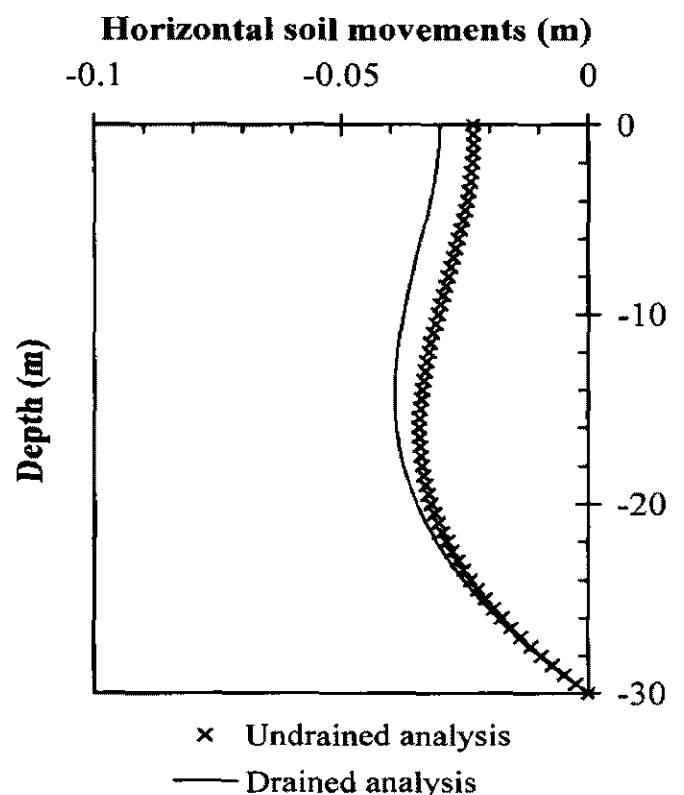
(a) Maximum horizontal soil movements at a distance of 1m from the wall.



(b) Maximum horizontal soil movements at a distance of 5m from the wall.



(c) Maximum horizontal soil movements at a distance of 10m from the wall.



(d) Maximum horizontal soil movements at a distance of 20m from the wall.

Figure 8.15: Maximum horizontal soil movements at different distances from the diaphragm wall when the excavation base has reached 10m depth.

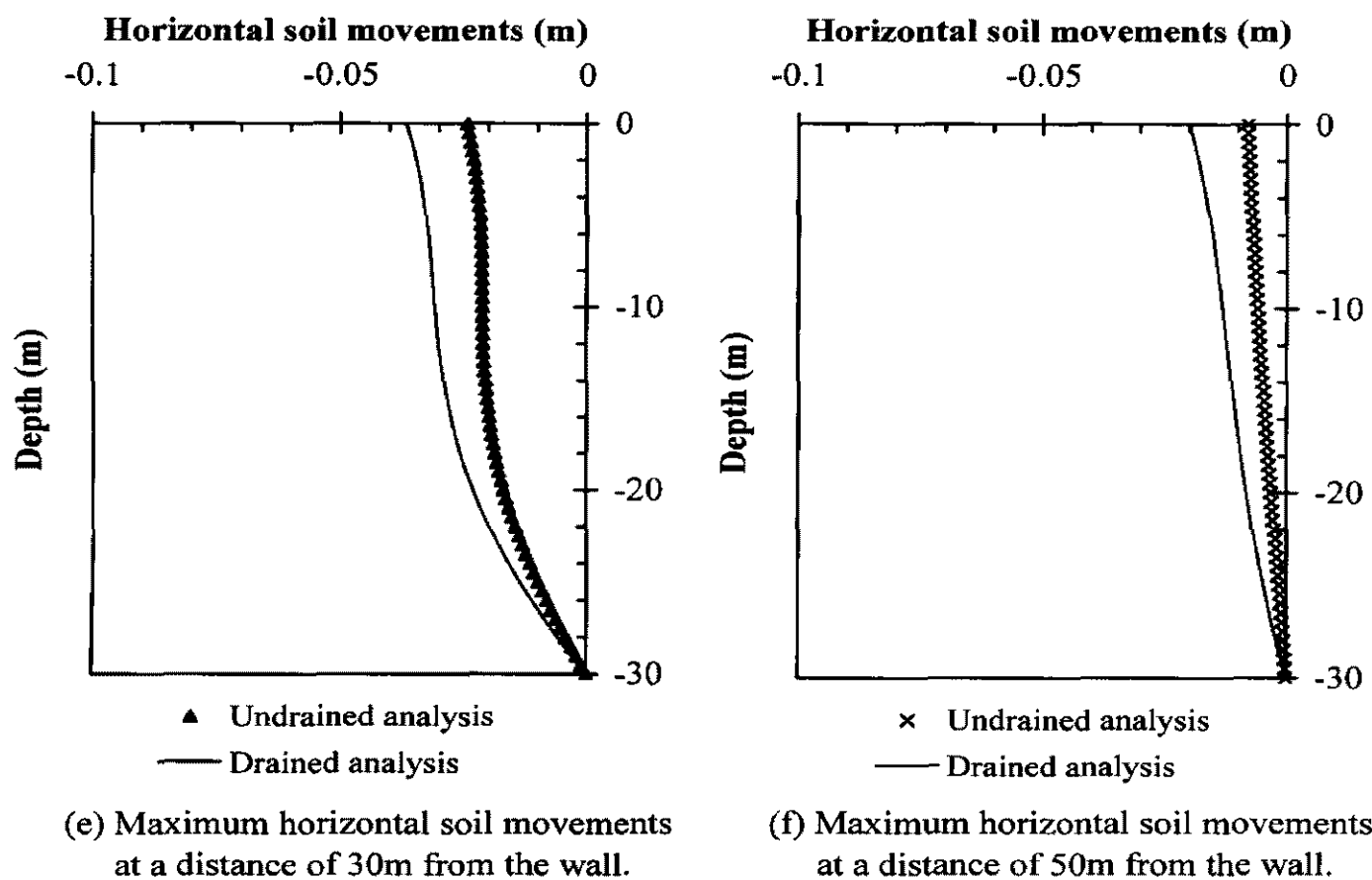


Figure 8.15: Maximum horizontal soil movements at different distances from the diaphragm wall when the excavation base has reached 10m depth (Continued).

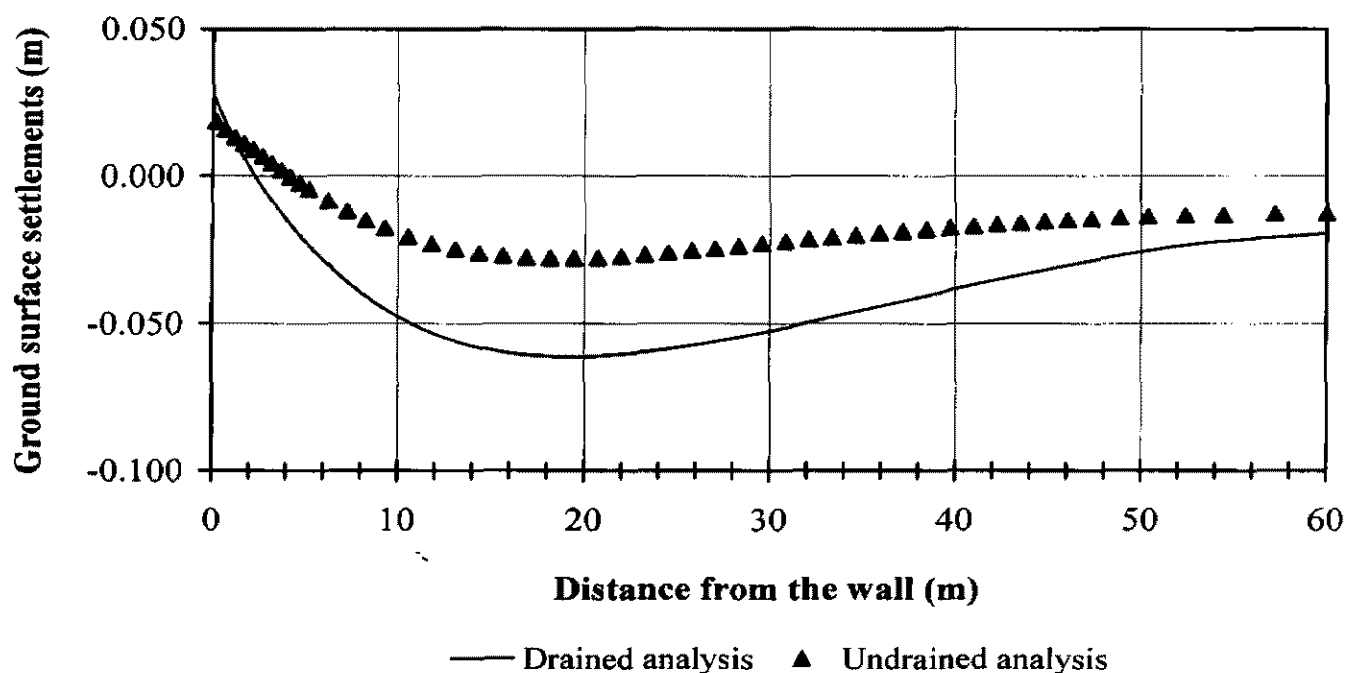
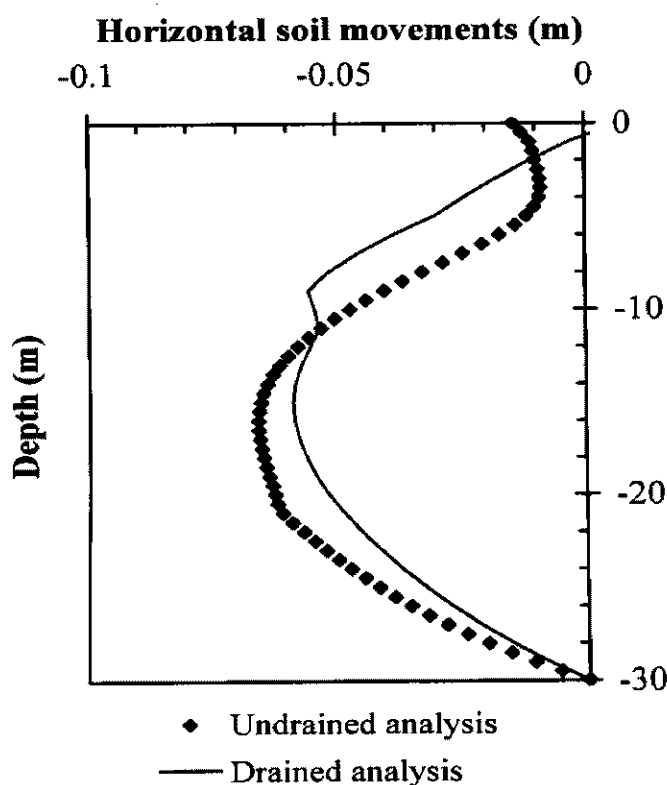
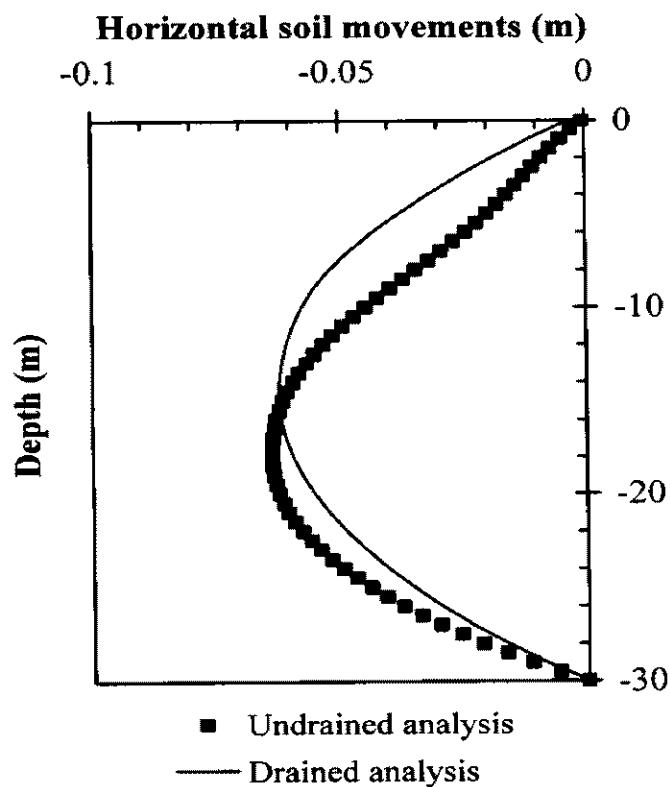


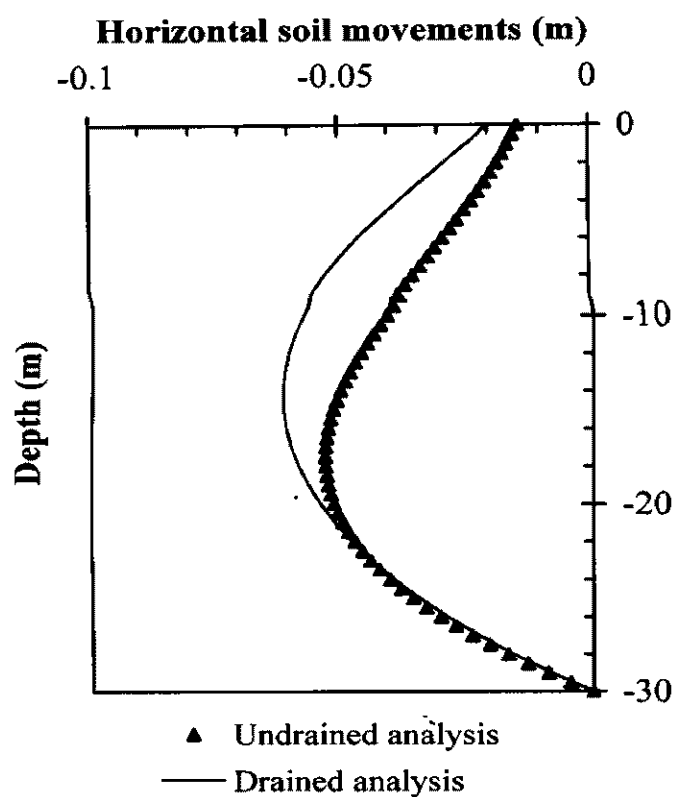
Figure 8.16: Maximum vertical soil movements when the excavation base has reached 10m depth (Diaphragm wall).



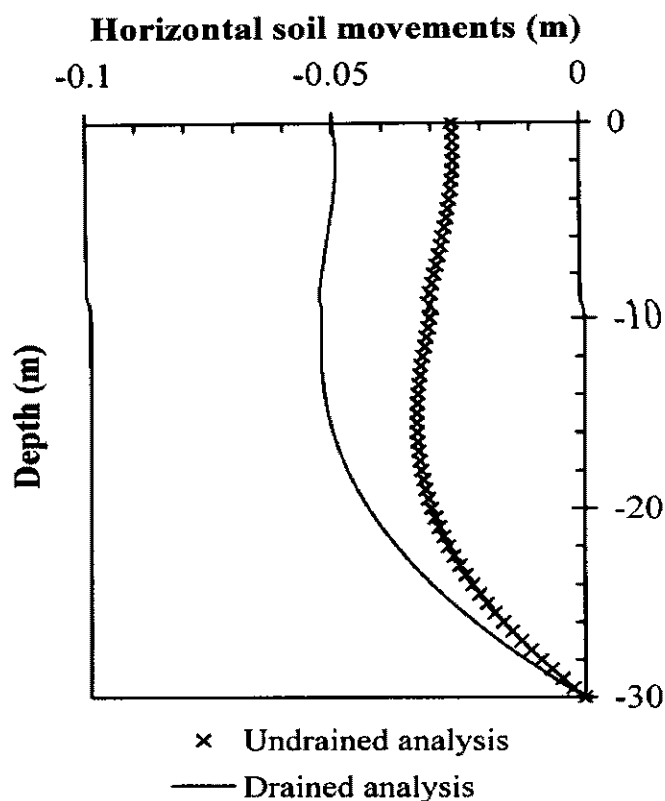
(a) Maximum horizontal soil movements at a distance of 1m from the wall.



(b) Maximum horizontal soil movements at a distance of 5m from the wall.



(c) Maximum horizontal soil movements at a distance of 10m from the wall.



(d) Maximum horizontal soil movements at a distance of 20m from the wall.

Figure 8.17: Maximum horizontal soil movements at different distances from the sheet pile wall when the excavation base has reached 10m depth.

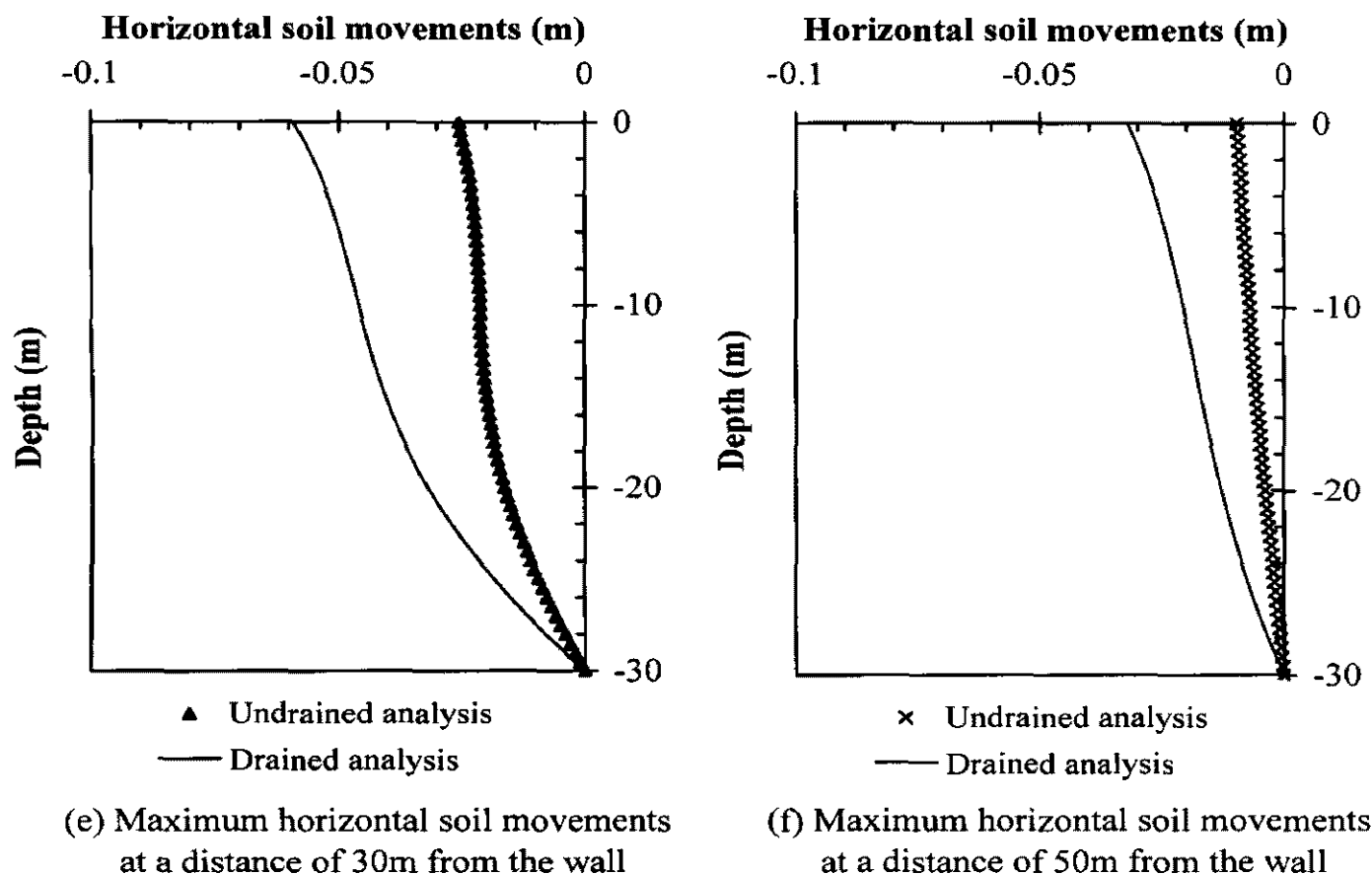


Figure 8.17: Maximum horizontal soil movements at different distances from the sheet pile wall when the excavation base has reached 10m depth (Continued).

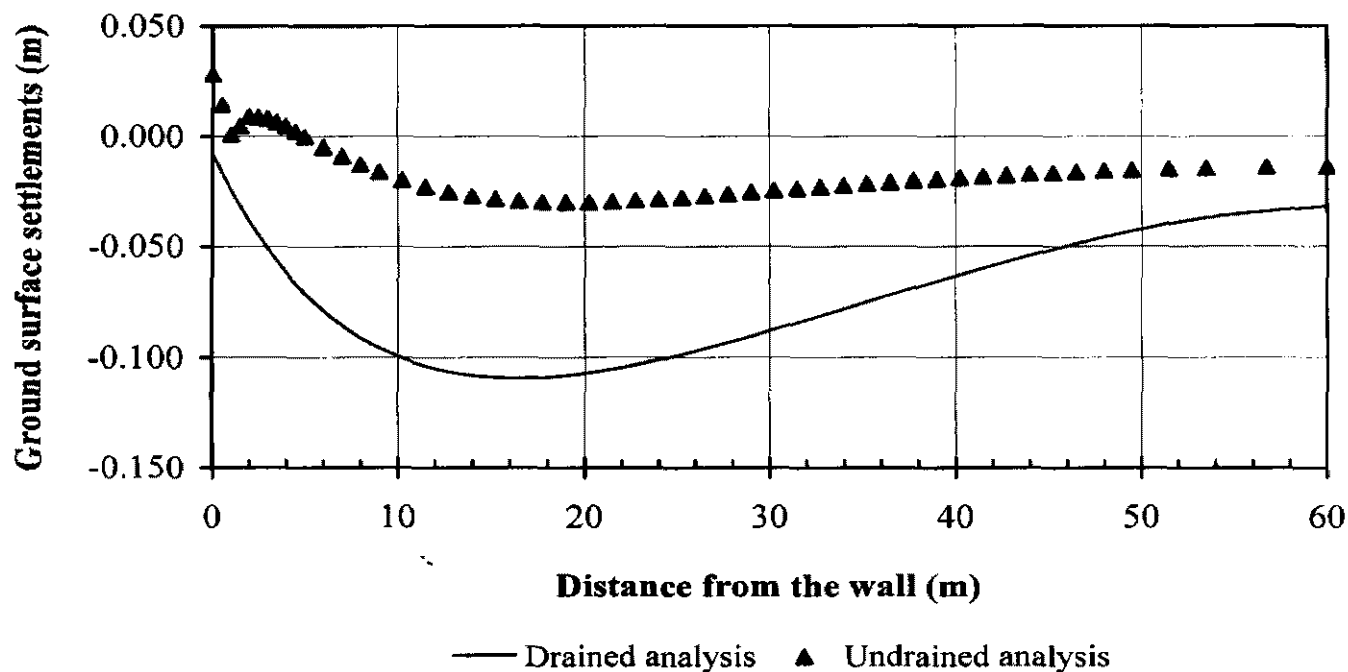
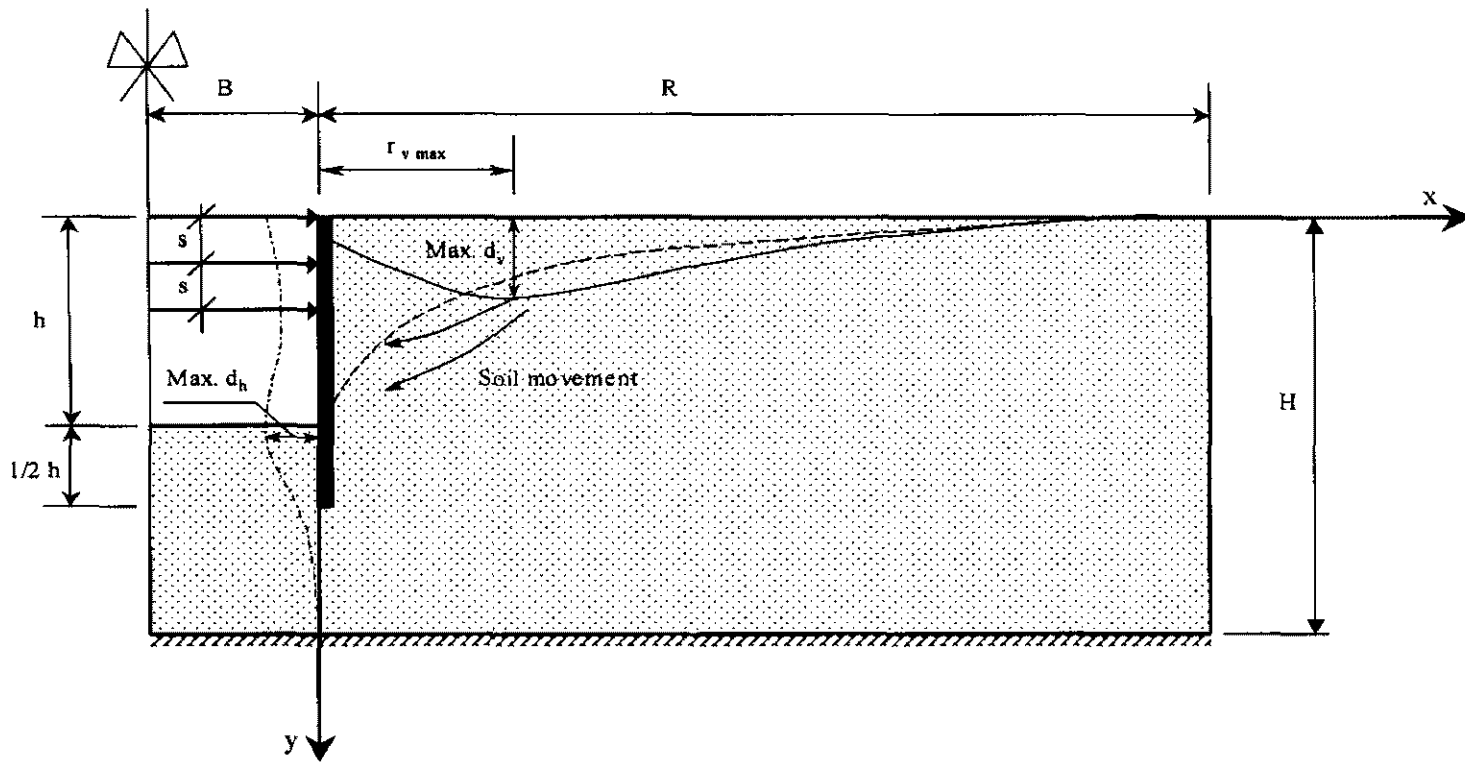
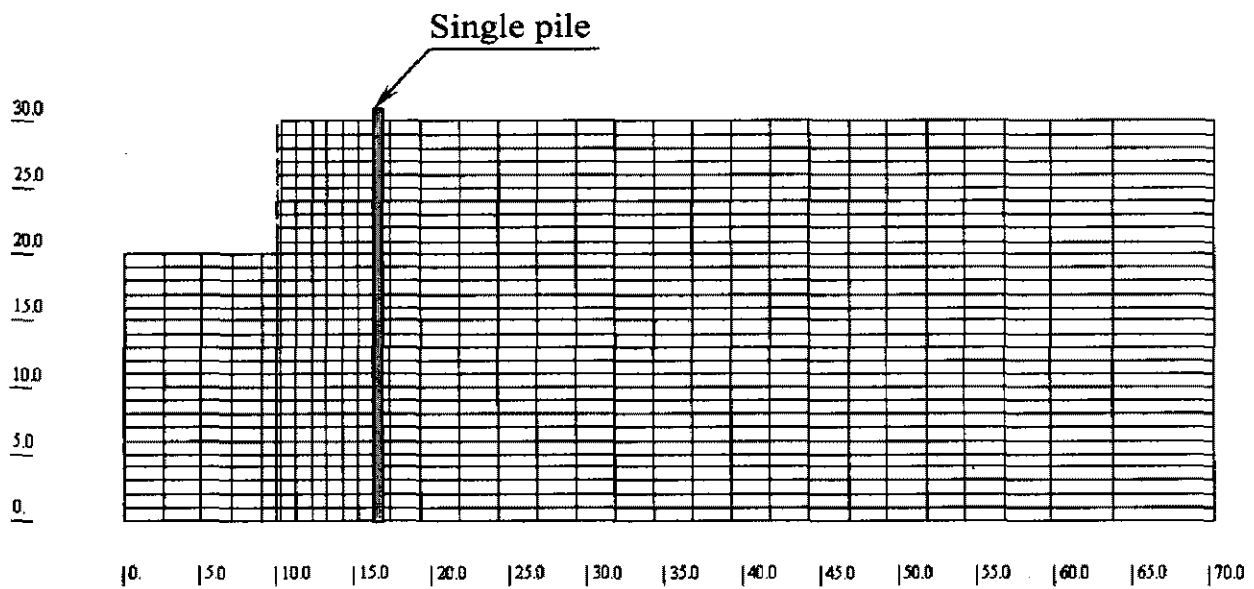


Figure 8.18: Maximum vertical soil movements when the excavation base has reached 10m depth (Sheet pile wall).



(a)



(b)

Figure 8-19: Notation used for parametric study of excavation-induced soil movements and a schematic diagram of the case where a single pile exists at a distance from the wall.

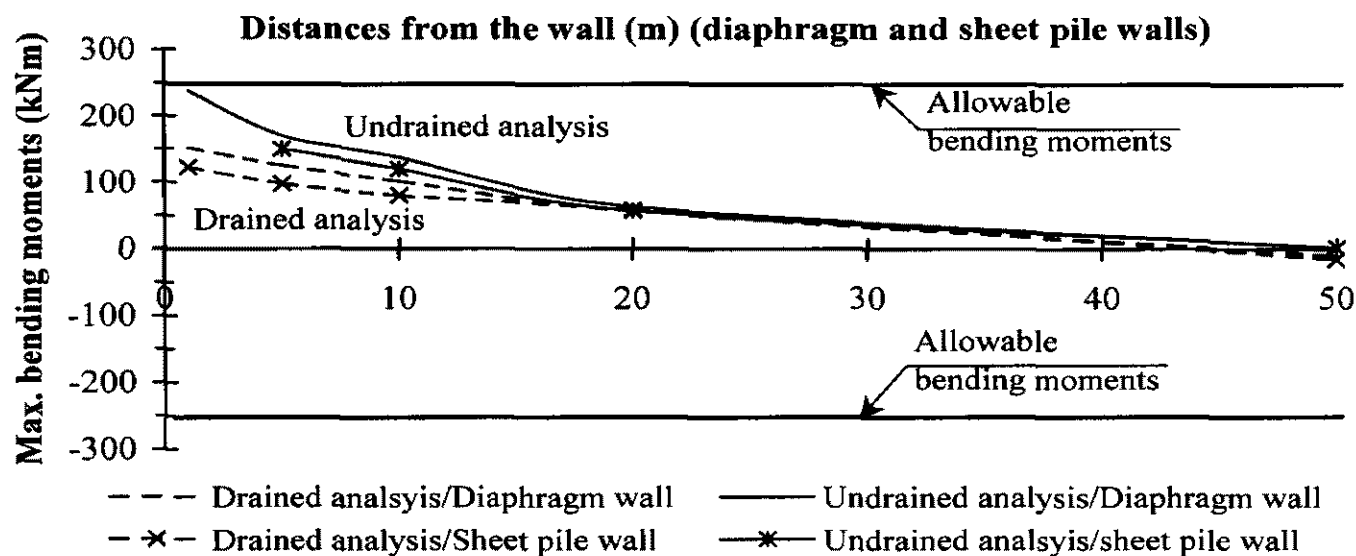
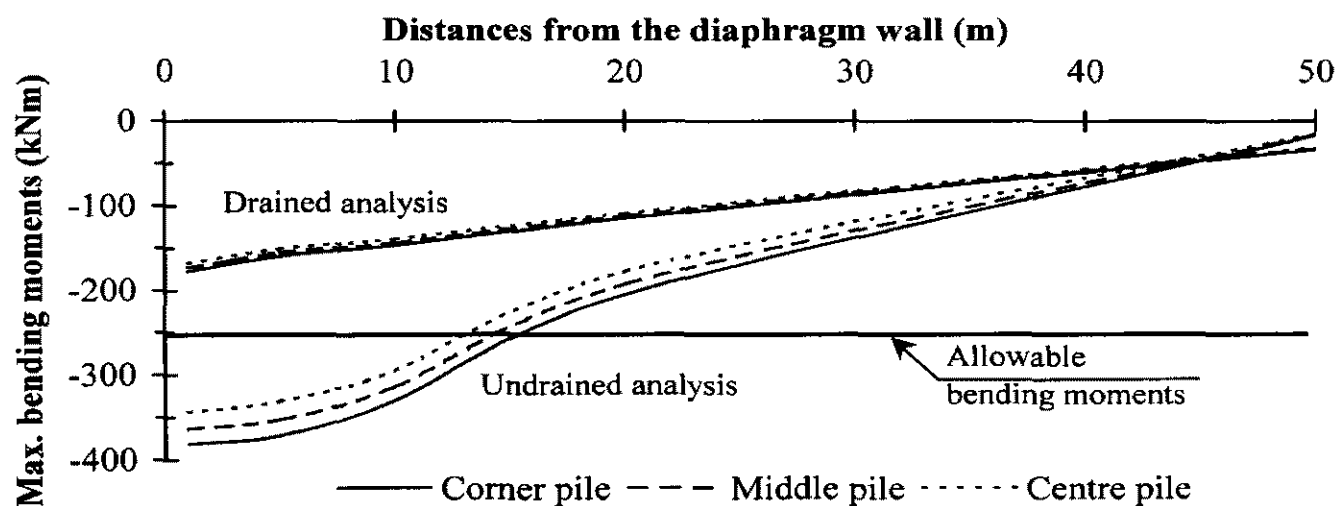
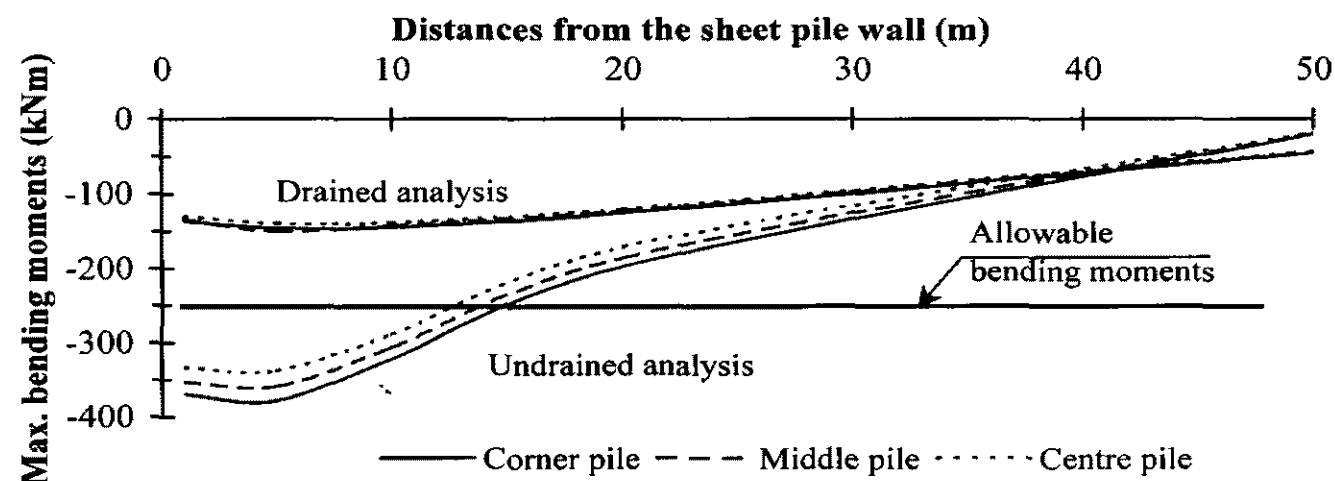
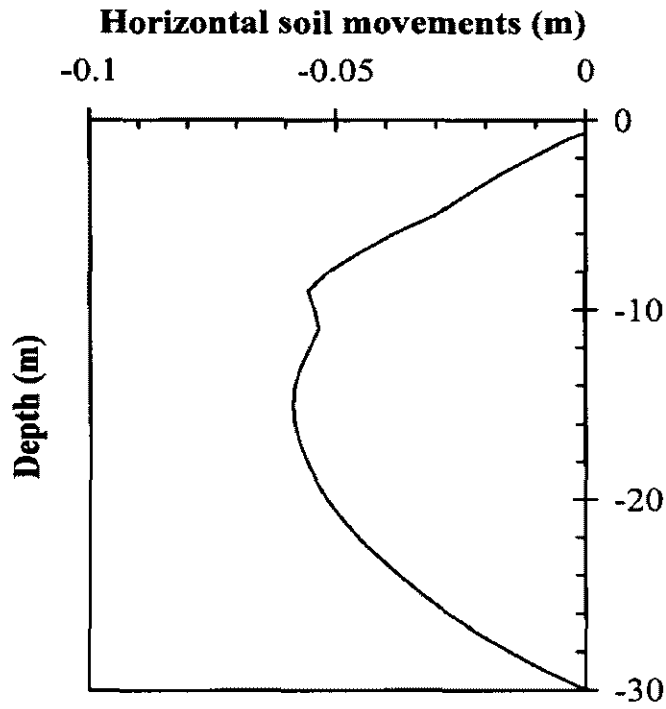
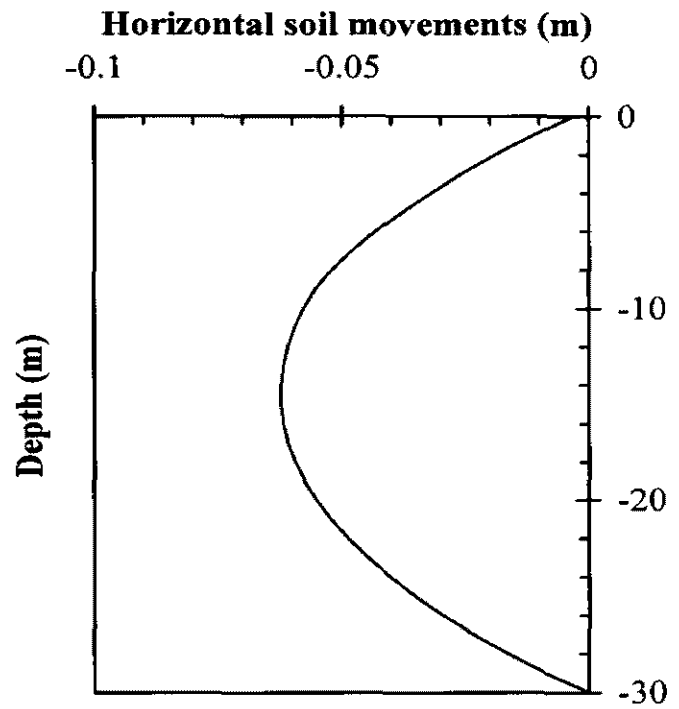


Figure 8.20: Maximum bending moments for a single pile.

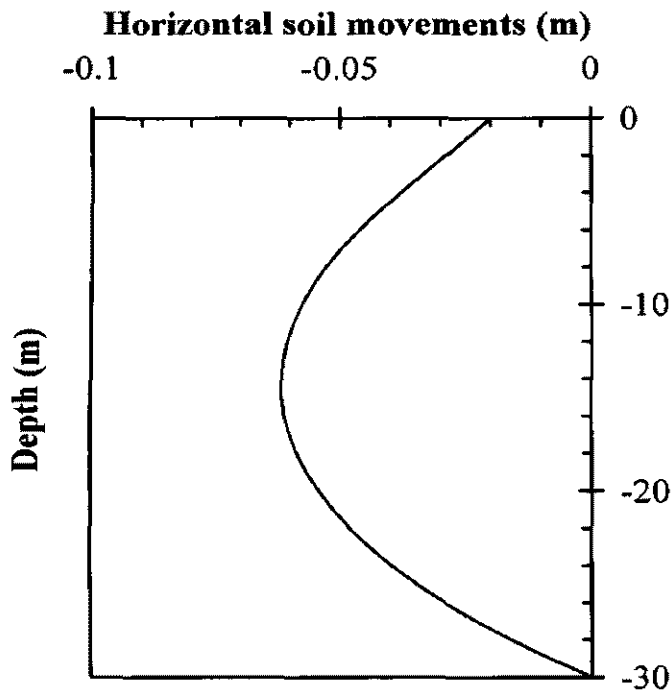
Figure 8.21: Maximum bending moments for a 3x3 pile group, $s/d = 3$ existing behind a diaphragm wall.Figure 8.22: Maximum bending moments for a 3x3 pile group, $s/d = 3$, existing behind a sheet pile wall.



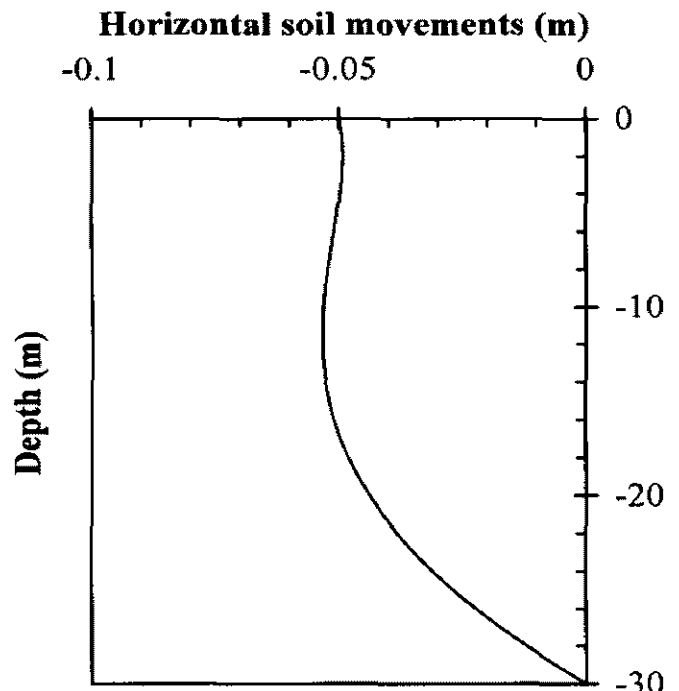
(a) Maximum horizontal soil movements at a distance of 1m from the wall



(b) Maximum horizontal soil movements at a distance of 5m from the wall



(c) Maximum horizontal soil movements at a distance of 10m from the wall



(d) Maximum horizontal soil movements at a distance of 20m from the wall

Figure 8.23: Maximum horizontal soil movements at different distances from the sheet pile wall when the excavation base has reached 10m depth.

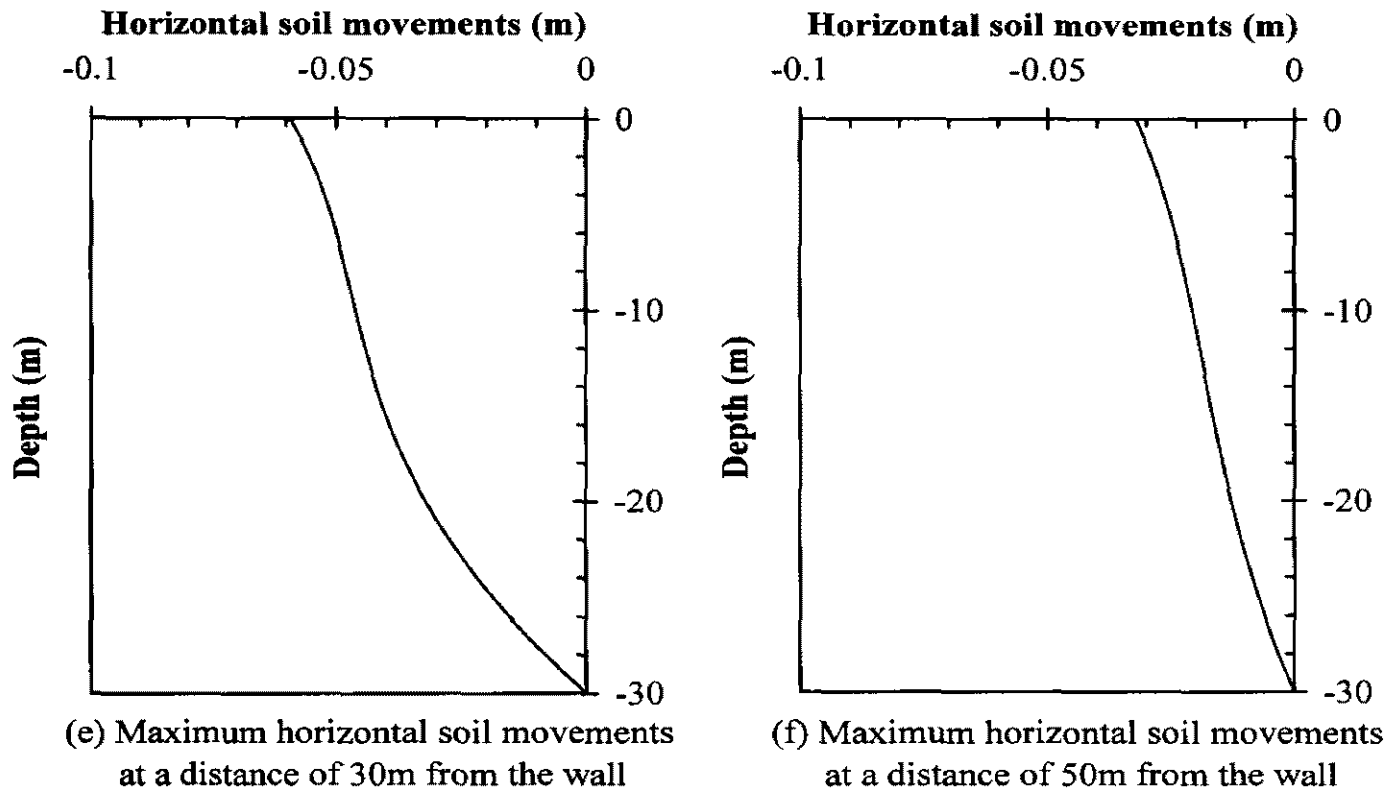


Figure 8.23: Maximum horizontal soil movements at different distances from the sheet pile wall when the excavation base has reached 10m depth (Continued).

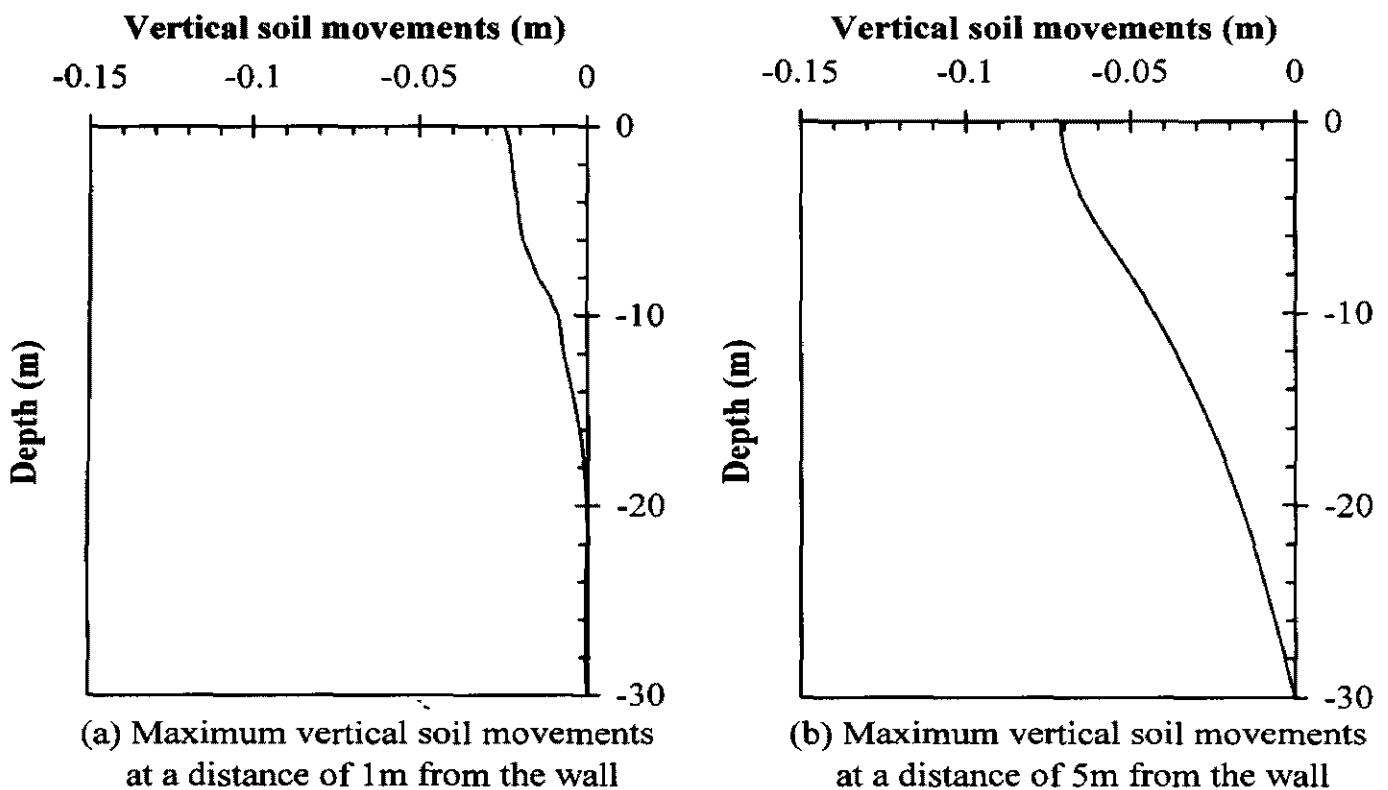
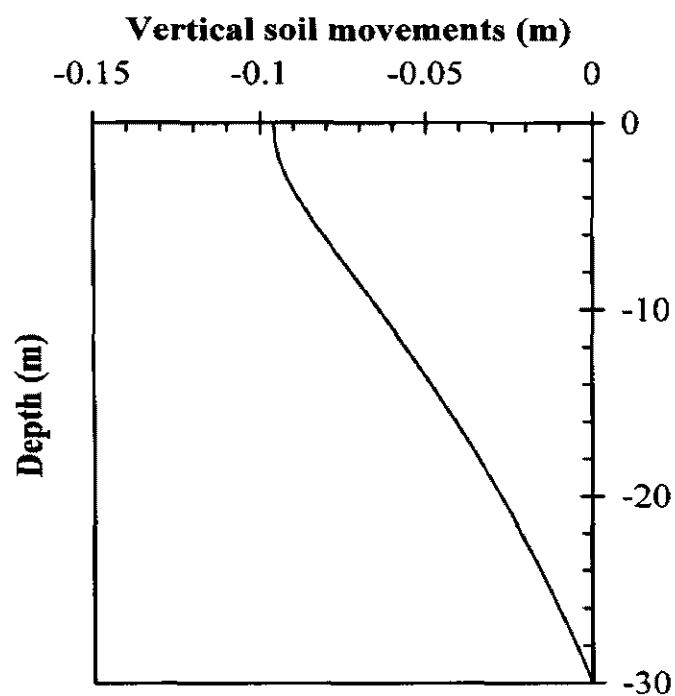
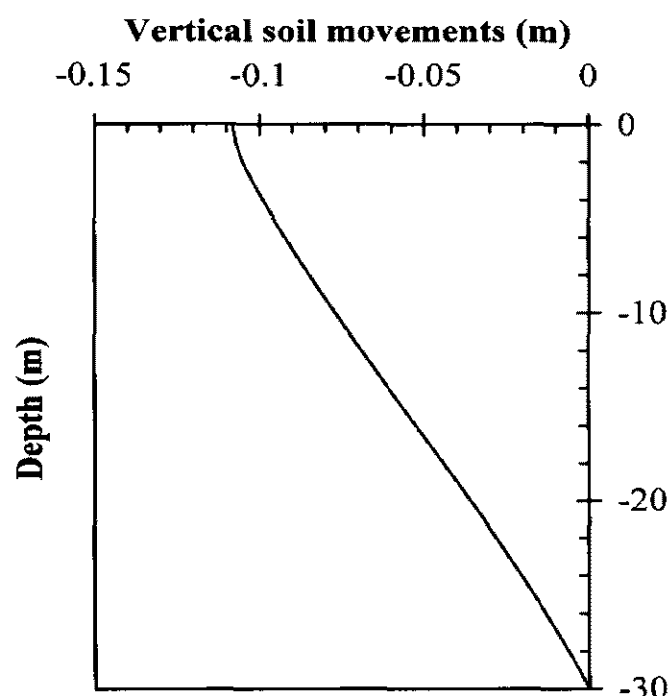


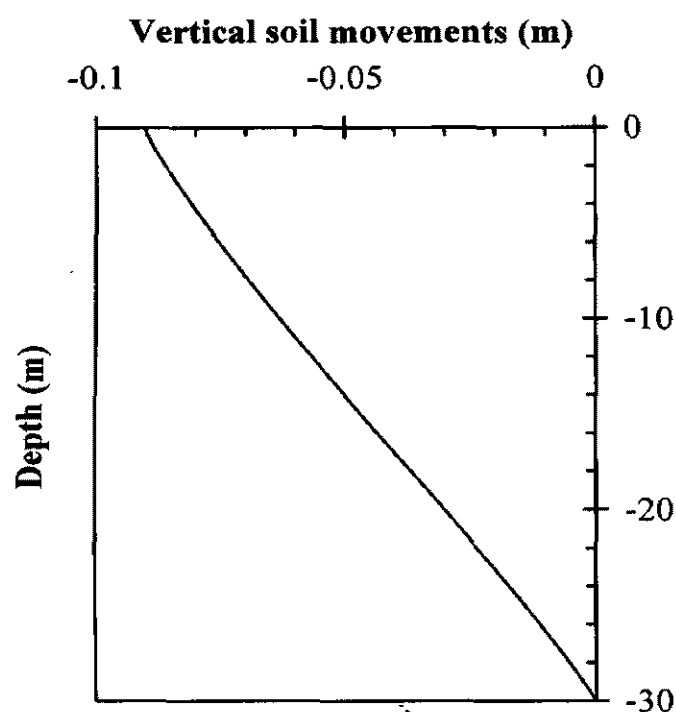
Figure 8.24: Maximum vertical soil movements at different distances from the sheet pile wall when the excavation base has reached 10m depth.



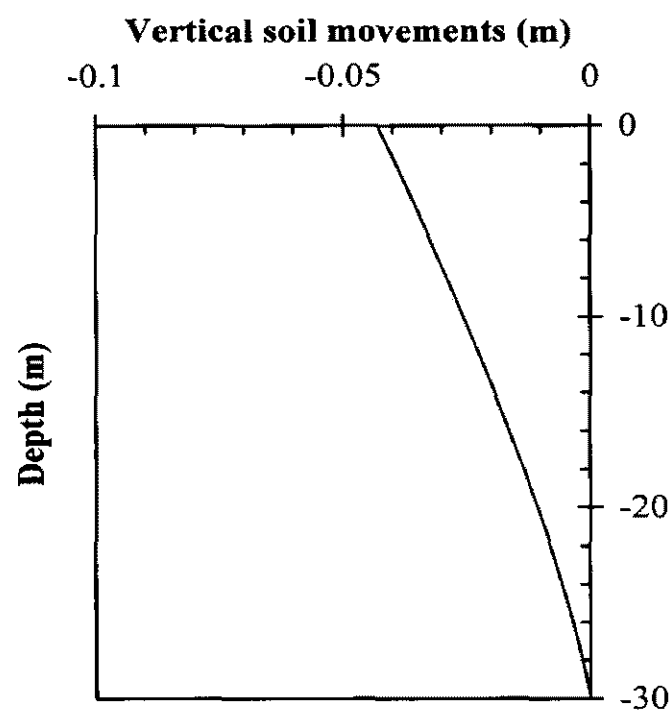
(c) Maximum vertical soil movements at a distance of 10m from the wall



(d) Maximum vertical soil movements at a distance of 20m from the wall



(e) Maximum vertical soil movements at a distance of 30m from the wall



(f) Maximum vertical soil movements at a distance of 50m from the wall

Figure 8.24: Maximum vertical soil movements at different distances from the sheet pile wall when the excavation base has reached 10m depth (Continued).

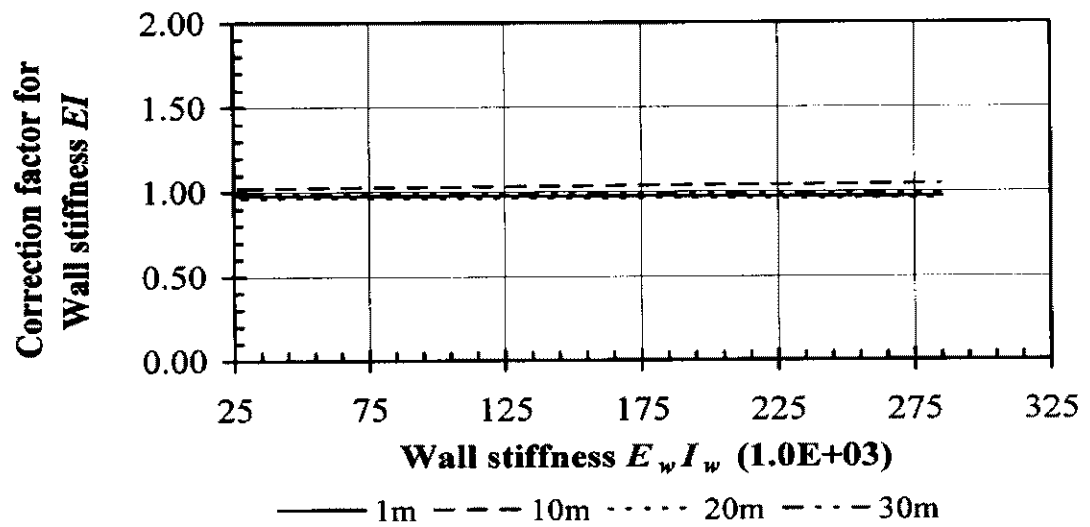


Figure 8.25: Correction factor for wall stiffness for horizontal soil movements.

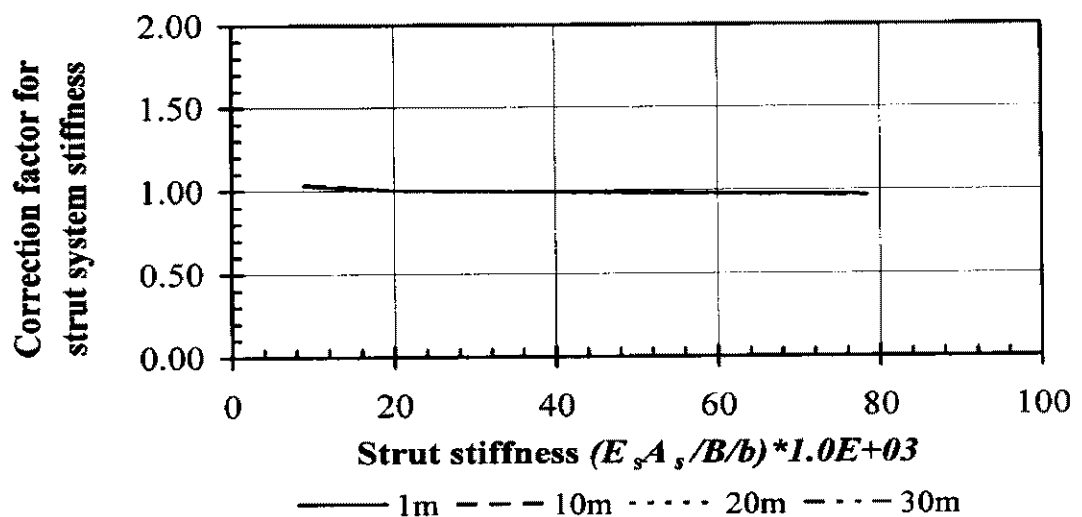


Figure 8.26: Correction factor for strut stiffness for horizontal soil movements.

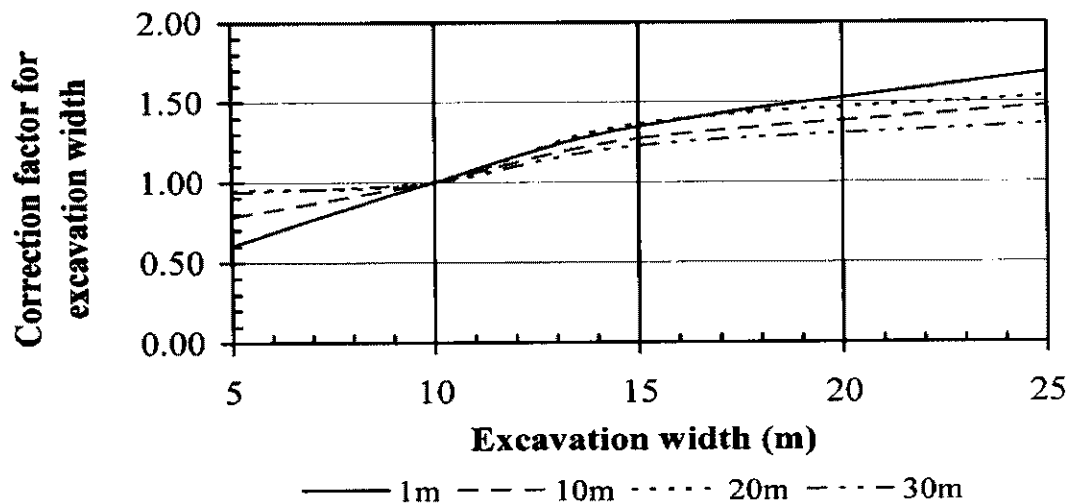


Figure 8.27: Correction factor for excavation width for horizontal soil movements.

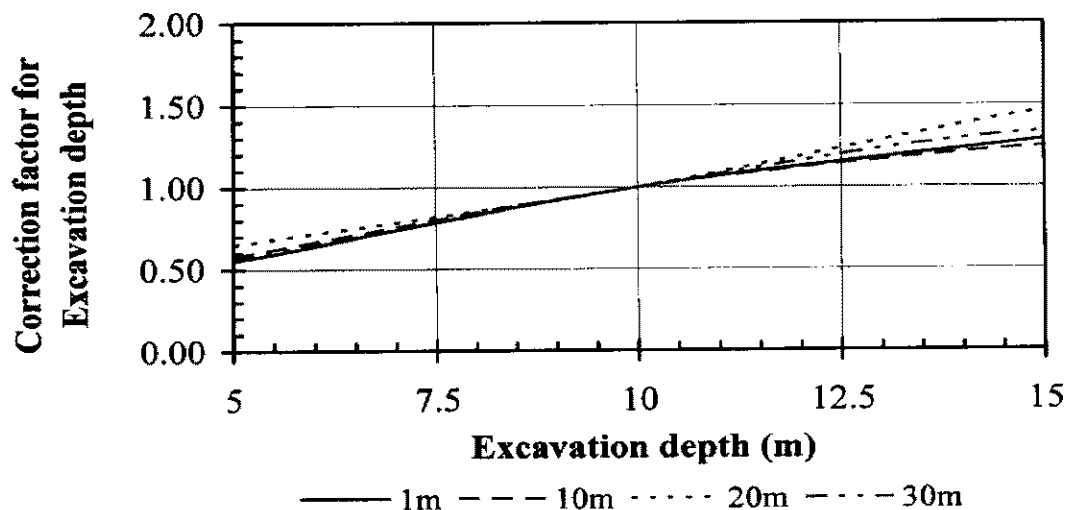


Figure 8.28: Correction factor for excavation depth for horizontal soil movements.

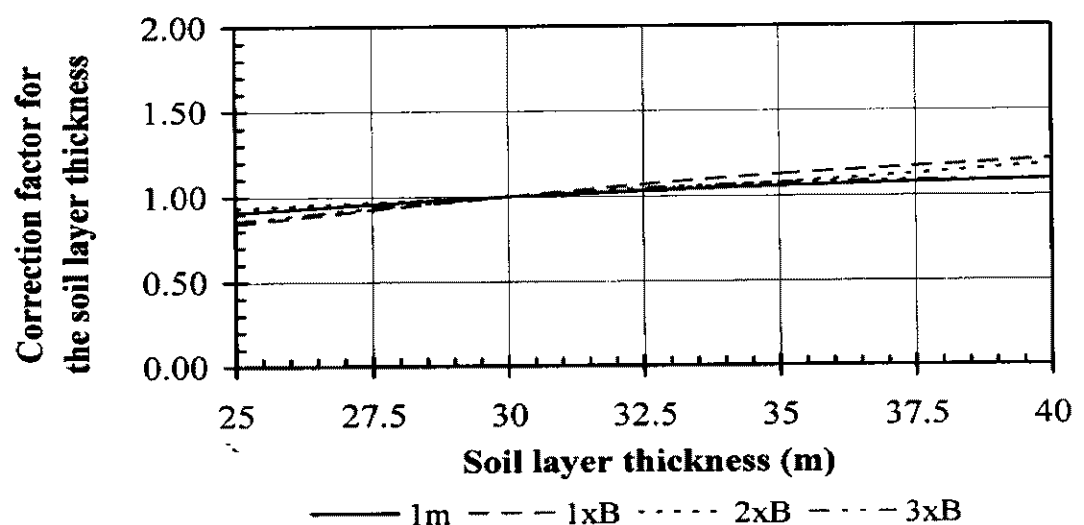


Figure 8.29: Correction factor for the homogeneous soil layer thickness for horizontal soil movements.

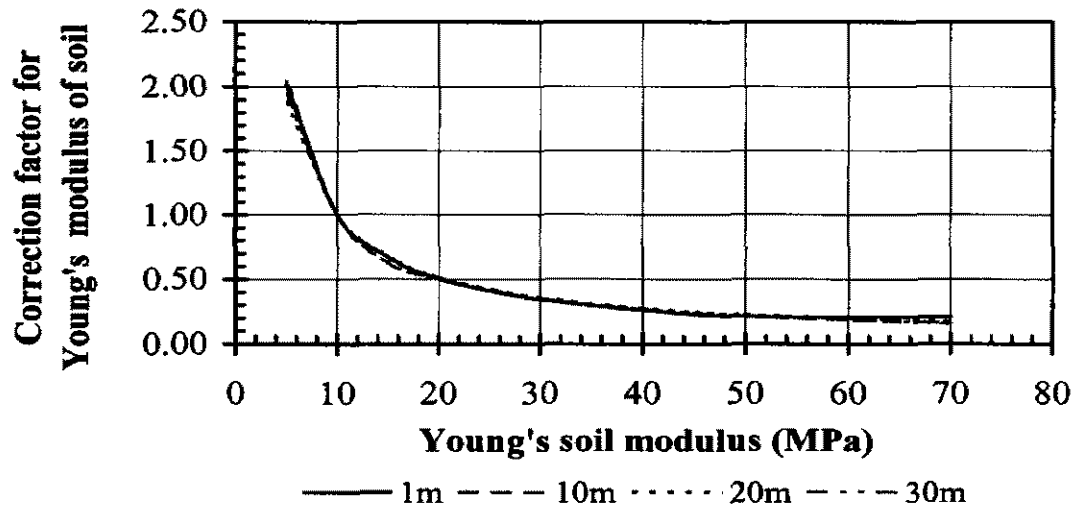


Figure 8.30: Correction factor for Young's modulus of soil for horizontal soil movements.

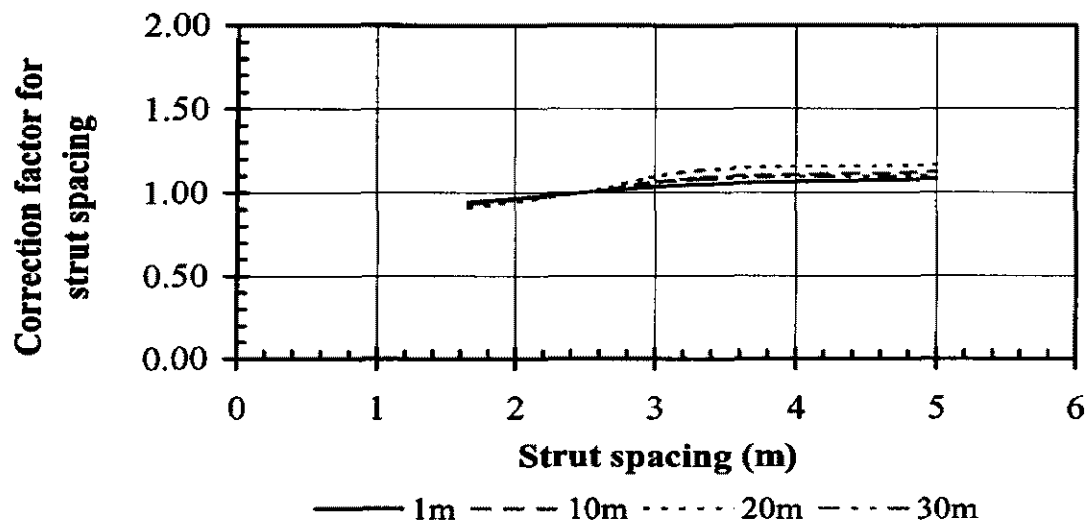


Figure 8.31: Correction factor for strut spacing for horizontal soil movements.

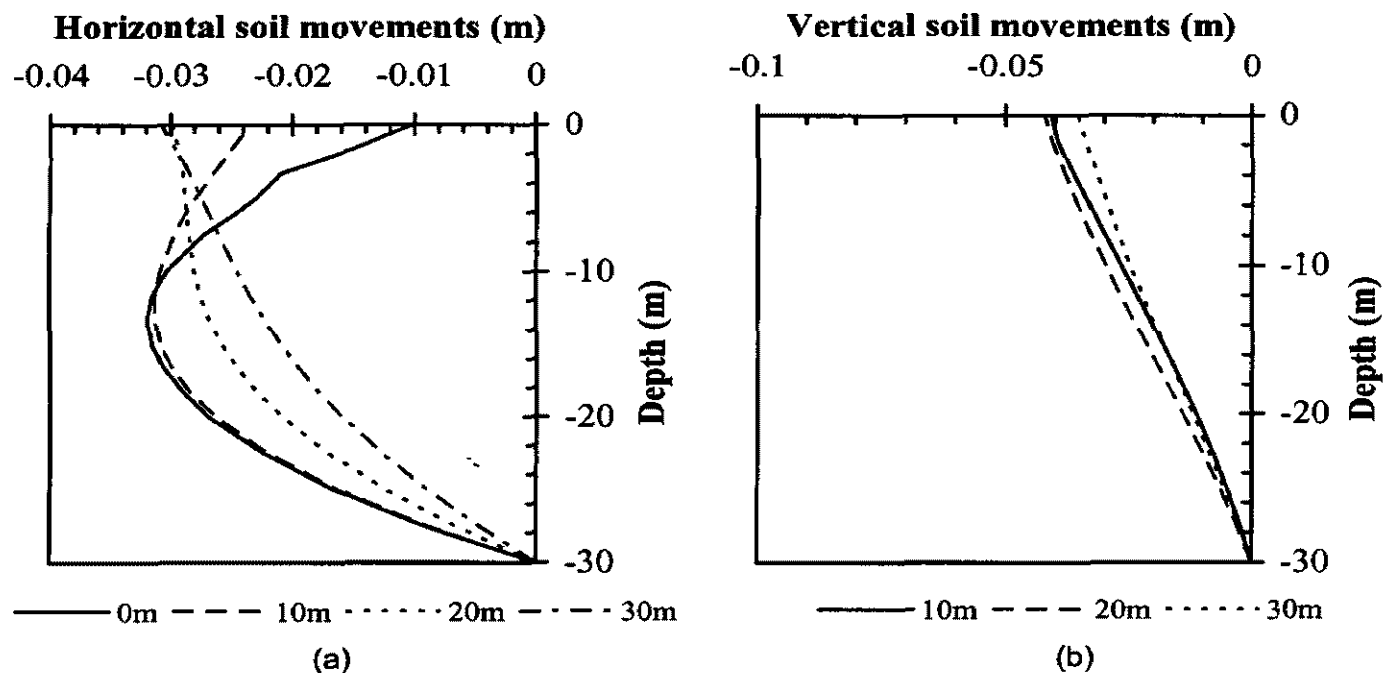


Figure 8.32: Horizontal and vertical soil movements at different distances from the wall.

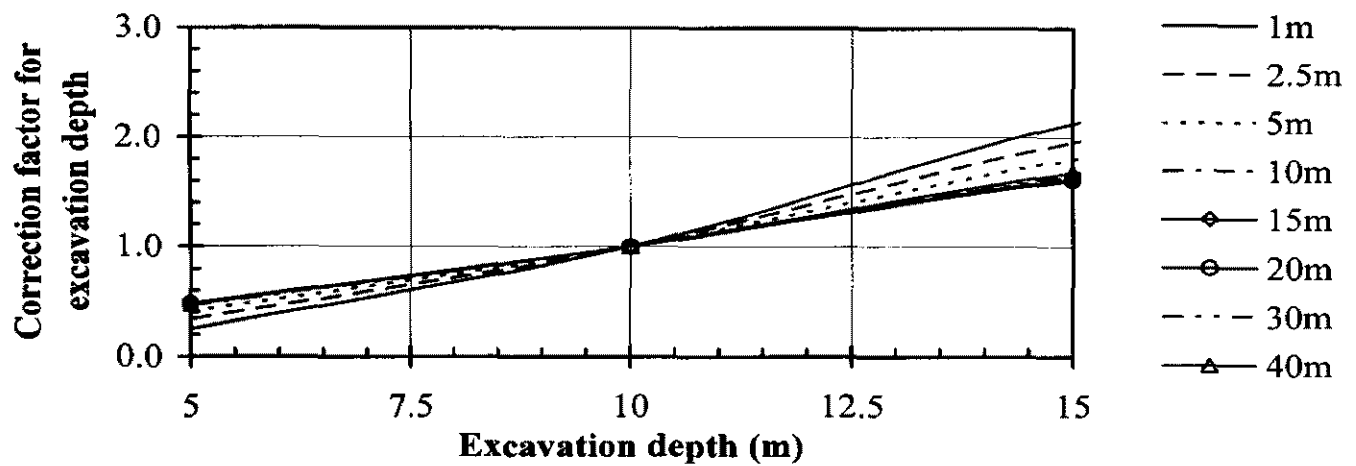


Figure 8.33: Correction factor for excavation depth for vertical soil movements.

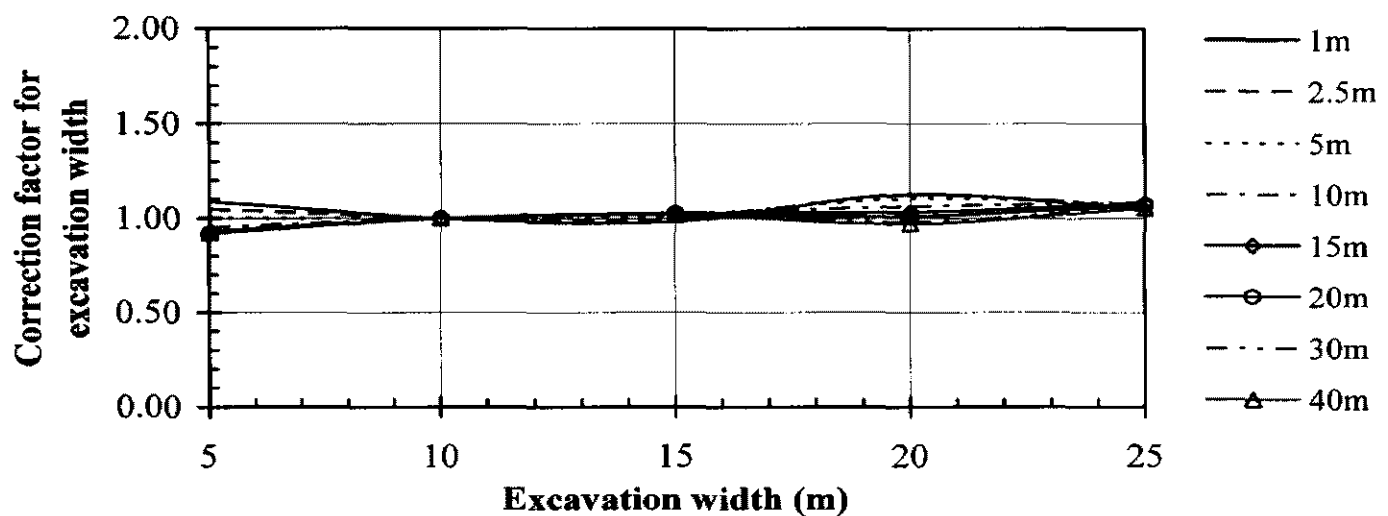


Figure 8.34: Correction factor for excavation width for vertical soil movements.

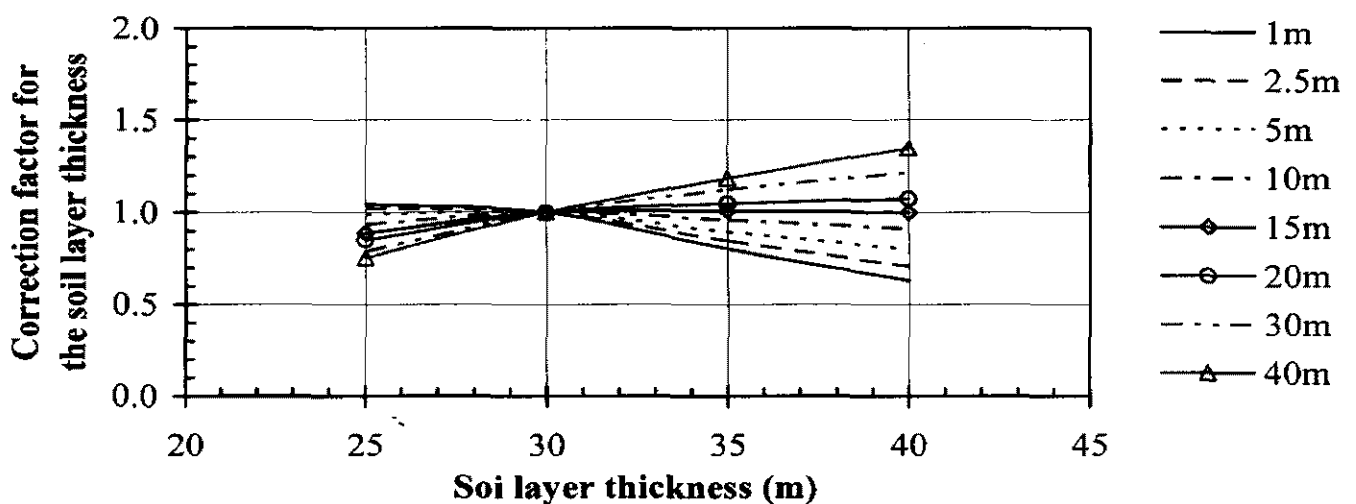


Figure 8.35: Correction factor for the soil layer thickness for vertical soil movements.

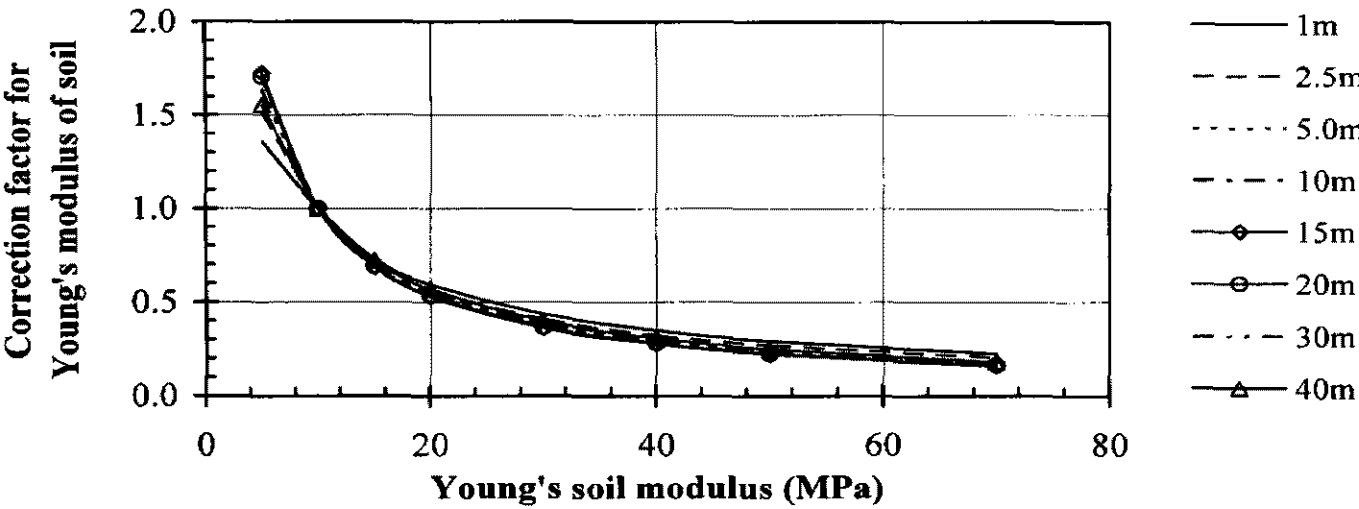


Figure 8.36: Correction factor for Young's soil modulus for vertical soil movements.

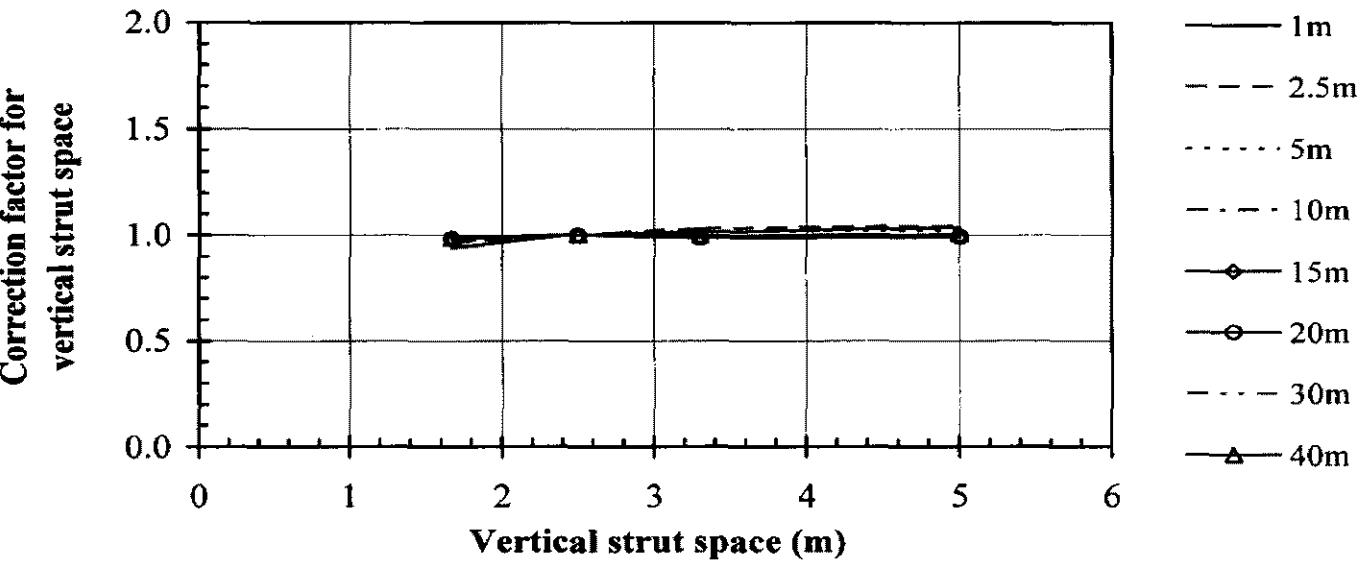


Figure 8.37: Correction factor for vertical strut spacing for vertical soil movements.

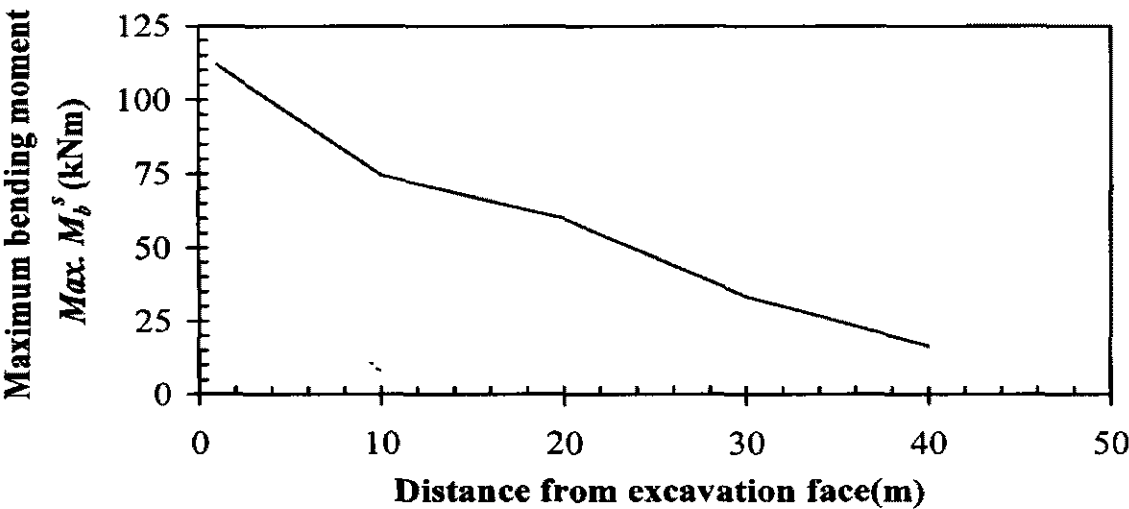


Figure 8.38: Basic maximum bending moments versus distance from excavation face.

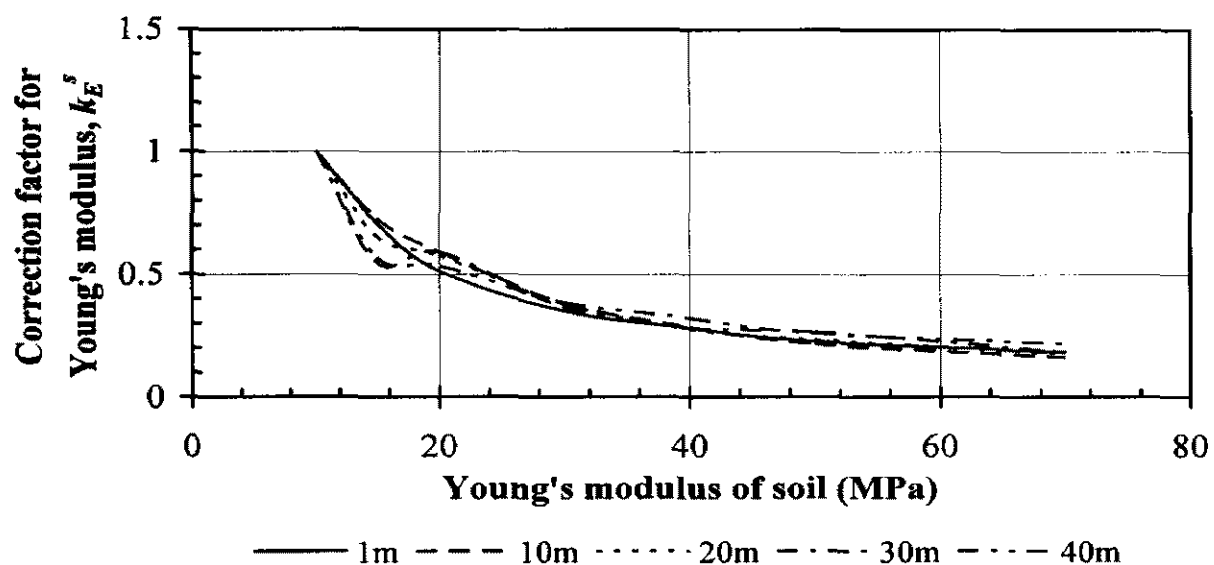


Figure 8.39: Maximum bending moment correction factor for Young's modulus of soil for pile at different distances from retaining wall.

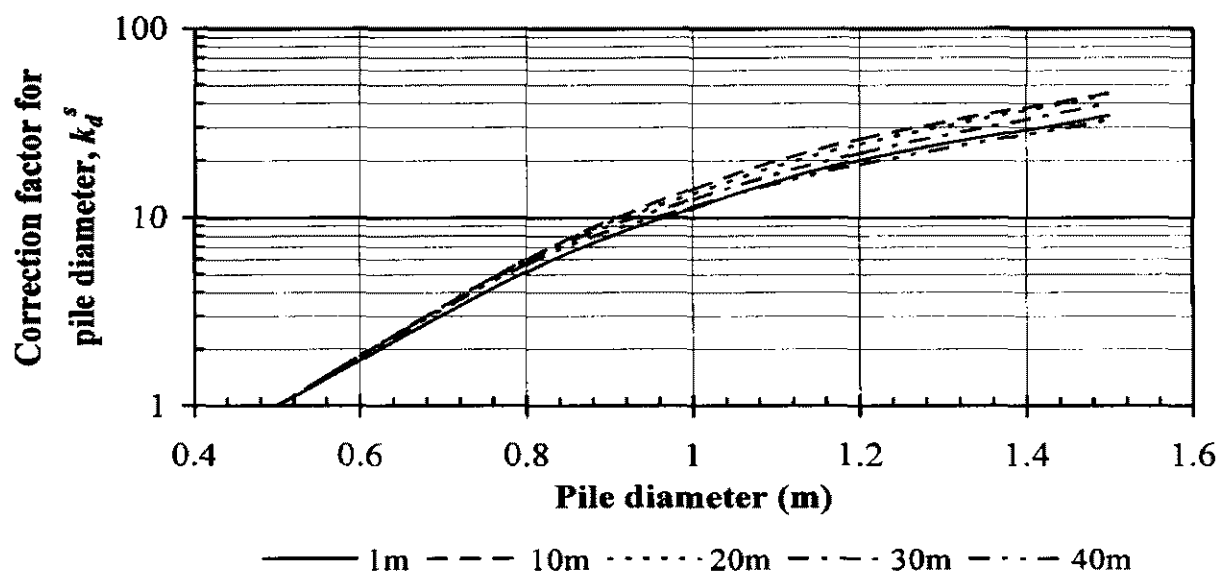


Figure 8.40: Maximum bending moment correction factor for pile diameter at different distances from retaining wall.

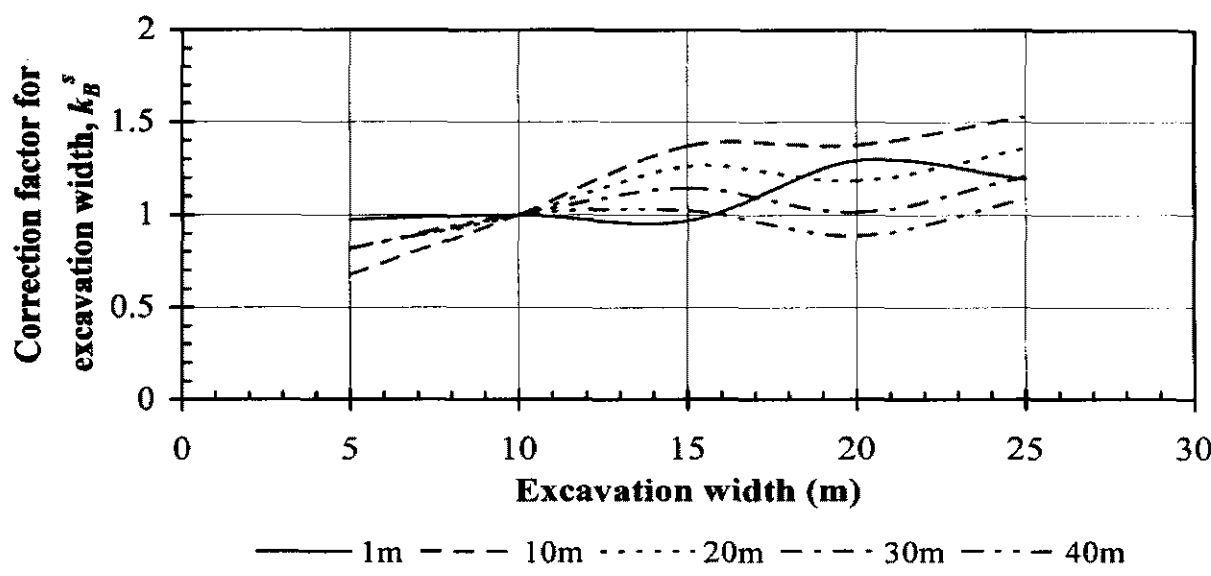


Figure 8.41: Maximum bending moment correction factor for excavation width at different distances from retaining wall.

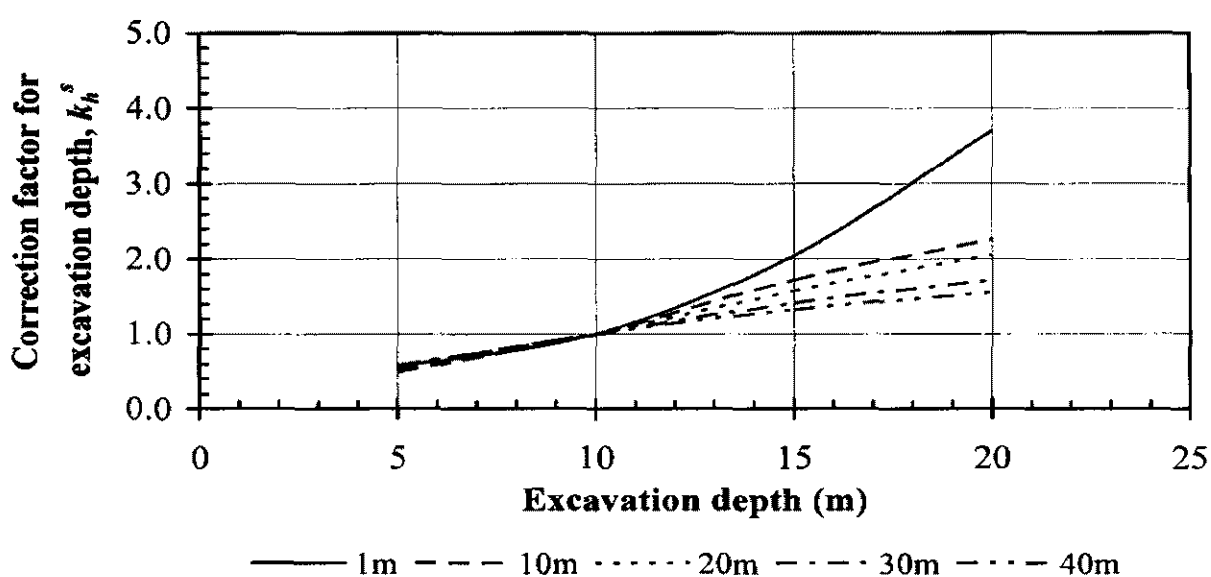


Figure 8.42: Maximum bending moment correction factor for excavation depth at different distances from retaining wall.

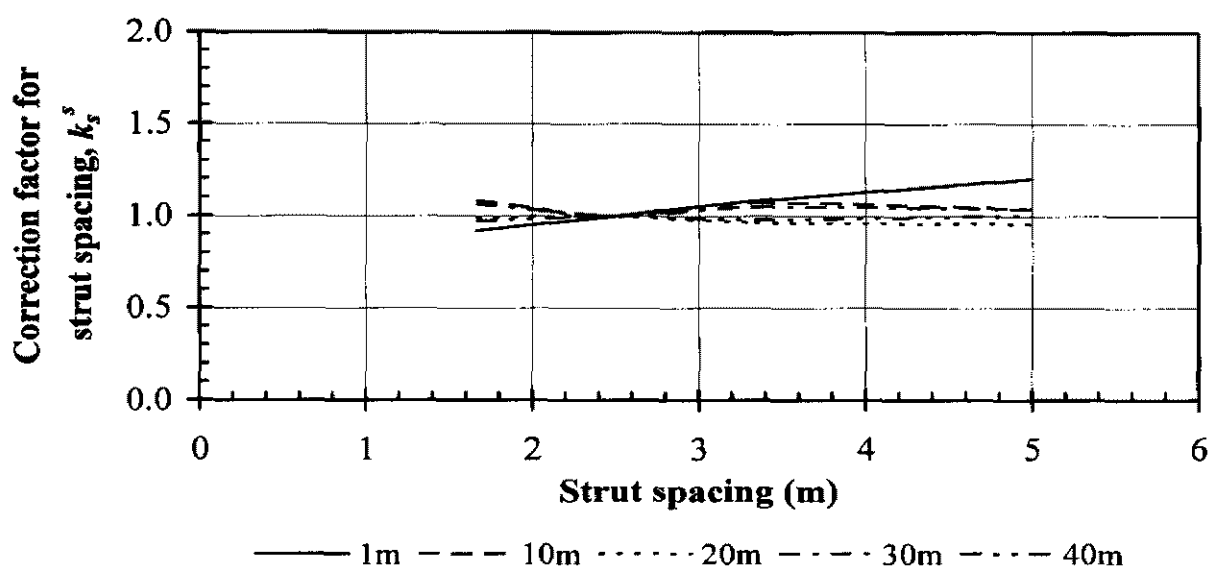


Figure 8.43: Maximum bending moment correction factor for strut spacing at different distances from retaining wall.

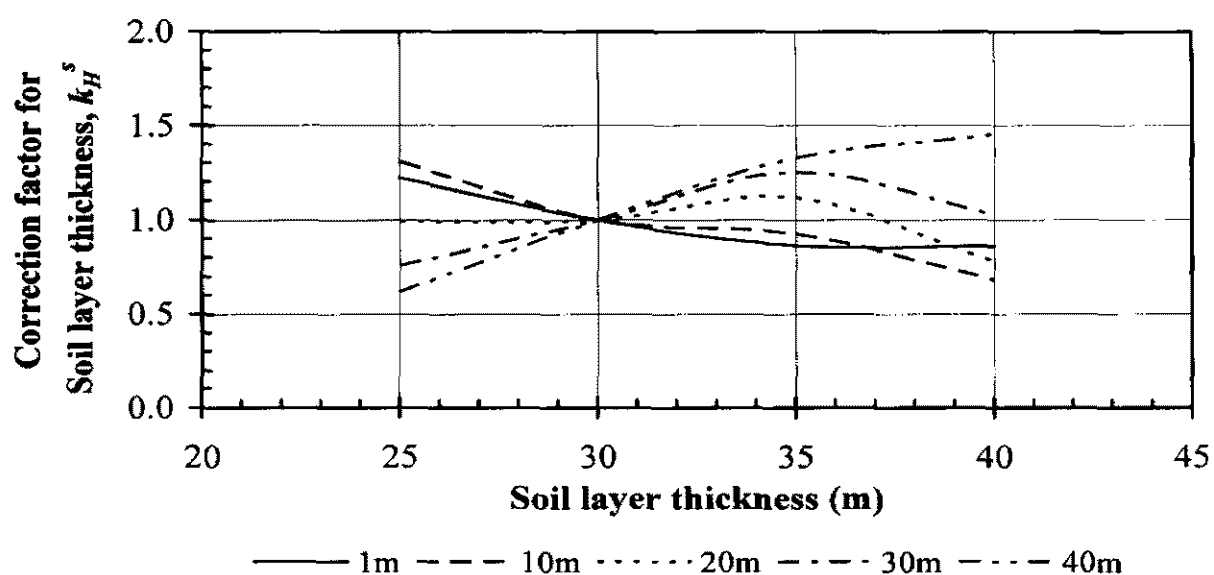


Figure 8.44: Maximum bending moment correction factor for the homogeneous soil layer thickness at different distances from retaining wall

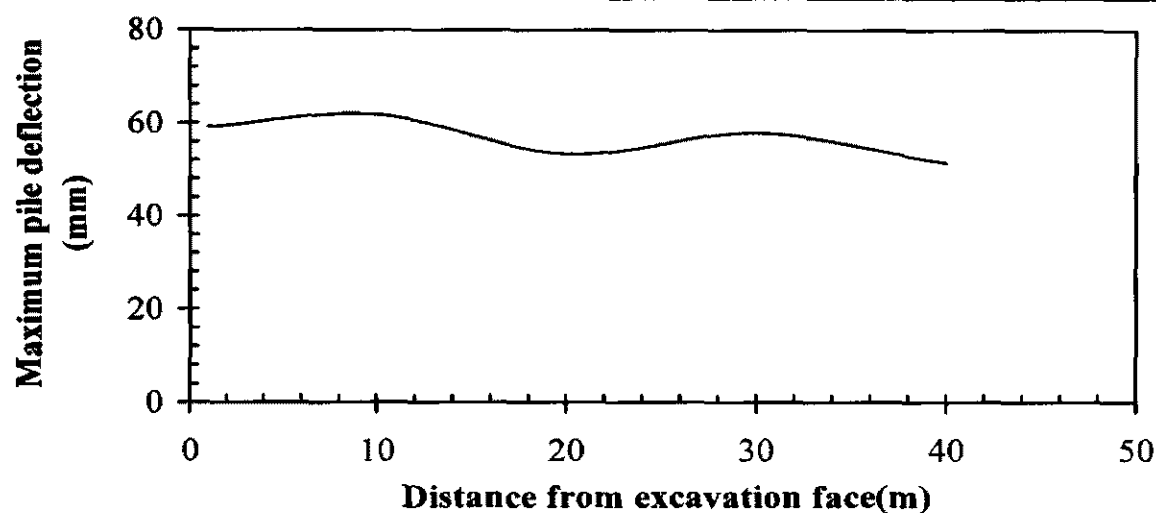


Figure 8.45: Basic maximum pile deflection versus distance from excavation face.

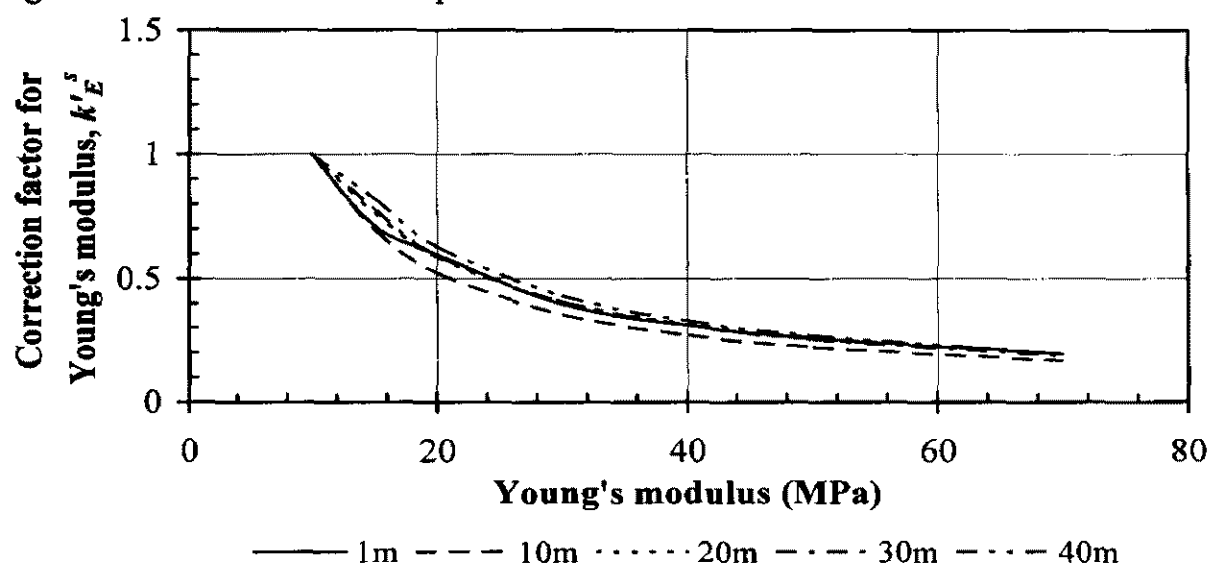


Figure 8.46: Maximum horizontal pile deflection correction factor for Young's modulus at different distances from retaining wall.

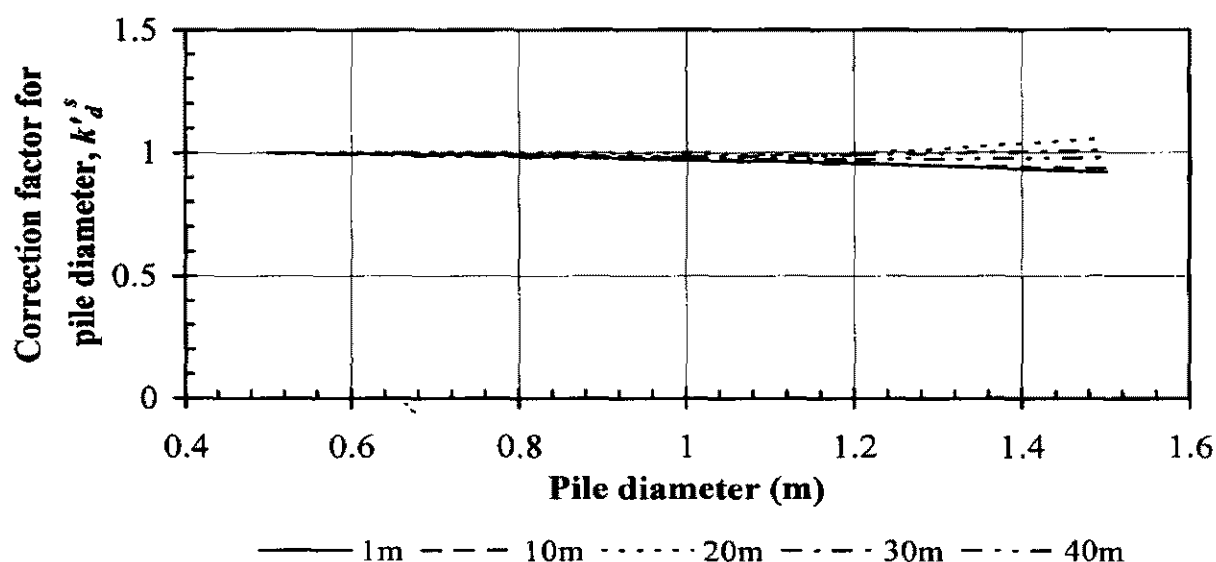


Figure 8.47: Maximum horizontal pile deflection correction factor for pile diameter at different distances from retaining wall.

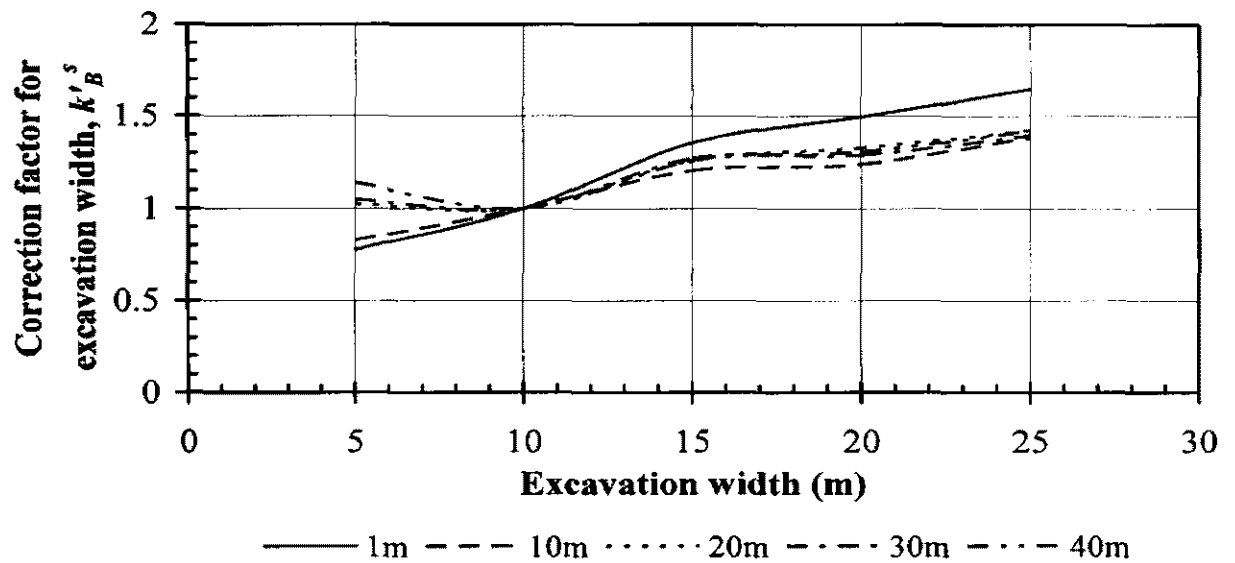


Figure 8.48: Maximum horizontal pile deflection correction factor for excavation width at different distances from retaining wall.

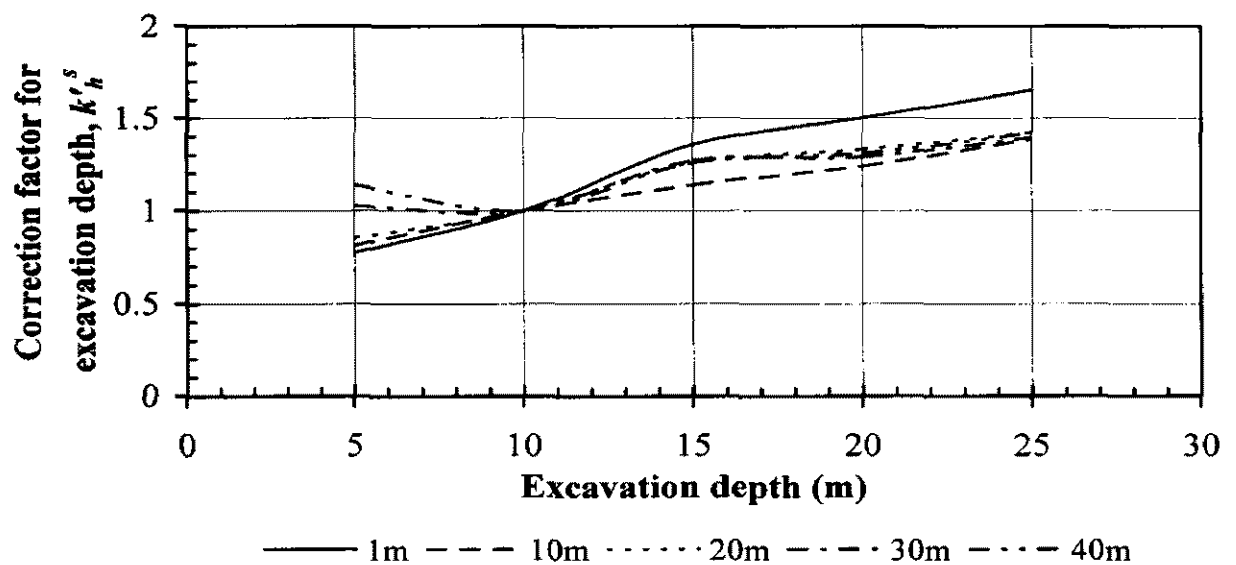


Figure 8.49: Maximum horizontal pile deflection correction factor for excavation depth at different distances from retaining wall.

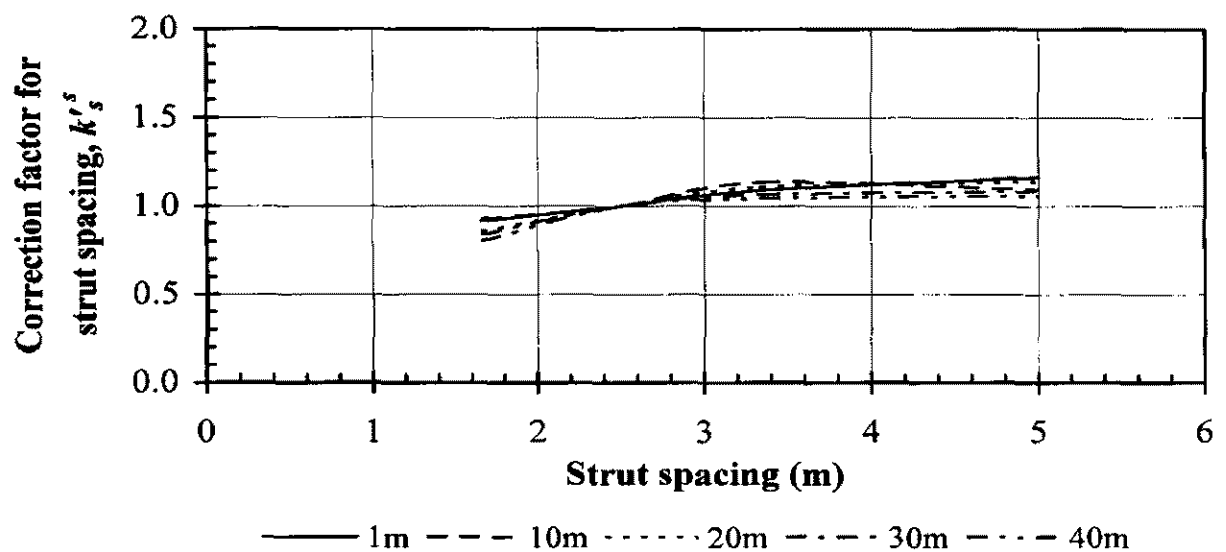


Figure 8.50: Maximum horizontal pile deflection correction factor for strut spacing at different distances from retaining wall.

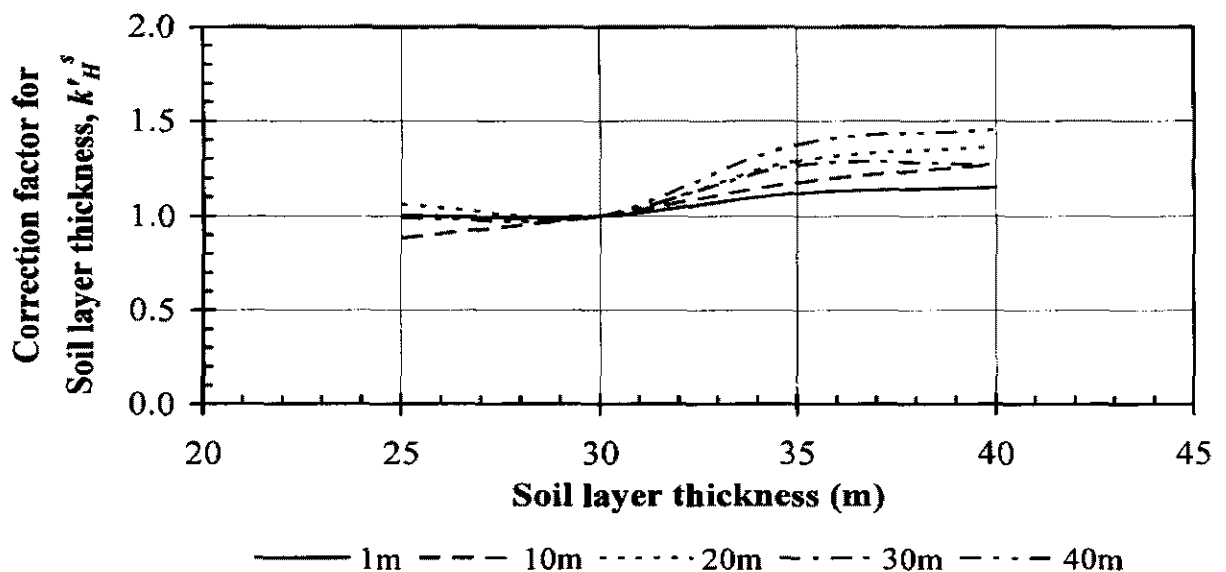


Figure 8.51: Maximum horizontal pile deflection correction for the homogeneous soil layer thickness at different distances from retaining wall.

CHAPTER 9

SUMMARY AND CONCLUSIONS

9.1 SUMMARY AND CONCLUSIONS

Soil movements due to lowering of ground water table in the ground water exploitation and the excavation processes has been investigated in this study. The influences of the soil movements on piled foundations have also been considered in both situations. The outcomes of this study are presented as follow:

In Chapter 2, a review of some of the existing theories that have been developed by several researchers involving analysis of soil behaviour due to ground water withdrawal has been presented. The available methods for calculating soil behaviour due to groundwater extraction has been categorised into three classes that are (1) Analytical methods; (2) Transform methods; (3) Numerical methods. Advantages and disadvantages of each method have been discussed. Both analytical and transform methods have a common limitation that is they can not be applied to problem in which there is drawdown of the water table. The drawdown of the water table can be analyzed by applying numerical methods. Numerical methods can be sub-classified into two types (1) coupled and (2) uncoupled methods. Uncoupled numerical methods are those methods in which the fluid flow equation is calculated separately. The change in pore pressures resulted from solving the fluid flow equation are then used as input data to compute the soil deformations. On the other hand, in coupled methods, pore pressure changes and soil deformations are computed simultaneously in both the saturated and the unsaturated zone. By doing that coupled methods can take into account the effects of the pore pressure changes on the deformation of the soil and vice versa. Of these coupled methods, a fully coupled approach that was developed by His, Cater, and Small (1992) has been employed to study soil behaviour due to lowering of ground water table.

The geological and hydrological conditions of the Quaternary sediments in the Hanoi area have been presented in Chapter 3. The sediment types, ages and distributions have been introduced. In general, the Quaternary sediments in the Hanoi area can be classified into two sediment types: the coarse-particle sediments lying below and the relatively fine particle sediments above. These sediments are usually young, normally consolidated, and uncemented sediments. The geotechnical properties of some of the

sediments in the Hanoi area have also presented. However, there is still very little information on the mechanical properties of the soil. The hydrological of the Hanoi soils have also been provided. In the Hanoi area, the division of the aquifer and aquitard system includes three water-bearing layers, namely qh , qp^1 , and qp^2 , and two impervious layers. Among these aquifers, the Lower Pleistocene Aquifer (qp^1) is the only main water source supplying water for the whole Hanoi area. It covers most of the Red River Delta with thickness ranging between 3m and 40m. The geological and hydrological of the Hanoi area were used to investigate two case studies of soil behaviour analysis due to ground water pumping in Chapter 5. It was also used as references in accessing soil parameters required in the pile analyses.

Chapter 4 has reviewed the design and the installation process of 6 benchmarks around the Hanoi area. The benchmarks were installed to monitor the total soil settlements and the lowering of the ground water table, to investigate the subsoil layers underlying the area. The collected measured data from six benchmarks have been used to carry out investigation on the effects of the ground water pumping on the Hanoi soil. For ground movement measurements, two types of devices were installed namely measured pointed and benchmarks to monitor the movements of typical soil layers and to observe the total movements of the ground surface respectively. Two types of devices were also installed in the benchmarks in Hanoi. They are the hydraulic and pneumatic piezometers. The former is usually used for gauging pore pressure in sandy soil type and the former is applied for monitoring pore pressure in clayey soil type. Data of ground water drawdown and the ground surface settlement have been collected and presented. Layout of soil layers beneath of each benchmark station and thickness has also been introduced with the depth where these measurement devices were installed.

In Chapter 5, comparisons of soil behaviour due to ground water pumping of two typical case studies from the finite element analysis results and the measured data were presented. Two typical case studies are the Phap Van case study and the Ngoc Ha case study with two different types of soil conditions respectively, one covered with a thick layer of clay soil and the other covered with a thick layer of sand.

Comparisons of the analysis results with the measured data have been carried out. Reasonably good predictions of the drawdown of the water table and of surface settlements could be made for the Hanoi area. The prediction of the ground movements in the future due to an assumed ground water pumping rate were also performed and presented. In fact from the finite element analysis of two case studies, horizontal soil movement due to ground water pumping is significant and the maximum horizontal soil movements in two typical case studies can be approximate 8% to 12% of the maximum vertical soil movement. Parametric study of the ground behaviour due to ground water pumping has been carried out. The relationships between parameters have been investigated, which are ground water pumping rate, radial distribution of the pumped wells, and soil layer thicknesses of two soil layers. It was demonstrated that the maximum horizontal soil movements are normally located at in the vicinity of edge of the pumped water area.

Study of pile behaviour due to ground water pumping has been conducted in Chapter 6. A short review of pile responses due to external movement imposed soil movements have been presented. Three boundary element programs that used to analyse pile behaviour due to external vertical and horizontal imposed soil movements, namely PALLAS, PIES and GEPAN, have been introduced. In this study, PALLAS and PIES have been used as two main programs. Due to the fact that the current version of GEPAN only allows the user to analyse a purely elastic pile behaviour analysis, it is only used at a limiting state. Hypothetical case studies of behaviour of single piles and pile group existing near the Phap Van ground water station have been conducted. The soil movements estimated from Chapter 5 were utilized as the source of induced external imposed at the pile soil interface for the pile behaviour analysis input data. The parametric study of pile behaviour due to external imposed soil movements have been presented and discussed. This study demonstrates that damage to an existing pile because could occur because of a significant induced bending moment due to ground water pumping.

Chapter 7 was presented to clarify the capability of finite element program EF4 so that it can be used to predict excavation with lowering of ground water table induced

soil movements. A brief review of available approaches for excavation induced soil movement prediction has been presented. The available approaches were classified in three categories (1) empirical approaches (2) total stress approaches (3) and effective stress approaches. Fully coupled finite element program EF4 was employed to analyse three difference case studies that are the Davidson Avenue and Rankin Street excavation, large cavitation channel excavation at President's Island, Memphis, Tennessee, and underground parking garage excavation at Post Office Square, Boston. It was again shown that the finite element program EF4 could be used to carry out excavation analysis with lowering of the ground water table.

In Chapter 8, soil movement results of the finite element analyses of excavation have been used as input data for the pile behaviour analysis programs. Three case studies have been investigated and presented. Comparisons between the calculated results of excavation induced soil movements and pile behaviour with the measured data of both centrifuge and full-scale tests indicated that the combination of the finite element analysis and the boundary element analysis can be used to predict pile behaviour near excavation with reasonable accuracy. The approximate method to model a row of piles as an equivalent wall near excavation needs to be used carefully. Parametric study of the drained excavation with ground water table drawdown has been carried out. The excavation with lowering of ground water table parametric study demonstrated that the soil deformation due to excavation with lowering of ground water table depends much on the deformation characteristics of the soil, the excavation geometry and the depth of the homogeneous soil layer where excavation is carried out. A parametric study of pile behaviour near an excavation with drawdown of water table has been conducted. Comparison between this drained pile behaviour study and the undrained pile behaviour study of Chen and Poulos (1997) has been shown that the undrained analysis can produce a greater induced pile bending moment than the drained analysis.

9.2 FUTURE RESEARCH

A recommendation for future research is a need for more appropriate characterisation of the ground conditions in geological, geotechnical, as well as hydrological aspects

in the Hanoi area. The soil investigations should be carried out to the depth of gravel and pebbles layers to understand the distribution as well as the geological and hydrological properties of this layer.

From the finite element analysis, apart from the vertical soil movement, horizontal soil movement due to ground water pumping is significant. Hence, measured of horizontal soil movements due to ground water pumping is required.

In this study, the effects of pile head conditions have not been carefully considered. Hence a further study need to focus on pile behaviour with different pile head conditions such as, a group of piles with raft on top. More researches also need to focus on the relationship between a new construction and the existing structures to understand the interaction between them. This requires effort to investigate existing structure state and the ground condition in which it presents.

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