ON THE DESIGN AND ANALYSIS OF ANCHORED SHEET PILE WALLS

By

WEI ZHANG

Bachelor of Science

Hohai University

Nanjing, China

1983

Submitted to the Faculty of the
Graduate College of the
Oklahoma State University
in partial fulfillment of
the requirements for
the Degree of
MASTER OF SCIENCE
May, 1992

ON THE DESIGN AND ANALYSIS OF

ANCHORED SHEET PILE WALLS

Thesis Approved:

Mete Öner

Thesis Adviser

Vernon C. Matzen

Donald R. Snethen

Thomas C. Collins

Dean of the Graduate College

ACKNOWLEDGEMENTS

I wish to express sincere appreciation to Dr. Mete Oner, my academic advisor, for his constant encouragement, wise advice and invaluable support throughout my graduate program and thesis preparation. He has always been a source of assistance, guidance and understanding during the period of my graduate studies. Many thanks also go to Dr. Donald R. Snethen and Dr. Vernon A. Mast for serving on my graduate committee. Their suggestions and directions were very helpful throughout the study.

I appreciate Mr. Chungcheng Lai, my colleagues in the graduate program, for his careful discussion and assistance in computer data preparation for this research.

I would like to dedicate my thesis to my wife, Qin Wang, for her love, understanding, and support. Without her, this thesis would not have been possible. I would like to, also, dedicate this thesis to my parents, Guobin Zhang and Xiuying Feng, and my brothers, Qi Zhang and Cheng Zhang, and my son, Tianchen Zhang. In many ways, I am proud of them and enjoy being with them.

TABLE OF CONTENTS

Chapter Page

LIST OF TABLES

Table		Page

LIST OF FIGURES

Figure Page

CHAPTER I

INTRODUCTION

There are two conventional methods for the design of anchored sheet pile walls. These are the free-earth support (FES) and fixed-earth support (FxES) methods. These methods have been used for a long time because of their simplicity. The FES method is much simpler of the two, and it gives smaller depths of penetration (i.e., it is more economical). Therefore, it appears to be favored in practice. However, it is known that this method gives large bending moments. Therefore, the moment calculated by FES is reduced by Rowe's (1952) empirical curves before selecting a sheet pile section. A computer program WALSHT (Dawkins,1988) that uses these methods was found to be a convenient tool for the purposes of this study.

Because of the somewhat questionable simplifying assumptions made in the conventional methods it was found desirable to analyze a number of typical sheet pile walls by the finite element method (FEM). The FEM is the most sophisticated method of stress and deformation analysis available today. The main advantage of the FEM is that the number of simplifying assumptions made in its derivation is a minimum. The finite element method is a complete method of analysis which gives the stresses everywhere in the soil and around the sheet pile as the excavation progresses, rather than making arbitrary assumptions about active and passive pressures as in the free-earth-support method. The computer program, FEMSSI (Oner, 1989) was used in this study. This is a finite element analysis program specially developed to analyze nonlinear soil-structure interaction problems such as a sheet pile wall. The method requires expertise, detailed soil property data, and extensive engineer time and computer resources. Due to these factors FEM is not currently considered to be a routine design method, but a research tool.

In this thesis, the free earth support method was used to design sheet pile walls in 27 typical soil profiles. Then, the finite element method was used to evaluate the results given by the free earth support method in each case. This thesis reports the results of these analyses, comparisons, and the practical conclusions derived from them.

CHAPTER II

APPLICATION OF FREE EARTH SUPPORT METHOD

General

In general, the design of a sheet pile wall involves the following, after the determination of the governing environmental factors that includes the design-basis loadings, and soil profile and properties:

(1) Selection of the wall type (cantilever versus anchored), and calculation of the required penetration depth,

(2) Determination of the pile bending moment (and shear) and the selection of a pile section from among the commercially available sections,

(3) Design of an anchorage system, if necessary.

In this chapter, the selections of soil profile and properties, the basis of the free earth support method (FES), and the program (WALSHT) used are discussed. Then, the results given by FES are presented and discussed.

Selection of Soil Properties and Wall Friction

Soil properties and wall friction are very important factors in sheet pile wall design. The unit weight and the shear strength parameters of the soils involved, together with geometric factors govern lateral earth pressures. The design cases studied in this thesis involved cohesionless and cohesive soils. The factor of the undrained and drained conditions in cohesive soils were considered. So a brief description of soil behavior is presented in this section.

The "Sand" and "Clay" Idealization

It has become common to simplify the soil classification by a broad division as "sands" and "clays". In this thesis, the sand-clay division is still used, but in a slightly different sense. The term "sand" is taken as a soil which has a zero cohesion intercept, with a sufficiently high permeability, as nonplastic silts, clean sands and gravel. "Clay" is taken as a soil with a significant amount of fines such that it behaves differently under drained and undrained loading conditions (Oner, 1989).

Shear Strength of Sands

The shear strength of sand is usually be given by a ϕ angle. This angle is called by a number of different names, such as angle of internal friction, friction angle, angle of shear resistance (or strength), all of which mean the same thing. The cohesion intercept, if any, is usually neglected in practice since doing so is conservative. There are typical friction angle values listed as "recommended for preliminary designs" by

Hough (1957), and a list of actually measured values (Oner, 1989).

Shear Strength of Clays

The shear strength of clays is considerably more complicated than sands because of their (1) lower permeability, (2) higher void ratios, and (3) interaction of water with particles.

Since the undrained condition may be expected to occur under "fast" loading in the field, it represents the "short term"; in time, drainage will occur and the drained strength will govern (the "long term" condition). The general approach in solving problems involving clay is that, unless the choice is obvious, both undrained and drained conditions are analyzed separately. The more critical condition governs the design. Total stresses are used in an analysis with undrained shear strength (since pore pressures are "included" in the undrained shear strength), and effective stresses are used in a drained case; thus such analyses are usually called total and effective stress analyses.

The undrained shear strength of a normally consolidated clay is usually expressed by only a cohesion intercept, it is labeled c_u to indicate that ϕ was taken as zero. c_u decreases dramatically with water content; therefore, in design, the engineer may consider the fully saturated condition even if a clay is partly saturated in the field. The c_u value increases with depth (or effective stress) and this is commonly expressed with the ratio "c_u/p" (p denotes the effective vertical stress). The undrained shear strength of many over consolidated soils is further complicated due to the presence of fissures; this leads to a lower field strength than what tests on small laboratory samples indicate.

On basis of stress-strain behavior, the drained shear strength of normally consolidated clays is similar to that of loose sand (c'=0), except that the ϕ angle is generally lower. The drained shear strength of over consolidated clays is similar to that of dense sand (again with lower ϕ angles), where the soil exhibits both a peak strength and a "residual" strength.

Wall Friction

It is known that the wall friction has an important effect on lateral earth pressures, especially on passive pressures. This produces a significant effect in design. Laboratory passive pressure model tests have shown, for walls rotating about the base, that the wall friction angle can be as large as the ϕ angle at the top, but gradually decreases to zero at the bottom, the average being about 2/3 ϕ (Oner, 1989).

Selection of Soil properties and wall friction

In this research, all the soil properties and wall friction parameters were provided by Dr. Oner. The soil properties used in this research are given in Table 2.1. The wall friction angles were taken as 2/3 ϕ in all cases.

TABLE 2.1

SOIL PROPERTIES

Soil Type	Symbol	Cohesion (c)	Friction Angle (ϕ)	Moist Soil Density (γ_{wet})	Saturated Soil Density (γ_{sat})
Loose Sand	L	0	30	97	123
Medium-Dense Sand	D	0	36	110	131
Soft Clay, Undrained	S	c_u * Profile	0	95	110
Medium-Stiff clay Undrained	M	c_u ** Profile	0	110	120
Soft Clay, Drained	T	0	25	95	110
Medium Clay, Drained	N	0	30	110	120

* based on $c_u/p = 0.25$ below G.W.T.

** based on $c_u/p = 0.40$ below G.W.T.

Cases Studied

Thirty one profiles with loose sand, dense sand, soft clay (undrained and drained conditions), and medium-stiff clay (undrained and drained) were considered. All cases were comprehensively studied by both FEM and FES. The profiles and the case names are given in Table 2.2.

TABLE 2.2

THE PROFILES AND THE CASES NAMES

LL40	DL40	ML40	SL40,SL30	NL40	TL40
LD40	DD40	MD40	SD40	ND40	TD40
LM40	DM40	MM30,MM40	SM30		
LS30	DS30	MS30	SS30,SS20		
LN40	DN40			NN30	TN40
LT40	DT40			NT40	TT30

Note: L = Loose sand,
D = Dense sand,
S = Soft clay,
M = Medium-stiff clay,
T = Soft clay under drained condition,
N = Medium-stiff clay under drained condition.

The case names in the above table indicate the soil types and the wall free height; the first letter stands for soil type behind the wall and the second letter stands for foundation soil type, and the number is the height of excavation in sheet pile wall. The depth of the anchor was taken as one fourth of the excavated depth. The ground water table was also assumed to be at the same elevation as the anchor. The sheet pile wall profile is as shown in Fig. 2.1.

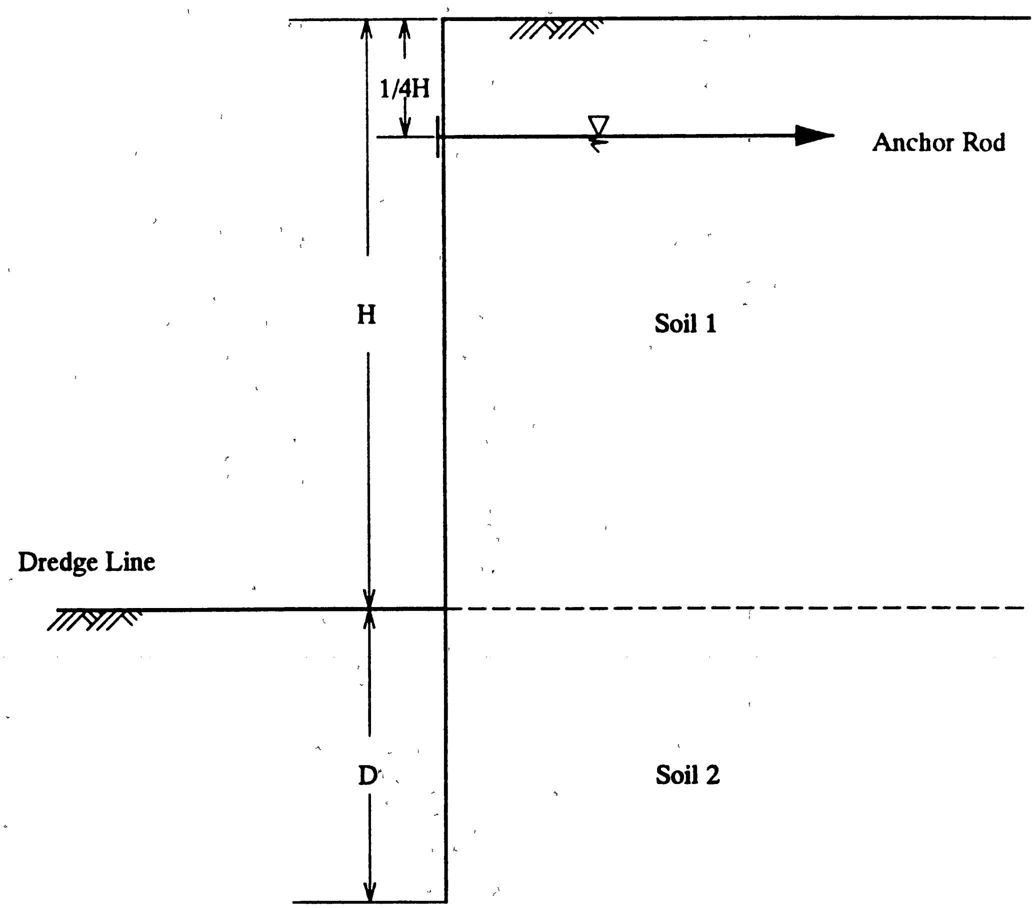

Figure 2.1 The sheet pile wall profile

The Free Earth Support Method

Principles

The free earth support method assumes that the piling is rigid and it may rotate at the anchor-rod level, with failure occurring by rotation about the fixed anchor rod. Passive pressure develops in the soil in front of the piling, and active pressure develops behind the wall. The assumed pressure diagrams and identification of terms are illustrated in Fig.2.2.

The formulation is based on the moment equilibrium about the anchorage point. This equation can be solved for the unknown depth of penetration, D. Once the depth has been found the pressures below the dredge line are determined; then the anchor force and bending moment are obtained from static.

The Design Program

The free earth support method was coded into a design program WALSHT (Dawkins, 1988). It is a very convenient and powerful program for using the FES to design anchored sheet pile walls. The program also includes Rowe's moment reduction calculations and it offers some aid for the selection of a sheet pile section.

Applying WALSHT program to the design of sheet pile walls involves the following steps: (1) setting up input data, (2) running the WALSHT program, and (3) examining the results to select the height of the sheet pile wall, penetration depth, and a suitable section to be used.

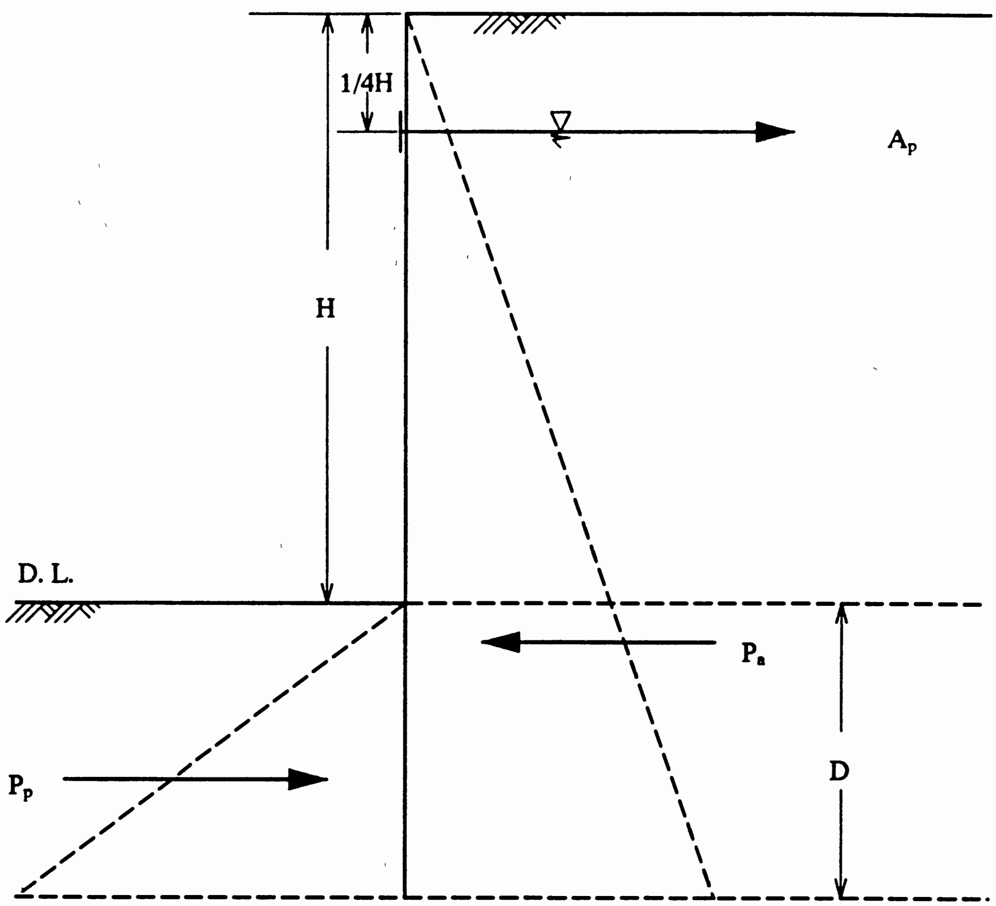

Figure 2.2 Presumed earth pressure on an anchored wall

Input Data preparation

There are two means for input data preparation. (1) by hand, using a text editor; or (2) interactively, by answering the questions of the program. In the latter case the program will (optionally) write the data entered to a correctly formatted file. This file can later be edited to correct the mistakes, if any, using a text editor.

The cases considered in this study involved continuously varying c_u profiles, while WALSHT program accepts constant c_u values for each soil layer. There, to represent variable c_u profiles, a large number of soil layers (with a different c_u in each) were used. The format of input data of the WALSHT program is given as follows.

INPUT DATA OF WALSHT PROGRAM

. Heading: One to four lines text. Start with '

. Anchored design:

CONTROL ANCHORED DESIGN {number of anchor} {factor of safety}

. Wall Structure

WALL {elevation of top of pile from dredge line} {elevation of anchor position from dredge line}

. Surface position

SURFACE RIGHTSIDE {number of ranges} {X1} {X2}

SURFACE LEFTSIDE {number of ranges} {X1} {X2}

. Soil characteristics

SOIL RIGHTSIDE STRENGTHS {number of soil data}

{Saturated Soil Density} {Moist Soil Density} {Angle Of Internal Friction} {Cohesion} {Angle Of Wall Friction} {Adhesion} {Elevation Of Each Point} {Slop Of Each Point}

SOIL LEFTSIDE STRENGTHS {number of ranges}

{Saturated Soil Density} {Moist Soil Density} {Angle Of Internal Friction} {Cohesion} {Angle Of Wall Friction} {Adhesion} {Elevation Of Each Point} {Slop Of Each Point}

. Water

WATER ELEVATIONS {water density} {X1} {X2}

. Control

FINISH

A typical input data is as follows:

```
1000  'lm40 case
1010  '
1020  '
1030  CONTROL ANCHORED DESIGN 1 1.5
1040  WALL 40.0 30.0
1050  SURFACE RIGHTSIDE 1 0 40.0
1060  SURFACE LEFTSIDE 1 0 0
1070  SOIL RIGHTSIDE STRENGTHS 13
1080   97.0    97.0  30    0.00  20 0   30.0  0
1090  123.5   123.5  30    0.00  20 0   25.0  0
1100  123.5   123.5  30    0.00  20 0   20.0  0
1110  123.5   123.5  30    0.00  20 0   15.0  0
1120  123.5   123.5  30    0.00  20 0   10.0  0
1130  123.5   123.5  30    0.00  20 0    5.0  0
1140  123.5   123.5  30    0.00  20 0    0.0  0
1150  120.5   120.5   0 1175.00   0 0   -5.0  0
1160  120.5   120.5   0 1291.00   0 0  -10.0  0
1170  120.5   120.5   0 1407.00   0 0  -15.0  0
```

```
1180   120.5   120.5   0  152 120.5   120.5   0  1639.00   0 0  -25.0  0
1200   120.5   120.5   0  1755.00   0 0
1210   SOIL LEFTSIDE STRENGTHS  6
1220   120.5   120.5   0  1175.00   0 0   -5.0  0
1230   120.5   120.5   0  1291.00   0 0  -10.0  0
1240   120.5   120.5   0  1407.00   0 0  -15.0  0
1250   120.5   120.5   0  1523.00   0 0  -20.0  0
1260   120.5   120.5   0  1639.00   0 0  -25.0  0
1270   120.5   120.5   0  1755.00   0 0
1280   WATER ELEVATIONS 62.5 30.0 30.0
1290   FINISH
```

Results

The output of the WALSHT program consists of several parts. Useful information for designing sheet pile walls includes the echo of the input data, soil pressures, results given by the free earth support method, Rowe's moment reduction and available sections. A sample output of WALSHT program is given in Appendix A.

Discussion of the Results

Penetration Depth

Penetration depth is an important factor in FES design. Thirty one cases with loose sand, dense sand, soft clay (undrained, and drained), medium-stiff clay (undrained, and drained) were designed by the free earth support method.

Table 2.3 shows the results given by FES for loose sand foundation with different backfill soils. The ratio of the excavation depth to the total sheet pile height, α = D/(H+D), ranges from 0.74 to 0.77. The average α ratio is 0.75. These results show that the penetration depth depends on the foundation soil, the soils behind the wall seems to have little influence on the penetration depth. For this loose sand foundation soil (γ_{wet}=95 pcf; γ_{sat}=123 pcf; ϕ=30^0), the α ratio in anchored sheet pile wall design can be taken as 0.74. Since the smallest α ratio gives the larger penetration depth,D, the design of penetration is on safe side.

These results are in agreement with the result for uniform loose sand given by Rowe (1956). Rowe showed that, for a uniform loose sand, the ratio of excavation depth to pile height is 0.73 (his anchor level was at 0.2 (H+D).

TABLE 2.3

LOOSE SAND FOUNDATION SOIL

Case	Excavation depth (H) (ft)	Penetration Depth (D) (ft)	Height of pile (ft)	The Ratio α=H/(H+D)	Section Selected
LL40	40	13.03	53.03	0.75	PZ35
DL40	40	13.23	53.23	0.75	PZ35
SL40	40	14.20	54.20	0.74	PZ40
SL30	30	10.65	40.65	0.74	PZ35
ML40	40	12.18	52.18	0.77	PZ22
TL40	40	12.57	52.57	0.76	PZ27
NL40	40	13.18	53.18	0.75	PZ27

Table 2.4 also shows the results for dense sand foundation soil with different backfill soils given by FES. The variation of the ratio α for dense sand with different

soils behind the wall ranges from 0.80 to 0.84, average being 0.82. For this typical dense sand foundation soil (γ_{wet}=110 pcf; γ_{sat}=131 pcf; ϕ=36°), the α ratio in anchored sheet pile wall design can be taken as 0.80.

TABLE 2.4

DENSE SAND FOUNDATION SOIL

Case	Excavation depth (H) (ft)	Penetration Depth (D) (ft)	Height of pile (ft)	The Ratio α=H/(D+H)	Section Selected
LD40	40	8.58	48.58	0.82	PZ40
DD40	40	8.44	48.44	0.83	PZ35
SD40	40	9.85	49.85	0.80	PZ40
MD40	40	7.82	47.82	0.84	PLZ25
TD40	40	8.50	48.50	0.82	PZ38
ND40	40	8.68	48.68	0.82	PZ38

The results in both Table 2.5 and 2.6 for soft clay foundation soils (undrained, and drained) with different backfill soils given by FES. The variation of the α ratio for soft clay as foundation soil with different soils behind the wall ranges from 0.54 to 0.58, average being 0.57. The variation of the α ratio for soft clay (drained) as foundation soil with different soils behind the wall ranges from 0.64 to 0.67, average being 0.66. The soft clay (γ_{wet}=95 pcf; γ_{sat}=110 pcf; ϕ=25°; c_u/p=0.25) as foundation soils, the α ratios in anchored sheet pile wall design can be taken as 0.54 (undrained) and 0.64 (drained) for safe side consideration.

TABLE 2.5

SOFT CLAY FOUNDATION SOIL (UNDRAINED)

Case	Excavation depth (H) (ft)	Penetration Depth (D) (ft)	Height of pile (ft)	The Ratio α=H/(H+D)	Section Selected
LS30	30	23.52	53.52	0.56	PZ32
DS30	30	25.49	55.49	0.54	PZ32
SS30	30	23.52	53.52	0.56	PZ40
MS30	30	21.40	51.40	0.58	PZ27

TABLE 2.6

SOFT CLAY FOUNDATION SOIL (DRAINED)

Case	Excavation depth (H) (ft)	Penetration Depth (D) (ft)	Height of pile (ft)	The Ratio α=D/(D+H)	Section Selected
LT40	40	20.97	60.97	0.66	PZ27
DT40	40	22.04	62.04	0.64	PZ23
TT30	30	14.71	44.71	0.67	PZ40
NT40	40	21.25	61.25	0.65	PLZ25

The results of medium-stiff clay (undrained and drained) as foundation soils show in Table 2.7 and 2.8. The variation of the α ratio for medium-stiff clay (undrained) as foundation soil with different soils behind the wall ranges from 0.80 to 0.88, average being 0.86. The variation of the α ratio for medium-stiff clay (drained) as foundation soil with different soils behind the wall ranges from 0.75 to 0.76, average being 0.75. The medium-stiff clay (γ_{wet}=110 pcf; γ_{sat}=120 pcf; ϕ=30°; c_u/p=0.40) as foundation soils, the α ratios in anchored sheet pile wall design can be selected as 0.80 (undrained) and 0.75 (drained) for safe side consideration.

TABLE 2.7

MEDIUM-STIFF CLAY FOUNDATION SOIL (UNDRAINED)

Case	Excavation depth (H) (ft)	Penetration Depth (D) (ft)	Height of pile (ft)	The Ratio α=H/(D+H)	Section Selected
LM40	40	6.66	46.66	0.86	PZ32
DM40	40	5.67	45.67	0.88	PLZ25
SM30	30	7.49	37.49	0.80	PZ27
MM40	40	5.36	45.36	0.88	PZ27
MM30	30	4.04	34.04	0.88	PZ22

TABLE 2.8

MEDIUM-STIFF CLAY FOUNDATION SOIL (DRAINED)

Case	Excavation depth (H) (ft)	Penetration Depth (D) (ft)	Height of pile (ft)	The Ratio α=H/(D+H)	Section Selected
LN40	40	13.44	53.44	0.75	PLZ23
DN40	40	13.66	53.66	0.75	PLZ23
TN40	40	12.97	52.97	0.76	PZ27
NN40	40	13.64	53.64	0.75	PLZ25
NN30	30	10.23	40.23	0.75	PZ22

From above tables, the α ratios for the same foundation soil with different backfill soils is nearly same, but the α ratio for different foundation soils are quite different. So the results confirm that the penetration depth depends on the foundation soil, the soils behind the wall seems to have little influence on the penetration depth.

The wall friction also influences penetration depth. Agreement was reached with the results of a parametric study by Kovacs et al. (1974) using the free earth support method are given in Fig. 2.3. The studied profile had a uniform sand (γ_{wet}=105 pcf; γ'=60 pcf; ϕ=32°; c=0) and excavated depth (H=25 ft). It is seen in this figure that using a wall friction angle equal to 2/3 ϕ may reduce the penetration by a factor of two. In TT30 case, the penetration depth reduces by 37.56%, the maximum bending moment reduces by 40.21%, and the anchor force reduces by 28.32%. In addition, a much lighter section could be used where when the wall friction was taken into account (Table 2.9).

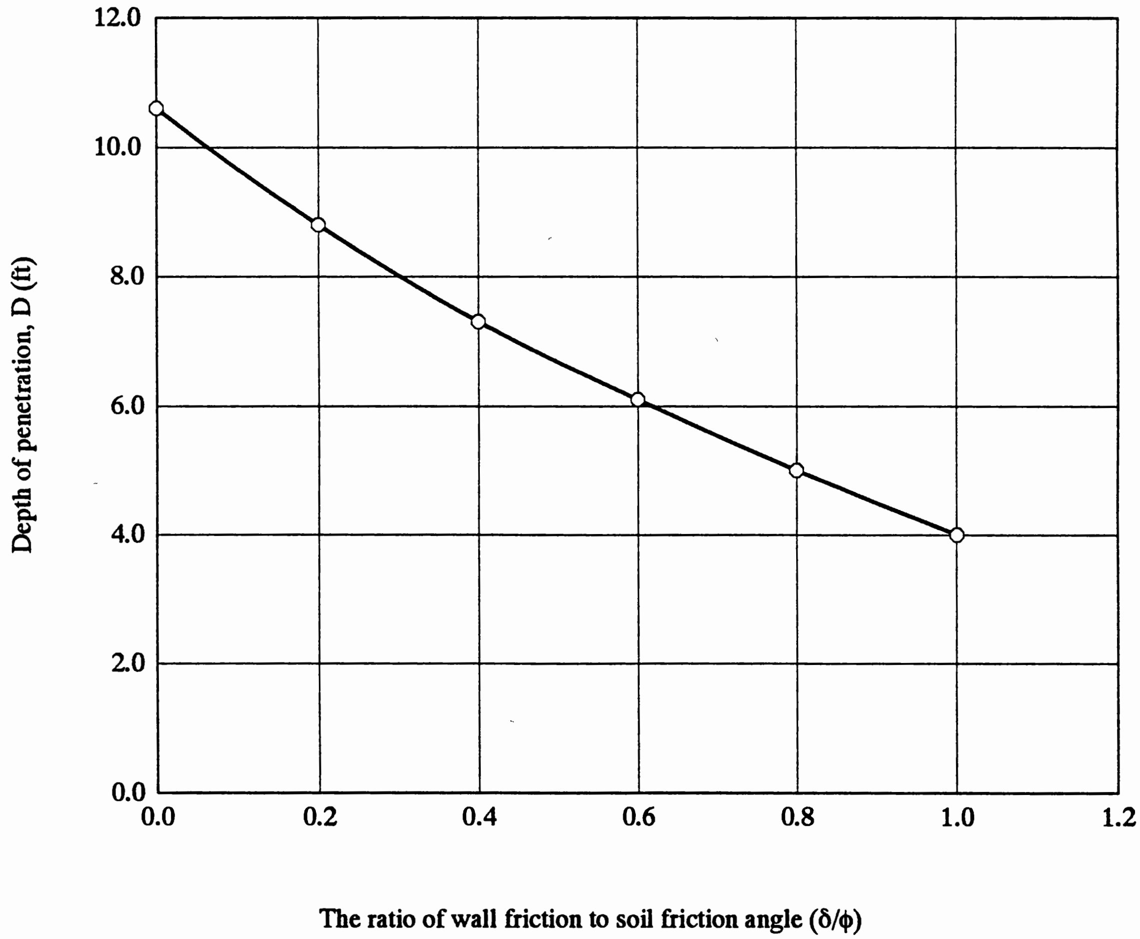

Figure 2.3 Effect of Wall Friction on Depth of Penetration

TABLE 2.9

WALL FRICTION EFFECT

Wall Friction	Penetration Depth	Maximum Moment(k-ft)	Anchor Force (k)	Section Selected
(1) 0	23.56	101.88	10.5	PZ40
(2) 2/3(phi)	14.71	60.91	7.526	PZ22
[(1)-(2)]/(1)	37.56%	40.21%	28.32%	

Rowe's Moment Reduction

Rowe(1952) proposed moment reduction for sheetpiling designs based on the free-earth method. Rowe's "moment reduction curve" attempts to correct the major shortcoming of the free earth support method, namely the unrealistically large bending moments, so that this simpler method can be used in design. Rowe's curves are given in Fig.2.4. These curves give a moment reduction ratio based on wall flexibility and soil stiffness. The maximum moment calculated by free earth support method is corrected by this factor for the purpose of selection a pile section.

In Rowe's curves, the pile flexibility is expressed in terms of a ρ parameter:

$$\rho = (H+D)^4 / EI$$

Where H is the free height and D is the actual penetration in feet, E is the Young modulus of steel in psi, and I is the (area-) moment of inertia of a trial pile section in in^4 per ft width of wall, The soil stiffness is expressed by the following parameters:

For sands: D_r (relative density); curves given for loose (D_r=0) and dense

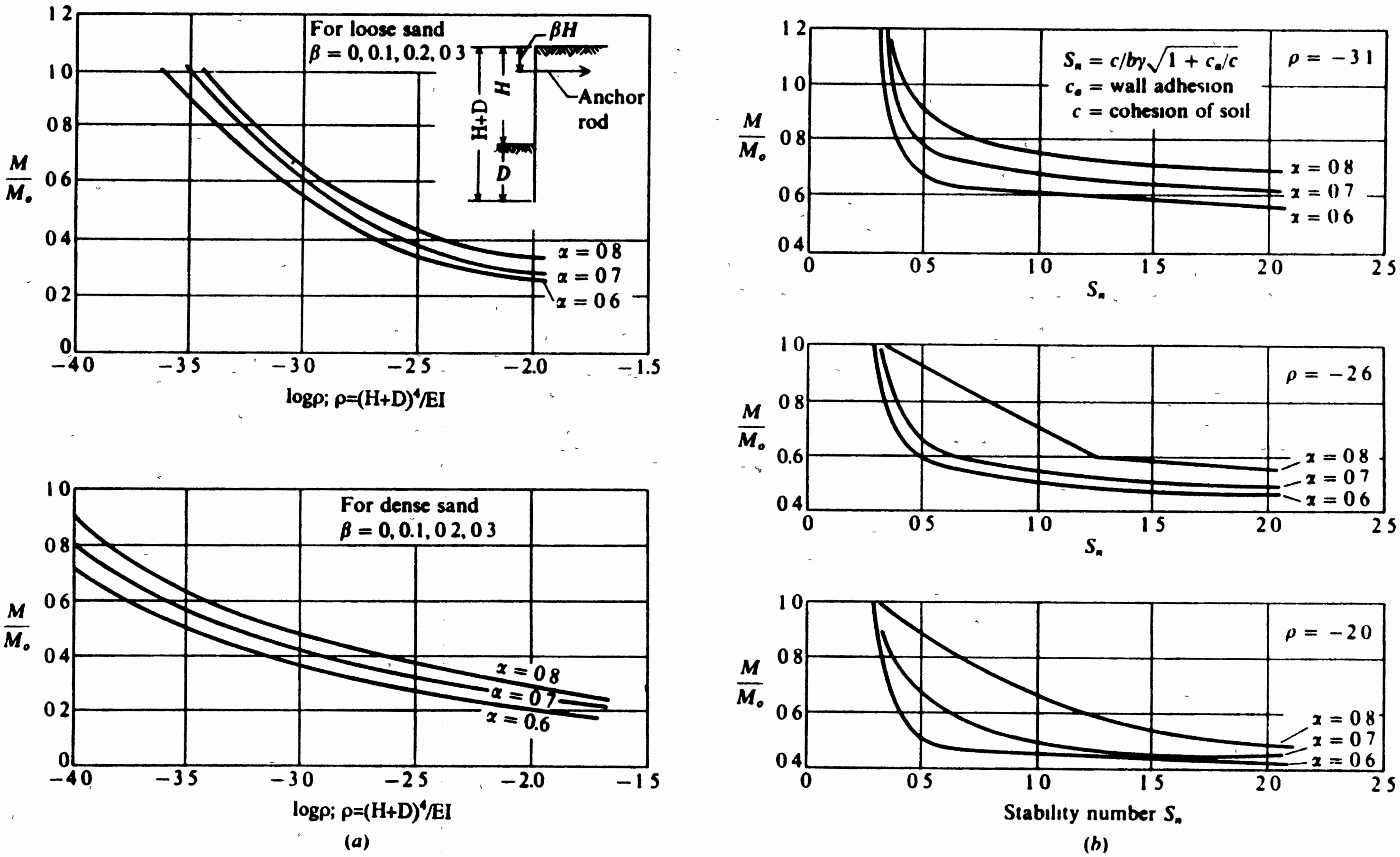

Figure 2.4 Rowe's moment-reduction curves for use with the free-earth support method (a) Sheet piles in sand [After Rowe (1952).] (b) Sheet piles in clay. [After Rowe (1957).]

(D_r=100), see Fig. 2.4 (a). For clays (undrained): Stability number $S_n = 1.25(c_u/p)$, see Fig. 2.4 (b). In this research, total 31 design cases were studied by using FES and the program WALSHT. The results given by the free earth support method with WALSHT program show that Rowe's reduction factors were not applied in about half of the cases as discussed below.

a. Reduction was not applied if $\log \rho = \log [(H+D)^4/EI]$ less than -3.5 or greater than -1.5 because this is the experimental range in Rowe's curves. In SL30 case $\log \rho$ was -3.58 and program WALSHT did not apply Rowe's reduction for this case.

b. Reduction was not applied if the ratio $\alpha = H/(H+D)$ was less than 0.6 or greater than 0.8. The α was slightly greater than 0.8 in cases where dense sand was the foundation soil (see Table 2.10), and the α factor was less than 0.6 in cases where foundation soil was an undrained soft clay (Table 2.11).

TABLE 2.10

CASES FOR WHICH REDUCTION WAS NOT APPLIED

IN DENSE SAND FOUNDATION SOIL WHERE $\alpha > 0.8$

Case	Excavation depth (H) (ft)	Penetration Depth (D) (ft)	Height of pile (ft)	The Ratio $\alpha = H/(D+H)$	Section Selected
LD40	40	8.58	48.58	0.82	PZ40
DD40	40	8.44	48.44	0.83	PZ35
SD40	40	9.85	49.85	0.80	PZ40
MD40	40	7.82	47.82	0.84	PLZ25
TD40	40	8.50	48.50	0.82	PZ38
ND40	40	8.68	48.68	0.82	PZ38

TABLE 2.11

CASES FOR WHICH REDUCTION WAS NOT APPLIED

IN UNDRAINED SOFT CLAY WHERE $\alpha < 0.6$

Case	Excavation depth (H) (ft)	Penetration Depth (D) (ft)	Height of pile (ft)	The Ratio $\alpha = H/(H+D)$	Section Selected
LS30	30	23.52	53.52	0.56	PZ32
DS30	30	25.49	55.49	0.54	PZ32
SS30	30	23.52	53.52	0.56	PZ40
MS30	30	21.40	51.40	0.58	PZ27

The variation of the ratio $\alpha = H/(H+D)$ for soft clay foundation (undrained) with different backfill soils ranges from 0.54 to 0.58, the average being 0.56. The ratio in all cases is less than 0.6, and program WALSHT refused to apply any reduction factor. This leads to an uneconomical section selection. The program could use $\alpha = 0.6$ in such cases and apply a reduction factor on the bending moment; this would sill be conservative.

c. WALSHT does not apply a reduction factor if the stability number S_n is less than 0.5. These cases are listed in Table 2.12.

TABLE 2.12

WALSHT DOES NOT APPLY A REDUCTION FACTOR

WHERE THE STABILITY NUMBER $S_n < 0.5$

Case	c_u/p_o	$S_n=1.25c_u/p_o$
LS30, DS30, SS30, MS30	0.25	0.31
LM40, DM40, SM30, MM30, MM40	0.40	0.50

d. Reduction factors are not available for drained clay cases because Rowe's tests were all undrained. Because of this, no moment reduction was applied in all cases with clay soil under drained condition. These cases are LT40, DT40, TT30, NT30, LN40, DN40, TN40, NN30.

CHAPTER III

APPLICATION OF THE FINITE ELEMENT METHOD

General

The finite element method (FEM) is a powerful analytical technique. Armed with a suitable stress-strain (constitutive) model for the soils involved, it is capable of predicting the entire stress and deformation field for a soil-structure interaction problem. Therefore, FEM can be applied to any complex soil-structure interaction problem with confidence.

The FEMSSI program was employed to evaluate and analyze the sheet pile walls designed by the free earth support method. The input data preparation for FEMSSI program, the output of the program, and evaluation of the results are presented in this chapter.

Pre-processing

Input data preparation is a critical step for a finite element analysis. The input data required is very long and complicated so that mistakes are usually made if input data is prepared by hand. To facilitate data preparation for FEM another computer program is used; such a program is called a pre-processor. In this research the program

GENERECT (Oner, 1990) was used to generate the required FEMSSI data files.

GENERECT Program

The GENERECT program is a rectangular grid data generation for FEMSSI (Oner, 1990). It generates finite element grids with the following characteristics:

(1) The grid is symmetric, with a beam in the middle that is used to represent a sheet pile wall;

(2) Nodes & elements are numbered from top left corner, row-wise, which minimizes the band-width.

The program (GENERECT) reads simple data from a very short data file called "the GEN file" here.

GEN file

A GEN file contains the following information:

(1) Sheet pile wall characteristics including the total length of the pile, excavation depth, penetration depth, and the pile section;

(2) Geological conditions including the depth of different soil layers, ground water condition, and soil properties;

(3) Finite element mesh size, division choices, and the boundary conditions.

The sheet pile wall characteristics of 27 cases as determined by the free earth support method for finite element method analyses are shown in Table 3.1. The soil properties used in FEM are given as Table 3.2. A typical sample profile is shown in

Fig. 3.1. The profile has two types of soils. One behind the wall, called the "backfill" and the other below the dredge line, called the "foundation soil" in the following.

The influence area concerned was taken as follows: in the horizontal direction, 12 times of excavation depth (12H) from the pile on both sides, and in the vertical direction, 6 times of excavation depth (6H) below the dredge line. The boundary conditions are such that on the two sides, X direction is fixed, Y direction is free, and on the bottom, both directions are fixed.

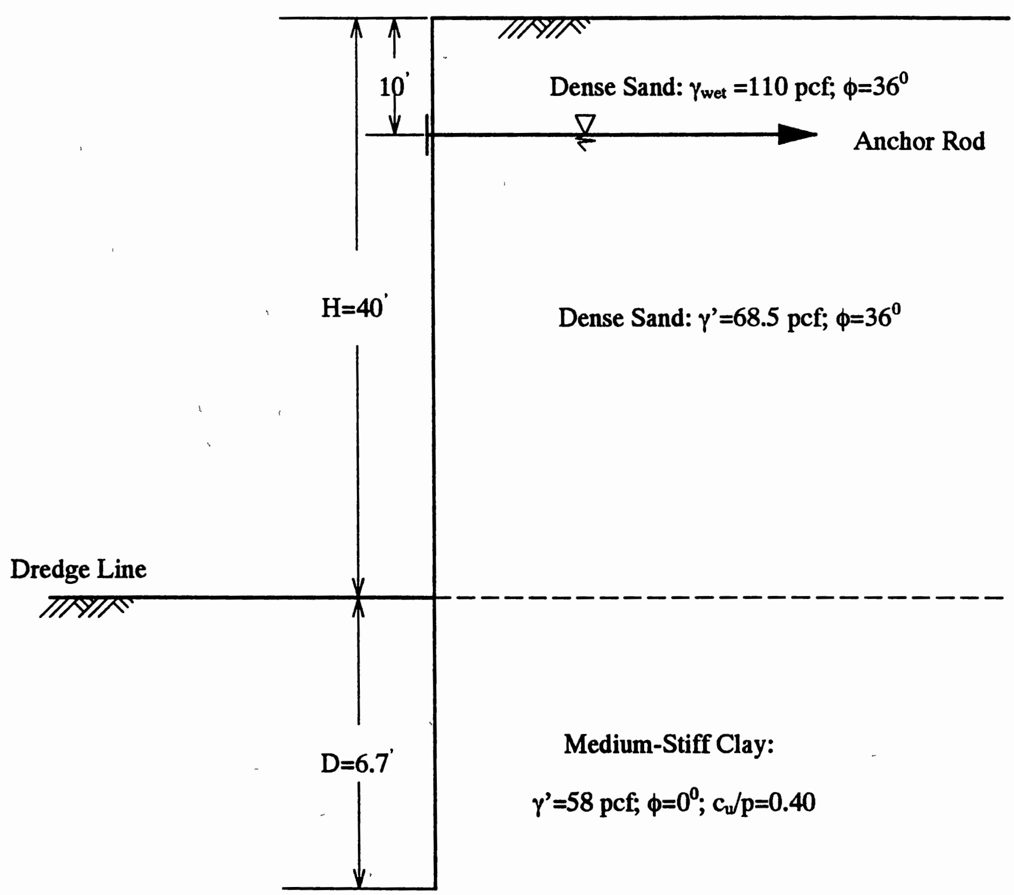

Figure 3.1 The DM40 case sample profile

TABLE 3.1

THE SHEET PILE WALL CHARACTERISTICS

Case	Penetration Depth (ft)	Excavation Height (ft)	The Length of Pile (ft)	Pile Section
LL40	13.0	40	53.0	PZ35
DL40	13.5	40	53.5	PZ35
LD40	8.6	40	48.6	PZ40
DD40	8.5	40	48.5	PZ35
SL40	15.0	40	55.0	PZ40
SL30	11.0	30	41.0	PZ35
ML40	12.2	40	52.2	PZ22
SD40	10.0	40	50.0	PZ40
MD40	7.8	40	47.8	PLZ25
TL40	12.6	40	52.6	PZ27
NL40	13.2	40	53.2	PZ27
TD40	8.5	40	48.5	PZ38
ND40	8.7	40	48.7	PZ38
LM40	6.7	40	46.7	PZ32
DM40	5.7	40	45.7	PLZ25
SS30	23.6	30	53.6	PZ40
MS30	22.0	30	52.0	PZ27
SM30	8.0	30	38.0	PZ27
MM40	5.4	40	45.4	PZ27
LT40	21.0	40	61.0	PZ27
DT40	22.0	40	62.0	PLZ23
LN40	13.5	40	53.5	PLZ23
DN40	13.7	40	53.7	PLZ23
NT40	21.3	40	61.3	PZ27
TN40	13.0	40	53.0	PZ27
NN30	17.1	30	47.1	PLZ23
TT30	23.6	30	53.6	PZ40

TABLE 3.2

SOIL PROPERTIES

Soil	γ_{wet} (pcf)	γ' (pcf)	c_u/p	ϕ	K_0	ν_i	m	n
L	97	60.5	0	30	0.50	0.30	120	0.5
D	110	68.5	0	36	0.41	0.25	200	0.5
S	95	48	0.25	0	0.96	0.49	250	0
M	110	58	0.40	0	0.96	0.49	500	0
T	95	48	0	25	0.577	0.35	15	0.9
N	110	58	0	30	0.5	0.30	30	0.6

Note:
L = Loose sand;
D = Dense sand;
S = Soft clay;
M = Medium-stiff clay;
T = Soft clay under drained condition;
N = Medium-stiff clay under drained condition;
γ_{wet} = Wet soil density;
γ' = Effective soil density;
c_u/p = The ratio of undrained shear strength to effective vertical stress;
ϕ = Soil friction angle;
K_0 = The coefficient of lateral earth pressure at rest;
ν_i = Initial Poisson's ratio
m,n = Nonlinear parameters

Example

DM40 case is used as an example. The input GEN file is given below. The lines starting with a single quote mark are comments (that GENERECT ignores). The actual numeric data consists of only 20 lines. Compared with the finite element data file that

FEM program requires, this GEN file is very small (about 1%).

```
DM40 CASE (DENSE BACKFILL,MEDIUM-STIFF CLAY FOUNDATION) GRID12
'---nodes in x & y directions & number of layers to excavate
14,30,11
'---extra nodes in the middle (# beams + 1)
 15
'---xi (from left to right)
 0,2,5,10,20,35,55,80,110,160,225,300,380,480
'---yi (from bottom to top node)
-160,-130,-105,-80,-60,-40,-20,0,22,40,55,65,70,72,73,74.3,76,78,80
82,85,90,95,100,105,110,112.5,115,117.5,120
'---Beam
PZ25
'---Links
DEFAULT
 '---Soil types
3
'Layers Of Soil Type
4, 7, 18
'---friction - adhesion
.44523, 0
.44523, 0
0,4052
'---Soil props:  c,  phi,  gamma,   Ko,   nui, nuf,  m,  n
0.0, 36.0, 110.0, 0.4122, 0.3,0.49, 200,0.5
0.0, 36.0,  68.5, 0.4122, 0.3,0.49, 200,0.5
0.0, 0.0,  58.0, 0.96, 0.49,0.49, 500,0.0
'---Number of points (depths) in Cu table
2
'ElevCu, Cu   table
-160,6836
80,  1268
'---Node from the top where anchor should be placed & AnchorStiff
5, 200000
'---Boundary condition codes (side, corner, bottom)
1, 3, 3
```

The input data for FEMSSI program is the output of GENERECT. The example of input data (LM40) as shown in an Appendix B.

Discussion of Results

A total of 27 cases were analyzed by running the FEMSSI program using WES supercomputer (Oner, 1991). The results of two cases (TT30 and NN30 cases) can not be used since the penetration depth given by the free earth support method is too large due to the oversight that the design was done without considering the wall friction. For the remaining twenty five cases results are given in Table 3.3. These results were extracted from the FEMSSI output files which included extremely detailed information about stresses and deformations everywhere in the system. The extracted results given in Table 3.3 are the maximum bending moment in the pile, the anchor force, and the maximum lateral displacement of the wall.

TABLE 3.3

THE RESULTS EXTRACTED FROM THE FEMSSI OUTPUT

Case	The Length of Pile (ft) (H+D)	Maximum Moment (k-ft)	Anchor Force (k)	Displacement (ft)	Pile Section
LL40	53.0	68.11	11.45	0.3572	PZ35
DL40	53.5	52.58	9.708	0.3407	PZ35
LD40	48.6	64.21	9.324	0.1801	PZ40
DD40	48.5	46.72	6.627	0.1715	PZ35
SL40	55.0	92.79	23.26	0.3975	PZ40
SL30	41.0	53.45	13.59	0.2315	PZ35
ML40	52.2	18.90	23.08	0.4038	PZ22
SD40	50.0	76.99	19.57	0.2414	PZ40
MD40	47.8	18.49	16.85	0.2108	PLZ25
TL40	52.6	88.47	14.15	0.5704	PZ27
NL40	53.2	73.59	13.57	0.5051	PZ27
TD40	48.5	88.19	13.29	0.3382	PZ38
ND40	48.7	72.91	11.71	0.2926	PZ38
LM40	46.7	38.93	14.52	0.1952	PZ32
DM40	45.7	27.46	14.69	0.1674	PLZ25
SS30	53.6	75.68	21.03	0.3403	PZ40
MS30	52.0	23.44	25.26	0.3369	PZ27
SM30	38.0	30.11	13.35	0.1429	PZ27
MM40	45.4	13.49	22.15	0.1772	PZ27
LT40	61.0	110.4	24.12	3.083	PZ27
DT40	62.0	52.48	26.32	3.287	PLZ23
LN40	53.5	74.22	12.8	1.189	PLZ23
DN40	53.7	61.18	10.61	1.309	PLZ23
NT40	61.3	133.8	23.96	3.087	PZ27
TN40	53.0	105.4	14.6	1.179	PZ27

CHAPTER IV

DISCUSSION OF RESULTS AND COMPARISON

In this chapter, the pile bending moments, and the issue of moment reduction, as well as anchor forces and section selection are discussed.

Bending Moments

All twenty five cases with different soil types show that the maximum bending moment value given by the free-earth-support on the sheetpile is larger than that given by the finite element method. In other words, the ratio of the maximum bending moment given by FEM to that by FES is always less than one. The ratios range from 0.25 to 0.95. The average value of the ratio is 0.64 (see Table 4.1).

TABLE 4.1

COMPARISON OF THE MAXIMUM BENDING MOMENTS

Case	The Length of Pile (ft) (H+D)	Maximum Moment		The Ratio of M_{FEM}/M_{FES}	Pile Section
		FEM	FES		
LL40	53.0	68.11	105.70	0.64	PZ35
DL40	53.5	52.58	95.18	0.55	PZ35
LD40	48.6	64.21	85.95	0.75	PZ40
DD40	48.5	46.72	76.06	0.61	PZ35
SL40	55.0	92.79	166.84	0.56	PZ40
SL30	41.0	53.45	70.52	0.76	PZ35
ML40	52.2	18.90	80.87	0.23	PZ22
SD40	50.0	76.99	143.04	0.54	PZ40
MD40	47.8	18.49	65.47	0.28	PLZ25
TL40	52.6	88.47	109.98	0.80	PZ27
NL40	53.2	73.59	110.23	0.67	PZ27
TD40	48.5	88.19	93.01	0.95	PZ38
ND40	48.7	72.91	90.55	0.81	PZ38
LM40	46.7	38.93	73.98	0.53	PZ32
DM40	45.7	27.46	62.05	0.44	PLZ25
SS30	53.6	75.68	100.00	0.75	PZ40
MS30	52.0	23.44	52.92	0.44	PZ27
SM30	38.0	30.11	55.56	0.54	PZ27
MM40	45.4	13.49	54.14	0.25	PZ27
LT40	61.0	110.40	148.17	0.75	PZ27
DT40	62.0	52.48	145.05	0.36	PLZ23
LN40	53.5	74.22	107.92	0.69	PLZ23
DN40	53.7	61.18	98.09	0.62	PLZ23
NT40	61.3	133.8	153.73	0.87	PZ27
TN40	53.0	105.4	111.58	0.94	PZ27

Moment Reduction

It has long been recognized that the bending moments given by the free earth support method are too large. Rowe (1952, 1957) proposed methods to reduce the calculated moment depending on the soil type and pile flexibility. In the current practice of anchored sheet pile wall design, a reduction is allowed on the maximum bending moment value obtained by free-earth-support method according to Rowe's curves. Since this leads to economical designs it is desirable to verify and perhaps extend these curves for cases not covered by the original curves utilizing the results of the finite element analyses.

To compare the FEM results with Rowe's curves, the ratio of the sheet pile bending moment values given by FEM and FES, M_{FEM}/M_{FES}, are calculated and plotted against Rowe's flexibility number (log ρ). These are discussed in the following for different foundation soil types.

Loose Sand Foundation Soil

There are seven cases, LL40, DL40, SL40, SL30, ML40, TL40, and NL40, in this group. The ratios of M_{FEM}/M_{FES} to log ρ for these cases are given in Table 4.1.

TABLE 4.1

THE RATIO OF M_{FEM}/M_{FES} TO LOG ρ

FOR LOOSE SAND FOUNDATION SOIL

Case	Depth (ft)	Section	H (ft)	I (in^4/ft)	M_{FEM}/M_{FES}	Log ρ
LL40	13.03	PZ35	53	361.2	0.64	-3.12
DL40	13.23	PZ35	53.2	361.2	0.55	-3.12
SL40	14.2	PZ40	54.2	490.9	0.56	-3.22
SL30	10.65	PZ35	40.7	361.2	0.76	-3.58
ML40	12.18	PZ22	52.2	84.38	0.31	-2.52
TL40	12.57	PZ27	52.6	184.2	0.80	-2.84
NL40	13.18	PZ27	53.2	184.2	0.67	-2.82

For uniform loose sand (LL40, α = 0.75) case, the maximum moment reduction factors from both FEM analysis and Rowe's curves seems to have slight difference (see Fig.4.1). It shows that the design by using Rowe's curves is little more conservative comparing with the result from FEM analysis. It is because of wall friction effect. In Rowe's design the wall friction effect was not considered in the calculation of passive earth pressure (δ = 0), but the wall friction angle (δ =2/3 ϕ) was calculated in passive earth pressure in this research. It is recalled that the wall friction influences penetration depth (in Chapter II), and when a wall friction angle equals to 2/3 ϕ, the penetration depth reduces by about 38 % (in TT30 case). In LL40 case the results show that when the ratio of M_{FEM}/M_{FES} = 0.64 at log ρ = -3.12, the ratio α = 0.75 (see Table 2.3 and

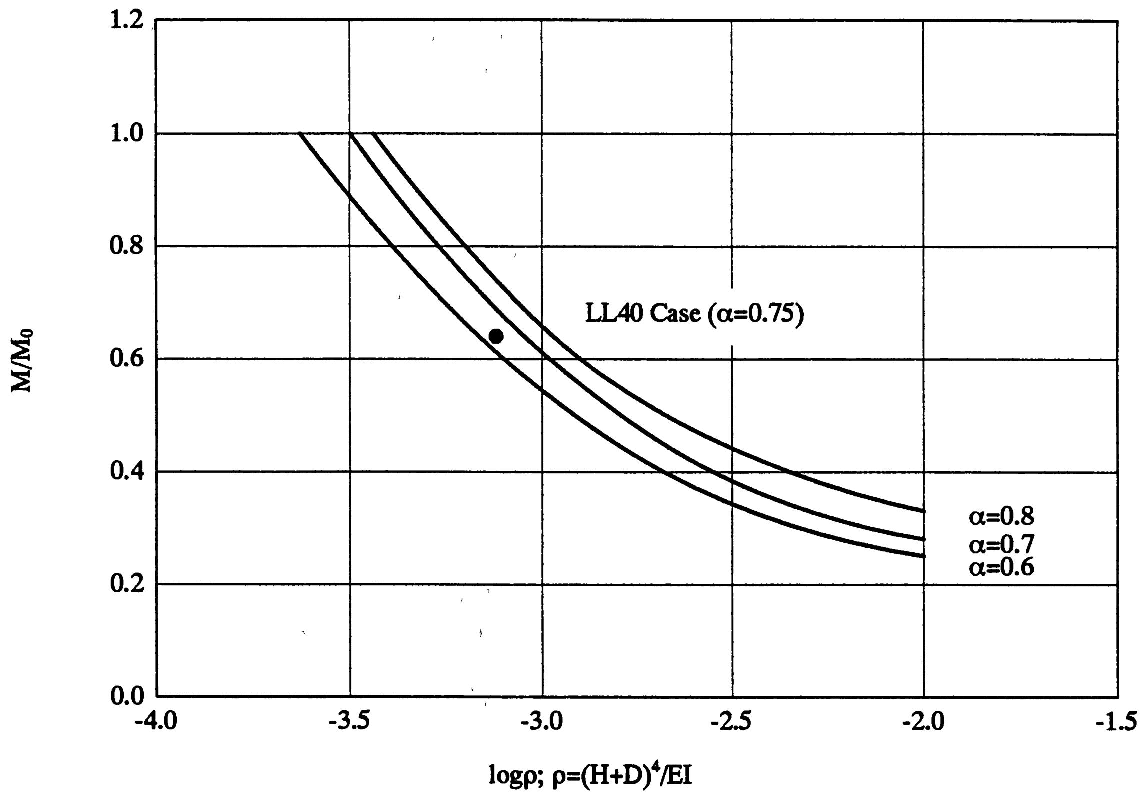

Figure 4.1 Rowe's moment-reduction curves for sheet pile in loose sand

4.1). In Rowe's curves, when log ρ = -3.12 and the moment reduction factor, M/M_0, equals 0.64, the α ratio is 0.65 (see Fig. 4.1). In other words, when the moment reduction factors and pile flexibility are same, the penetration depth from FEM analysis is less than that from Rowe's curves. When the penetration depths were calculated according to the α ratios (0.65 for δ = 0, and 0.75 for δ = 2/3 ϕ), the comparison shows that the penetration depth also reduces 38 %. So for uniform loose sand (LL40) case, the results given by Rowe's reduction curve obtained from a number of tests in uniform loose sand.

From Rowe's curves, the pattern of moment reduction curves is same when the α ratio ranges from 0.60 to 0.8. It will be recalled that the penetration depth given by FES is nearly the same if the foundation soil is the same, essentially independent of the soil behind the wall (see discussion in Chapter II). For loose sand foundation soil (γ_{wet}=95 pcf; γ_{sat}=123 pcf; ϕ=30°), the ratio of the excavation depth to the total length of pile, α, ranges from 0.74 to 0.77, the average value being 0.75. So the pattern of moment reduction curves in Rowe's curves can be used in the extending reduction curves. To be on the safe side, the smallest value, 0.74, should be chosen in design since smaller α values give larger penetration depth.

The moment reduction curves, M_{FEM}/M_{FES} against log ρ, for loose sand foundation soil cases are given in Fig. 4.2. Fig. 4.2 shows that different reduction curves were obtained for each different backfill soils with the same foundation soil (loose sand). Since the result from the finite element analysis agrees with Rowe's curve for the uniform loose sand (LL40) case, the moment reduction curve for loose sand backfill and

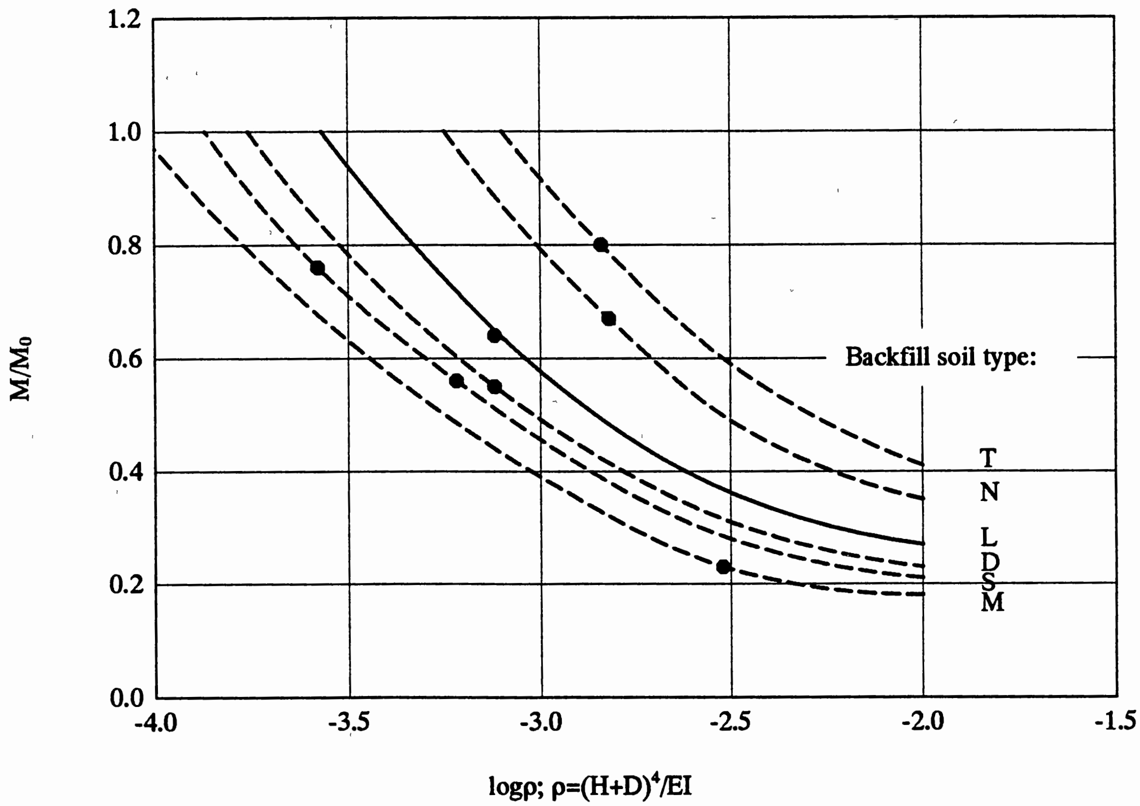

Figure 4.2 Moment-reduction curves for loose sand as foundation soil (α=0.74)

T: drained soft clay; N: drained medium-stiff clay; L: loose sand; D: dense sand; S: soft clay, undrained, M: medium-stiff clay, undrained loose sand foundation soil can be considered to be very reliable and used as a guide in extending the reduction curves (Fig. 4.2). These curves were drawn through the points representing each case with a different backfill soil considering the pattern of Rowe's curve for the loose sand case.

Dense Sand as Foundation Soil

In the category of dense sand foundation soil there are six cases (LD40, DD40, SD40, MD40, TD40, and ND40). The moment reduction curves (values of the ratio M_{FEM}/M_{FES} versus log ρ) for these cases are given in Table 4.2.

TABLE 4.2

THE RATIO OF M_{FEM}/M_{FES} TO Log ρ

FOR DENSE SAND FOUNDATION SOIL

Case	Depth (ft)	Section	H (ft)	I (in^4/ft)	M_{FEM}/M_{FES}	Log ρ
LD40	8.58	PZ40	48.6	490.9	0.75	-3.41
DD40	8.44	PZ35	48.4	361.2	0.61	-3.28
SD40	9.85	PZ40	49.9	490.9	0.54	-3.36
MD40	7.82	PLZ25	47.8	223.3	0.37	-3.09
TD40	8.5	PZ38	48.5	280.8	0.95	-3.17
ND40	8.68	PZ38	48.7	280.8	0.81	-3.16

For uniform dense sand case (DD40), the agreement observed with the Rowe's reduction curve is quite good (Fig. 4.3). Therefore the moment reduction curve for dense sand backfill and dense sand foundation soil is known, and this can be taken as a basis for extending Rowe's reduction curves. The possible extensions to reduction curves are shown in Fig. 4.4 for other backfill soils. These curves were determined with both the tendency of Rowe's reduction curve for dense sand backfill soil and each results from the FEM.

The ratio of the excavation depth to the total length of pile, α, is between 0.80 and 0.84 for this group, the average being 0.82 (see Discussion in Chapter II). To be on the safe side, the α ratio should be taken as 0.80 as a design basis.

Soft clay as foundation soil (undrained)

There are very few cases in this category (LS30, DS30, SS30, MS30). The ratio of the maximum FEM value to the maximum FES value of bending moment is given in Table 4.3. Since the distribution of the curves for different foundation soil seems having certain pattern, the moment reduction curve may be determined by both curve pattern and the FEM results. The moment reduction curves are shown in Fig.4.5.

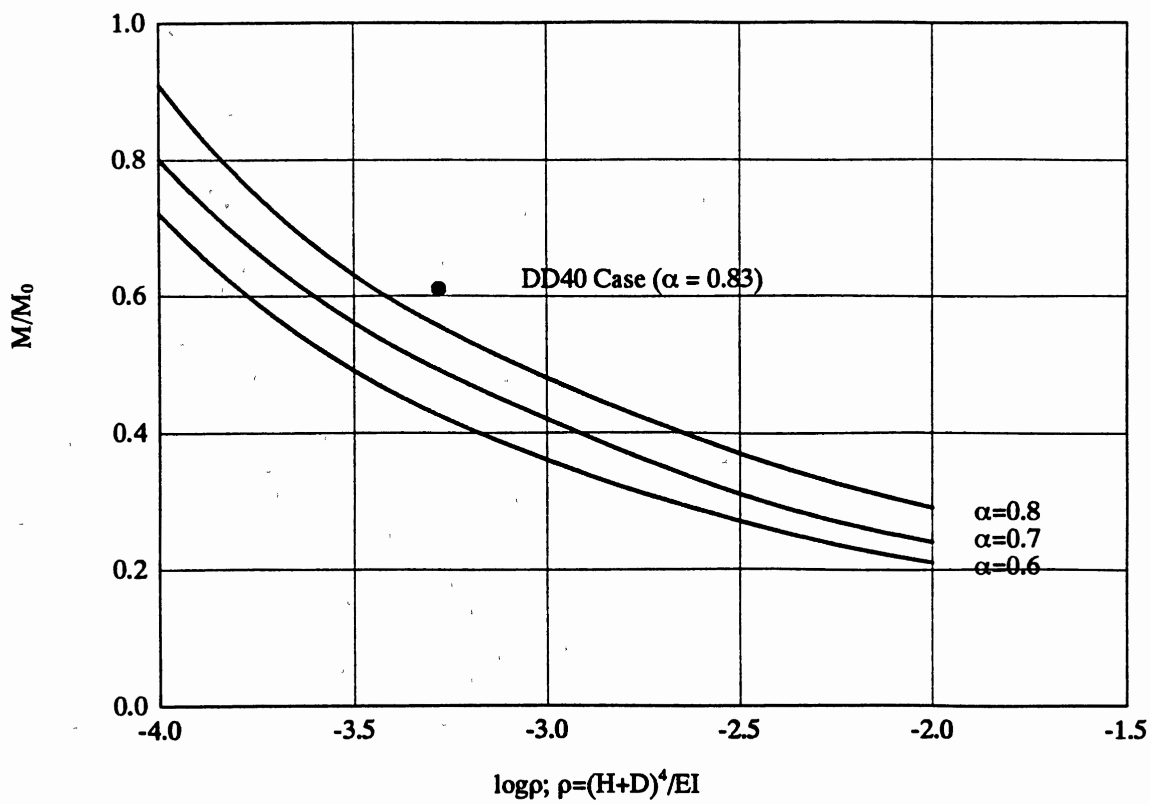

Figure 4.3 Rowe's moment-reduction curves for sheet pile in dense sand

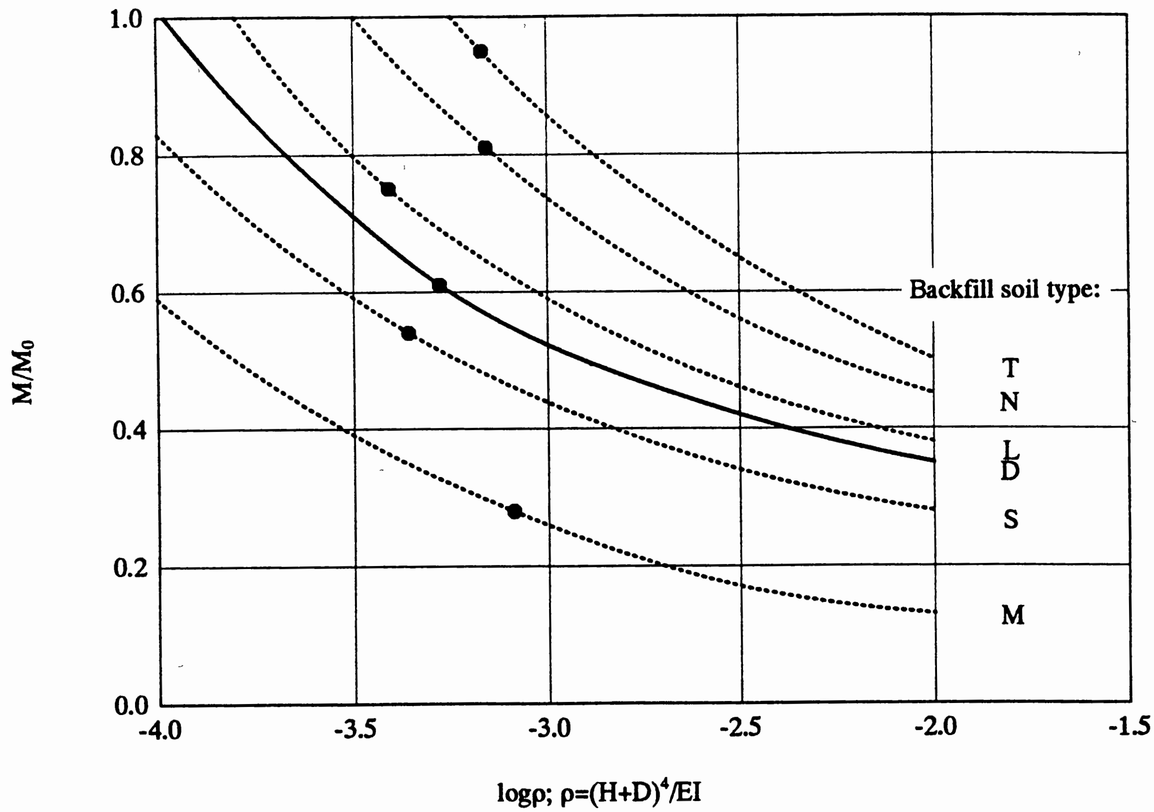

Figure 4.4 Moment-reduction curves for dense sand as foundation soil (α=0.80)

T: drained soft clay; N: drained medium-stiff clay; L: loose sand; D: dense sand;

S: soft clay, undrained, M: medium-stiff clay, undrained

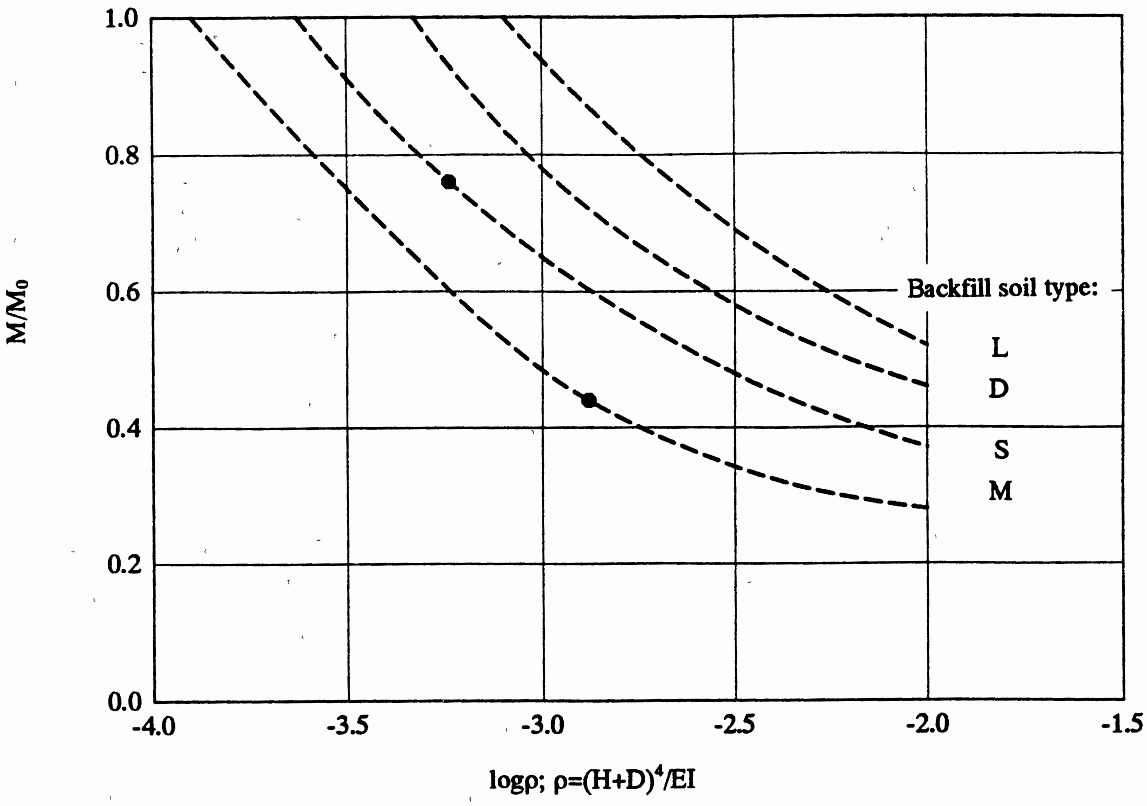

Figure 4.5 Moment-reduction curves for soft clay (undrained) as foundation soil
L: loose sand; D: dense sand; S: soft clay, undrained, M: medium-stiff clay, undrained

TABLE 4.3

THE RATIO OF M_{FEM}/M_{FES} TO Log ρ FOR SOFT CLAY

FOUNDATION SOIL (UNDRAINED)

Case	Depth (ft)	Section	H (ft)	I (in^4/ft)	M_{FEM}/M_{FES}	Log ρ
SS30	23.52	PZ40	53.5	490.9	0.76	-3.24
MS30	21.4	PZ27	51.4	184.2	0.65	-2.88

Soft Clay as foundation soil (drained)

Four cases are in this group (LT40, DT40, TT30, NT40). The results and the maximum moment ratios (M_{FEM}/M_{FES}) is given in Table 4.4. The moment reduction curves are also given in Fig.4.6. Since the displacement of the pile is too large (over 3 ft) to be accepted by engineer (see Table 4.13), the curves in Fig. 4.6 just shows the tendency of each curves.

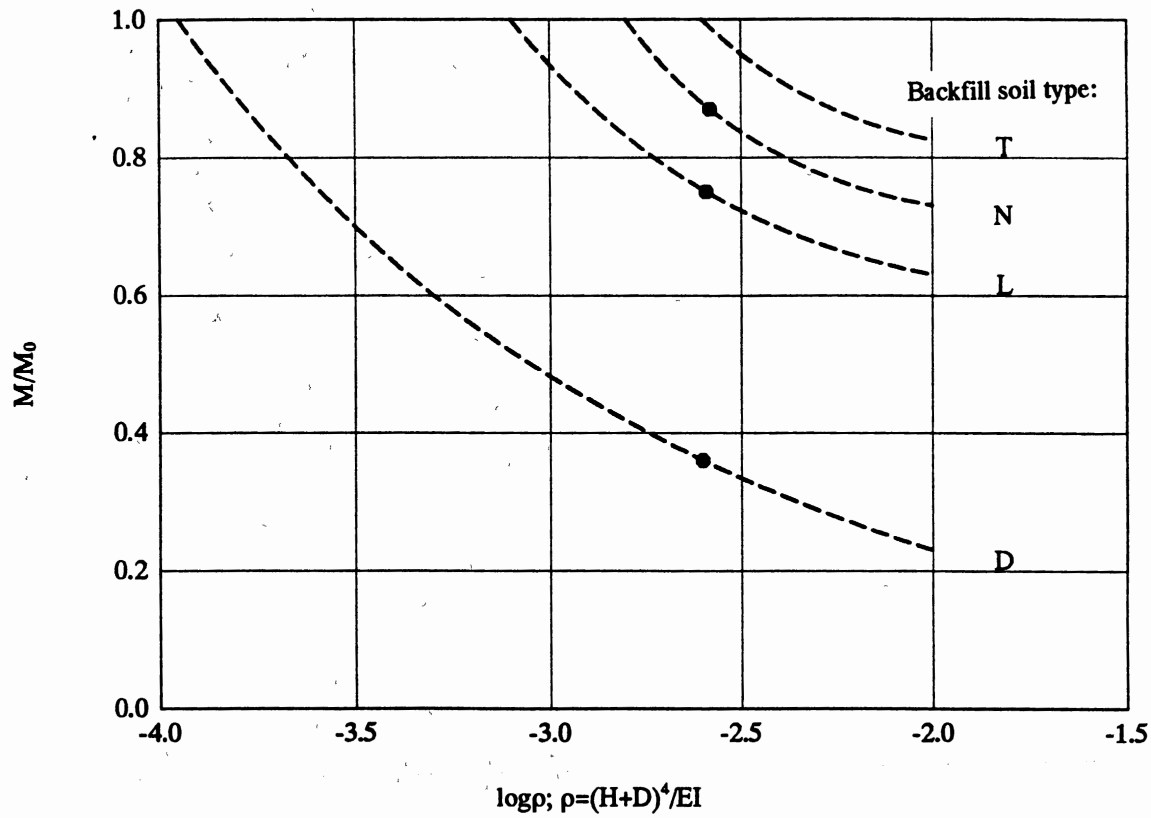

Figure 4.6 Moment-reduction curves for soft clay (drained) as foundation soil

T: drained soft clay; N: drained medium-stiff clay; L: loose sand; D: dense sand;

TABLE 4.4

THE RATIO OF M_{FEM}/M_{FES} TO Log ρ FOR SOFT CLAY

FOUNDATION SOIL (DRAINED)

Case	Depth (ft)	Section	H (ft)	I (in^4/ft)	M_{FEM}/M_{FES}	Log ρ
LT40	20.97	PZ27	61	184.2	0.75	-2.59
DT40	22.04	PLZ23	62	203.8	0.36	-2.60
NT40	21.25	PZ27	61.3	184.2	0.87	-2.58

Medium-Stiff clay as foundation soil (undrained)

Four cases (LM40, DM40, SM30, MM40) are in this design group. The maximum moment ratios (M_{FEM}/M_{FES}) is given in Table 4.5. The moment reduction curves are also given in Fig.4.7.

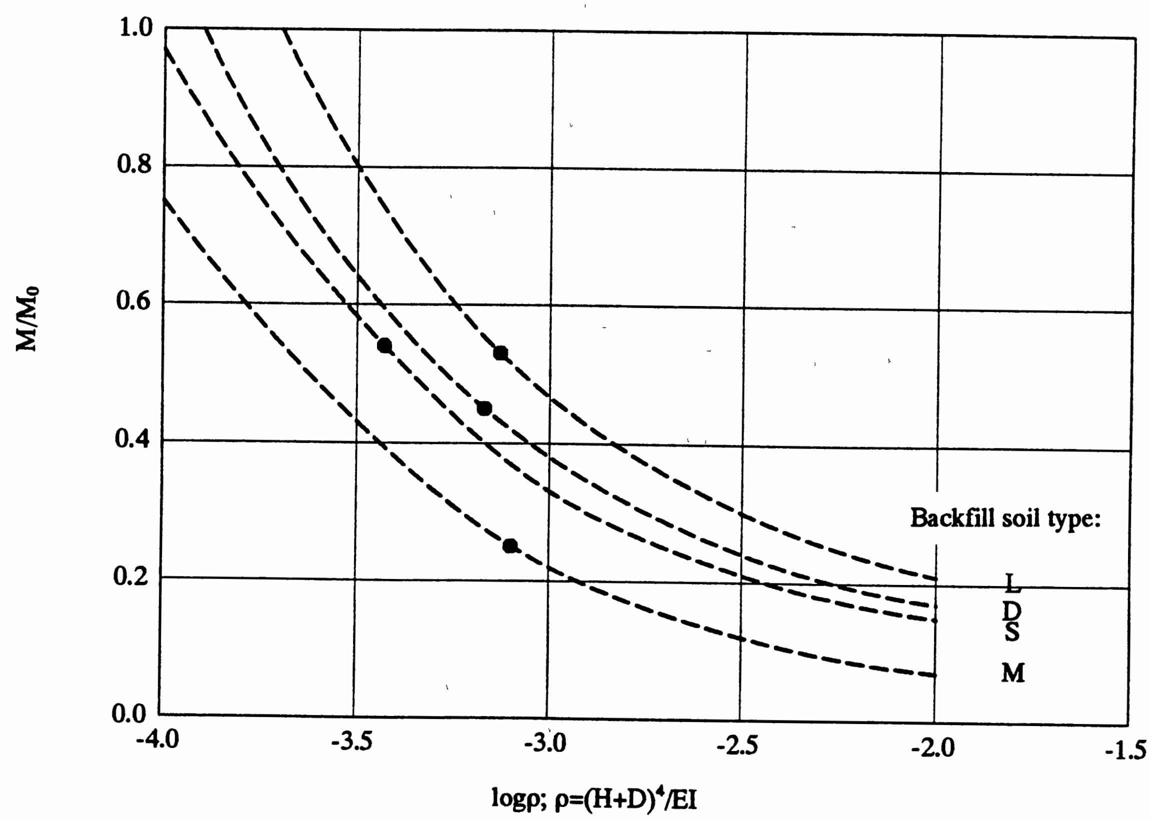

Figure 4.7 Moment-reduction curves for medium-stiff clay (undrained) foundation soil
L: loose sand; D: dense sand; S: soft clay, undrained, M: medium-stiff clay, undrained

TABLE 4.5

THE RATIO OF M_{FEM}/M_{FES} TO Log ρ FOR MEDIUM-STIFF CLAY FOUNDATION SOIL (UNDRAINED)

Case	Depth (ft)	Section	H (ft)	I (in^4/ft)	M_{FEM}/M_{FES}	Log ρ
LM40	6.66	PZ32	46.7	220.4	0.53	-3.13
DM40	5.67	PLZ25	45.7	223.3	0.45	-3.17
SM30	7.49	PZ27	37.5	184.2	0.54	-3.43
MM40	5.36	PZ27	45.4	184.2	0.25	-3.10

Medium- Stiff clay as foundation soil (drained)

There are three cases (LN40, DN40, and TN40) in this category. The maximum moment ratios (M_{FEM}/M_{FES}) is given in Table 4.6. The moment reduction curves are also given in Fig. 4.8.

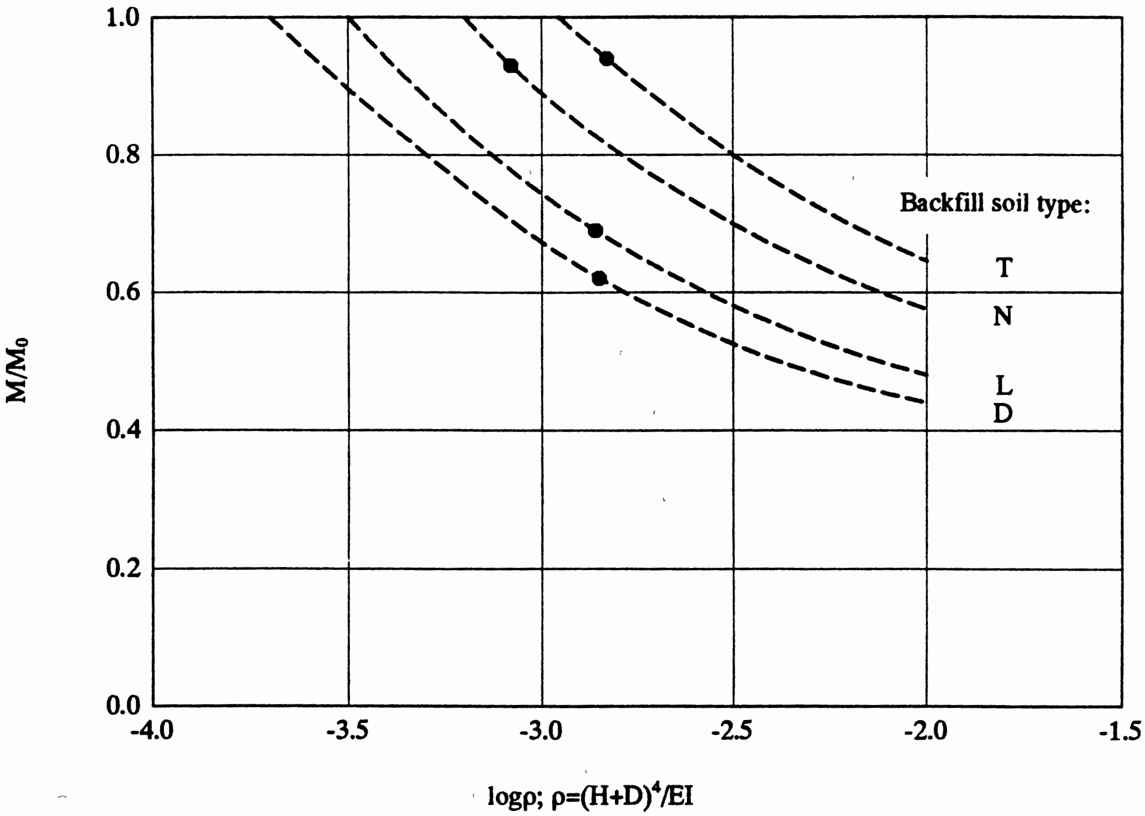

Figure 4.8 Moment-reduction curves for medium-stiff clay (drained) foundation soil

T: drained soft clay; N: drained medium-stiff clay; L: loose sand; D: dense sand;

TABLE 4.6

THE M_{FEM}/M_{FES} RATIOS TO Log ρ FOR MEDIUM-STIFF CLAY FOUNDATION SOIL (DRAINED)

Case	Depth (ft)	Section	H (ft)	I (in^4/ft)	M_{FEM}/M_{FES}	Log ρ
LN40	13.44	PLZ23	53.4	203.8	0.69	-2.86
DN40	13.66	PLZ23	53.7	203.8	0.62	-2.85
TN40	12.97	PZ27	53.0	184.2	0.94	-2.83

The medium-stiff clay (γ_{wet}=110 pcf; γ_{sat}=120 pcf; ϕ=30°; c_u/p=0.40) as foundation soils, the α ratios in anchored sheet pile wall design can be selected as 0.80 (undrained) and 0.75 (drained) for safe side consideration.

All new bending moment reduction curves have the same characteristics as the shear strength (ϕ and c_u) in backfill soil increase, the moment reduction ratios decrease. In other words, when the shear strengths in backfill soils are larger, (the resultant active earth pressures are smaller,) the bending moments given by FES are much larger than the values given by FEM.

Anchor Force (A_p)

The anchor force is one of the critical factors in designing an anchored sheet pile wall. It is gradually recognized that the anchor force given by the free earth support method may be smaller than the actual force acting on the tie rod. There have been various attempts to explain the difference. For example, Sowers and Sowers (1967), based on their experience with a large number of case histories, concluded that anchorage failure is a common cause of sheet pile wall failure.

In this section, the results given by both FES and FEM, and the factors influencing the anchor force will be discussed.

Results

In this study it was found that the anchor forces in most of cases given by FEM are larger than the results from the FES analysis. In two cases, LD40 and DD40, the anchor forces given by FEM are smaller than those given by FES analysis. The ratio of the value from FEM analysis to the one from FES in anchor force is between 0.78 and 4.95 (see Table 4.7). The FEM results show that the anchor force according to FES method is not safe in most cases.

TABLE 4.7

THE RATIOS ($A_{p\text{-}FEM}/A_{p\text{-}FES}$) OF ANCHOR FORCES

Case	Anchor Force (K)		Ratio of A_p-FEM/ A_p-FES
	FEM	FES	
LL40	11.45	10.69	1.07
DL40	9.71	9.50	1.02
LD40	9.32	9.75	0.96
DD40	6.63	8.52	0.78
SL40	23.26	15.93	1.46
SL30	13.59	8.96	1.52
ML40	23.08	7.15	3.23
SD40	19.57	14.73	1.33
MD40	16.85	6.36	2.65
TL40	14.15	11.79	1.20
NL40	13.57	11.45	1.19
TD40	13.29	10.93	1.22
ND40	11.71	10.47	1.12
LM40	14.52	9.08	1.60
DM40	14.69	7.77	1.89
SS30	21.03	10.78	1.95
MS30	25.26	5.10	4.95
SM30	13.35	7.94	1.68
MM40	22.15	5.74	3.86
LT40	24.12	12.61	1.91
DT40	26.32	11.63	2.26
LN40	12.80	10.81	1.18
DN40	10.61	9.61	1.10
NT40	23.96	13.4	1.79
TN40	14.60	11.81	1.24

Factors That Influence Anchor Force

From the data in Table 4.7, it is observed that the anchor force is the function of degree of fixity in the foundation soil, the actual earth pressures behind the wall and the passive earth pressures below the dredge line. The degree of fixity means that the degree of soil under dredge line holding sheet pile wall.

Influence of degree of fixity. The anchor force is inversely proportional to the degree of fixity. The higher the degree of fixity, the less is the displacement under the dredge line. A small amount of lateral displacement under the dredge line seems to cause a large increase in the anchor force.

Dense sand has a higher degree of fixity than loose sand does. With the same type of soil behind the wall the value of the anchor force for dense sand foundation soil case is less than the value for loose sand foundation soil (see Table 4.8).

TABLE 4.8

THE ANCHOR FORCE COMPARISON BETWEEN DENSE SAND AND LOOSE SAND FOUNDATION SOILS

Case	Anchor Force (k)		$A_{p\text{-}FEM}/A_{p\text{-}FES}$	Maximum displacement (ft)
	FEM	FES		
LD40	9.32	9.75	0.96	0.1801
DD40	6.63	8.55	0.78	0.1715
LL40	11.45	10.69	1.07	0.3572
DL40	9.71	9.50	1.02	0.3407

In LD40, and DD40 cases, the values of anchor forces given by FES are larger than the results from FEM analysis. The reason is that the strength of the sands above the tie-rod behind the wall is not completely used when the displacement is smaller. Generally speaking, in sand cases the results given by FES are nearly the same as the results from FEM analysis. For uniform dense sand the results are conservative. That may explain why many major failures of piling did not happen in sand cases according to FES design.

For soft clay as foundation soil, the degree of fixity is much lower that the anchor force values given by FEM are as much as about 2-5 times of the those given by FES (Table 4.9).

TABLE 4.9

THE $A_{p\text{-}FEM}/A_{p\text{-}FES}$ RATIOS IN SOFT CLAY FOUNDATION SOILS

Case	Anchor Force (k)		$A_{p\text{-}FEM}/A_{p\text{-}FES}$	Maximum displacement (ft)
	FEM	FES		
SS30	21.03	10.78	1.95	0.3403
MS30	25.26	5.10	4.95	0.3369
LT40	24.12	12.61	1.91	3.083
DT40	26.32	11.63	2.26	3.287
NT40	23.96	13.40	1.79	3.087

There are many failures of sheet pile walls in soft clay. This kind of failure of sheet pile walls were reported by Feld (1953), Sowers and Sowers (1967), LaGatta and Shields (1984), and Rieke, Crowser, and Schroeder (1988). Various explanations for the causes were also reported, but the explanation that the design anchor force was not large enough seems to be supported by these findings.

Active earth pressure influence on anchor force. The anchor force given by FEM is much larger than that of FES when the soil behind the wall is cohesive (see Table 4.10 and Table 4.11).

TABLE 4.10

THE $A_{p\text{-}FEM}/A_{p\text{-}FES}$ RATIOS IN THE SOFT CLAY BACKFILL SOIL

Case	Anchor Force (k)		$A_{p\text{-}FEM}/A_{p\text{-}FES}$	Maximum displacement (ft)
	FEM	FES		
SS30	21.03	10.78	1.95	0.3403
SL40	23.26	15.93	1.46	0.3975
SL30	13.59	8.96	1.52	0.2315
SD40	19.57	14.73	1.33	0.2414
SM30	13.35	7.94	1.68	0.1429

For these cases the anchor force from FEM analysis is 1.33 to 1.95 times the one given by FES (the average being 1.59).

TABLE 4.11

THE $A_{p\text{-}FEM}/A_{p\text{-}FES}$ RATIOS IN MEDIUM-STIFF CLAY BACKFILL SOIL

Case	Anchor Force (k)		$A_{p\text{-}FEM}/A_{p\text{-}FES}$	Maximum displacement (ft)
	FEM	FES		
MS30	25.26	5.10	4.95	0.3369
MM40	22.15	5.74	3.86	0.1772
ML40	23.08	7.15	3.23	0.1772
MD40	16.85	6.36	2.65	0.2108

The anchor force differences in the cases of medium-stiff clay behind the wall are much larger. The value of anchor force from FEM analysis is 2.7 to 5 times of the result given by FES.

It seems that the assumption of active earth pressure development in cohesive soils above the anchor level is not very accurate. It will be recalled that the effect of cohesion parameter is a reduction in active earth pressures in FES calculations.

Passive earth pressure influence on anchor force

Passive earth pressure has a significant influence on anchor force. In the free earth support method, the anchor force is inversely proportional to the passive earth pressure below the dredge line (see Fig. 4.9). When passive earth pressure assumed is larger than the actual passive earth pressure, the anchor force given by FES is smaller than the actual anchor force.

It will be recalled that Coulomb method over-estimates passive earth pressures for positive wall friction. In FES design, the passive earth pressure calculated by Coulomb method was reduced using a factor safety of 1.5 on shear strength; still the K_p values used are too large.

The results from both FES design and FEM analyses seem to show that the passive earth pressures assumed in FES calculation in cohesionless soils are correct. The anchor forces from both FEM and FES are nearly same. However, those in cohesive soils are different from actual passive earth pressures. In other words, the actual passive earth pressures in cohesive soils are less than assumed passive earth pressures, although the factor safety 1.5 was considered in FES calculation. In cohesive soil, the effect of

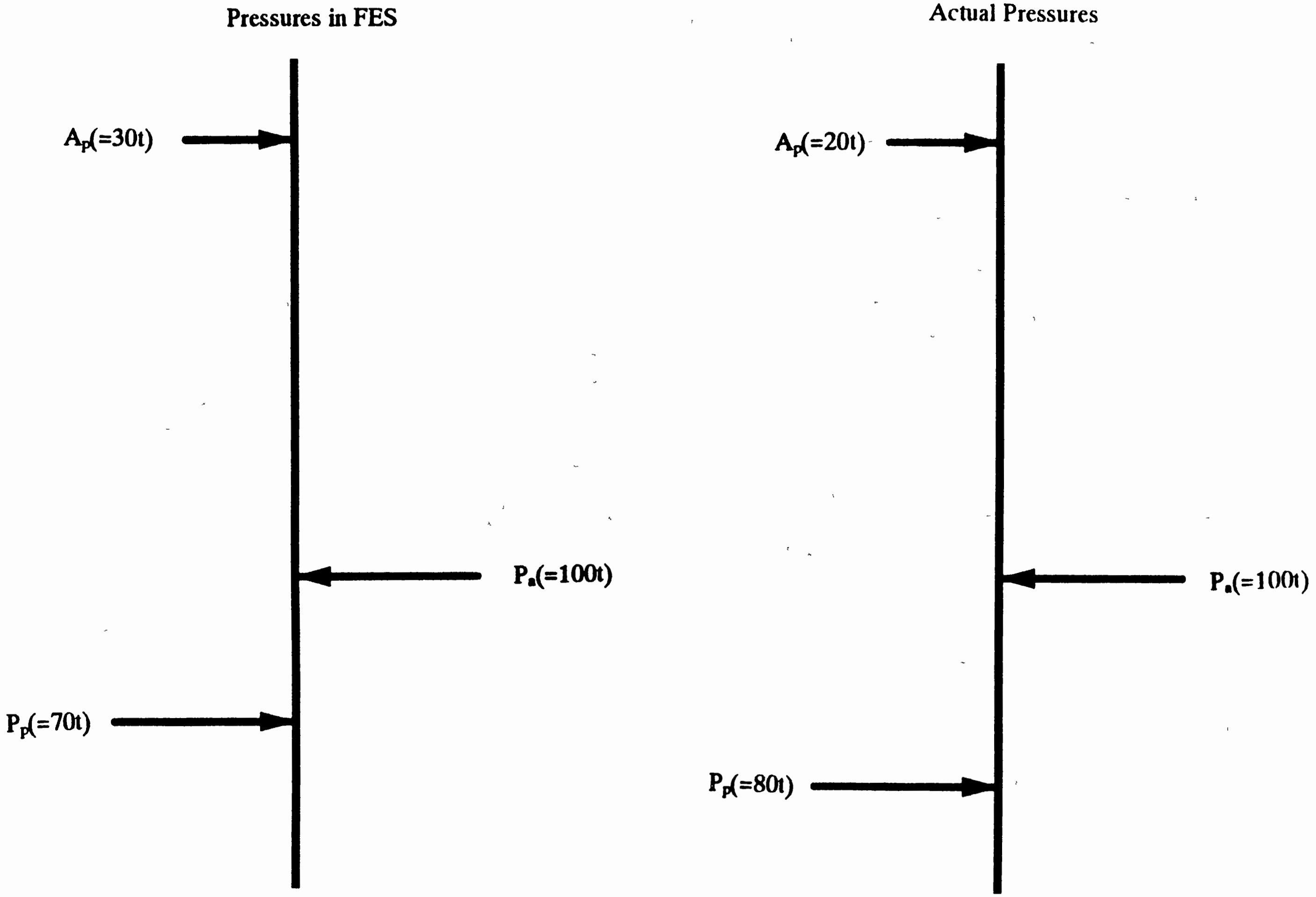

Figure 4.9 The Relationship between Passive earth pressure and Anchor Force on An Anchored Sheet Pile Wall

cohesion parameter is increased in passive earth pressure in FES calculation. The increase in passive earth pressure is significant. Therefore, the anchor forces given by FES were less than the actual anchor forces which were given by FEM (see Table 4.12).

TABLE 4.12

THE $A_{p\text{-}FEM}/A_{p\text{-}FES}$ IN COHESIVE FOUNDATION SOILS

Case	Anchor Force (k)		$A_{p\text{-}FEM}/A_{p\text{-}FES}$	Maximum Displacement (ft)
	FEM	FES		
LM40	14.52	9.08	1.60	0.1952
DM40	14.69	7.77	1.89	0.1674
SS30	21.03	10.78	1.95	0.3403
MS30	25.26	5.10	4.95	0.3369
SM30	13.35	7.94	1.68	0.1429
MM40	22.15	5.74	3.86	0.1772

Explanation for Pressure Concentration

There are two explanations for the observation that the actual anchor force is larger than the one given by FES, or in other words, pressures around the anchor point may be larger than active pressure. One is "arching" effect, which is a result of a complex soil-structure interaction where soil deformation plays the major role. Other is a local effect, which is the result of a passive pressure tendency above the anchor, considering that the wall rotates backwards and moves into the soil above the anchor.

Deflection on [illegible]
machine [illegible]
[illegible]
[illegible]

Both "arching" effect and local effect may occur in anchored sheet pile wall, but they may not produce a significant effect. Rowe (1952, 1955, 1957) conducted an extensive research with small models of anchored sheet piles in sand and clay. He found that some "arching" effect occurred when the anchor was not allowed to yield. A small anchor yield, less than H/1000, was sufficient to destroy the arching effect, and the active pressures became equal to Coulomb values with a wall friction angle of 2/3 phi. Moreover, the local effect can not always explain the pressure concentration. When the anchorage was near surface, the pressure concentration was below the anchor rod (Rowe, 1952).

However, one explanation, "passive earth pressure effect", has been found during this research. The "passive earth pressure effect" means that in the free earth support method, the anchor force is inversely proportional to the passive earth pressure below the dredge line. When passive earth pressure assumed is larger than actual passive earth pressure, the anchor force from FES is smaller than actual anchor force. The increase of anchor force equals to the difference between assumed and actual resultant passive earth pressures. This principle can be simply illustrated by the Fig. 4.9.

The "passive earth pressure effect" may be a major cause of pressure concentration around an anchor.

Whatever may be the reason, the result that the anchor forces (as found from FEM) are on the average, about 80% larger than conventional FES calculations indicate, is in agreement with field observations. Casagrande (1973) reviewed several cases where earth pressures were estimated from inclinometer measurements and recommended that "tie-rods and anchorage should be designed for not less than twice the forces used in conventional design."

Displacement

The displacement of the system is another important factor in designing a sheet pile wall. The results from FEM analyses show that the lateral wall displacement in all cases with both cohesionless and cohesion soils is reasonable, but the displacement in all cohesive soils under drained condition is extremely large, which should not be acceptable to a designer. Table 4.13 shows the cases where the lateral pile displacement was found to be greater than a few inches.

TABLE 4.13

LATERAL PILE DISPLACEMENT

Case	Section	The length of pile (ft)	Maximum Displacement (ft)	Log ρ
LT40	PZ27	61.0	3.083	-2.59
DT40	PLZ23	62.0	3.287	-2.60
LN40	PLZ23	53.4	1.189	-2.85
DN40	PLZ23	53.7	1.309	-2.85
NT40	PZ27	61.3	3.087	-2.58
TN40	PZ27	53.0	1.179	-2.83

The displacements shown in Table 4.13 are between 1.2 and 3.3 feet. Such large displacements of piling should be taken as failure and can not be accepted as good design. The FES calculations do not give any indication that this may be the case.

If a sheet pile wall is a permanent structure, then the long-term (drained) conditions should be considered in design. When a cohesive soil is under drained condition, the cohesion of the soil is lost so that the strength of the soil decreases. Therefore, the passive pressure in front of piling may not be large enough to support the wall. As a result, large wall displacements develop.

Section Selection

Sheet pile sections are selected by (1) finding the maximum bending moment by the free earth support method, (2) reducing the maximum moment by using Rowe's reduction factor, and then (3) comparing that with the allowable moment of available pile sections.

It will be recalled that there are limitations in applying Rowe's reduction curves (discussed in Chapter II). In some cases, the bending moment given by FES can not be reduced due to those limitations, and the original moment is still used for section selection. Obviously, this leads to a conservative selection.

In other cases, the maximum bending moment given by FES reduced with Rowe's factor is still too large compared with the result from FEM analysis. Here are two typical examples which show that there is no section selection available according to FES design with Rowe's reduction factor, but a section of pile can still be selected with the FEM result. Some characteristics of available pile sections are given in Table 4.14 for use in the following discussion.

TABLE 4.14

ALLOWABLE MOMENTS FOR THE AVAILABLE PILE SECTIONS

(based on f_s = 24 ksi)

Section	Section Modulus in^3/ft	Moment of Inertia in^4/ft	Allowable Moment k-ft/ft
PZ22	18.05	84.38	36.1
PLZ23	30.20	203.8	60.4
PZ27	30.20	184.2	60.4
PLZ25	32.80	223.3	65.4
PZ35	48.44	361.2	96.9
PZ40	60.70	490.9	121.4

Example one (SL40 case): The maximum moment given by FES is equal to 166.84 k-ft/ft. Rowe's reduction factor is 0.86, therefore the reduced moment is equal to 143.48 k-ft/ft. The ratio of allowable moment to the reduced moment is less than one (0.85) if the section with the highest capacity, PZ40, is selected. So no section was available for SL40 case according to common design practice. Nevertheless PZ40 was selected for use in FEM analysis. The maximum moment from FEM analysis was 92.79 k-ft/ft. If it is assumed that FEM is the most reliable method available to engineers today, the value of 92.79 may be taken as the correct value of M_{max}. The ratio of allowable moment (PZ40) to the maximum moment is 1.31. So section PZ40 can safely be used in SL40 case.

Example two (SD40 case): The maximum moment given by FES is 143.04 k-ft/ft. The selected section was PZ40. The design moment is also 143.04 k-ft/ft since Rowe's reduction is not applicable ($\alpha > 0.8$). The ratio of the allowable moment (PZ40) to the reduced moment is 0.85 (less than one). So no section is available for SD40 case according to FES design program WALSHT. However, the maximum moment from FEM analysis was 76.99 k-ft/ft. So the ratio of allowable moment (PZ40) to the maximum moment is in fact 1.58. So section PZ40 can safely be used in SD40 case.

CHAPTER V

CONCLUSIONS

In this study 27 anchored sheet pile wall cases were designed by FES method, and each case was analyzed by FEM (FEMSSI program) for this research. The findings may be summarized as follows.

Penetration Depth

Penetration depth, as calculated by FES method, depends on the characteristics of the foundation soil. Comparing the penetration depths in various cases where the foundation soil is the same but the "backfill" soil is different, it is concluded that the soil behind the wall has little influence on the penetration depth.

Bending Moments

1. The bending moment values given by FES method are larger than those from FEM analysis. The ratio of the moment from FEM to the moment given by FES is between 0.25 and 0.94, the average being 0.64.

2. The results are in good agreement with Rowe's experimental reduction curves for uniform loose sand, and dense sand cases. However, Rowe's curves are not comprehensive. Not applying the Rowe's reduction factor in some cases (due to the

limits of Rowe's tests) leads to over-conservative results.

3. It is proposed that Rowe's reduction curves be extended. New reduction curves were suggested for composite soil profiles involving loose sand, dense sand, soft clay (drained, and undrained), and medium-stiff clay (drained, and undrained). These curves are given in Figures 4.2, 4.4, 4.5, 4.6, 4.7, and 4.8.

4. All new bending moment reduction curves have the same characteristics as the shear strength (ϕ and c_u) in backfill soil increase, the moment reduction ratios decrease. In other words, when the shear strengths in backfill soils are larger, (the resultant active earth pressures are smaller,) the bending moments given by FES are much larger than the values given by FEM.

Anchor force

1. The anchor forces in most cases studied are larger than those given by FES. The ratio of the anchor force from FEM to those given by FES is between 0.78 and 4.95, the average being 1.79. This is in agreement by the findings in the field (on actual anchored sheet pile walls) that FES method gives too low anchor forces (A_p).

2. For cohesionless profiles, the anchor forces given by FEM and FES are nearly the same. One exception is the uniform dense sand profile, for which the anchor force given by FES is on safe side.

3. For cohesive soil behind the wall, the anchor forces given by FES are much smaller than the results from FEM analyses. The anchor forces given by FEM average 1.57 (soft clay) and 3.68 (medium-stiff clay) times those given by FES.

4. Active earth pressure given by Coulomb method may be smaller than actual earth pressure in cohesive soils. It may reduce the anchor force in FES design.

5. Passive earth pressure given by Coulomb value with 1.5 safety factor may still larger than actual passive earth pressure in cohesive soils. It may reduce the anchor force in FES design.

6. The "passive earth pressure effect" may explain the cause of pressure concentration around the anchor. The "passive earth pressure effect" implies that the anchor force is inversely proportional to the passive earth pressure. Since the passive earth pressure assumed in conventional (FES) calculations is larger than the actual pressure, the design anchor force is smaller than the actual anchor force. The increase in anchor force is equal to the difference between the assumed and the actual passive earth pressures.

Deformations

In cases where a cohesive soil is the foundation soil, it was found that under the long term (drained) conditions, the wall displacement was found (by FEM) to be unacceptablely large (1.2 to 3.3 ft). This situation was not clear from the conventional FES design calculations.

BIBLIOGRAPHY

(1) Allen, R.L., J.M. Duncan, and R.T. Sancio, "An Engineering Manual for Sheet Pile Walls," Department of Civil Engineering, Virginia Tech., Nov. 1987

(2) Anderson, W.F., T.H. Hanna, and M.N. Abdel-Malek, "Overall Stability of Anchored Retaining Walls," Journal of Geotechnical Engineering, American Society of Civil Engineers, Vol.109, No. 11, Nov., 1983, pp. 1416-1433

(3) Baggett, J.K., and S. Buttling, "Design and In-situ Performance of a Sheet Pile Wall," Proceedings of the Ninth International Conference on Soil Mechanics and Foundation Engineering, 1972, Tokyo, Vol. 2, pp.3-7

(4) Baumann, P., "Analysis of Sheet-Pile Bulkheads," Transaction, ASCE, Vol. 100, 1935, pp. 707-740

(5) Boghrat, A., "Use of STABL Program in Tied-back Wall Design," Journal of Geotechnical Engineering, American Society of Civil Engineers, Vol. 115, No. 4, April, 1989, pp. 546-552

(6) Bransby, P.L., and G. W. E. Milligan, "Soil Deformation Near Cantilever Sheet Pile Walls," Geotechnique, Vol. 25, No. 2, June, 1975, pp. 175-195

(7) Broms,B., and H. Stille, "Failure of Anchored Sheet-Pile Walls," Journal of Geotechnical Engineering Division, American Society of Civil Engineers, Vol. 102, No. 3, March, 1976, pp.225-251

(8) Casagrande, L., " Comments on Conventional Design of Retaining Structures," Journal of Soil Mechanics and Foundations Division, American Society of Civil Engineers, Vol. 99, No. 2, Feb., 1973, pp. 181-198

(9) Clough, G.W., "Application of the Finite Element Method to Earth-Structure Interaction, State-of-the Art," Proc. Symposium, Application of the Finite Element Method in Geotechnical Engineering, Vicksburg, MS, 1972, Vol. 3, pp. 1057-1116

(10) Clough, G.W., and J.M. Duncan, "Finite Element Analyses of Retaining Wall Behavior," Journal of Soil Mechanics and Foundations Divisions, American Society of Civil Engineers, Vol. 97, No. 12, Dec., 1971, pp.1657-1673

(11) Dawkins, W.P., "On Soil-Structure Interaction Analysis of Sheet Pile Walls," USAE Corps of Engineers, Jan., 1978

(12) Dawkins, W.P., "Computer Program CBEAMC. Analysis of Beam-Columns with Nonlinear Supports," USAE Waterways Experiment Station, Vicksburg, MS, June, 1980

(13) Dawkins, W.P., "Computer Program WALSHT: A User's Guide," USAE Waterways Experiment Station, Vicksburg, MS, 1985

(14) Hanna, T.H., and I.I. Kurdi, "Studies on Anchored Flexible Retaining Walls in Sand," Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol. 100, No. 10, Oct., 1974, pp.1107-1122

(15) Ichihara, M., H. Matsuzawa, M. Kawamura, and T. Takeuchi, "Influence of Seepage Flow on Earth Pressure Against Retaining Wall," Soils and Foundations, Vol. 22, No. 2, June, 1982, pp. 15-28

(16) Kollbrunner, C.F., "Calculation of Sheet Pile Walls in Cohesive Soil," Proc., 2nd. International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, 1948, Vol. 1, pp.108-112

(17) Kovacs, W.D., J.H. Martin, and F.A. Gerig, "Sheet Pile Design with Variable Soil Parameters," Journal of Soil Mechanics and Foundations Division, American Society of Civil Engineers, Vol. 100, No. 2, Feb., 1974, pp. 200-205

(18) Masih, R., "Graphical Solution for Sheet Pile Embedment," Journal of Geotechnical Engineering, American Society of Civil Engineers, Vol. 110, No. 4, April, 1984, pp. 534-538

(19) Matsuo, M., S. Kenmochi, and H. Yagi, "Experimental Study on Earth Pressure of Retaining Wall by Fied Tests," Soils and Foundations, Vol. 18, No. 3, Sept., 1978, pp. 27-41

(20) Miigan, G.W.E., "Soil Deformations near Anchored Sheet Pile Walls," Geotechnique, Vol. 33, No. 1, Mar., 1983, pp.41-55

(21) Oner, M. "Computer Program FEMSSI. Finite Element Analysis Method for Soil-Structure Interaction", Oklahoma State University, Stillwater, Aug., 1989.

(22) Oner, M. "Theoretical Manual for Sheet Pile Walls," (Initial, Partial Draft), Oklahoma State University, Stillwater, Aug., 1989

(23) Richart, F.E., "Analysis for Sheet-Pile Retaining Wall," Transactions,

American Society of Civil Engineers, Vol. 122, 1957, pp. 113-1132

(24) Rowe, P.W., "Sheet Pile Walls Subject to Line Resistance Above the Anchorage," Proceedings, Institution of Civil Engineers, London, Vol. 7, No. 6233, Aug., 1957, pp. 879-896

(25) Rowe, P.W., "A Theoretical and Experimental Analysis of Sheet Pile Walls," Proceedings, Institution of Civil Engineers, London, Vol. 4, Part I, Jan., 1955, pp.32-69

(26) Rowe, P.W., "Sheet Pile Walls at Failure," Proceedings, Institution of Civil Engineers, London, Vol. 5, No. 3, 1956, pp. 276-315

(27) Rowe, P.W., "The Flexibility Characteristics of Sheet Pile Walls," Structural Engineer, Vol. 34, No. 5, May, 1955, pp. 150-158

(28) Rowe, P.W., "Sheet Pile Walls Encastre at the Anchorage," Proceedings, Institution of Civil Engineers, London, Part 1, Vol. 4, No. 1, Jan., 1955, pp.70-87

(29) Rowe, P.W., "Anchored Sheet Pile Walls," Proceedings, Institution of Civil Engineers, London, Part 1, Vol. 4, No. 5788, 1952, pp. 27-70

(30) Sowers, B.G., and G.F. Sowers, "Failures of Bulkhead and Excavation Bracing," Civil Engineering, American Society of Civil Engineers, Vol. 37, No. 1, Jan., 1967, pp. 72-77

(31) Terzaghi, K., "Large Retaining Wall Tests (A series of five papers reporting fundamental results)," Engineering News Record, Vol. 112, Nos. 5-23, Feb.-March, 1934

(32) Tschebotarioff, G.P., "Large Scale Earth Pressure Tests with Model Flexible Bulkheads," Report to Bureau of Yards and Docks, U.S. Navy, Princeton University, 1949

(33) Tschebotarioff, G.P., "Soil Mechanics, Foundations, and Earth Structures," McGraw-Hill, Inc., New York, 1951

(34) Turabi, D.A., and A. Balla, "Distribution of Earth Pressure on Sheet-pile Walls," Journal of the Soil Mechanics and Foundation Division, Proc. American Society of Civil Engineers, Vol. 94, No. 6, Nov., 1968, pp. 1271-1301

(35) United States Steel, "Steel Sheet Piling Design Manual," USS, Pittsburg, PA, Feb. 1974

APPENDIXES

APPENDIX A

SAMPLE OUTPUT FROM WALSHT PROGRAM

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS BY CLASSICAL METHODS
DATE: 91/07/04 TIME: 6.30.20

INPUT DATA

I.--HEADING:
MM40 CASE
II.--CONTROL
ANCHORED WALL DESIGN

LEVEL 1 FACTOR OF SAFETY FOR ACTIVE PRESSURES = 1.00
LEVEL 1 FACTOR OF SAFETY FOR PASSIVE PRESSURES = 1.50

III.--WALL DATA
ELEVATION AT TOP OF WALL = 40.00 (FT)
ELEVATION AT ANCHOR = 30.00 (FT)

IV.--SURFACE POINT DATA

IV.A--RIGHTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	40.00

IV.B-- LEFTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	.00

V.--SOIL LAYER DATA

V.A.--RIGHTSIDE LAYER DATA
LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURES = DEFAULT
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURES = DEFAULT

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH-ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH-ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
110.00	110.00	.00	440.0	.00	.0	30.00	.00	DEF	DEF
120.50	120.50	.00	498.0	.00	.0	25.00	.00	DEF	DEF
120.50	120.50	.00	614.0	.00	.0	20.00	.00	DEF	DEF
120.50	120.50	.00	730.0	.00	.0	15.00	.00	DEF	DEF
120.50	120.50	.00	846.0	.00	.0	10.00	.00	DEF	DEF
120.50	120.50	.00	962.0	.00	.0	5.00	.00	DEF	DEF
120.50	120.50	.00	1078.0	.00	.0	.00	.00	DEF	DEF
120.50	120.50	.00	1194.0	.00	.0	-5.00	.00	DEF	DEF
120.50	120.50	.00	1310.0	.00	.0	-10.00	.00	DEF	DEF
120.50	120.50	.00	1426.0	.00	.0	-15.00	.00	DEF	DEF
120.50	120.50	.00	1542.0	.00	.0	-20.00	.00	DEF	DEF
120.50	120.50	.00	1658.0	.00	.0	-25.00	.00	DEF	DEF
120.50	120.50	.00	1774.0	.00	.0			DEF	DEF

V.B.-- LEFTSIDE LAYER DATA
LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURES = DEFAULT
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURES = DEFAULT

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH-ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH-ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
120.50	120.50	.00	1194.0	.00	.0	-5.00	.00	DEF	DEF

120.50	120.50	.00	1310.0	.00	.0	-10.00	.00	DEF	DEF
120.50	120.50	.00	1426.0	.00	.0	-15.00	.00	DEF	DEF
120.50	120.50	.00	1542.0	.00	.0	-20.00	.00	DEF	DEF
120.50	120.50	.00	1658.0	.00	.0	-25.00	.00	DEF	DEF
120.50	120.50	.00	1774.0	.00	.0			DEF	DEF

VI.--WATER DATA

UNIT WEIGHT = 62.50 (PCF)
RIGHTSIDE ELEVATION = 30.00 (FT)
LEFTSIDE ELEVATION = 30.00 (FT)
NO SEEPAGE

VII.--SURFACE LOADS
NONE

VIII.--HORIZONTAL LOADS
NONE

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS BY CLASSICAL METHODS
DATE: 91/07/04 TIME: 6.31.00

SUMMARY OF RESULTS FOR
ANCHORED WALL DESIGN

I.--HEADING

MM40 CASE

II.--SUMMARY

RIGHTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS AND THEORY OF ELASTICITY EQUATIONS FOR SURCHARGE LOADS.

LEFTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS AND THEORY OF ELASTICITY EQUATIONS FOR SURCHARGE LOADS.

METHOD :	FREE EARTH	EQUIV. BEAM	FIXED EARTH
WALL BOTTOM ELEV. (FT) :	-5.36	-10.92	-12.44
PENETRATION (FT) :	5.36	10.92	12.44
MAX. BEND. MOMENT (LB-FT) :	-54142.	-44447.	-39051.
AT ELEVATION (FT) :	12.00	14.00	15.00
MAX. SCALED DEFL. (LB-IN3):	1.1682E+10	-7.4067E+09	7.7103E+09
AT ELEVATION (FT) :	13.00	40.00	14.00
ANCHOR FORCE (LB) :	5744.	5168.	4822.

(NOTE: PENETRATION FOR EQUIVALENT BEAM METHOD DOES NOT INCLUDE INCREASE PRESCRIBED BY DRAFT EM 1110-2-2906.)

(NOTE: DIVIDE SCALED DEFLECTION BY MODULUS OF ELASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN**4 TO OBTAIN DEFLECTION IN INCHES.)

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS BY CLASSICAL METHODS
DATE: 91/07/04 TIME: 6.31.00

COMPLETE RESULTS FOR
ANCHORED WALL DESIGN
BY FREE EARTH METHOD

I.--HEADING
MM40 CASE

II.--RESULTS (ANCHOR FORCE = 5744. (LB))

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	SCALED DEFLECTION (LB-IN3)	NET PRESSURE (PSF)
40.00	0.	0.	-1.0609E+10	.00
39.00	0.	0.	-9.5483E+09	.00
38.00	0.	0.	-8.4874E+09	.00
37.00	0.	0.	-7.4265E+09	.00
36.00	0.	0.	-6.3656E+09	.00
35.00	0.	0.	-5.3046E+09	.00
34.00	0.	0.	-4.2437E+09	.00
33.00	0.	0.	-3.1828E+09	.00
32.00	0.	0.	-2.1219E+09	.00
31.00	18.	55.	-1.0610E+09	110.00
30.00	147.	220.	0.0000E+00	220.00
30.00	147.	-5524.	0.0000E+00	104.00
29.00	-5316.	-5391.	1.0596E+09	162.00
28.00	-10617.	-5200.	2.1100E+09	220.00
27.00	-15698.	-4951.	3.1421E+09	278.00
26.00	-20501.	-4644.	4.1471E+09	336.00
25.00	-24967.	-4279.	5.1168E+09	394.00
25.00	-24967.	-4279.	5.1168E+09	162.00
24.00	-29156.	-4088.	6.0433E+09	220.00
23.00	-33125.	-3839.	6.9195E+09	278.00
22.00	-36816.	-3532.	7.7385E+09	336.00
21.00	-40170.	-3167.	8.4940E+09	394.00
20.00	-43131.	-2744.	9.1800E+09	452.00
20.00	-43131.	-2744.	9.1800E+09	220.00
19.00	-45756.	-2495.	9.7916E+09	278.00
18.00	-48103.	-2188.	1.0324E+10	336.00
17.00	-50114.	-1823.	1.0774E+10	394.00
16.00	-51730.	-1400.	1.1137E+10	452.00
15.00	-52895.	-919.	1.1410E+10	510.00
15.00	-52895.	-919.	1.1410E+10	278.00
14.00	-53666.	-612.	1.1592E+10	336.00
13.00	-54101.	-247.	1.1682E+10	394.00
12.00	-54142.	176.	1.1678E+10	452.00
11.00	-53730.	657.	1.1581E+10	510.00
10.00	-52809.	1196.	1.1391E+10	568.00
10.00	-52809.	1196.	1.1391E+10	336.00
9.00	-51436.	1561.	1.1110E+10	394.00
8.00	-49669.	1984.	1.0739E+10	452.00
7.00	-47450.	2465.	1.0284E+10	510.00
6.00	-44720.	3004.	9.7457E+09	568.00
5.00	-41423.	3601.	9.1307E+09	626.00
5.00	-41423.	3601.	9.1307E+09	394.00
4.00	-37616.	4024.	8.4442E+09	452.00
3.00	-33357.	4505.	7.6928E+09	510.00
2.00	-28588.	5044.	6.8838E+09	568.00
1.00	-23250.	5641.	6.0254E+09	626.00
.00	-17287.	6296.	5.1270E+09	684.00
.00	-17287.	6296.	5.1270E+09	-1140.00
-1.00	-11562.	5156.	4.1987E+09	-1140.00
-2.00	-6976.	4016.	3.2503E+09	-1140.00
-3.00	-3531.	2876.	2.2896E+09	-1140.00
-4.00	-1225.	1736.	1.3226E+09	-1140.00

-5.00	-92.	499.	3.5341E+08	-1333.33
-5.36	0.	0.	0.0000E+00	-1403.80

(NOTE: DIVIDE SCALED DEFLECTION BY MODULUS OF ELASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN**4 TO OBTAIN DEFLECTION IN INCHES.)

III.--SOIL PRESSURES

ELEVATION (FT)	< LEFTSIDE PRESSURE (PSF)> PASSIVE	ACTIVE	<RIGHTSIDE PRESSURE (PSF)> ACTIVE	PASSIVE
40.00	0.	0.	0.	587.
39.00	0.	0.	0.	697.
38.00	0.	0.	0.	807.
37.00	0.	0.	0.	917.
36.00	0.	0.	0.	1027.
35.00	0.	0.	0.	1137.
34.00	0.	0.	0.	1247.
33.00	0.	0.	0.	1357.
32.00	0.	0.	0.	1467.
31.00	0.	0.	110.	1577.
30.00+	0.	0.	220.	1687.
30.00-	0.	0.	104.	1764.
29.00	0.	0.	162.	1822.
28.00	0.	0.	220.	1880.
27.00	0.	0.	278.	1938.
26.00	0.	0.	336.	1996.
25.00+	0.	0.	394.	2054.
25.00-	0.	0.	162.	2209.
24.00	0.	0.	220.	2267.
23.00	0.	0.	278.	2325.
22.00	0.	0.	336.	2383.
21.00	0.	0.	394.	2441.
20.00+	0.	0.	452.	2499.
20.00-	0.	0.	220.	2653.
19.00	0.	0.	278.	2711.
18.00	0.	0.	336.	2769.
17.00	0.	0.	394.	2827.
16.00	0.	0.	452.	2885.
15.00+	0.	0.	510.	2943.
15.00-	0.	0.	278.	3098.
14.00	0.	0.	336.	3156.
13.00	0.	0.	394.	3214.
12.00	0.	0.	452.	3272.
11.00	0.	0.	510.	3330.
10.00+	0.	0.	568.	3388.
10.00-	0.	0.	336.	3543.
9.00	0.	0.	394.	3601.
8.00	0.	0.	452.	3659.
7.00	0.	0.	510.	3717.
6.00	0.	0.	568.	3775.
5.00+	0.	0.	626.	3833.
5.00-	0.	0.	394.	3987.
4.00	0.	0.	452.	4045.
3.00	0.	0.	510.	4103.
2.00	0.	0.	568.	4161.
1.00	0.	0.	626.	4219.
.00+	0.	0.	684.	4277.
.00-	1592.	0.	452.	4432.
-1.00	1650.	0.	510.	4490.
-2.00	1708.	0.	568.	4548.
-3.00	1766.	0.	626.	4606.
-4.00	1824.	0.	684.	4664.
-5.00+	1882.	0.	742.	4722.
-5.00-	2037.	0.	510.	4877.
-6.00	2095.	0.	568.	4935.

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS BY CLASSICAL METHODS
DATE: 91/07/04 TIME: 6.31.00

PRELIMINARY DESIGN DATA FOR
FREE EARTH DESIGN IN CLAY

I.--HEADING
MM40 CASE
II.--DESIGN PARAMETERS

WALL HEIGHT RATIO (ALPHA) = .88
ANCHOR HEIGHT RATIO (BETA) = .22
STABILITY NUMBER = .28

SHEET PILE DATA:

SHEET PILE NAME	<SECTION PROPERTIES> (PER FOOT OF WALL) SECTION MODULUS (IN**3)	MOMENT OF INERTIA (IN**4)	ALLOWABLE STRESS (PSI)	MODULUS OF ELASTICITY (PSI)
PZ40	60.70	490.80	24000.	2.90E+07
PZ38	46.80	380.80	24000.	2.90E+07
PZ35	48.50	361.20	24000.	2.90E+07
PZ32	38.30	220.40	24000.	2.90E+07
PZ27	30.20	184.20	24000.	2.90E+07
PZ22	18.10	84.40	24000.	2.90E+07
PLZ25	32.80	223.25	24000.	2.90E+07
PLZ23	30.20	203.75	24000.	2.90E+07

III.--PRELIMINARY DESIGN DATA

SHEET PILE	LOG(H**4/EI)	ROWE'S MOMENT REDUCTION COEF.	RATIO OF ALLOWABLE MOMENT TO FREE EARTH MOMENT
PZ40	-3.53	1.0 (***)	2.24
PZ38	-3.42	1.0 (***)	1.73
PZ35	-3.39	1.0 (***)	1.79
PZ32	-3.18	1.0 (***)	1.41
PZ27	-3.10	1.0 (***)	1.12
PZ22	-2.76	1.0 (***)	.67
PLZ25	-3.18	1.0 (***)	1.21
PLZ23	-3.14	1.0 (***)	1.12

*** REDUCTION NOT APPLICABLE DUE TO ALPHA GREATER THAN 0.8.

*** REDUCTION NOT APPLICABLE DUE TO STABILITY NUMBER LESS THAN 0.5.

APPENDIX B

THE INPUT DATA FOR FEMSSI PROGRAM (ABBREVIATED)

(THE OUTPUT OF GENERECT)

```
DM40L CASE (DENSE SAND BACKFILL ON MEDIUM-STIFF CLAY FOUNDATION) LARGE GRID12
GENERAL
840,798,12,2000
COORDINATES
1,-480,120
2,-380,120
3,-300,120

 .   .   .
 .   .   .
 .   .   .

838,300,-160
839,380,-160
840,480,-160
CONNECTIVITY
 1 ,1,30,31,2
 2 ,2,31,32,3
 3 ,3,32,33,4

  .    .   .    . .
  .    .   .    . .
  .    .   .    . .

 796 ,363,364,-1,0
 797 ,392,393,-1,0
 798 ,421,422,-1,0
SOIL PROPERTIES FOR 3 TYPE(S)
 1 ,0.0, 36.0, 110.0, 0.4122, 0.3,0.49, 200,0.5
 2 ,0.0, 36.0,  68.5, 0.4122, 0.3,0.49, 200,0.5
 3 ,0.0, 0.0,  58.0, 0.96, 0.49,0.49, 500,0.0
CU TABLE
    1 ,      0.0
    2 ,      0.0
    3 ,      0.0

    .        .
    .        .
    .        .

  285 ,      0.0
  286 ,      0.0
  287 ,   1291.2
  288 ,   1291.2

    .       .
    .       .
    .       .

  752 ,   6488.0
  753 ,   6488.0
  754 ,   6488.0
TYPE NUMBERS 754
1,104,1,1
105,286,1,2
287,754,1,3
BEAM 14
755,768,1,1,4.176E+09,.1,0
LINK 30
769,772,1,1,2,1E+09,100000,.44523,0,0,0
784,787,1,1,1,1E+09,100000,.44523,0,0,0
773,779,1,1,2,1E+09,100000,.44523,0,0,0
788,794,1,1,1,1E+09,100000,.44523,0,0,0
780,783,1,1,2,1E+09,100000,0,4052,0,0
795,798,1,1,1,1E+09,100000,0,4052,0,0

TO BE EXCAVATED LATER 143
1,13,1
```

```
27,39,1
53,65,1
79,91,1
105,117,1
131,143,1
157,169,1
183,195,1
209,221,1
235,247,1
261,273,1
BOUNDARY NODES 85
1,407,29,1
29,435,29,1
436,787,27,1
462,813,27,1
814,840,26,3
815,839,1,3

STEP 1
GRAVITATE 754 ELEMENTS
1,754,1
 1.0

STEP 2 : EX LAYER 1
ANCHOR 1
131,131,1,200000,0
SUBSTEPS 4
OUTD 0
OUTS 0
EXCAVATE
NODE 14
1,14,1
SOIL 13
1,13,1
LINK 1
769,769,1

   .
   .
   .

STEP 12 : EX LAYER 11
SUBSTEPS 4
EXCAVATE
NODE 14
291,304,1
SOIL 13
261,273,1
LINK 1
779,779,1
```

APPENDIX C

FEMSSI OUTPUT FILE (ABBREVIATED)

```
PROGRAM  F E M S S I  OUTPUT

[ DL40L CASE DENSE BACKFILL ON LOOSE FOUNDATION SOIL LARGE GRID12 (12H)

# NODAL POINTS     =  902
# ELEMENTS         =  862
# OPERATION STEPS  =   12
  ATMOSPHERIC PRES.= 2000.0

NODE COORDINATES
NODE        X           Y
   1   -480.00     120.00
   2   -380.00     120.00
   3   -300.00     120.00
   .      .           .
   .      .           .
   .      .           .

 900    300.00    -160.00
 901    380.00    -160.00
 902    480.00    -160.00

ELEMENT      CORNER NODES
   1     1   30   31    2
   2     2   31   32    3
   3     3   32   33    4
   .     .    .    .    .
   .     .    .    .    .
   .     .    .    .    .
 860   479  480   -1    0
 861   508  509   -1    0
 862   537  538   -1    0

SOIL MODEL PARAMETERS...
TYPE     C      FI  GAMMA     KO      NUI      NUF       M        N
  1   0.00  36.00 110.00   0.412   0.250   0.490   200.0   0.500
  2   0.00  36.00  68.50   0.412   0.250   0.490   200.0   0.500
  3   0.00  30.00  60.50   0.500   0.300   0.490   120.0   0.500

SOIL TYPE NUMBERS FOR 806 ELEMENTS
 FROM  TO STEPS  TYPE
   1  104    1    1
 105  286    1    2
 287  806    1    3

BEAM ELEMENTS:  18
 FROM  TO STEPS INIT       E           A           I
 807  824    1    1  0.4176E+10  0.1000E+00  0.1742E-01

LINK ELEMENTS:  38
 FROM  TO STEP  INIT  R/L  SN        ST      MU      ADHSN   FN-INIT  FT-INIT
825  828    1    1    2  1.00E+09  1.00E+05 0.45    0.00       0.00     0.00
844  847    1    1    1  1.00E+09  1.00E+05 0.45    0.00       0.00     0.00
829  835    1    1    2  1.00E+09  1.00E+05 0.45    0.00       0.00     0.00
848  854    1    1    1  1.00E+09  1.00E+05 0.45    0.00       0.00     0.00
836  843    1    1    2  1.00E+09  1.00E+05 0.36    0.00       0.00     0.00
855  862    1    1    1  1.00E+09  1.00E+05 0.36    0.00       0.00     0.00

ELEMENTS TO BE EXCAVATED LATER: 143
 FROM   TO STEPS
   1   13    1
  27   39    1
  53   65    1
  79   91    1
 105  117    1
 131  143    1
 157  169    1
 183  195    1
 209  221    1
```

```
 235  247    1
 261  273    1

BOUNDARY CONDITIONS AT 89 NODES
 FROM  TO STEP    X-DIR   Z-DIR   ROTN
   1  523   29   FIXED   FREE    FREE
  29  551   29   FIXED   FREE    FREE
 552  849   27   FIXED   FREE    FREE
 578  875   27   FIXED   FREE    FREE
 876  902   26   FIXED   FIXED   FREE
 877  901    1   FIXED   FIXED   FREE

HALF BAND WIDTH=   62
ROTATIONAL D.O.F.:   19

+---------------------------------------+
|         OPERATION STEP  1 OF 12       |
+---------------------------------------+

[ STEP 1

TURN GRAVITY ON 806 ELEMENTS
 FROM  TO STEPS
   1  806    1
FRACTION OF KO STRESS:      1.00

=== SUBSTEP  1 OF  1 ===


FORCES IN THE LINK ELEMENTS
 ELT.  LINK       FN          FT
 825    1    7.0853E+01  5.6843E-09
 826    2    2.8341E+02  1.4211E-08
 827    3    5.6681E+02  2.2737E-08
  .     .        .          .
  .     .        .          .
  .     .        .          .
 860   36    3.7599E+03 -3.1264E-08
 861   37    3.3826E+03 -3.9790E-08
 862   38    2.9787E+03 -2.5580E-08

STRESSES AT THE END OF STEP  1, SUBSTEP  1 OF  1
ELEMENT  X-STRESS  Y-STRESS  XY-STRESS   P.PRES.    G-MOD.      M-MOD.  F
    1       56.68    137.50     0.00     0.00  0.3082E+05  0.1049E+06  0.630
    2       56.68    137.50     0.00     0.00  0.3082E+05  0.1049E+06  0.630
    3       56.68    137.50     0.00     0.00  0.3082E+05  0.1049E+06  0.630

    .         .         .          .       .        .           .        .
    .         .         .          .       .        .           .        .
    .         .         .          .       .        .           .        .
  804     8383.75  16767.50     0.00     0.00  0.1737E+06  0.6949E+06  0.612
  805     8383.75  16767.50     0.00     0.00  0.1737E+06  0.6949E+06  0.612
  806     8383.75  16767.50     0.00     0.00  0.1737E+06  0.6949E+06  0.612


+---------------------------------------+
|         OPERATION STEP  2 OF 12       |
+---------------------------------------+

[ STEP 2 : EX LAYER 1

NEW ANCHORS:   1
 FROM  TO STEPS   X-SPRING    Y-SPRING
 131  131    1  2.0000E+05  0.0000E+00

NUMBER OF SUBSTEPS:  8
DISPLACEMENT OUTPUT FREQUENCY:  0
STRESS OUTPUT FREQUENCY:  0
```

```
REMOVED NODES:  14
 FROM  TO STEPS
   1   14    1

REMOVED SOIL ELEMENTS:  13
 FROM  TO STEPS
   1   13    1

REMOVED LINKS:   1
GENERATED NODAL LOADS
 LINK NODE    FX         FY         NODE    FX         FY

=== SUBSTEP  1 OF  8 ===
  .      .      .
  .      .      .

+---------------------------------------+
|        OPERATION STEP 12 OF 12        |
+---------------------------------------+
[ STEP 12 : EX LAYER 11

NUMBER OF SUBSTEPS:  8

REMOVED NODES:  14
 FROM  TO STEPS
 291  304    1

REMOVED SOIL ELEMENTS:  13
 FROM  TO STEPS
 261  273    1

REMOVED LINKS:   1
GENERATED NODAL LOADS
 LINK NODE    FX         FY         NODE    FX         FY

=== SUBSTEP  1 OF  8 ===
 .
 .
 .
DISPLACEMENTS
 NODE      STEP        STEP        TOTAL       TOTAL
 NUM      X-DISP.     Z-DISP.     X-DISP.     Z-DISP.
  15   0.1257E-02  0.5300E-02  0.9549E-01  0.7484E+00
  16   0.1304E-02  0.5262E-02  0.9585E-01  0.6857E+00
  17   0.1243E-02  0.4038E-02  0.9694E-01  0.6289E+00
  .         .           .           .           .
  .         .           .           .           .
  .         .           .           .           .
 900   0.0000E+00  0.0000E+00  0.0000E+00  0.0000E+00
 901   0.0000E+00  0.0000E+00  0.0000E+00  0.0000E+00
 902   0.0000E+00  0.0000E+00  0.0000E+00  0.0000E+00

DISPLACEMENT OF BEAM ELEMENTS
ELT NODE  X-DISP.   Y-DISP.  ROTATION NODE  X-DISP.    Y-DISP.   ROTATION
 807  15  9.549E-02  7.484E-01 -1.431E-02   44  5.972E-02  7.484E-01 -1.431E-02
 808  44  5.972E-02  7.484E-01 -1.431E-02   73  2.394E-02  7.484E-01 -1.433E-02
 809  73  2.394E-02  7.484E-01 -1.433E-02  102 -1.201E-02  7.484E-01 -1.446E-02
 810 102 -1.201E-02  7.484E-01 -1.446E-02  131 -4.854E-02  7.485E-01 -1.482E-02
 811 131 -4.854E-02  7.485E-01 -1.482E-02  160 -1.237E-01  7.485E-01 -1.493E-02
 812 160 -1.237E-01  7.485E-01 -1.493E-02  189 -1.951E-01  7.485E-01 -1.340E-02
 813 189 -1.951E-01  7.485E-01 -1.340E-02  218 -2.557E-01  7.485E-01 -1.067E-02
 814 218 -2.557E-01  7.485E-01 -1.067E-02  247 -3.007E-01  7.486E-01 -7.252E-03
 815 247 -3.007E-01  7.486E-01 -7.252E-03  276 -3.281E-01  7.486E-01 -3.786E-03
 816 276 -3.281E-01  7.486E-01 -3.786E-03  305 -3.367E-01  7.486E-01 -1.962E-03
 817 305 -3.367E-01  7.486E-01 -1.962E-03  334 -3.396E-01  7.486E-01 -9.667E-04
 818 334 -3.396E-01  7.486E-01 -9.667E-04  363 -3.407E-01  7.486E-01 -1.919E-04
 819 363 -3.407E-01  7.486E-01 -1.919E-04  392 -3.405E-01  7.486E-01  3.676E-04
 820 392 -3.405E-01  7.486E-01  3.676E-04  421 -3.393E-01  7.486E-01  7.375E-04
 821 421 -3.393E-01  7.486E-01  7.375E-04  450 -3.376E-01  7.486E-01  9.541E-04
```

822	450	-3.376E-01	7.486E-01	9.541E-04	479 -3.356E-01	7.486E-01	1.056E-03
823	479	-3.356E-01	7.486E-01	1.056E-03	508 -3.334E-01	7.486E-01	1.091E-03
824	508	-3.334E-01	7.486E-01	1.091E-03	537 -3.318E-01	7.486E-01	1.097E-03

FORCES IN THE BEAM ELEMENTS

ELT.	NODE	FX	FY	MOMENT	NODE	FX	FY	MOMENT
807	15	4.018E+01	-7.824E+01	2.146E-02	44	-4.018E+01	7.824E+01	1.004E+02
808	44	-5.728E+02	-4.368E+02	-1.004E+02	73	5.728E+02	4.368E+02	-1.332E+03
809	73	-1.946E+03	-1.279E+03	1.332E+03	102	1.946E+03	1.279E+03	-6.196E+03
810	102	-3.507E+03	-1.867E+03	6.196E+03	131	3.507E+03	1.867E+03	-1.496E+04
811	131	5.342E+03	-2.017E+03	1.496E+04	160	-5.342E+03	2.017E+03	1.175E+04
812	160	4.195E+03	-2.105E+03	-1.175E+04	189	-4.195E+03	2.105E+03	3.272E+04
813	189	2.825E+03	-2.255E+03	-3.272E+04	218	-2.825E+03	2.255E+03	4.684E+04
814	218	1.147E+03	-2.435E+03	-4.684E+04	247	-1.147E+03	2.435E+03	5.258E+04
815	247	-8.639E+02	-2.246E+03	-5.258E+04	276	8.639E+02	2.246E+03	4.826E+04
816	276	-2.692E+03	-1.928E+03	-4.826E+04	305	2.692E+03	1.928E+03	4.018E+04
817	305	-3.969E+03	-1.816E+03	-4.018E+04	334	3.969E+03	1.816E+03	3.225E+04
818	334	-4.067E+03	-9.513E+02	-3.225E+04	363	4.067E+03	9.513E+02	2.411E+04
819	363	-3.762E+03	-2.019E+02	-2.411E+04	392	3.762E+03	2.019E+02	1.659E+04
820	392	-3.136E+03	2.614E+02	-1.659E+04	421	3.136E+03	-2.614E+02	1.032E+04
821	421	-2.443E+03	6.828E+02	-1.032E+04	450	2.443E+03	-6.828E+02	5.434E+03
822	450	-1.729E+03	1.073E+03	-5.434E+03	479	1.729E+03	-1.073E+03	1.975E+03
823	479	-7.051E+02	1.505E+03	-1.975E+03	508	7.051E+02	-1.505E+03	5.648E+02
824	508	-3.766E+02	1.118E+03	-5.648E+02	537	3.766E+02	-1.118E+03	5.337E-06

FORCES IN THE LINK ELEMENTS

ELT.	LINK	FN	FT
836	23	1.7491E+03	6.3662E+02
837	24	2.4137E+03	8.7853E+02
838	25	2.7702E+03	1.0083E+03
839	26	2.9154E+03	1.0611E+03
840	27	3.0288E+03	1.1024E+03
841	28	3.4441E+03	1.2517E+03
842	29	2.5454E+03	6.1993E+02
843	30	2.2653E+03	-2.1620E+02
844	5	-4.0158E+01	-1.3409E+00
845	6	6.1300E+02	-2.7116E+02
846	7	1.3729E+03	-6.1126E+02
847	8	1.5614E+03	-6.8849E+02
848	16	8.5892E+02	-3.8242E+02
849	17	1.1472E+03	-5.1077E+02
850	18	1.3697E+03	-6.0983E+02
851	19	1.6779E+03	-7.4705E+02
852	20	2.0110E+03	-8.9536E+02
853	21	1.8278E+03	-8.1378E+02
854	22	1.2768E+03	-5.6848E+02
855	31	1.8472E+03	-6.7232E+02
856	32	2.1087E+03	-7.6752E+02
857	33	2.1449E+03	-7.8068E+02
858	34	2.2217E+03	-8.0864E+02
859	35	2.3154E+03	-8.4274E+02
860	36	2.4200E+03	-8.8082E+02
861	37	2.2168E+03	-8.0685E+02
862	38	1.8888E+03	-6.8746E+02

ANCHOR FORCES

ANCH.	NODE	X-FORCE	Y-FORCE
1	131	-9.708E+03	0.000E+00

STRESSES AT THE END OF STEP 12, SUBSTEP 8 OF 8

ELEMENT	X-STRESS	Y-STRESS	XY-STRESS	P.PRES.	G-MOD.	M-MOD.	F
14	138.55	146.47	116.40	0.00	0.6676E+04	0.1782E+06	0.990
15	281.25	147.76	102.99	0.00	0.2485E+04	0.2947E+05	0.960
.	.	.	.	.	.	.	.
.	.	.	.	.	.	.	.
.	.	.	.	.	.	.	.
804	8278.26	16674.18	179.15	0.00	0.1694E+06	0.6618E+06	0.620
805	8320.31	16715.94	92.73	0.00	0.1711E+06	0.6671E+06	0.617
806	8337.62	16725.87	28.90	0.00	0.1720E+06	0.6697E+06	0.615

VITA

Wei Zhang

Candidate for the Degree of

Master of Science

Thesis: ON THE DESIGN AND ANALYSIS OF ANCHORED SHEET PILE WALLS

Major Field: Civil Engineering

Biographical:

Personal Data: Born in Nanjing, China, January 19, 1962, son of Guobin Zhang and Xiuying Feng.

Education: Graduated from Gaozhi Senior High School, Nanjing, China, in June 1979; received Bachelor of Science Degree in Hydrogeology and Engineering Geology from Hohai University, Nanjing, China, in July, 1983; completed requirements for the Master of Science degree at Oklahoma State University in May, 1992.

Professional Experience: Assistant Engineer, The First Institute of Hydrogeology and Engineering Geology, Nanjing, China, August, 1983, to November, 1988; Engineer, The First Institute of Hydrogeology and Engineering Geology, Nanjing, China, December, 1988, to December, 1989; Research Assistant and Teaching Assistant, School of Civil Engineering, Oklahoma State University, January, 1990, to December, 1991.