University of Louisville

ThinkIR: The University of Louisville's Institutional Repository

Electronic Theses and Dissertations

12-2011

Comparative evaluation of conventional soldier pile/lagging retention systems versus soil nailing retention systems.

Abdul Amir Al-Helu 1966-
University of Louisville

Follow this and additional works at: <https://ir.library.louisville.edu/etd>

Part of the [Civil and Environmental Engineering Commons](#)

Recommended Citation

Al-Helu, Abdul Amir 1966-, "Comparative evaluation of conventional soldier pile/lagging retention systems versus soil nailing retention systems." (2011). *Electronic Theses and Dissertations*. Paper 30.
<https://doi.org/10.18297/etd/30>

This Doctoral Dissertation is brought to you for free and open access by ThinkIR: The University of Louisville's Institutional Repository. It has been accepted for inclusion in Electronic Theses and Dissertations by an authorized administrator of ThinkIR: The University of Louisville's Institutional Repository. This title appears here courtesy of the author, who has retained all other copyrights. For more information, please contact thinkir@louisville.edu.

**COMPARATIVE EVALUATION OF CONVENTIONAL SOLDIER
PILE/LAGGING RETENTION SYSTEMS VERSUS SOIL NAILING
RETENTION SYSTEMS**

By

Abdul Amir Al-Helu

A Dissertation

Submitted to the Faculty of the
Graduate School of the University of Louisville

In Partial Fulfillment of the requirements

For the Degree of

Doctor of Philosophy

Department of

Civil and Environmental Engineering

University of Louisville

Louisville, Kentucky

December, 2011

Copyright 2011 by Abdul Amir Al-Helu

All rights reserved

**COMPARATIVE EVALUATION OF CONVENTIONAL SOLDIER
PILE/LAGGING RETENTION SYSTEMS VERSUS SOIL NAILING
RETENTION SYSTEMS**

By

Abdul Amir Al-Helu

A Dissertation Approved on

September 28, 2011

By the following Dissertation Committee:

Dr. D. Joseph Hagerty
Dissertation Director

Dr. Ibrahim N. Imam

Dr. C. Robert Ullrich

Dr. James C. Watters

Dr. Terence A. Weigel

DEDICATION

This dissertation is dedicated to my mother Maliha Al-Helu who passed away before seeing me graduate as she wished.

ACKNOWLEDGEMENTS

I would like to thank Dr. Hagerty for his guidance and enthusiasm throughout this study. Thank you to the other members of this committee, Dr. Ullrich, Dr. Weigel, Dr. Watters, and Dr. Imam. Many thanks go to Mr. Richard Chilton who helped me very much to estimate job pricing, including materials and labor costs for both retention methods and for his encouragement to finish the PhD program.

I would like to thank my family and my brothers for their encouragement.

ABSTRACT

COMPARATIVE EVALUATION OF CONVENTIONAL SOLDIER
PILE/LAGGING RETENTION SYSTEMS VERSUS SOIL NAILING
RETENTION SYSTEMS

ABDUL AMIR AL-HELU

9/28/2011

An investigation was made into the relative costs of excavation retention by two systems: conventional soldier pile-wale-tieback support, and support by soil nailing. The investigation included design of excavation supports by the two aforementioned systems for fifteen actual excavations, the majority of which were accomplished in the Louisville, Kentucky area. The design was optimized to arrive at minimum cost solutions. In all cases, the soil nailing system was much less costly than the conventional system. Several factors appeared to make soil nailing more economical than conventional retention systems, if deformations and settlement of soil nailing systems can be tolerated: lower cost for construction materials; quicker mobilization of installation equipment; faster construction time; smaller construction space requirements; and less need for heavy equipment.

TABLE OF CONTENTS

1. INTRODUCTION	1
1.1 General	1
1.2 Fundamental Evaluation	2
1.3 Conventional Retention Systems.....	2
1.4 Purpose of Study.....	4
2. EARTH RETENTION SYSTEMS	5
2.1 General	5
2.2 Soldier Piles and Lagging.....	6
2.3 Internal Bracing versus Tiebacks	7
2.4 Conventional Tied Systems versus Soil Nailing	8
2.4.1 Installation of Conventional Systems.....	10
2.4.2 Installation of Soil Nails	16
2.4.3 Advantages of Conventional Systems	20
2.4.4 Disadvantages of Conventional Systems.....	20
2.4.5 Advantages of Soil Nailing Systems	21
2.4.6 Disadvantages of Soil Nailing Systems.....	22
2.4.7 Summary on Technical Aspects	23
3. COSTS OF RETENTION SYSTEMS	25
3.1 Analyzed Projects	25

3.2	Assumptions in Analyses	25
3.3	Method of Analysis	26
4.	RESULTS OF ANALYSES	66
4.1	Cost of Conventional Systems	66
4.2	Cost of Soil Nailing Systems	67
4.3	Factors Causing Differences in Cost	69
4.3.1	Construction Management Approach	69
4.3.2	Project Size	72
4.3.3	Project Location	91
4.3.4	Adjusted Analysis	95
5.	CONCLUSIONS AND RECOMMENDATIONS	97
5.1	General Conclusions	97
5.2	Factors Affecting Cost.....	98
5.2.1	Effect of Project Size	98
5.2.2	Effect of Project Location	99
5.3	Recommendations	99
	REFERENCES.....	100
	APPENDIX A:	101
	APPENDIX B:	138
	APPENDIX C:	223
	APPENDIX D	266
	APPENDIX E	294
	APPENDIX F	311
	CURRICULUM VITA	337

LIST OF FIGURES

Figure 1	Typical Wood Lagging Wall.....	12
Figure 2	Tieback through Soldier Pile Detail.....	13
Figure 3	Tieback with Common Wale Detail.....	13
Figure 4	Lagging Stud Detail	14
Figure 5	Typical Soil Nail Wall Construction Sequence.....	18
Figure 6	Tieback Bonded Length and Free Length.....	27
Figure 7	Two-tieback Wall Vertical Spacing	28
Figure 8	Three-tieback Wall Vertical Spacing	29
Figure 9	Four-tieback Wall Vertical Spacing	29
Figure 10	Step 1 of Retention System Design Process part A (all dimensions in feet).....	31
Figure 11	Case 5 Part A Step 1 Design Calculation	32
Figure 12	Step 2 of Retention System Design Process part A (all dimensions in feet)	33
Figure 13	Case 5 part A step 2 design calculation	34
Figure 14	Case 5 part A step 2 design calculation	35
Figure 15	Step 3 of Retention System Design Process part A (all dimensions in feet).....	36
Figure 16	Case 5 part A step 3 Design Calculation 1 st run	37

Figure 17 Case 5 part A step 3 Design Calculation 2 nd run	38
Figure 18 Step 4 of Retention System Design Process part A (all dimensions in feet).....	39
Figure 19 Case 5 part A step 4 Design Calculation	40
Figure 20 Step 1 of Retention System Design Process part B (all dimensions in feet).....	41
Figure 21 Case 5 part B step 1 Design Calculation	42
Figure 22 Step 2 of Retention System Design Process part B (all dimensions in feet).....	43
Figure 23 Case 5 part B step 2 Design Calculation.....	44
Figure 24 Case 5 part B step 2 Design Calculation	45
Figure 25 Step 3 Final Stage of Design Process part B (all dimensions in feet).....	46
Figure 26 Case 5 part B step 3 Design Calculation	47
Figure 27 Typical Soil Nailing Cross Section	51
Figure 28 Case 5 part A Soil Nailing Calculation 1 st run	52
Figure 29 Case 5 part A Soil Nailing Calculation 2 nd run.....	54
Figure 30 Case 5 part A Soil Nailing Calculation 3 rd run	56
Figure 31 Case 5 part A Final Soil Nail Section (all dimensions in feet).....	58
Figure 32 Case 5 part B Soil Nailing Calculation	59
Figure 33 Case 5 Part B Final Soil Nail Section (all dimensions in feet).....	61
Figure 34 Case 5 part A Soil Nailing Cost	63
Figure 35 Case 5 part B Soil Nailing Cost	64
Figure 36 Total Cost Comparison, Conventional and Soil Nailing Systems.	68

Figure 37 Unit Cost Comparison, Conventional Systems and Soil Nailing Systems	68
Figure 38 Cost Ratio (Soil Nailing / Conventional).....	69
Figure 39A Total Cost of Conventional Systems and Soil Nailing Systems versus Retained Area	74
Figure 39B Total Cost of Conventional Systems and Soil Nailing Systems versus Retained Area without case 15 (71,820 sq ft).....	74
Figure 40A Unit Cost of Conventional Systems and Soil Nailing Systems versus Retained Area.....	75
Figure 40B Unit Cost of Conventional Systems and Soil Nailing Systems versus Retained Area without case 15 (71,820 sq ft).....	75
Figure 41A Unit Cost of Steel Pile and Tiebacks versus Retained Area	76
Figure 41B Unit Cost of Steel Pile and Tiebacks versus Retained Area without case 15 (71,820 sq ft).....	77
Figure 42A Cost of Nail, Plate, Nut and Shotcrete versus Retained Area	77
Figure 42B Unit Cost of Nail, Plate, Nut and Shotcrete versus Retained Area without case 15 (71,820 sq ft).....	78
Figure 43A Total Cost of Conventional Systems and Soil Nailing Systems versus Depth of Cut (depth of retained excavation).....	79
Figure 43B Total Cost of Conventional Systems and Soil Nailing Systems versus Depth of Cut (depth of retained excavation) without case 15.....	80
Figure 44A Unit Cost of Conventional Systems and Soil Nailing Systems versus Depth of Cut (depth of retained excavation).....	80

Figure 44B Unit Cost of Conventional Systems and Soil Nailing Systems versus	
Depth of Cut (depth of retained excavation) without case 15.....	81
Figure 45A Unit Cost of Tiebacks versus Depth of Cut (depth of retained	
excavation).....	82
Figure 45B Unit Cost of Tiebacks versus Depth of Cut (depth of retained	
excavation) without case 15.....	82
Figure 46 Unit Cost (\$/Linear foot) of Conventional and Soil Nailing versus	
Depth of Cut (depth of retained excavation).....	83
Figure 47 Unit Cost (Case 3) for Each Conventional System Component	85
Figure 48 (Case 3) Unit Cost for Each Soil Nailing System Component	85
Figure 49 Unit Cost (Case 7) for Each Conventional System Component.....	86
Figure 50 (Case 7) Unit Cost for Each Soil Nailing System Component	86
Figure 51 Unit Cost (Case 13) for Each Conventional System Component.....	87
Figure 52 (Case 13) Unit Cost for Each Soil Nailing System Component.....	87
Figure 53 Unit Cost (Case 14) for Each Conventional System Component	88
Figure 54 (Case 14) Unit Cost for Each Soil Nailing System Component	88
Figure 55A Unit Cost of Conventional Systems and Soil Nailing Systems versus	
Retained area in Louisville, KY	92
Figure 55B Unit Cost of Conventional Systems and Soil Nailing Systems versus	
Retained area in Louisville, KY without case 15 (17,820 sq ft).....	92
Figure 56 Unit Cost of Conventional Systems and Soil Nailing Systems versus	
Retained area Outside of Louisville, KY	93
Figure 57 Total Cost of Conventional System with Location versus Retained	
Area without case 15.....	94

Figure 58	Total Cost of Soil Nailing System with Location versus Retained	
	Area without case 15 (71,820 sq ft).....	94
Figure 59	Unit Cost of Conventional System with Location versus Depth of	
	Cut (depth of retained excavation).....	95
Figure 60	Unit Cost of Soil Nailing System with Location versus Depth of	
	Cut (depth of retained excavation).....	96

LIST OF TABLES

Table 1	Soil Properties	7
Table 2	Case 5 Conventional System Materials Takeoff	48
Table 3	Case 5 Conventional System Materials and Operation Cost	49
Table 4	Case 5 Soil Nailing Materials Takeoff part A and part B.....	62
Table 5	Case 5 Soil Nailing Materials takeoff	62
Table 6	Case 5 part A, and part B Soil Nailing Cost	65
Table 7	Costs for Conventional System	66
Table 8	Cost of Soil Nailing Systems	67
Table 9	Unit Cost \$/Linear Foot versus Depth of Cut (depth of retained excavation) for Conventional and Soil Nailing	84
Table 10	Conventional System Unit Cost for Components and Operations, \$/sqft...	90
Table 11	Soil Nailing System Unit Cost for Components and Operations, \$/sqft.....	91

LIST OF ILLUSTRATIONS

Illustration 1	Tieback through Common Wale.....	14
Illustration 2	Wood Lagging Detail.....	15
Illustration 3	Tieback through Steel Pile.....	15
Illustration 4	New Construction in Excavated Area.....	16

CHAPTER I

1. INTRODUCTION

1.1 General

Deep excavations are becoming increasingly common for the construction of buildings, road tunnels, mass rapid transit systems and other facilities at densely built-up areas within city and suburban areas. Soil or rock excavation can be considered as deep excavation if the excavation is typically more than 14 feet deep. Such works could affect nearby structures and the foundations of those structures because of ground movement associated with stress changes. Deep excavation dictates careful design and planning especially when constructed in urban areas. Retaining and support system selection in deep excavation can have significant impact on time, cost and performance of the completed excavation and construction project. Hence, there is a need for design and execution of cost-effective systems for retention of deep excavations. The Author, in this research, will make a comparison of costs for fifteen cases on the basis of design and installation of a conventional (H-pile and lagging) retention system versus design and installation of a soil nailing system. The hypothesis is that soil nailing is more economical for earth retention in excavations than is conventional H-pile, lagging, wales and tiebacks, for the situations examined in this study.

1.2 Fundamental Evaluation

A consideration of fundamental soil-structure interaction would indicate that the soil nailing system would be cheaper than the conventional retaining system for conditions encountered on typical projects, because the soil nailing system is a passive system that uses the *in situ* soil as part of the retaining system. The soil is allowed to move sufficiently to mobilize the shearing resistance of the soil mass. Then, the resistance to further movement offered by the soil shear strength combines with the soil nail system support to hold the mass of soil in place. The installed system will start working to hold the soil against further movement using the effects of the nails, shotcrete and wire mesh in conjunction with the soil internal resistance. In contrast, the conventional system is built as an active pressure system with stiff members that push against the retained soil with the effects of the steel soldier piles, tiebacks, and steel wales. The key consideration is that the soil nailing system allows a certain amount of movement of the retained soil mass, and that movement may not be acceptable on some projects. If that movement is not acceptable under project conditions, the conventional system will be required and may be more costly than the soil nailing system. The analysis in the following sections will show a cost comparison for 15 actual projects. First, an explanation of retaining system characteristics is given.

1.3 Conventional Retention Systems

The most popular retaining systems for deep excavations can be classified as soldier piles and lagging walls, sheet piling walls, soil mix walls, and diaphragm

walls. The lateral force members for these retaining systems can be of different types: soil tiebacks; rock anchors; internal struts and braces. All conventional systems have similar characteristics in that they include some sort of vertical membrane (lagging boards between vertical soldier piles, interlocked sheet piles; or overlapping piles or drilled shafts) held in place by components that provide resistance to lateral earth-pressure forces (tie-backs drilled into soil behind the membrane; anchors drilled and secured in rock behind and below the membrane, or horizontal struts bearing on horizontal wales arranged to spread load from struts to membrane). In rare circumstances (not to be treated in this research) vertical piles or drilled shafts are embedded below the excavation bottom and extend as cantilevers up to the top of the excavation. In all of these systems, some part of the membrane is installed before excavation is begun. Components used to resist lateral earth-pressure forces are installed soon after the membrane is in place (tie-backs or anchors) or are installed as excavation proceeds (wales and struts). By far the most common retention system is soldier piles with horizontal wooden lagging retained by soil tie backs. Appendix A contains sketches illustrating various conventional retention systems.

In contrast to conventional retention systems, soil nailing involves installation of tensile members into the soil behind the outer edges of an excavation without a membrane that extends from top to bottom of the excavation. Shallow excavations are made, and then tensile members are installed at a shallow angle to the horizontal through the exposed short sides of the excavation. Vertical sheets of drainage fabric are placed against the soil face to provide outlets for any water that would collect under the cover to be placed over the soil face. After the tensile members are in place,

some sort of reinforcing system is placed over the exposed soil face and a relatively thin coating of gunite or shotcrete is sprayed over the reinforcing bars. Then, a plate is placed over the exposed end of each tensile element, and secured in place with a lock nut threaded on the end of the tensile member. After all the exposed soil is covered in this fashion, the excavation is deepened; the increments of excavation depth are kept uniform from top to bottom of the excavation. Soil nailing thus is a “top down” construction method. Soil nailing has become very popular for retention of deep excavations, and has been claimed to be more economic than conventional retention systems.

1.4 Purpose of study

The purpose of this study is to make a cost comparison between two retention systems and examine which of these two systems is the more economic; the first system is the most commonly used conventional system, soldier piles (H-pile sections), wood lagging, and tieback anchors. The other system is the soil nailing system, with low-strength steel tendons as tensile anchors, welded wire mesh to act as reinforcing between tensile members to hold the face of the wall, and shotcrete to cover the face of the wall.

CHAPTER 2

EARTH RETENTION SYSTEMS

2.1 General

Excavation with vertical sides requires lateral support, to prevent sidewall collapse and to reduce lateral movement and settlement of the surrounding ground. Retention systems also are evaluated against the tendency for base failure and bottom heave in soft clays.

The settlement and lateral movement depend on

- 1- Soil properties
- 2- General procedure of excavation
- 3- Adequacy of the bracing
- 4- Workmanship

The settlement near an open cut can be reduced only if the inward movement of the sheeting and the bottom heave can be substantially reduced. Experience shows that using a heavy and stiff section of soldier piles or sheet piles usually is not sufficient to limit lateral movements and settlements (Clough and O'Rourke, 1990). If soil stiffness and strength are low, lateral supports or anchors must be spaced closely to reduce the magnitude of the lateral movement of a wall. In other words, movements can be substantially reduced by installing layers of struts or anchors relatively close in vertical spacing. The most important variable, however, that

determines the amount of movement is not the stiffness of the wall sections, or the stiffness of the lateral force members, or the vertical spacing of bracing, but the properties of the surrounding soil. The combination of the four parameters listed above has driven practice to the use of soldier pile and lagging systems restrained by soil tie-backs/anchors. Such systems have been chosen for analysis in this research.

In this study, the Author provides an economic comparison between two types of retention systems: soldier pile (H-pile) and wood lagging, restrained by soil anchors (conventional method), versus the soil nailing retention method. The Author studied fifteen cases for which he already had designed a conventional retention system (and that had been installed) and used a present cost analysis to compare costs for design and construction of those systems, to the costs associated with design and installation of soil nailing systems for those fifteen cases. Soil strata in most of the fifteen cases are layers of sandy clay and sandy silt soils. Soil properties are listed in Table 1 on the next page for each case. For context, the deepest cut in this study was 42 feet.

2.2 Soldier Piles and Lagging

The most common system for retaining open excavations in the United States is the use of soldier piles and lagging; the components of such a system include soldier piles, wooden lagging boards, lateral load-spreader beams (wales) and tiebacks. Soldier piles are driven or predrilled at intervals along the edges of the excavation. If a predrilled approach is used, holes are drilled, grout is placed and H-

piles are installed in the hole. Soldier piles and wood lagging systems have been used commonly for many years,

Table 1. Soil Properties

	1st Layer	Soil unit weight	Friction angle	Cohesion	2nd Layer	Soil unit weight	Friction angle	Cohesion	3rd Layer	Soil unit weight	Friction angle	Cohesion	Depth to hard pan below excavation level
	ft	pcf	degree	psf	ft	pcf	degree	psf	ft	pcf	degree	lb/sq.ft	ft
Case 1	8.5	120	25	700	15	125	30	1500					60
Case 2	15	110	20	500	20	120	30	1000					50
Case 3	20	125	30	550									35
Case 4	8	120	25	650	10	125	30	1100	25	130	35	1600	40
Case 5	11.5	115	30	500	10	120	30	1000	20	125	25	1500	35
Case 6	11	120	20	600	16	125	25	1500	18	130	30	1700	35
Case 7	13	110	20	750	20	125	30	1100					12
Case 8	15	30	115	800	28	120	28	1500					29
Case 9	17	115	28	550	19	130	33	1200					23
Case 10	22	120	30	1000	18	125	33	1600					15
Case 11	18	125	30	1100									32
Case 12	21	120	28	550	25	125	32	1300					24
Case 13	15	115	20	1100									40
Case 14	13	110	28	500	17	120	30	1100					28
Case 15	10	110	20	550	13	120	30	1000	19	125	30	1500	20

thereby providing a vibration-free installation. As the excavation in front of the wall proceeds, wood lagging is installed between the soldier piles in lifts. Tieback anchors are installed and stressed to provide lateral restraint as needed. Some type of lateral load-spreader is used to distribute the concentrated load from the anchor to the soldier piles.

2.3 Internal Bracing versus Tiebacks

In the case of internal bracing systems, the lateral earth pressure is transferred between opposing walls through compressive struts. Rakes resting on a foundation mat or rock offer another internal bracing alternative. Usually, the struts are either pipe or I- beam sections and usually are preloaded to provide a very stiff system.

Installation of the bracing struts is carried out by excavating soil locally around the strut and only continuing the general excavation when preloading is complete. The struts rest on a series of wale beams that distribute the strut load to the wall. Preloading ensures a rigid contact between interacting members and is accomplished by inserting a hydraulic jack on each side of an individual strut between the wale beam and a special jacking plate welded to the strut. Internal bracing makes sense in narrow excavations and in cases where tieback installation is not practicable. The struts can bend excessively under their own weight if the excavation spacing is very large. A clear benefit of using struts is that no openings are required in the wall membrane for tieback installation. In contrast to internally braced systems, using tieback retention systems will give a clear, open excavation that facilitates construction within the retention system. In addition, if any lateral movement occurs in the retained wall, the movement could be eliminated by increasing load in the adjacent tieback.

2.4 Conventional Tied Systems versus Soil Nailing Systems

A principal difference between these systems lies in the character of the tensile elements used to resist lateral soil-pressure forces. Soil nails are usually shorter than tie-backs, with much lower design loads than tieback anchors. In length, soil nails usually are between 70 percent and 100 percent of the wall height. Grouted tieback anchors are usually longer than soil nails; tiebacks are usually 100 percent to 125 percent of the wall height in length. Sometimes, tieback anchors are much longer to reach rock or soil with sufficient strength to support the design load.

Soil nails are usually installed at closer spacing than are tiebacks. Nails may be installed to support an area of approximately 20 to 30 square feet per nail, while tiebacks may be installed to support an area of approximately 120 square feet per tie. Tiebacks are active support members; they push on the retained soil harder than the earth and surcharge pressures would push on the wall. Soil nails are passive support members. They provide their support as the soil mass begins to strain to mobilize resistance to shear. Soil nailing cannot be used in sandy soils or soils that cannot stand in a vertical cut of 4 to 6 feet without support until wire mesh and shotcrete are installed.

Soil nails and tieback anchors typically have similar corrosion protection details. Every tieback anchor is usually proof tested or performance tested. Most soil nails are not tested. Usually a small percentage of the nails are tested or non-production nails are tested. Soil nail tendons are usually Grade 60 or 75 thread bar tendons. Tieback tendons are usually either thread bar tendons (Grade 160) or multi-strand tendons (Grade 270). A soil nail wall does not have soldier piles (H-piles). Shotcrete and nails are the sole support. There may be a second or permanent facing of shotcrete or even a precast facing attached to the nails. Tied back walls often use soldier beams, lagging, and tiebacks with a precast or cast-in-place concrete permanent facing when required.

Construction experience suggests that soil nail walls become economical when the retained soils have a little cohesion and when it may be expensive to install soldier piles. Uncontrolled ground water can preclude the use of soil nails and

conventional retaining systems; dewatering may be required to permit use of either of these systems.

2.4.1 Installation of Conventional Systems

The membrane components of the conventional system under study in this research are H-piles and wood lagging. The H-pile sections usually are 10BP42 or 12BP53 sections installed at 8 to 10 feet center-to-center around the perimeter of the excavation. The timber lagging is 3-inch-thick to 4-inch-thick rough-cut boards. Bridging between H-piles and anchors is accomplished with horizontal wales consisting of double C-section channels. A drilled and grouted tieback is the most common anchor. Usually the wood lagging in the upper 8 feet of the retained wall, or down to the first row of tiebacks, will be installed behind the soldier pile flanges to accomplish a fast installation. The lagging at larger depths of the wall will be installed on the front face of the piles by welding studs and plates to the piles to hold the wood lagging. Welding will slow the work in the lower parts of the excavations; therefore, the installation cost of the wood lagging for the upper 8 feet is lower than the cost at lower elevations.

To install the kind of system analyzed in this research, the first thing done is to predrill or drive the steel beam (soldier pile) into the ground on a base line around the excavation perimeter down to the required depth (depth of the excavation plus the required depth below the bottom as needed for stability according to the design calculations). Then the excavation is advanced down to a depth of 4 to 6 feet, or down to the upper tieback elevation, depending on the type of the soil [Can the soil

maintain a vertical face without support until lagging (wood, steel, concrete, etc.) is installed?]. When excavation level reaches a tieback elevation, the process of tieback installation starts.

The tieback installation starts with drilling a hole 4 inches to 6 inches in diameter at an angle of about 20 to 30 degrees below the horizontal plane as deep as the design requires. The depth of tieback installation depends on the soil (strength) properties, and the design load. Then, a hollow high-tensile-strength threaded steel reinforcing bar (175 ksi) is inserted into the hole and grout is injected through the center of the rebar. The grouted part of the rebar is called the bonded zone and the upper part of the rebar (not grouted) is called a free zone. The bonded zone must be behind the failure zone in the soil to make the system stable. The failure zone is assumed to extend up from the bottom of the H-piles at a steep angle to the horizontal; the angle depends on the shearing resistance of the soil. A stability analysis must be done to ensure that the mass of soil retained by the piles, lagging and tiebacks will not fail along a shearing surface that extends below the bottom of the excavation in a global shear failure.

Depending on the design, a wale(s) may be installed, or the tieback can be installed through the steel H-pile. If a wale design has been used, then a steel wedge will be welded to each steel H-pile at an angle that is the same as the tieback angle. Then, the wale will be mounted on these wedges and the tieback will go through the wale usually between two channel section. The tieback will be secured through a steel plate and held with nuts against the plate that, in turn, bears against the wale. The tieback then is stressed with hydraulic jacks to provide a pre-stressed lateral

resistance. This sequence is shown in Fig. 1. Also, Fig 2 shows the arrangement for a tieback installed through a steel H-pile. Details of the connections are shown in Fig. 3 and Fig. 4. Each tieback will be proof tested to 120 percent of the design load per the Prestressed Testing Institute recommendations. Illustrations 1 through 4 show typical scenes from a project where a conventional retention system was installed.

Figure 1. Typical Wood Lagging Wall

Figure 2. Tieback through Soldier Pile Detail

Figure 3. Tieback with Common Wale Detail

Figure 4. Lagging Stud Detail

Illustration 1. Tieback through Common Wale (courtesy of RAM Engineering)

Illustration 2. Wood Lagging Detail (courtesy of RAM Engineering)

Illustration 3 Tieback through Steel Pile (courtesy of RAM Engineering)

Illustration 4 New Construction in Excavated Area (courtesy of RAM Engineering)

2.4.2 Installation of Soil Nails.

Soil nailing is an in-situ soil reinforcement technique, which has been used during the last 28 years, mainly in France and Germany, to retain excavations or stabilize slopes. The fundamental concept of soil nailing consists of placing in the ground passive inclusions, closely spaced, to increase the overall shear strength of the in-situ soil, to restrain its displacements and limit its lateral expansion during and after excavation (Lazarte et al., 2003).

The typical sequence of construction for a soil nail wall using solid steel nail bars as assumed for study in this research is described below and shown schematically in Figure 5.

Step 1. Excavation:

Initial excavation is carried out to a depth for which the face of the excavation has the ability to remain unsupported for a short period, typically on the order of 24 to 48 hours. The depth of the excavation lift is usually between 3 and 6 feet and reaches slightly below the elevation where nails will be installed. The width of the excavated platform or bench must be sufficient to provide access to the installation equipment.

Step 2. Drilling Nail Holes:

Nail holes are drilled to a specified length, diameter, inclination, and horizontal spacing from the excavated platform. Nails can be driven, but for this research, the more common technique of drilling nail holes has been assumed.

STEP 1. EXCAVATE SMALL CUT

STEP 2. DRILL NAIL HOLE

**STEP 3. INSTALL AND GROUT NAIL
(INCLUDES STRIP DRAIN INSTALLATION)**

**STEP 4. PLACE TEMPORARY FACING
(INCLUDES SHOTCRETE,
REINFORCEMENT,
BEARING PLATE, HEX NUT, AND
WASHERS INSTALLATION)**

**STEP 5. CONSTRUCTION OF
SUBSEQUENT LEVELS**

**STEP 6. PLACE FINAL FACING
ON PERMANENT WALLS
(INCLUDES BUILDING
OF TOE DRAIN)**

Figure 5. Typical Soil Nail Wall Construction Sequence.

Modified after Porterfield et al. (1994).

Step 3. Nail Installation and Grouting:

Nail bars are placed in the pre-drilled holes. The bars are most commonly solid, although hollow steel nails can be used and have seen increased usage. Centralizers are placed around the nails prior to insertion to help maintain alignment within the hole and allow sufficient protective grout coverage over the nail bar. A grout pipe (tremie) is also inserted in the drill hole at this time. When corrosion protection requirements are high, corrugated plastic sheathing can be used to provide an additional level of corrosion protection. The drill hole then is filled with cement grout through the tremie pipe. The grout is commonly placed under gravity or low pressure. Prior to Step 4 (facing placement), geocomposite drainage strips are installed on the excavation face approximately midway between each set of adjacent nails. The drainage strips then are unrolled to the next wall lift. The drainage strips extend to the bottom of the excavation where collected water is conveyed via a toe drain away from the soil nail wall.

Step 4. Construction of Temporary Shotcrete Facing:

A temporary facing system then is constructed to support the open-cut soil section before the next lift of soil is excavated. The most typical temporary facing consists of a lightly reinforced shotcrete layer commonly containing 6x6 welded wire mesh (WWM), which is placed at approximately the middle of the facing thickness. Following appropriate curing time for the temporary facing, a steel bearing plate is placed over the nail head protruding from the drill hole. A hexagonal nut and washers are installed to secure the nail head against the bearing plate. The hexagonal nut is

tightened to a required minimum torque after the temporary facing has sufficiently cured. Curing usually requires a minimum of 24 hours.

2.4.3 Advantages of Conventional Systems

The conventional methods of retention have some advantage over the soil nailing methods, such as:

1. The lateral deformation of the retained wall is less than the deformation of a soil nailing wall; this deformation is on the order of 0.08 percent to 0.1 percent of the height of the wall compared to 0.2 percent to 0.4 percent of the height of the wall in soil nailing systems (Stocker and Riedinger, 1990).
2. Conventional systems are more commonly used, so more workers will be knowledgeable about conventional systems and can perform this type of retention work with little training.
3. Conventional systems require fewer anchors and that means they are faster to install (less drilling time).

2.4.4 Disadvantage of Conventional Systems

1. This system requires heavy construction equipment and extra cost for mobilization and demobilization of such equipment.
2. Conventional systems require a larger easement for tieback installation, because tiebacks are drilled deep in the ground and adjacent underground utilities or underground structures may be in the way, or adjacent property

owner(s) may not give permission to install tiebacks under these properties.

Soil nails are more plentiful, but typically are shorter than tiebacks.

3. A conventional system is not sufficiently flexible to adapt to complex job geometry.
4. Conventional systems typically consist of components that are expensive (long complex tiebacks, long H-piles, wales) compared to the costs of soil nailing components.
5. The mode of failure is sudden without large prior deformations that signal impending failure.
6. After installation, conventional systems are difficult to change to conform to different design parameters.

2.4.5 Advantages of Soil Nailing Systems (Stocker and Riedinger, 1990)

1. The components of these systems are relatively low cost compared to components of conventional retaining systems.
2. Soil nailing systems can be adapted easily to different site conditions and soil profiles.
3. Soil nailing systems can be modified easily in order to change or optimize the initial design during construction.
4. The resultant soil nail structure has greater flexibility than conventional systems and the capacity to withstand larger total and differential settlements.
5. The failure mode is less dangerous (a soil nailing system does not collapse suddenly without large deformation).

6. Right-of-way (easements) requirements are less than for conventional systems, as the nails typically are shorter than the tieback anchors.
7. Installation of soil nails involves smaller equipment and causes little environmental disturbance.
8. The bottom of the wall is placed at the bottom of the excavation with no need for the embedment required for stability of soldier piles.

2.4.6 Disadvantages of Soil Nailing Systems

1. Soil nailing cannot be used in loose granular soil or very soft fine-grained soil that cannot sustain a vertical cut 3 to 6 feet high without support until shotcrete is placed.
2. Reliable drainage systems are difficult to construct because the elements of the drainage system are flexible sheets of fabric placed against exposed soil with little security against movement or blockage.
3. Soil nailing systems exhibit larger lateral deformations and consequent settlement behind excavation faces than do anchored tieback walls (0.2 percent of excavation height for granular soils and 0.4 percent of excavation height for fine-grained soils).
4. Aesthetically, the shotcrete face of a nailed system is not satisfying. Additional measures have to be taken to cover the shotcrete with surface treatments.

5. Soil nailing in very low shear strength soil may require a very high soil nail density.
6. Soil nailing in sensitive soils and expansive soils for permanent long-term applications is not recommended.

2.4.7 Summary on Technical Aspects

The design of a soil nailing system involves selection of the size, type, and spacing of the nails. It also includes selection of a facing and an appropriate drainage system. The design criteria of the nailed soil system include:

1. Stability with respect to the potential failure of the soil, the inclusions (excessive tension or bending), and their interaction with the soil (pullout failure or plastic flow of the soil between the inclusions).
2. Stability with respect to the nail-facing connection for a facing system with only modest strength.
3. Tolerable displacements with respect to expected structure performance.
4. Environmental and architectural aspects.

Soil nailing design requires an evaluation and analysis for every step during which soil nails are installed. The design procedure requires obtaining a factor of safety for each lift by finding maximum tension and maximum shear force on a slip surface. When the excavation is advanced, the analysis for wall stability must be repeated again, for each lift. Therefore, use of a computer program for design is recommended, especially when the wall is inclined, the soil has several layers of different properties, and the ground surface is not horizontal. Several computer

programs are available, such as the SNAIL PROGRAM by the California Department of Transportation, Division of New Technology, Materials and Research, Office of Geotechnical Engineering User's Manual (not dated), and Nail M15 by Dr. S. Bang, South Dakota School of Mines and Technology, User's Manual, (obtained by personal contact). The Author used the Nail M15 Program for design of the systems included in this research, because of experience with excellent analytical results, verified with hand calculations, over a span of several years.

CHAPTER 3

COSTS OF RETENTION SYSTEMS

3.1 Analyzed Projects

Fifteen retention system projects were selected for analysis; those systems have been designed and constructed, using conventional retention systems. Eleven systems were built around the Louisville, Kentucky area; four of these projects were done outside of Louisville. Therefore, the mobilization and demobilization costs were considered, and an out-of-state allowance was included in accounting for the total cost. These projects were analyzed using present worth and current cost approaches for both conventional retention and soil nailing, and several cost comparison studies were done. Appendix B contains design results of conventional systems and Appendix C contains design results for soil nailing systems. Material takeoff calculations are given in Appendix D. Costs for conventional systems are given in detail in Appendix E and costs for soil nailing systems are given in detail in Appendix F.

3.2 Assumptions in Analyses

All conditions on the analyzed sites were assumed to be the same when designing and constructing both systems, conventional and soil nailing, using current

dollar cost for all components. Actual cost figures were available for the conventional systems that had been built, but those costs were calculated again using current unit costs for material, equipment rental and labor.

3.3 Method Of Analysis

To show the steps that have been followed to arrive at the final design for each case, the design steps for case 5 have been selected for illustration as an example, and the other fourteen cases will follow the same procedure. The rationale given in the following paragraphs allows development of a conceptual model of a typical conventional wall.

From experience and PTI recommendations, the length of the bonded zone should not exceed 30 feet, because tiebacks would not get any extra bonded strength even if their length exceeds 30 feet; the tiebacks would fail in tension rather than pull out.

To determine how long the free length will be, two methods are available to define the failure zone or the slip surface: the first method is to draw a line from the bottom of the wall (toe) to the surface with an outer angle of $(45 + \phi/2)$; where ϕ is the angle of internal friction. This line was moved 5 feet into the retained soil. This line defines the boundary between the soil that will move with the wall and the soil that will remain stable, and is taken to be the boundary between the free length and the bonded length. The other method is to draw a logarithmic spiral slip surface based on the soil properties to choose the most critical line that defines the boundary between the moving soil and the soil that will remain stable. By experience, the first method will

give a line located farther from the wall and, thus, more conservative, as shown in Fig. 6 on the next page.

Figure 6. Tieback Bonded Length and Free Length

The total length of tieback is the summation of the bonded length, free length and 5 feet of tie tail, that extends beyond the wall.

To determine tieback vertical spacing, an empirical approach is used. From experience it is known that if the project requires a single tieback, the best tieback location is about (0.35 to 0.4) of the wall height, H , from the top of the wall, but not more than 10 feet. At any greater spacing, a heavy soldier pile section will be required for the initial cantilever stage during construction. Also, a bench two feet deep will be required below the tieback location to allow tieback installation, therefore even if the top cantilever section of the wall is 10 feet, the total height to be supported is 12 feet. If the project requires installation of two tiebacks, the location of the upper tieback will be about (0.275 to 0.3) of the total wall height from top of

the wall, and lower tieback location will be about (0.3 to 0.4) of the total wall height up from the bottom of the wall. These values have been determined from experience. The designer must check the load on the tieback for the situation during construction just before lower tieback installation to check tieback design load, soldier pile section modulus, and soldier pile embedment in the ground to determine the requirements at this intermediate stage, versus the completed stage, for the configuration as shown in Fig. 7 below.

Figure 7. Two-tieback Wall Vertical Spacing

When project design requires installation of three tiebacks, the upper tieback location will be about $(0.2 \text{ to } 0.25) H$ from the top of the wall, with the same spacing for the lower tieback from the bottom of the wall, and the remaining length between upper and lower tiebacks will be divided into two equal segments for midpoint tieback location, as shown in Fig. 8.

Figure 8. Three-tieback Wall Vertical Spacing

When projects design requires installation of four tiebacks, upper tieback location will be at $(0.175 \text{ to } 0.2) H$ and the lower tieback will be the same distance from the bottom of the excavation, and then the spacing between upper and lower tiebacks will be divided into three equal segments for location of the two remaining tiebacks, as shown in Fig. 9.

Figure 9. Four-tieback Wall Vertical Spacing

To determine the required soldier pile section, the designer calculates the maximum moment in the pile at all construction stages and at the final stage, and then calculates the section modulus (S_x) required to limit bending stress in the pile to the appropriate code value. Then, the designer chooses the available steel section that will provide at least that section modulus.

An evaluation for tieback design load needs to be conducted to get an economical design; high tensile strength steel rebar with a diameter of 1-1/4 inch will hold up to 100 kips of design load and 120 kips of the proof load, which experience has shown the author to be adequate for all depths of excavation considered in this study. Therefore, author tried to use 1-1/4 inch-diameter rebar for tiebacks and not to go to 1-3/8 inch-diameter rebar, because the larger rebar will entail higher steel cost and installation cost.

If the design requires a tieback for each bay (soldier pile to soldier pile), then an evaluation needs to be conducted to find which method will be more economic; to install the tieback through pile penetration or to install the tieback through the wale. Usually, if the selected soldier pile has adequate section modulus remaining after cutting a hole through pile flanges to install a tie back through the pile, then this method will be more economical than using a wale.

If the tieback capacity is sufficient to hold more than one bay load (1-1/2 or 2 bays) then using a wale will be the more economic design rather than penetrating each soldier pile for a tieback, because the cost of tiebacks is much higher than the cost of wales.

To illustrate fully these design steps, the situation of Case 5 will be described, where the required walls are 37.5 feet high (Part A) and 26 feet high (Part B).

CASE 5 PART A 37.5 FT WALL DESIGN STEPS:

Step 1: Assume that steel soldier piles will be driven on a base line to a depth of 2 feet below the bottom of the excavation. Then, the excavation will be advanced to a depth about 18 inches to 2 feet below the first tie location. Run design calculation as a cantilever wall to size the system components, as shown on the following page.

Figure 10. Step 1 of Retention System Design Process part A (all dimensions in feet)

CASE 5 PART A STEP 1

H	10
SCH.	300
SPACING	8.5
Ka1=	0.33
Ka2=	0.33
Kp	3
GAMMA 1	120
GAMMA SUB. 2	120
(BEAM WIDTH) b	1

TRY DIFF. VALUE OF D TO MAKE THIS EQ. =0

ASSUME d= 16.2 ZERO SHEAR POINT
D=1.2xd = 19.44 FT
total length= 29.44 FT
use total length = 30 FT

P1	8415.00	←
P2	16830.00	←
P3	34834.54	←
P4	140502.60	→
Y1	5.000	
Y2	3.333	
Y3	9.332	
Y4	10.885	

SEM. OF Fx = 0

3+Pg2-P4 SHOULD = 0

SUM. OF Fx = -80423.064 this value should = 0

also satisfy sum. of moment =0

P1	8415.00
P2	16830.00
P3	34834.54
P4	140502.60

	FORCE	ARM		MOMENT
P1	8415.00	21.20	+	178398.00
P2	16830.00	19.53	+	328746.00
P3	34834.54	6.87	+	239227.81
P4	140502.60	5.32	-	746807.04
				=====
				-435.23

this value (govern) should approx. = 0

find zero shear point

X= 8.755 FT

P1	8415.00	←
P2	16830.00	←
P3,x	14953.95	←
P4,x	40176.01	→

P1+P2+P3,x-P4,x = 22.94 try diff. value of x to make this value close to zero

find Max Moment At X= 8.755 ft

	FORCE	ARM		MOMENT
P1	8415.00	13.755	+	115748.325
P2	16830.00	12.088	+	203446.650
P3,x	14953.95	4.831	+	72237.397
P4,x	40176.01	5.983	-	240370.549
				=====
				151061.82

Max. Moment= 151.06 k-ft

Sx= 48.99 in^3 Gr-50

Figure 11. Case 5 Part A Step 1 Design Calculation

Step 2: In the construction process, the field personnel will drill and install the first tieback, and then advance the excavation to about 18 inches to 2 feet below the second tieback location. Run a design calculation with the first tieback in place, to size the retention system, as shown in the following three figures.

Figure 12. Step 2 of Retention System Design Process part A (all dimensions in feet)

CASE 5 PART A STEP 2

DEPTH OF EXCA	20.75	FT
FRICTION- PHEE	30	DEGREE
SOIL UNIT WEIGHT	120	#/C.F
SURCHARGE S_{cu}	300	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	1	FT
PILE EFF. FACTOR	3	
K_a	0.33	
K_p	3	
TIE LOCATION	8	FT
TIE ANGLE $X=$	30	DEGREE
P_b FACTOR =	22	
ALLOWABLE STRESS =	37	KSI

$\cos X=$	0.8660	
$P_b =$	456.5	PSF
$P_q =$	99	PSF
LENGTH OF upper triangle	4.15	FT
LENGTH OF rectangle	12.45	FT

FORCE

6732
8051.51875
14938.9625
10729.125
33370.15
8051.51875
16813.656
4585.086
616.50072

PILE PENETR. $D=$	5.58	FT
PAS.PRESSUp1=	6026.4	PSF
ACT.PRESSUp1=	821.7	PSF
ACT.PRESSUp2=	220.968	PSF
DIST.FROM DRE- DGE LINE TO TIE	12.75	FT

try diff.value of D

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

ARM		MOMENT
4	+	26928
5.2195	+	42024.90212
1.925	+	28757.50281
6.375	-	68398.17188
4.3	-	143491.645
9.9695	-	80269.61618
16.4328	+	276295.4463
15.54	-	71252.23644
16.4328	-	10130.83303
		=====

TOTAL MOMENT@TIE LOCATION= 463.3 (THIS VALUE SHOULD APPROX.=0)

the (D)should be=	6.70	FT
used		
USE $D=$	18.00	FT
$F.S=$	3.23	HENCE OK
TOTAL LENGTH=D+H=	38.75	FT

Figure 13. Case 5 part A step 2 design calculation

TO FIND TIEBACK FORCE		
MAKE SUM. FOR Fx=0		
F1=	17461.125	+
F2=	16103.0375	+
F3=	48309.1125	+
F4=	4585.086	+
F5=	16813.656	-
F6=	616.50072	+
Fx/ONE BAY=	70.26	KIP
Fx/ONE FT=	8.27	KIP/FT
Fx/ BAY =	70.26	KIP
TIE FORCE/1 BAY=	81.13	KIP
PROOFTEST LOAD=	97.36	KIP

FIND MAX. MOMENT

1 @TIEBACK LOCATION = -8 FT

FORCE
6732
8051.51875
14938.9625

ARM
4
5.2195
1.925

MOMENT
26928.00
42024.90
28757.50

MAX. MOMENT= 97.71 KIP.FT

2 @ X-DISTANCE FROM
THE TOP OF THE PILE

FIND X WHEN SUM.OF Fx =0

(ZERO SHEAR POINT)

X= 16.58553 FT

FORCE
13956.72
70261.21
8051.52
48252.96

ARM
8.2927648
8.5855296
13.80503
6.2177648

MOMENT
115739.82
603229.66
111151.45
300025.58

MAX. MOMENT= 76.31 KIP.FT

USE MAX. MOMENT= 97.71 KIP.FT
MOD. SECTION = 31.69 CUB. IN (Gr-50)

Figure 14. Case 5 part A step 2 design calculation (cont.)

Step 3: In the construction process, the field crew will drill and install the second tieback, and then advance the excavation to about 18 inches to 2 feet below the third tieback location. Run a design calculation with two tiebacks in place to size the retention system.

Figure 15. Step 3 of Retention System Design Process part A (all dimensions in feet)

H=	31.5	FT
S=	8.5	FT
SCH	300	PSF
TIE 1@	8	FT
TIE 2@	18.5	FT
TIE @	30	DEGREE
Ka=	0.33	FT
Pb FACTOR =	22	
upper triangle =	6.3	ft
lower triangle=	6.3	ft

Fx1= 70.64 KIP/FT
Fx2= 104.30 KIP/FT

DL1= 81.57 KIP/FT
DL2= 120.44 KIP/FT

PROOF TIE 1 =	98	KIPS/FT
PROOF TIE 2 =	145	KIPS/FT

<u>FORCE</u>	<u>ARM</u>		<u>MOMENT</u>
18.56	3.80	+	70.51
6.73	4.00	+	26.93
10.01	0.85	+	8.51
			=====
			105.95

<u>FORCE</u>	<u>ARM</u>		<u>MOMENT</u>
5.30	10.10	+	53.54
18.56	9.05	+	167.92
48.79	3.48	+	168.59
70.64	5.25	-	370.88
			=====
			13.17

Figure 16. Case 5 part A step 3 Design Calculation 1st run

The resultant design is not acceptable, because the second tieback design load will exceed 80 percent of the ultimate allowable load for (1¼-inch-diameter) reinforcing bar per PTI recommendations. If (1 3/8-inch-diameter) rebar were used that will increase the cost of tiebacks; therefore, redesign the system to keep the load in the second tiebacks around 100 kips (the proof load will be about 120 kips, or 120 percent of the design load). To do so, rerun the design calculation. Assume the surcharge load during construction will be limited to about 50 pounds per square foot rather than 300 pounds per square foot until the proof load test was performed, and

assume the tieback load will be locked at 75 percent of the design load. On the final stage design, use the normal surcharge of 300 pounds per square foot.

CASE 5 PART A STEP 3

TO KEEP 2nd TIE ABOUT 100 KIPS TO USE 1.25 INCH REBAR
CONTROL SURCHARGE DURING CONSTRUCTION

H=	31	FT
S=	8.5	FT
SCH	50	PSF
TIE 1@	8	FT
TIE 2@	18.5	FT
TIE @	30	DEGREE
Ka=	0.33	FT
Pb FACTOR =	22	
upper triangle =	6.2	ft
lower triangle =	6.2	ft

W= 5.93725 K/1 VERTICAL FT

Fx1= 60.70 KIP/FT

Fx2= 87.42 KIP/FT

COS X= 0.8660255

DL1= 70.09 KIP/FT

DL2= 100.94 KIP/FT

OK

PROOF LOAD 1.2 X DL

PROOF TIE 1 = 84 KIPS/FT

PROOF TIE 2 = 121 KIPS/FT

MAX MOMENT 1 @ TIE 1 LOCATION

FORCE	ARM		MOMENT
17.97	3.87	+	69.49
1.12	4.00	+	4.49
10.43	0.90	+	9.39
			=====
			83.37

FIND ZERO SHEAR POINT

X IS BELOW 0.2 H X= 7.05 FT

FORCE	ARM		MOMENT
0.87	10.15	+	8.83
17.97	9.12	+	163.83
41.86	3.53	+	147.55
60.70	5.25	-	318.66
			=====
			1.54

USE MAX. MOMENT = 83.37 FT-KIP

Sx= 27.04 IN³/FT Gr-50

Figure 17. Case 5 part A step 3 Design Calculation 2nd run

Step 4: Field crew will drill and install the third tieback, and then advance the excavation to the planned bottom elevation as the final stage. Run a design calculation for the final stage and use the design that will satisfy the final stage condition and conditions during construction.

Figure 18. Step 4 of Retention System Design Process part A (all dimensions in feet)

CASE 5 PART A STEP 4 (FINAL)

H=	37.5	FT
S=	8.5	FT
SCH	300	PSF
TIE 1@	8	FT
TIE 2@	18.75	FT
TIE 3@	29.5	FT
Ka=	0.33	FT
TIE @	30	DEGREE
Pb FACTOR =	22	
upper triangle =	7.5	ft
lower triangle =	7.5	ft

W= 7.854 K/1 VERTICAL FT

Fx1= 78.75 KIPS
Fx2= 84.43 KIPS
Fx3= 78.75 KIPS

COS X= 0.8660255

DL1= 90.93 KIP
DL2= 97.49 KIP
DL3= 90.93 KIP

PROOF LOAD 1.2 X DL

PROOF TIE 1 = 109.12 KIPS
PROOF TIE 2 = 116.99 KIPS
PROOF TIE 3 = 109.12 KIPS

MAX MOMENT 1 @ TIE 1 LOCATION

FORCE
6.73
29.92

ARM
4.00
2.67

MOMENT
26.93
79.79
=====

X IS BELOW 0.2 H X1= 5.88 FT

FORCE
6.31
26.30
46.14
78.75

ARM
9.63
8.38
2.94
5.38

MOMENT
60.75
220.24
135.54
423.28
=====

MAX MOMENT 2 @ TIE 2 LOCATION

FORCE
6.31
26.30
88.36
78.75

ARM
15
13.75
5.625
10.75

MOMENT
94.67
361.58
497.01
846.57
=====

MAX MOMENT 3 BETWEEN 2nd AND 3rd TIE

X2 IS BELOW 0.2 H X2= 16.63

FORCE
6.31
26.30
130.57
78.75
84.43

ARM
20.38
19.13
8.31
16.13
5.38

MOMENT
128.59
502.93
1085.39
1269.85
453.81
=====

USE MAX MOM = 106.71 FT-KIP

Sx= 38.81 IN^3 Gr-50

Figure 19. Case 5 part A step 4 Design Calculation

For case 5 part B with 26 feet depth, the same steps will be followed.

Step 1: Assume steel soldier piles will be driven on a base line to the designed depth (2 feet below bottom of excavation), and then the excavation will be advanced to a depth about 18 inches to 2 feet below the first tie location. Run a design calculation for a cantilever wall to size the system components.

Figure 20. Step 1 of Retention System Design Process part B (all dimensions in feet)

CASE 5 PART B STEP 1

Input Data

H	9.5
SCH.	300
SPACING	8.5
Ka1=	0.33
Ka2=	0.33
Kp	3
GAMMA 1	120
GAMMA SUB. 2	120
(BEAM WIDTH) b	0.83

TRY DIFF. VALUE OF D TO MAKE THIS EQ. =0

ASSUME d= 16.63 ZERO SHEAR POINT
D=1.2xd = 19.96 FT
total length= 29.46 FT
use total length = 30 FT

P1	7994.25	←
P2	15189.08	←
P3	29212.76	←
P4	123258.08	→

Y1	4.750
Y2	3.167
Y3	9.635
Y4	11.144

SEM. OF Fx = 0

P1+P2+P3+Pq2-P4 SHOULD = 0

SUM. OF Fx = -70861.9952 this value should = 0

also satisfy sum. of moment =0

P1	7994.25
P2	15189.08
P3	29212.76
P4	123258.08

	FORCE	ARM	MOMENT
P1	7994.25	21.38	+ 170917.07
P2	15189.08	19.80	+ 300693.05
P3	29212.76	7.00	+ 204357.14
P4	123258.08	5.49	- 676135.11
			=====
			-167.84

this value (govern) should approx. = 0

find zero shear point

X= 9 FT

P1	7994.25	←
P2	15189.08	←
P3,x	12424.10	←
P4,x	35609.48	→

1+P2+P3,x-P4,x = -2.05 try diff. value of x to make this value close to zero

id Max Moment At X= 9 ft

	FORCE	ARM	MOMENT
P1	7994.25	13.750	+ 109920.938
P2	15189.08	12.167	+ 184800.413
P3,x	12424.10	4.992	+ 62018.465
P4,x	35609.48	6.101	- 217248.581
			=====
			139491.23

Max. Moment= 139.49 k-ft

Sx= 45.24 in^3 Gr-50

Figure 21. Case 5 part B step 1 Design Calculation

Step 2: The field crew will drill and install the first tieback, and then advance the excavation to a depth about 18 inches to 2 feet below the second tieback location. Run a design calculation with a single tieback in place to size the retention system.

Figure 22. Step 2 of Retention System Design Process part B (all dimensions in feet)

CASE 5 PART B STEP 2

DEPTH OF EXCA	20.5	FT
FRICTION- PHEE	30	DEGREE
SOIL UNIT WEIGHT	120	#/C.F
SURCHARGE S _{cu}	300	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	1	FT
PILE EFF. FACTOR	3	
K _a	0.33	
K _p	3	
TIE LOCATION	7.5	FT
TIE ANGLE X=	30	DEGREE
P _b FACTOR =	22	
ALLOWABLE STRESS =	37	KSI

COS X=	0.8660	
P _b =	451	PSF
P _q =	99	PSF
LENGTH OFupper triangle	4.1	FT
LENGTH OF rectangle	12.3	FT

FORCE

6311.25
7858.675
13033.9
10939.5
34118.15
7858.675
18228.294
4716.558
668.37078

PILE PENETR. D=	5.81	FT
PAS.PRESSUp1=	6274.8	PSF
ACT.PRESSUp1=	811.8	PSF
ACT.PRESSUp2=	230.076	PSF
DIST.FROM DRE- DGE LINE TO TIE	13	FT

try diff.value of D

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

ARM		MOMENT
3.75	+	23667.1875
4.753	+	37352.28228
1.7	+	22157.63
6.5	-	71108.75
4.45	-	151825.7675
10.253	-	80574.99478
16.8346	+	306866.0382
15.905	-	75016.85499
16.8346	-	11251.75473

TOTAL MOMENT@TIE LOCATION= 267.0 (THIS VALUE SHOULD APPROX.=0)

the (D)should be=	6.97	FT
used		
USE D=	7.00	FT
F.S=	1.20	HENCE OK
TOTAL LENGTH=D+H=	27.50	FT

Figure 23. Case 5 part B step 2 Design Calculation

TO FIND TIEBACK FORCE
MAKE SUM. FOR $F_x=0$

F1=	17250.75	+
F2=	15717.35	+
F3=	47152.05	+
F4=	4716.558	+
F5=	18228.294	-
F6=	668.37078	+

F_x /ONE BAY=	67.28	KIP
F_x /ONE FT=	7.91	KIP/FT

F_x / BAY =	67.28	KIP
TIE FORCE/1 BAY=	77.68	KIP
PROOFTEST LOAD=	93.22	KIP

FIND MAX. MOMENT

1 @TIEBACK LOCATION = -7.5 FT

FORCE
6311.25
7858.675
13033.9

ARM
3.75
4.753
1.7

MOMENT
23667.19
37352.28
22157.63

MAX. MOMENT= 83.18 KIP.FT

2 @ X-DISTANCE FROM
THE TOP OF THE PILE

FIND X WHEN SUM.OF $F_x=0$

(ZERO SHEAR POINT)

X= 16.071756 FT

FORCE
13524.38
67276.78
7858.68
45893.73

ARM
8.0358781
8.5717561
13.324756
5.9858781

MOMENT
108680.29
576680.19
104714.93
274714.25

MAX. MOMENT= 88.57 KIP.FT

USE MAX. MOMENT= 88.57 KIP.FT
MOD. SECTION = 28.73 CUB. IN (Gr-50)

Figure 24. Case 5 part B step 2 Design Calculation (cont.)

Step 3: The field crew will drill and install the second tieback, and then advance the excavation to the planned bottom elevation as the final stage. Run a design calculation for the final stage, and then use the design that will satisfy the final stage conditions and conditions during construction.

Figure 25. Step 3 Final Stage of Design Process part B (all dimensions in feet)

CASE 5 PART B STEP 3 (FINAL)

H=	26	FT
S=	8.5	FT
SCH	300	PSF
TIE 1 @	7.5	FT
TIE 2 @	18.5	FT
TIE @	30	DEGREE
Ka=	0.33	FT
Pb FACTOR =	22	
upper triangle =	5.2	ft
lower triangle =	5.2	ft

W= 5.7035 K/1 VERTICAL FT

Fx1= 61.50 KIP/FT
Fx2= 61.50 KIP/FT

COS X= 0.8660255

DL1= 71.02 KIP
DL2= 71.02 KIP

PROOF LOAD 1.2 X DL

PROOF TIE 1 =	85	KIP	1.5	128	KIP
PROOF TIE 2 =	85	KIP	1.5	128	KIP

	FORCE	ARM		MOMENT
MAX MOMENT 1 @ TIE 1 LOCATION	12.64	4.03	+	50.99
	6.31	3.75	+	23.67
	11.18	1.15	+	12.86
				=====
				87.51

FIND ZERO SHEAR POINT

X IS BELOW 0.2 H X= 7.80 FT

FORCE	ARM		MOMENT
4.38	10.40	+	45.51
12.64	9.53	+	120.51
44.49	3.90	+	173.50
61.50	5.50	-	338.27
			=====
			1.25

USE MAX. MOMENT = 87.51 FT-KIP
Sx= 28.38 IN³/FT Gr-50

Figure 26. Case 5 part B step 3 Design Calculation

The next step will be the take off for all the components that are required.

Table 2. Case 5 Conventional System Materials Takeoff

CONVENTIONAL SYSTEM MATERIALS TAKEOFF									
AREA=	24,318	Sq ft				FT	Each	Total ft	Total steel pounds
					HP 12X53	30	42	1,260.0	
TIE BACK	190	Each	1.25" rebar						
					pound		===	=====	=====
WALES	3	Each	10 ft		53		42	1,260.0	66,780.0
WALES	30	Each	20 ft						
12x53 Beam Penetration	130	Each			HP 10x42	10	1	10	
					HP 10x42	25	2	50	
					HP 10x42	33	1	33	
					HP 10x42	28	46	1288	
					HP 10x42	20	1	20	
					HP 10x42	6.5	1	6.5	
					pound		=====	=====	
					42		52.00	1,407.5	59,115.0
									=====
									125,895.00

Area is the total retained area in square feet.

Pile length is the total length of the soldier piles, in feet.

Total pounds are the weight of the soldier piles in pounds.

Soldier piles used in this case are steel piles 10x42 and steel piles 12x53.

Wales are the double c-section steel channels through which tiebacks are inserted and which rest on soldier piles. The length of wale is determined by the support condition (if the wale is used to hold one bay (soldier pile to soldier pile) or used to hold two bays (three consecutive soldier piles)).

Beam penetration is the operation of punching a hole through both flanges of the soldier piles to install tiebacks through soldier piles.

Then the next step is to calculate the cost of the project as follows:

Table 3. Case 5 Conventional System Materials and Operation Cost

	<u>rate \$</u>	<u>per</u>	<u>unit</u>	<u>rate x unit</u>	<u>\$/sft</u>
driving in earth	10.00	ft	2,667.50	26,675.00	1.10
steel cost beam	0.52	lb	125,895.0	65,465.40	2.69
wale 10' long	450.00	ea.	3	1,350.00	0.06
wale 20' long	550.00	ea.	30	16,500.00	0.68
wale 10' uninstal	200.00	ea.	3	600.00	0.02
wale 20' uninstal	250.00	ea.	30	7,500.00	0.31
tieback cost	1,100.00	ea.	190	209,000.00	8.59
pent. throu. 10" beam	100.00	ea.	0	0.00	0.00
pent. throu. 12" beam	125.00	ea.	130	16,250.00	0.67
wood lagging down to 1 st tie	4.10	sft	6,309.0	25,866.90	1.06
wood lagging below	6.76	sft	18,009.0	121,740.84	5.01
mob/demob + shop dwg	1.00	ea.	10,000.00	10,000.00	0.41
OTA	100.00	M / DAY	0	0.00	0.00
set pile in hole	5.00	\$/ft	2,667.50	13,337.50	0.55
				=====	=====
				514,285.64	21.15
Area =	24,318	sft	Cost =	21.15	\$/sft

Then the same procedure will be followed to calculate the cost for the soil nailing system. Experience and research have developed typical parameters for soil nailing that allow development of a conceptual model of a typical soil nailing wall.

Usually the soil nailing wall is inclined about 5 degrees from the vertical plane, toward the retained soil. The first nail row will be about 2 feet from the top of the wall. Vertical and horizontal spacing will be on the order of 4 to 6 feet each. Uppermost nail length is about 80 to 100 percent of the wall height, H, while lowest

nail length is about 60 to 80 percent of the wall height and the length of nails between uppermost and lowest will be linearly proportioned between those two lengths. Nail steel rebar is usually No. 8 low strength (75 ksi); the nails are inserted in a drilled hole four inches in diameter and grouted with low pressure. The holes are installed at an angle of about 20 degrees from the horizontal plane.

The desired safety factor for sliding through the (bottom of the excavation) is 1.5 as a minimum, and 2.0 for deep-seated rotation (global failure). Permanent retention of the soil face between nails is accomplished by installing No. 4 steel rebar horizontally and vertically as a wale or spreader mat behind a nail plate. A drainage blanket will be installed between nails vertically from the top of the wall toward the bottom of excavation. Fig. 27 shows a typical section through a soil nailing wall. Soil nailing design parameters that could be changed to get the optimal design are: steel nail parameters (length, size, strength), drilled-hole diameters, vertical and horizontal spacing of nails, and nail inclination angle from horizontal plane. Also, surcharge load can be changed during design of intermediate stages in construction if it could be controlled.

Figure 27. Typical Soil Nailing Cross Section

Case 5 soil nailing design procedure:

Step 1: Run design calculation for part A, with a wall height of 37.5 feet, and try different nail patterns (patterns commonly used are spacing vertically and horizontally between 4 feet by 4 feet and 6 feet by 6 feet. In this case, a pattern of 6 feet by 6 feet will be used and the length of the upper nail was assumed to be 25 ft and the lower nail length was assumed to be 20 ft, based on experience.

* CASE 5 A 1ST RUN 6'X6'
 < OUTPUT FILE > = CAS5A1ST.OUT

INPUT DATA

SOIL PROPERTIES

FOUNDATION	SOIL UNIT WEIGHT	=	125.000 pcf
	COHESION OF SOIL	=	1500.000 psf
	FRICTION ANGLE OF SOIL	=	25.000 deg
	UNDRAINED SU AT EXCAVATION LEVEL	=	3000.000 psf
	DEPTH TO HARDPAN FROM EXCAVATION	=	40.000 ft
	SU AT THE TOP OF HARDPAN	=	5000.000 psf
LAYER 1	HEIGHT TO TOP OF LAYER FROM TOE	=	16.000 ft
	SOIL UNIT WEIGHT	=	125.000 pcf
	COHESION OF SOIL	=	1500.000 psf
	FRICTION ANGLE OF SOIL	=	25.000 deg
LAYER 2	HEIGHT TO TOP OF LAYER FROM TOE	=	26.000 ft
	SOIL UNIT WEIGHT	=	120.000 pcf
	COHESION OF SOIL	=	1000.000 psf
	FRICTION ANGLE OF SOIL	=	30.000 deg
LAYER 3	HEIGHT TO TOP OF LAYER FROM TOE	=	37.500 ft
	SOIL UNIT WEIGHT	=	115.000 pcf
	COHESION OF SOIL	=	500.000 psf
	FRICTION ANGLE OF SOIL	=	30.000 deg

WALL & LOAD CONDITION

WALL INCLINATION ANGLE	=	5.000 deg
SURCHARGE LOAD	=	300.000 psf
HOR. ACCELERATION COEFF.	=	.000

NAIL PROPERTIES

CROSS SECTIONAL AREA OF NAIL	=	.790 si
GROUT DIAMETER	=	4.000 in
YIELD STRENGTH OF NAIL	=	60.000 ksi
NAIL INCLINATION ANGLE	=	20.000 deg
NAIL SKEW ANGLE AT SURFACE	=	.000 deg
DEPTH TO UPPERMOST NAIL	=	2.000 ft
VER. SPACING OF NAIL	=	6.000 ft
HOR. SPACING OF NAIL	=	6.000 ft
TOTAL LENGTH OF UPPERMOST NAIL	=	25.000 ft
TOTAL LENGTH OF LOWERMOST NAIL	=	20.000 ft
TOTAL UNBONDED LENGTH OF NAIL	=	2.000 ft

Figure 28. Case 5 part A Soil Nailing Calculation 1st run

O U T P U T R E S U L T S

I=	2	A=	.133	F.S.=	4.495	T=	11184.2900	lb/ft
I=	3	A=	.200	F.S.=	2.813	T=	15993.8400	lb/ft
I=	4	A=	.267	F.S.=	2.215	T=	18200.2800	lb/ft
I=	5	A=	.333	F.S.=	1.910	T=	18989.5800	lb/ft
I=	6	A=	.400	F.S.=	1.731	T=	18868.8800	lb/ft
I=	7	A=	.467	F.S.=	1.615	T=	18113.5700	lb/ft
I=	8	A=	.533	F.S.=	1.539	T=	16945.2800	lb/ft
I=	9	A=	.600	F.S.=	1.492	T=	15501.8300	lb/ft
I=	10	A=	.667	F.S.=	1.461	T=	13892.5600	lb/ft
I=	11	A=	.733	F.S.=	1.447	T=	12168.1600	lb/ft
I=	12	A=	.800	F.S.=	1.444	T=	10400.0600	lb/ft
I=	13	A=	.867	F.S.=	1.458	T=	8694.9650	lb/ft
I=	14	A=	.933	F.S.=	1.487	T=	7359.0290	lb/ft
I=	15	A=	1.000	F.S.=	1.518	T=	6300.6500	lb/ft
I=	16	A=	1.067	F.S.=	1.551	T=	5532.0060	lb/ft
I=	17	A=	1.133	F.S.=	1.585	T=	4815.3470	lb/ft
I=	18	A=	1.200	F.S.=	1.625	T=	4349.9400	lb/ft
I=	19	A=	1.267	F.S.=	1.664	T=	3979.5310	lb/ft
I=	20	A=	1.333	F.S.=	1.703	T=	3635.9900	lb/ft
I=	21	A=	1.400	F.S.=	1.742	T=	3317.1270	lb/ft
I=	22	A=	1.467	F.S.=	1.783	T=	3021.0720	lb/ft
I=	23	A=	1.533	F.S.=	1.824	T=	2746.1790	lb/ft
I=	24	A=	1.600	F.S.=	1.867	T=	2490.9690	lb/ft
I=	25	A=	1.667	F.S.=	1.911	T=	2324.8800	lb/ft
I=	26	A=	1.733	F.S.=	1.957	T=	2225.7320	lb/ft
I=	27	A=	1.800	F.S.=	2.004	T=	2133.0070	lb/ft
I=	28	A=	1.867	F.S.=	2.051	T=	2046.2080	lb/ft
I=	29	A=	1.933	F.S.=	2.100	T=	1964.8910	lb/ft
I=	30	A=	2.000	F.S.=	2.148	T=	1888.6440	lb/ft

H/LT= 1.500 H/LB= 1.875

NO. OF NAILS (INCLUDING THE LOWERMOST) = 6

F.S. AGAINST SLIDING THROUGH TOE = 1.44 **NOT GOOD < 1.5

**F.S. AGAINST DEEP SEATED ROTATION = 4.32

1 TH NAIL FORCE =	136.9021	lb/ft
2 TH NAIL FORCE =	476.3413	lb/ft
3 TH NAIL FORCE =	954.5281	lb/ft
4 TH NAIL FORCE =	1724.9260	lb/ft
5 TH NAIL FORCE =	2749.4870	lb/ft
6 TH NAIL FORCE =	4357.8750	lb/ft

Figure 28. Case 5 part A Soil Nailing Calculation 1st run (cont.)

Step 2: Rerun design calculation and change the nail properties; the upper nail length will be 30 ft and the lower nail length will be 20 ft, but keep the nail pattern as 6 ft by 6ft. Results are shown in Fig. 29.

```

CASE 5A 2ND RUN 6ft x 6ft
< OUTPUT FILE > = CASE5A2ndRUN.OUT
***I N P U T   D A T A***

S O I L   P R O P E R T I E S

FOUNDATION   SOIL UNIT WEIGHT           =      125.000 pcf
              COHESION OF SOIL           =      1500.000 psf
              FRICTION ANGLE OF SOIL      =       25.000 deg
              UNDRAINED SU AT EXCAVATION LEVEL =    3000.000 psf
              DEPTH TO HARDPAN FROM EXCAVATION =    40.000 ft
              SU AT THE TOP OF HARDPAN     =    5000.000 psf

LAYER   1     HEIGHT TO TOP OF LAYER FROM TOE =    16.000 ft
              SOIL UNIT WEIGHT           =      125.000 pcf
              COHESION OF SOIL           =      1500.000 psf
              FRICTION ANGLE OF SOIL      =       25.000 deg

LAYER   2     HEIGHT TO TOP OF LAYER FROM TOE =    26.000 ft
              SOIL UNIT WEIGHT           =      120.000 pcf
              COHESION OF SOIL           =      1000.000 psf
              FRICTION ANGLE OF SOIL      =       30.000 deg

LAYER   3     HEIGHT TO TOP OF LAYER FROM TOE =    37.500 ft
              SOIL UNIT WEIGHT           =      115.000 pcf
              COHESION OF SOIL           =       500.000 psf
              FRICTION ANGLE OF SOIL      =       30.000 deg

W A L L   &   L O A D   C O N D I T I O N

              WALL INCLINATION ANGLE      =       5.000 deg
              SURCHARGE LOAD              =      300.000 psf
              HOR. ACCELERATION COEFF.    =       .000

N A I L   P R O P E R T I E S

              CROSS SECTIONAL AREA OF NAIL =       .790 si
              GROUT DIAMETER              =       4.000 in
              YIELD STRENGTH OF NAIL      =      60.000 ksi
              NAIL INCLINATION ANGLE      =      20.000 deg
              NAIL SKEW ANGLE AT SURFACE  =       .000 deg
              DEPTH TO UPPERMOST NAIL     =       2.000 ft
              VER. SPACING OF NAIL        =       6.000 ft
              HOR. SPACING OF NAIL        =       6.000 ft
              TOTAL LENGTH OF UPPERMOST NAIL =    30.000 ft
              TOTAL LENGTH OF LOWERMOST NAIL =    20.000 ft
              TOTAL UNBONDED LENGTH OF NAIL =     2.000 ft

```

Figure 29. Case 5 part A Soil Nailing Calculation 2nd run

O U T P U T R E S U L T S

I=	2	A=	.133	F.S.=	4.437	T=	12660.2600	lb/ft
I=	3	A=	.200	F.S.=	2.826	T=	17933.9200	lb/ft
I=	4	A=	.267	F.S.=	2.248	T=	20408.6800	lb/ft
I=	5	A=	.333	F.S.=	1.949	T=	21413.1600	lb/ft
I=	6	A=	.400	F.S.=	1.773	T=	21463.4100	lb/ft
I=	7	A=	.467	F.S.=	1.661	T=	20819.1600	lb/ft
I=	8	A=	.533	F.S.=	1.588	T=	19762.9300	lb/ft
I=	9	A=	.600	F.S.=	1.541	T=	18412.4800	lb/ft
I=	10	A=	.667	F.S.=	1.513	T=	16870.5300	lb/ft
I=	11	A=	.733	F.S.=	1.497	T=	15221.9900	lb/ft
I=	12	A=	.800	F.S.=	1.494	T=	13501.0600	lb/ft
I=	13	A=	.867	F.S.=	1.500	T=	11764.9500	lb/ft
I=	14	A=	.933	F.S.=	1.511	T=	10039.9700	lb/ft
I=	15	A=	1.000	F.S.=	1.527	T=	8253.2720	lb/ft
I=	16	A=	1.067	F.S.=	1.561	T=	6749.6450	lb/ft
I=	17	A=	1.133	F.S.=	1.602	T=	5671.0580	lb/ft
I=	18	A=	1.200	F.S.=	1.646	T=	4969.5170	lb/ft
I=	19	A=	1.267	F.S.=	1.689	T=	4324.7430	lb/ft
I=	20	A=	1.333	F.S.=	1.736	T=	3876.8120	lb/ft
I=	21	A=	1.400	F.S.=	1.783	T=	3547.9620	lb/ft
I=	22	A=	1.467	F.S.=	1.828	T=	3245.5210	lb/ft
I=	23	A=	1.533	F.S.=	1.874	T=	2966.6840	lb/ft
I=	24	A=	1.600	F.S.=	1.919	T=	2709.1320	lb/ft
I=	25	A=	1.667	F.S.=	1.965	T=	2470.9180	lb/ft
I=	26	A=	1.733	F.S.=	2.010	T=	2250.3700	lb/ft
I=	27	A=	1.800	F.S.=	2.057	T=	2086.4750	lb/ft
I=	28	A=	1.867	F.S.=	2.106	T=	2001.4090	lb/ft
I=	29	A=	1.933	F.S.=	2.155	T=	1921.9540	lb/ft
I=	30	A=	2.000	F.S.=	2.205	T=	1847.6220	lb/ft

H/LT= 1.250 H/LB= 1.875

NO. OF NAILS (INCLUDING THE LOWERMOST) = 6

**F.S. AGAINST SLIDING THROUGH TOE = 1.49 still < 1.5

**F.S. AGAINST DEEP SEATED ROTATION = 4.32

1 TH NAIL FORCE =	893.6461	lb/ft
2 TH NAIL FORCE =	1314.5640	lb/ft
3 TH NAIL FORCE =	1750.4500	lb/ft
4 TH NAIL FORCE =	2288.8750	lb/ft
5 TH NAIL FORCE =	3012.4140	lb/ft
6 TH NAIL FORCE =	4241.113	lb/ft

Figure 29. Case 5 part A Soil Nailing Calculation 2nd run (cont.)

Step 3: Rerun design calculation to get the factor of safety for sliding through toe equal to 1.5 or greater. Change nail pattern to 5.5 feet by 5 feet and keep upper nail length 30 ft and lower nail 20 ft.

CASE 5 PART A 37.5 FT WALL

INPUT DATA

SOIL PROPERTIES

FOUNDATION	SOIL UNIT WEIGHT	=	125.000 pcf
	COHESION OF SOIL	=	1500.000 psf
	FRICTION ANGLE OF SOIL	=	25.000 deg
	UNDRAINED SU AT EXCAVATION LEVEL	=	3000.000 psf
	DEPTH TO HARDPAN FROM EXCAVATION	=	40.000 ft
	SU AT THE TOP OF HARDPAN	=	5000.000 psf
LAYER 1	HEIGHT TO TOP OF LAYER FROM TOE	=	16.000 ft
	SOIL UNIT WEIGHT	=	125.000 pcf
	COHESION OF SOIL	=	1500.000 psf
	FRICTION ANGLE OF SOIL	=	25.000 deg
LAYER 2	HEIGHT TO TOP OF LAYER FROM TOE	=	26.000 ft
	SOIL UNIT WEIGHT	=	120.000 pcf
	COHESION OF SOIL	=	1000.000 psf
	FRICTION ANGLE OF SOIL	=	30.000 deg
LAYER 3	HEIGHT TO TOP OF LAYER FROM TOE	=	37.500 ft
	SOIL UNIT WEIGHT	=	115.000 pcf
	COHESION OF SOIL	=	500.000 psf
	FRICTION ANGLE OF SOIL	=	30.000 deg

WALL & LOAD CONDITION

WALL INCLINATION ANGLE	=	5.000 deg
SURCHARGE LOAD	=	300.000 psf
HOR. ACCELERATION COEFF.	=	.000

NAIL PROPERTIES

CROSS SECTIONAL AREA OF NAIL	=	.790 si
GROUT DIAMETER	=	4.000 in
YIELD STRENGTH OF NAIL	=	60.000 ksi
NAIL INCLINATION ANGLE	=	20.000 deg
NAIL SKEW ANGLE AT SURFACE	=	.000 deg
DEPTH TO UPPERMOST NAIL	=	2.000 ft
VER. SPACING OF NAIL	=	5.500 ft
HOR. SPACING OF NAIL	=	5.000 ft
TOTAL LENGTH OF UPPERMOST NAIL	=	30.000 ft
TOTAL LENGTH OF LOWERMOST NAIL	=	25.000 ft
TOTAL UNBONDED LENGTH OF NAIL	=	2.000 ft

Figure 30. Case 5 part A Soil Nailing Calculation 3rd run

O U T P U T R E S U L T S

I=	2	A=	.133	F.S.=	4.102	T=	22879.6900	lb/ft
I=	3	A=	.200	F.S.=	2.896	T=	29925.6400	lb/ft
I=	4	A=	.267	F.S.=	2.421	T=	33272.0000	lb/ft
I=	5	A=	.333	F.S.=	2.166	T=	34650.8500	lb/ft
I=	6	A=	.400	F.S.=	2.007	T=	34955.6000	lb/ft
I=	7	A=	.467	F.S.=	1.903	T=	34417.8600	lb/ft
I=	8	A=	.533	F.S.=	1.835	T=	33306.4300	lb/ft
I=	9	A=	.600	F.S.=	1.787	T=	31881.5700	lb/ft
I=	10	A=	.667	F.S.=	1.756	T=	30204.3800	lb/ft
I=	11	A=	.733	F.S.=	1.738	T=	28361.3900	lb/ft
I=	12	A=	.800	F.S.=	1.729	T=	26433.6400	lb/ft
I=	13	A=	.867	F.S.=	1.729	T=	24424.6300	lb/ft
I=	14	A=	.933	F.S.=	1.736	T=	22403.3800	lb/ft
I=	15	A=	1.000	F.S.=	1.746	T=	20322.9200	lb/ft
I=	16	A=	1.067	F.S.=	1.777	T=	18370.5800	lb/ft
I=	17	A=	1.133	F.S.=	1.815	T=	16731.5800	lb/ft
I=	18	A=	1.200	F.S.=	1.851	T=	15343.4200	lb/ft
I=	19	A=	1.267	F.S.=	1.889	T=	14258.7000	lb/ft
I=	20	A=	1.333	F.S.=	1.929	T=	13332.1200	lb/ft
I=	21	A=	1.400	F.S.=	1.967	T=	12469.8500	lb/ft
I=	22	A=	1.467	F.S.=	2.007	T=	11722.2600	lb/ft
I=	23	A=	1.533	F.S.=	2.049	T=	11189.4900	lb/ft
I=	24	A=	1.600	F.S.=	2.092	T=	10690.9700	lb/ft
I=	25	A=	1.667	F.S.=	2.134	T=	10223.0400	lb/ft
I=	26	A=	1.733	F.S.=	2.177	T=	9782.7750	lb/ft
I=	27	A=	1.800	F.S.=	2.220	T=	9367.7940	lb/ft
I=	28	A=	1.867	F.S.=	2.264	T=	8976.1250	lb/ft
I=	29	A=	1.933	F.S.=	2.308	T=	8606.0970	lb/ft
I=	30	A=	2.000	F.S.=	2.353	T=	8256.2400	lb/ft

H/LT= 1.250 H/LB= 1.500

NO. OF NAILS (INCLUDING THE LOWERMOST) = 7

**F.S. AGAINST SLIDING THROUGH TOE = 1.73 good > 1.5

**F.S. AGAINST DEEP SEATED ROTATION = 4.32

1 TH NAIL FORCE =	958.8588	lb/ft
2 TH NAIL FORCE =	1616.2710	lb/ft
3 TH NAIL FORCE =	2323.0820	lb/ft
4 TH NAIL FORCE =	3266.6650	lb/ft
5 TH NAIL FORCE =	4331.3090	lb/ft
6 TH NAIL FORCE =	5805.6800	lb/ft
7 TH NAIL FORCE =	8131.7790	lb/ft

Figure 30. Case 5 part A Soil Nailing Calculation 3rd run (cont.)

Figure 31. Case 5 part A Final Soil Nail Section (all dimensions in feet)

Follow same steps for part B, with a wall height of 26 feet, and get the final design as shown on the following pages.

* CASE 5B 5.5 ft BY 5 ft
 < OUTPUT FILE > = CASE5B1.OUT

I N P U T D A T A

S O I L P R O P E R T I E S

FOUNDATION	SOIL UNIT WEIGHT	=	125.000 pcf
	COHESION OF SOIL	=	1500.000 psf
	FRICTION ANGLE OF SOIL	=	25.000 deg
	UNDRAINED SU AT EXCAVATION LEVEL	=	3000.000 psf
	DEPTH TO HARDPAN FROM EXCAVATION	=	51.000 ft
	SU AT THE TOP OF HARDPAN	=	5000.000 psf
LAYER 1	HEIGHT TO TOP OF LAYER FROM TOE	=	4.500 ft
	SOIL UNIT WEIGHT	=	125.000 pcf
	COHESION OF SOIL	=	1500.000 psf
	FRICTION ANGLE OF SOIL	=	25.000 deg
LAYER 2	HEIGHT TO TOP OF LAYER FROM TOE	=	14.500 ft
	SOIL UNIT WEIGHT	=	120.000 pcf
	COHESION OF SOIL	=	1000.000 psf
	FRICTION ANGLE OF SOIL	=	30.000 deg
LAYER 3	HEIGHT TO TOP OF LAYER FROM TOE	=	26.000 ft
	SOIL UNIT WEIGHT	=	115.000 pcf
	COHESION OF SOIL	=	500.000 psf
	FRICTION ANGLE OF SOIL	=	30.000 deg

W A L L & L O A D C O N D I T I O N

WALL INCLINATION ANGLE	=	5.000 deg
SURCHARGE LOAD	=	300.000 psf
HOR. ACCELERATION COEFF.	=	.000

N A I L P R O P E R T I E S

CROSS SECTIONAL AREA OF NAIL	=	.790 si
GROUT DIAMETER	=	4.000 in
YIELD STRENGTH OF NAIL	=	60.000 ksi
NAIL INCLINATION ANGLE	=	20.000 deg
NAIL SKEW ANGLE AT SURFACE	=	.000 deg
DEPTH TO UPPERMOST NAIL	=	2.000 ft
VER. SPACING OF NAIL	=	5.500 ft
HOR. SPACING OF NAIL	=	5.000 ft
TOTAL LENGTH OF UPPERMOST NAIL	=	20.000 ft
TOTAL LENGTH OF LOWERMOST NAIL	=	15.000 ft
TOTAL UNBONDED LENGTH OF NAIL	=	2.000 ft

Figure 32. Case 5 part B Soil Nailing Calculation

O U T P U T R E S U L T S

I=	2	A=	.133	F.S.=	5.083	T=	6332.8550 lb/ft
I=	3	A=	.200	F.S.=	3.227	T=	9333.3710 lb/ft
I=	4	A=	.267	F.S.=	2.579	T=	10772.4900 lb/ft
I=	5	A=	.333	F.S.=	2.248	T=	11473.3200 lb/ft
I=	6	A=	.400	F.S.=	2.055	T=	11642.7600 lb/ft
I=	7	A=	.467	F.S.=	1.934	T=	11478.3900 lb/ft
I=	8	A=	.533	F.S.=	1.858	T=	11094.0800 lb/ft
I=	9	A=	.600	F.S.=	1.810	T=	10570.1600 lb/ft
I=	10	A=	.667	F.S.=	1.783	T=	9935.7290 lb/ft
I=	11	A=	.733	F.S.=	1.769	T=	9205.3270 lb/ft
I=	12	A=	.800	F.S.=	1.767	T=	8447.3250 lb/ft
I=	13	A=	.867	F.S.=	1.775	T=	7658.7750 lb/ft
I=	14	A=	.933	F.S.=	1.790	T=	6869.3590 lb/ft
I=	15	A=	1.000	F.S.=	1.815	T=	6107.3180 lb/ft
I=	16	A=	1.067	F.S.=	1.860	T=	5454.3630 lb/ft
I=	17	A=	1.133	F.S.=	1.906	T=	4976.4740 lb/ft
I=	18	A=	1.200	F.S.=	1.951	T=	4581.8080 lb/ft
I=	19	A=	1.267	F.S.=	1.994	T=	4216.6960 lb/ft
I=	20	A=	1.333	F.S.=	2.039	T=	3938.5210 lb/ft
I=	21	A=	1.400	F.S.=	2.087	T=	3746.4290 lb/ft
I=	22	A=	1.467	F.S.=	2.135	T=	3565.6660 lb/ft
I=	23	A=	1.533	F.S.=	2.181	T=	3401.0820 lb/ft
I=	24	A=	1.600	F.S.=	2.227	T=	3246.7450 lb/ft
I=	25	A=	1.667	F.S.=	2.274	T=	3101.7320 lb/ft
I=	26	A=	1.733	F.S.=	2.325	T=	2962.2770 lb/ft
I=	27	A=	1.800	F.S.=	2.374	T=	2833.1790 lb/ft
I=	28	A=	1.867	F.S.=	2.424	T=	2711.6140 lb/ft
I=	29	A=	1.933	F.S.=	2.474	T=	2597.0300 lb/ft
I=	30	A=	2.000	F.S.=	2.527	T=	2488.2230 lb/ft

H/LT= 1.300 H/LB= 1.733

NO. OF NAILS (INCLUDING THE LOWERMOST) = 5

**F.S. AGAINST SLIDING THROUGH TOE = 1.72 good > 1.5

**F.S. AGAINST DEEP SEATED ROTATION = 6.16

1 TH NAIL FORCE =	428.5895 lb/ft
2 TH NAIL FORCE =	883.8430 lb/ft
3 TH NAIL FORCE =	1324.9810 lb/ft
4 TH NAIL FORCE =	2213.0530 lb/ft
5 TH NAIL FORCE =	3596.8580 lb/ft

Figure 32. Case 5 part B Soil Nailing Calculation (cont.)

Figure 33. Case 5 Part B Final Soil Nail Section (all dimensions in feet)

By experience, it is very practical to keep the same pattern for soil nailing even if there are two different excavation levels to keep commonality in the job site and avoiding any worker mistakes.

The next step is materials take off as shown in Table 4 and Table 5.

Table 4. Case 5 Soil Nailing Materials Takeoff part A and part B

part A		Quantity	Unit Length	Total Length
#8 rebar	1 st	79	30.00	2,370
	2 nd	78	29.50	2,301
	3 rd	77	28.50	2,195
	4 th	76	27.50	2,090
	5 th	75	26.50	1,988
	6 th	73	26.00	1,898
	7 th	71	25.00	1,775
		=====		=====
		529		14,616.00
part B		Quantity	Unit Length	Total Length
#8 rebar	1 st	80	20.00	1,600
	2 nd	79	19.00	1,501
	3 rd	78	17.50	1,365
	4 th	76	16.50	1,254
	5 th	75	15.00	1,125
		=====		=====
		388		6,845.00
#4 wales	18,412	ft		
6x6 wire mesh	24,318	sqft		
2 2" gunnite	24,318	sqft	297.22	cy
no. of nails	917			

Table 5. Case 5 Soil Nailing Materials takeoff

SOIL NAILING		
total linear ft of nails drill	21,461	ft
# 8 total linear ft of nails steel	21,461	ft
no. of soil nails	917	each
no. of soil nails plates & nuts	917	each
#4 wales	18,412	lft
6x6 wire mesh	24,318.00	sqft
shotcrete or gunnite 2, 2"	24,318.00	sqft
layout	40	man/day

Then the next step is to calculate the soil nailing cost, as shown in Figs. 34 and 35.

Case 5 Part A Soil Nailing Cost

1- Drill hole									
drill rate	6 each x 20'	each/hr	nail length	used length					
		6	20	27.5					
Equip. 24000/160 =		150							
fog / hr =		40							
drill string wear =		15							
labor 42.17+34.37=		80							
		=====							
		285	→	total	total\$/each	ratio	total\$/each		
				300	50	1.375	68.75		
spacing =	27.5	sft							
1- Drill hole						total/each / spacing=	2.50	\$/sft	
2- Furnish and install nails									
	rate/ hr =	4							
Equip									
grout mixer w/360 air=		6400							
small excavator=		3600							
		=====							
		10,000.00	/ 160 =	62.50	/ rate / hr =	15.63			
Labor 4 men crew									
	50+42+69=	161	→	175	/ rate / hr =	43.75			
F O G =	35			35	/ rate / hr =	8.75			
Matl - Bar= 2.75			→	3+CONT. 0.15	x nail length	86.625			
grout / ft =	0.83			0.83	x nail length	22.825			
					=====				
					sub. tot. =	177.58			
Plate and nuts						10			
					=====				
						187.58	/ spacing=	6.82	\$/sft
3- Place 6x6 mesh w/ miradrain									
	mesh mat= 20.45x1.06 / 8x15 =			0.18	\$/sft				
	miradrain @ 4' c-c = 210/150 = 1.4	/ 4 =		0.35	\$/sft				
	chairs for mesh @4sft/chair =			0.1	\$/sft				
	labor to place 4 @ 34.37 = 140/(6x4x15)			0.39	\$/sft				
				=====					
				1.02	→	1.1	\$/sft		
contin. walers @ dbl #4 and 4'									
	nail spacing= 5	ft							
	nail spacing= 0.416	x 0.5 mat and 0.5 lab	→	0.54	\$/sft				
4- Shotcrete									
	82.5x1.06+75/cy sub/place								
	162.45/cy @ 4" = 81 sft=2/sft* 1.1= 2.2 /sft	→	1.72	\$/sft	=====				
			12.68	\$/sft					
5- over-all supervision @ 750 sft/day 75x8/750= 0.8									
			→	0.8	\$/sft				
6- OTA 6 men x 100/day /750: 0.8									
			→	0	go to zero if job is in town	=====			
						13.48			
7- MOB AND DEMOB+ SHOP DRAWING (2500+2500+5000)/27000									
			→	0.37	=====				
				13.85	\$/sft				
	shaving excavation face			0.25	\$/sft	=====			
				14.10	\$/sft	sft	total cost		
						14,270	201,220		

Figure 34 Case 5 part A Soil Nailing Cost

Case 5 Part B Soil Nailing Cost

1- Drill hole									
drill rate	6 each x 20'	each/hr	nail length	used length					
		6	20	17.5					
Equip.24000/160 =	150								
fog / hr =	40								
drill string wear =	15								
labor 42.17+34.37=	80								
	=====								
	285			total	total\$/each	ratio	total\$/each		
				300	50	0.875	43.75		
spacing =	27.5	sft							
1- Drill hole							total/each / spacing=	1.59	\$/sft
2- Furnish and install nails									
	rate/ hr =	4							
Equip									
grout mixer w/360 air=	6400								
small excavator=	3600								
	=====								
	10,000.00	/ 160 =	62.50	/ rate / hr =	15.63				
Labor 4 men crew	50+42+69=	161	175	/ rate / hr =	43.75				
F O G =	35		35	/ rate / hr =	8.75				
Matl - Bar= 2.75			3+CONT. 0.15	x nail length	55.125				
grout / ft =	0.83		0.83	x nail length	14.525				
				=====					
				sub. tot. =	137.78				
Plate and nuts					10				
					=====				
					147.78	/ spacing=	5.37	\$/sft	
3- Place 6x6 mesh w/ miradrain									
mesh matl=	20.45x1.06 / 8x15	=	0.18	\$/sft					
miradrain @ 4' c-c =	210/150 =	1.4	/ 4 =	0.35	\$/sft				
chairs for mesh @4sft/chair =			0.1	\$/sft					
labor to place 4 @ 34.37 =	140/(6x4x15)		0.39	\$/sft					
			=====						
			1.02				1.1	\$/sft	
contin. walers @ dbl #4 and 4'									
nail spacing=	5	ft							
nail spacing=	0.416	x 0.5 mat and 0.5 lab					0.54	\$/sft	
4- Shotcrete									
82.5x1.06+75/cy sub/place									
162.45/cy @ 4" = 81 sft=2/sft* 1.1= 2.2 /sft							1.72	\$/sft	
							=====		
							10.32	\$/sft	
5- over-all supervision @ 750 sft/day 75x8/750= 0.8							0.8	\$/sft	
6- OTA 6 men x 100/day /750 sft 0.8							0	go to zero if job is in town	
							=====		
							11.12		
7- MOB AND DEMOB+ SHOP DRAWING (2500+2500+5000)/27000							0.37		
							=====		
							11.49	\$/sft	
shaving excavation face							0.25	\$/sft	
							=====		
							11.74	\$/sft	
								sft	total cost
								10,048	118,009

Figure 35. Case 5 part B Soil Nailing Cost

Next, calculate the total cost and unit cost for the project part A and part B together.

Table 6. Case 5 part A, and part B Soil Nailing Cost

	Soil Nailing Area Sqft	Unit cost \$/sqft	Total Cost \$
Part A	14,270	14.1	201,220
Part B	10,048	11.74	118,009
Total A+B	24,318	13.13	319,229

CHAPTER 4

RESULTS OF ANALYSES

Several cost comparison studies were done for both systems; total cost versus total cost; unit cost versus unit cost, and soil nailing cost as a percentage of conventional system cost, for all fifteen projects.

4.1 Cost of Conventional Systems

Table 7 below shows total cost, total square footage and the unit cost in dollars per square foot, of the conventional systems for the fifteen projects

Table 7. Costs for Conventional System

CASE NO.	Total cost \$	Area Sq ft	Unit cost \$/Sq ft
CASE 1	254,352	12,560	20.25
CASE 2	519,479	22,400	23.19
CASE 3	49,445	1,875	26.37
CASE 4	188,197	8,940	21.05
CASE 5	514,285	24,318	21.15
CASE 6	567,366	24,933	22.76
CASE 7	69,567	3,020	23.04
CASE 8	294,056	13,234	22.22
CASE 9	179,376	8,514	21.07
CASE 10	235,262	11,940	19.70
CASE 11	138,127	6,218	22.21
CASE 12	325,384	14,710	22.12
CASE 13	68,565	2,765	24.80
CASE 14	95,346	4,100	23.26
CASE 15	1,833,996	71,820	25.54

4.2 Cost of Soil Nailing Systems

Table 8 below shows total cost, total square footage, and unit cost in dollars per square foot of the soil nailing systems for the fifteen projects; the table also shows the soil nailing cost as a percentage of the conventional system cost.

Table 8. Cost of Soil Nailing Systems

CASE NO.	Total Cost, \$	Area, Sqft	Unit cost, \$/Sqft	Cost ratio (Soil nailing /Conventional)*100
CASE 1	156,258	12,560	12.44	61.43%
CASE 2	322,224	22,400	14.39	62.03%
CASE 3	20,531	1,875	10.95	41.52%
CASE 4	127,796	8,940	14.29	67.91%
CASE 5	319,229	24,318	13.13	62.07%
CASE 6	383,761	24,933	15.39	67.64%
CASE 7	31,056	3,020	10.28	44.64%
CASE 8	180,362	13,234	13.63	61.34%
CASE 9	127,401	8,514	14.96	71.02%
CASE 10	176,399	11,940	14.77	74.98%
CASE 11	74,648	6,218	12.01	54.04%
CASE 12	212,131	14,710	14.42	65.19%
CASE 13	30,266	2,765	10.95	44.14%
CASE 14	46,924	4,100	11.44	49.21%
CASE 15	1,124,953	71,820	15.66	61.34%

The results shown in Table 8 are shown graphically in Figures 36, 37, and 38.

Figure 36. Total Cost Comparison, Conventional and Soil Nailing Systems

Figure 37. Unit Cost Comparison, Conventional Systems and Soil Nailing Systems

Figure 38. Cost Ratio (Soil Nailing / Conventional)

4.3 Factors Causing Differences in Costs

4.3.1 Construction Management Approach

To compare the cost of both retention systems (Conventional and Soil nailing), there are three simple categories of factors related to the construction process should be compared:

- 1- Material Cost
- 2- Labor Cost
- 3- Equipment Cost

Conventional system materials include steel piles, high tensile strength steel rebars or tendons, steel wales, wood lagging, and welded studs with plates to hold wood lagging boards in place. Soil nailing system materials include welded wire mesh, shotcrete, low tensile strength steel rebar for nails, and no. 4 steel rebars as

wales. A comparison of the materials required to build both systems indicates that Soil Nailing materials are cheaper than conventional system materials. However, if a much larger quantity of cheaper materials were required for the soil nailing system than the quantity of more expensive materials involved in the conventional system, then the total costs of the two systems might be comparable or the soil nailing system might be more expensive. The analysis presented previously has used actual quantities of the various materials to get cost estimates that support the conclusion that material costs for soil nailing are lower than material costs for conventional retaining systems. This conclusion also is borne out by the data shown in Table 7 for conventional systems and Table 8 for soil nailing systems, showing material costs and costs for particular operations.

The second category of factors is the cost of labor to install both systems. Installing a conventional retention system requires driving steel piles in the ground with a vibratory hammer or predrilling holes and setting steel piles in those holes and then backfilling those holes with low strength concrete. Installing wood lagging behind steel pile flanges down to the first row of tiebacks, and after that drilling holes and installing tiebacks and then grouting the holes at high pressure are required for conventional systems. All tiebacks will be performance or proof tested to 120% of design load and locked at 75 percent of design load. When excavation advances below the first row of tiebacks, the installation of wood lagging requires welding studs on the steel pile flanges with plates to hold wood lagging boards; this procedure is labor-intensive and will slow the installation procedure. Some projects require wale installation which entails welding wedges on steel piles as seats for wales or the

designer may choose to install tiebacks through the steel piles (which require cutting holes in both pile flanges and welding stiffener plates between the flanges to prevent local buckling. In contrast, soil nailing system installation requires drilling holes for nails that are more shallow/shorter than tieback holes and grouting at low-pressure, laying wire mesh on the face of excavated soil and placing no. 4 rebars to form a wale. After the wale is in place, 4 inches of shotcrete are sprayed on the soil face in two lifts. Installing a drainage system is required for both systems. By comparing the labor cost required to install both systems it is obvious that the soil nailing system requires less labor and less costly labor than the conventional system.

The third category of factors to consider is the equipment costs for both systems; the conventional system requires steel piles to be driven into the ground with a vibratory hammer or cutting predrilled vertical holes, using a drilling machine to drill inclined holes for tiebacks and then using a high-pressure pump to grout the inclined holes. Installing a conventional system requires using excavator to lower heavy wales to proper depth, and a welding machine to finish wale installation and weld studs for wood lagging boards.

Soil nailing will require small, light-weight equipment to install the system. A drilling machine will be needed to drive short nails or drill short holes for nails. A pump will be required to grout the annuli around the nails at low pressure, as well as a mixer for shotcrete and a pneumatic system to blow the shotcrete onto the face of the excavation.

By comparing these equipment needs, it is obvious that soil nailing system equipment will be cheaper both in capital cost and operating expenses.

This study shows that the soil nailing system is cheaper than the conventional system. Therefore, a meaningful question will be asked, “Why is not Soil Nailing more widely used”? According to the construction management analysis, other factors influence the decision on which retention system should be used. Two factors determine what system to use:

1- Area of local practice: This description refers to how easily a local contractor can get project materials, hire labor and secure equipment by either purchase or rental, to install a retention system. Put another way, would it be necessary for a general contractor to rely on a specialist subcontractor to come into town and do the job?

2- Engineer’s recommendations: The design engineer’s state of knowledge and his experience with retention systems will influence what system is chosen. Engineers typically feel more comfortable with what they know and are slow to adopt new systems; contractors usually have to persuade engineers to adopt new methods or systems.

In analyzing the results of this research, two other inherent factors that could affect costs were investigated: project size; and project location.

4.3.2 Project Size

The overall variation in total system cost with size of retained area is shown in Figure 39 (Parts A and B), for conventional systems and soil nailing systems for linear fits to the generated data. Unit costs are shown in Figure 40 (Parts A and B), for both systems as a function of retained soil area. Conventional systems

ranged in unit cost between \$19.70/sqft and \$26.37/sqft, while soil nailing systems ranged between \$10.28/sqft and \$15.66/sqft, for unit costs. The linear correlation shown is better than the correlation obtained through fitting a polynomial function to the data, and the resultant relation is simpler, so only the linear fit lines are shown. The correlation between project size and unit cost is not as significant as the correlation between total cost and project size, because the unit cost (total cost/project size) already includes the project size as a determinant. The unit cost of tiebacks is strongly related to the project size, even though the project size already was used to obtain the unit cost. The unit cost of steel piles is insensitive to project size because the length of steel piles is closely related to the depth of excavation, and thus is strongly related to project size. The unit costs of soil nailing system components are insensitive to project size because the unit cost already reflects the strong relation between number of components and project size.

Figure 39A. Total Cost of Conventional Systems and Soil Nailing Systems versus Retained Area

Figure 39B. Total Cost of Conventional Systems and Soil Nailing Systems versus Retained Area without case 15 (71,820 sq ft)

Figure 40A. Unit Cost of Conventional Systems and Soil Nailing Systems versus Retained Area

If case 15 is eliminated (71,820 sq ft) and Fig. 40A is redrawn, a better fit to the generated data is obtained; see Fig. 40B.

Figure 40B. Unit Cost of Conventional Systems and Soil Nailing Systems versus Retained Area without case 15 (71,820 sq ft)

From Fig. 39B, linear equations were obtained to relate total cost to project size (where X is project size in square feet):

A: Total Cost without case 15

$$Y1 = 22.098X - 1,885.2 \dots\dots\dots \text{Conventional}$$

$$(R)^2 = 0.9925$$

$$Y2 = 14.75X - 10,332 \dots\dots\dots \text{Soil Nailing}$$

$$(R)^2 = 0.9871$$

$$\text{Slope of line, } M1 = 22.098 \dots\dots\dots \text{Conventional total cost}$$

$$\text{Slope of line, } M2 = 14.75 \dots\dots\dots \text{Soil Nailing total cost}$$

A comparison among slopes of the correlation lines shows that the conventional systems are more sensitive to the size of retained area than are the soil nailed systems.

Fig. 41 shows unit costs for steel beams and tiebacks as functions of retained area. Fig. 42 shows unit costs of (nail, plate and nut) and of shotcrete versus retained area.

Figure 41A. Unit Cost of Steel Pile and Tiebacks versus Retained Area

Figure 41B. Unit Cost of Steel Pile and Tiebacks versus Retained Area
without case 15 (71,820 sq ft)

Figure 42A. Unit Cost of Nail, Plate, Nut and Shotcrete versus Retained Area

Figure 42B. Unit Cost of Nail, Plate, Nut and Shotcrete versus Retained Area
without case 15 (71,820 sq ft)

In Fig. 41B, the correlation without Case 15 data was not as good as the correlation for data from all cases, but in Fig. 42B, a slightly improved correlation was obtained by deleting the data from Case 15.

From Figures 39B, 40B, 41B, and 42B a comparison among total cost, unit cost, cost of steel beams, and cost of tie backs for conventional systems can be made with costs of nails, nail drilling, nuts and shotcrete for soil nailing, for all the cases except Case 15.

Conventional systems (In the following equations, X is project size)

$$Y1 = 22.098X - 1,885.2 \dots \text{Total Cost}$$

$$Y3 = -8E-05X + 22.79 \dots \text{Unit Cost}$$

$$Y5 = 0.0002X + 4.17 \dots \text{Steel Pile Cost}$$

$$Y6 = -2E-05X + 3.79 \dots \text{Tie back Cost}$$

Soil Nailing Systems

$$Y2 = 14.75 X - 10,332 \dots\dots\dots \text{Total Cost}$$

$$Y4 = 0.0001X + 11.573 \dots\dots\dots \text{Unit Cost}$$

$$Y7 = 0.0001X + 6.18 \dots\dots\dots \text{Drill, Nail, and Nuts}$$

$$Y8 = \text{Constant, } 1.72 \dots\dots\dots \text{Shotcrete}$$

Fig. 43A shows total cost for both kinds of support systems versus depth of retained excavation, with results of a regression analysis based on a linear equation, and Fig. 43B shows data from all cases except Case 15, with linear regression analysis.

Figs. 44A and 44B show unit costs for the two retention systems versus depth of retained excavation, with results of a linear regression analysis in Fig. 44A and Fig. 44B (without Case 15).

Figure 43A. Total Cost of Conventional Systems and Soil Nailing Systems versus
Depth of Cut (depth of retained excavation)

Figure 43B. Total Cost of Conventional Systems and Soil Nailing Systems versus
Depth of Cut (depth of retained excavation) without case 15

Figure 44A. Unit Cost of Conventional Systems and Soil Nailing Systems versus
Depth of Cut (depth of retained excavation)

Figure 44B. Unit Cost of Conventional Systems and Soil Nailing Systems versus
Depth of Cut (depth of retained excavation) without case 15

Figs. 45A and 45B (without Case 15) show unit costs of tiebacks versus depth of excavation, with results of a linear regression analysis. A strong correlation is shown between cost of tiebacks and depth of cut because tieback length (and cost) is dependent on depth of cut.

Figure 45A. Unit Cost of Tiebacks versus Depth of Cut (depth of retained excavation)

Figure 45B. Unit Cost of Tiebacks versus Depth of Cut (depth of retained excavation)

without case 15

Figure 46 shows unit cost per linear foot of excavation rather than per square foot of excavation versus depth of cut, for all the data from all fifteen cases.

Figure 46. Unit Cost (\$/Linear foot) of Conventional and Soil Nailing versus
Depth of Cut (depth of retained excavation)

Fig. 46 shows that depth of cut is a strong influence on unit cost, and that portraying unit cost per linear foot of excavation wall is a better way of representing unit cost than using cost per square foot of retained area, for both methods of retention. Table 9 shows the numerical data on which the foregoing Fig. 46 was based.

Table 9. Unit Cost \$/Linear Foot versus Depth of Cut (depth of retained excavation)
for Conventional and Soil Nailing

CASE NO.	Conventional Unit Cost \$/L ft	Soil Nailing Unit Cost \$/L ft	depth of cut (ft)
CASE 1	475.90	292.36	23.5
CASE 2	742.11	460.32	32
CASE 3	395.57	164.25	15
CASE 4	631.54	428.85	30
CASE 5	666.17	413.51	31.5
CASE 6	637.16	430.97	28
CASE 7	437.68	195.39	19
CASE 8	533.27	327.09	24
CASE 9	695.26	493.81	33
CASE 10	482.74	361.96	24.5
CASE 11	322.10	174.08	14.5
CASE 12	553.00	360.52	25
CASE 13	334.77	147.77	13.5
CASE 14	430.22	211.73	18.5
CASE 15	1,072.51	657.87	42

An analysis of the results of the cost compilation showed that the costs of soil nailing in cases numbers. 3, 7, 13, and 14 were between 40 percent and 50 percent of the cost for conventional systems for those projects as shown in Figs. 47 through 54. Those four cases involved smaller excavations, and therefore, smaller areas of retained soil faces; the smaller retained areas made the unit costs of the conventional systems, in dollars per square feet, higher because the cost of mobilization and demobilization was divided by small areas. Material costs for the steel H-piles and for the tiebacks also were increased because of the small areas of the retained soil, as shown for case 3 in Figure 47, for case 7 in Figure 49, for case 13 in Figure 51, and for case 14 in Figure 53.

Comparable unit costs for soil nailing system components are shown in Figures 48, 50, 52, and 54.

Figure 47. Unit Cost (Case 3) for Each Conventional System Component

Figure 48. (Case 3) Unit Cost for Each Soil Nailing System Component

Figure 49. Unit Cost (Case 7) for Each Conventional System Component

Figure 50. (Case 7) Unit Cost for Each Soil Nailing System Component

Figure 51. Unit Cost (Case 13) for Each Conventional System Component

Figure 52. (Case 13) Unit Cost for Each Soil Nailing System Component

Figure 53. Unit Cost (Case 14) for Each Conventional System Component

Figure 54. (Case 14) Unit Cost for Each Soil Nailing System Component

By studying Table 10, on the next page, the Author found that the steel unit cost, tieback unit cost, and unit cost of the lagging below the upper 8 feet (that required welding studs on the soldier piles to hold lagging) are the most influential cost factors for the conventional retention system. Steel costs varied between \$ 2.69/sqft and \$ 4.57/sqft and tieback costs varied between \$ 3.42/sqft and \$ 12.38 /sqft. Unit costs for these two components are more variable than the unit costs of the rest of the retention system components. These variations depend on the project size (square footage) and the depth of the sheeted cut. When sheeted areas were small (2,000 to 4,000 square feet), the unit costs of steel and tiebacks increased, while the lagging unit cost did not vary significantly. Furthermore, when the required sheeted cut is deep and requires several layers of tiebacks, tieback unit costs will increase as in case 15 when the excavation required the deepest cut in this study (42 feet).

Table 10. Conventional System Unit Cost for Components and Operations, \$/sqft

	\$/sqft	\$/sqft	\$/sqft	\$/sqft	\$/sqft	\$/sqft	\$/sqft	\$/sqft	\$/sqft	\$/sqft	\$/sqft	\$/sqft	\$/sqft	\$/sqft	\$/sqft	\$/sqft
	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9	Case 10	Case 11	Case 12	Case 13	Case 14	Case 15	Avg. =
driving in earth	1.66	1.28	1.73	1.31	1.10	1.39	1.77	1.25	1.22	1.30	1.62	1.52	1.81	1.77	1.24	1.46
steel pile	4.57	3.51	3.77	2.86	2.69	3.83	3.87	2.73	3.37	3.59	3.54	4.16	3.95	3.87	3.41	3.58
wale 10' long	1.07	1.61	1.68	0.10	0.06	0.00	0.60	0.68	0.00	1.73	1.81	1.38	1.95	1.43	0.00	0.94
wale 20' long	0.04	0.05	0.00	1.54	0.68	0.44	0.00	1.08	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.26
wale 10' uninstall	0.48	0.71	0.75	0.04	0.02	0.00	0.26	0.30	0.00	0.77	0.80	0.61	0.87	0.63	0.00	0.42
wale 20' uninstall	0.02	0.02	0.00	0.70	0.31	0.20	0.00	0.49	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.12
tieback cost	5.15	8.79	3.42	6.76	8.59	8.38	4.76	7.47	7.70	4.43	5.63	6.42	4.34	5.38	12.38	6.64
penetrate 10" beam	0.00	0.00	0.00	0.00	0.00	0.00	0.33	0.00	0.67	0.00	0.00	0.00	0.00	0.00	0.00	0.07
penetrate 12" beam	0.00	0.00	0.00	0.00	0.67	0.74	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.41	0.19
wood lagging upper 8 ft	1.75	0.96	1.91	1.21	1.06	1.23	1.80	1.31	1.57	1.23	1.81	1.60	1.87	1.74	0.74	1.45
wood lagging below 8 ft	3.87	5.18	3.61	4.76	5.01	4.74	3.79	4.60	4.17	4.73	3.78	4.12	3.68	3.89	5.54	4.36
mob/demob + shop dwg	0.80	0.45	8.00	1.12	0.41	0.60	4.97	1.13	1.76	1.26	2.41	1.02	5.42	3.66	0.21	2.21
out of state allowance	0.00	0.00	0.64	0.00	0.00	0.51	0.00	0.54	0.00	0.00	0.00	0.53	0.00	0.00	0.00	0.15
set pile in hole	0.83	0.64	0.86	0.65	0.55	0.70	0.89	0.62	0.61	0.65	0.81	0.76	0.90	0.89	0.62	0.73
	===	===	===	===	===	===	===	===	===	===	===	===	===	===	===	===
Unit cost Total =	20.25	23.19	26.37	21.05	21.15	22.76	23.04	22.22	21.07	19.70	22.21	22.12	24.80	23.26	25.54	22.58

Table 11 shows the unit costs for the soil nailing system components; the most influential cost factors are the unit cost for nails and nail installation. Soil nail unit cost varied between \$4.98 / sqft and \$10.68 / sqft. These unit costs are low when project size is small (2,000 to 4,000 square feet) as in cases 3, 7, 13 and 14. The unit cost for other soil nailing components did not vary with project size.

Table 11. Soil Nailing System Unit Cost for Components and Operations, \$/sqft

	\$/sqft	\$/sqft	\$/sqft	\$/sqft	\$/sqft	\$/sqft	\$/sqft	\$/sqft	\$/sqft	\$/sqft	\$/sqft	\$/sqft	\$/sqft	\$/sqft	\$/sqft	\$/sqft
	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9	Case 10	Case 11	Case 12	Case 13	Case 14	Case 15	Avg. =
Drill and install nail, plate and nut	7.66	9.61	4.98	9.67	8.14	9.48	5.12	6.79	9.99	9.99	7.04	8.59	5.98	6.08	10.69	7.99
6x6 mesh	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10
#4 wale	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54
shotcrete	1.72	1.72	1.72	1.72	1.72	1.72	1.72	1.72	1.72	1.72	1.72	1.72	1.72	1.72	1.72	1.72
Supervision	0.80	0.80	1.00	0.80	0.80	0.80	0.80	0.80	0.80	0.80	0.80	0.80	0.80	0.80	0.80	0.81
out of state allowance	0.00	0.00	0.80	0.00	0.00	0.80	0.00	0.80	0.00	0.00	0.00	0.80	0.00	0.00	0.00	0.21
mob. and shop drawing	0.37	0.37	0.56	0.37	0.37	0.54	0.56	0.56	0.56	0.37	0.56	0.56	0.56	0.56	0.56	0.50
shaving face	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25
	===	===	===	===	===	===	===	===	===	===	===	===	===	===	===	===
Unit cost Total =	12.44	14.39	10.95	14.45	12.92	15.23	10.09	12.56	14.96	14.77	12.01	14.36	10.95	11.05	15.66	13.12

4.3.3 Project Location

Comparison of costs for projects done in Louisville, Kentucky with costs for projects done outside the Louisville area did not reveal a significant difference between the two locations of projects. Figures 55A and 55B (without Case 15) show unit costs for projects constructed in the Louisville, Kentucky area. Figure 56 shows unit costs for projects constructed outside of Louisville, for both conventional and soil nailing systems.

Figure 55A. Unit Cost of Conventional Systems and Soil Nailing Systems versus Retained area in Louisville, KY

Figure 55B. Unit Cost of Conventional Systems and Soil Nailing Systems versus Retained area in Louisville, KY without case 15 (17,820 sq ft)

Figure 56. Unit Cost of Conventional Systems and Soil Nailing Systems versus Retained area Outside of Louisville, KY

By studying Figures 55B, and 56, the Author concluded that the unit cost of jobs for both conventional systems and soil nailing systems constructed in the Louisville, Kentucky area were in the same cost range as costs for projects constructed outside the Louisville area.

Figure 57 shows total cost for all projects except Case 15, differentiated by location, for conventional systems, and Figure 58 shows total cost for soil nailing systems differentiated by location, except for Case 15.

Figure 57. Total Cost of Conventional System with Location versus Retained Area without case 15

Figure 58. Total Cost of Soil Nailing System with Location versus Retained Area without case 15 (71,820 sq ft)

4.3.4 Adjusted Analysis

The most critical intrinsic factor affecting the cost of both systems was the job size. To make comparisons more meaningful, the Author divided total cost by project linear feet to produce the plot of unit cost versus size of project shown in Figure 59 for conventional systems. A similar procedure produced the plot of unit costs versus size of project for soil nailing systems shown in Figure 60.

Figure 59. Unit Cost of Conventional System with Location versus Depth of Cut
(depth of retained excavation)

Figure 60. Unit Cost of Soil Nailing System with Location versus
Depth of Cut (depth of retained excavation)

Figures 59 and 60 show that the location will have less significance for the unit cost of both conventional and soil nailing systems than the project size.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 General Conclusions

Soil nailing can be used successfully and economically to retain deep excavations, provided the anticipated deformations and associated settlements are within acceptable limits as determined by project conditions.

Several factors are making soil nailing more economical than conventional retention systems, if the deformations and settlements of soil nailing systems can be tolerated: lower cost for materials; quicker mobilization; faster construction time; smaller construction space requirements; and less need for heavy equipment.

Soil nailing is advantageous in heterogeneous soils having boulders or rock mixed with softer soils, but that advantage is not necessarily related to project cost.

Soil nailing cannot be used in clayey soils of low strength, in poorly draining soils, in soils that cannot support a temporary vertical cut, and in soil profiles favoring deep-seated sliding surfaces.

In general, this research indicated that, if soil nailing can meet deformation and settlement criteria, soil nailing will be more economical than conventional retention systems for excavations up to 42 feet deep, for the conditions that existed in the fifteen project sites examined in this study. This conclusion supports the hypothesis stated at the beginning of this dissertation.

5.2 Factors Affecting Cost

Comparing cost between Conventional and Soil Nailing retention systems, three categories of factors need to be compared: Material, Labor, and Equipment costs. Conventional system material will cost more than soil nailing material. Labor cost to install a conventional system is higher than labor cost for a soil nailing system, a conventional system requires buying or renting heavier and more expensive equipment to install the system, compared with light and less expensive equipment for a soil nailing system. In other words, for most cases, soil nailing systems will be cheaper than conventional systems. However, there are other factors affecting which system will be used in a project: Area of local practice (option for local contractors to get project materials, labor, and equipment), and Engineer's recommendation and experience with retention system, which will influence what system is chosen.

Two intrinsic factors were investigated in comparing the costs for soil nailing systems and conventional systems: project size; and project location, but the project size was the dominant factor, while project location mattered little.

5.2.1 Effect of Project size

If the project has a small square footage of sheeted area, the cost of the Conventional system will be increased primarily because of the unit costs for steel H-piles and wales, steel tieback, and unit mobilization and demobilization costs. For projects that have a large square footage of sheeted area (more than 6,500 square feet), the average cost of soil nailing is about two-thirds of the cost of Conventional systems if all other conditions were the same.

5.2.2 Effect of Project Location

According to the data in this study, the location of the project did not have a significant effect on cost.

5.3 Recommendations

The Author recommends that more studies be conducted for situations and conditions not covered herein, such as a cost comparison study of both systems versus depth of cut, with data from cuts more than 42 feet deep. Also, more data should be gathered from cities other than Louisville, Kentucky, especially cities such as Chicago, Boston, New York, and San Francisco.

REFERENCES

Clough, G. W. and Thomas D. O'Rourke (1990). "Construction Induced Movements of Insitu Walls," *Geotechnical Special Publication, No. 25, Design and Performance of Earth Retaining Structures*, ASCE, New York, 18–21

Lazarte, C. A., Elias, V., Espinoza, D., and Sabatini, P. J. (2003). *Soil Nailing Walls, Geotechnical Engineering Circular No. 7*, Report No. 7, Report No. FHWA0-IF-03-017 Federal Highway Administration, Washington, D. C., 7-16

PTI Post-Tensioning Institution (1996). *Recommendations for Prestressed Rock and Soil Anchors 3rd Edition*

Porterfield, J. A., Cotton, D. M., and Byrne, R. J. (1994). "Soil Nailing Field Inspectors Manual, Project Demonstration 103," Publication No. FWHA-SA-93-068, Federal Highway Administration, Washington, D.C., 7-9

Stocker, M. and Riedinger, G., (1990). "The Bearing Behavior of Nailed Retaining Structures," *Geotechnical Special Publication, No. 25, Design and Performance of Earth Retaining Structures*, ASCE, New York, .612-628

Appendix A: Sketches of Systems

Case 1
Soldier pile wall profile

Case 1
soil nailing wall profile

Case 1
Soil nailing cross section

Case 2. Soldier Pile Profile

Case 2
Soil nailing cross section

Case 2
soil nailing wall profile

Case 3
Soil nailing cross section

Case 3
soil nailing wall profile

Case 3
Soldier pile wall cross section

Case 3
Soldier pile wall profile

Case 4
Soldier pile wall profile

Case 4
Soldier pile wall cross section A-A

Case 4
Soldier pile wall cross section B-B

Case 4
soil nailing wall profile

Case 4
Soil nailing cross section A-A

Case 4
Soil nailing cross section B-B

Case 5
Soil nailing cross section

Case 5
soil nailing wall profile

Case 5
Soldier pile wall cross section

Case 6
Soil nailing cross section

Case 6
Soldier pile wall cross section

Case 7
Soldier pile wall profile

Case 7
Soil nailing cross section

Case 7
Soldier pile wall cross section

Case 7
soil nailing wall profile

Case 8
Soil nailing cross section

Case 8
Soldier pile wall cross section

Case 9
Soldier pile wall cross section

Case 9
Soil nailing cross section

Case 9

Soldier pile wall profile

Case 10
Soldier pile wall cross section

Case 10
Soil nailing cross section

Case 10
Soldier pile wall profile

Case 10
soil nailing wall profile

Case 11
Soil nailing cross section

Case 11
Soldier pile wall cross section

Case 12
Soil nailing cross section

Case 12
Soldier pile wall cross section

Case 13
Soil nailing cross section

Case 13
Soldier pile wall cross section

Case 14
Soil nailing cross section

Case 14
Soldier pile wall cross section

Case 15
Soil nailing cross section

Case 15
Soldier pile wall cross section

Appendix B: Conventional System Design

CASE 1

DEPTH OF EXCA	23.5	FT
FRICTION- PHEE	25	DEGREE
SOIL UNIT WEIGHT	120	#/C.F
SURCHARGE Sou	300	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	1	FT
PILE EFF. FACTOR	2.5	
Ka	0.33	
Kp	3	
TIE LOCATION	9	FT
TIE ANGLE X=	30	DEGREE
Pb FACTOR =	22	
ALLOWABLE STRESS =	33	KSI

COS X= 0.8660
Pb = 517 PSF
Pg = 99 PSF
LENGTH OF upper triangle 4.7 FT
LENGTH OF rectangle 14.1 FT

FORCE

7573.5
10327.075
18898.35
12201.75
43086.1
10327.075
21736.125
8467.67
958.3895

PILE PENETR. D= 8.95 FT

PAS.PRESSUp1= 5255 PSF
ACT.PRESSUp1= 930.8 PSF
ACT.PRESSUp2= 275.22 PSF
DIST.FROM DRE-
DGE LINE TO TIE

try diff.value of D

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

ARM

4.5
5.851
2.15
7.25
4.9
11.351
19.087
17.975
19.087

MOMENT

34080.75
60423.71583
40827.1525
88462.8875
211023.99
117222.6283
414877.4179
118256.3683
18254.60830

TOTAL MOMENT@TIE LOCATION= -1211.1 (THIS VALUE SHOULD APPROX.=0)

the (D)should be= 8.34 FT
used

USE D= 9.00 FT
F.S= 1.29 HENCE OK

TOTAL LENGTH=D+H= 32.50 FT

CASE 1

DEPTH OF EXCA	23.5	FT
FRICTION- PHEE	25	DEGREE
SOIL UNIT WEIGHT	120	#/C.F
SURCHARGE Scu	300	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	1	FT
PILE EFF. FACTOR	2.5	
Ka	0.33	
Kp	3	
TIE LOCATION	9	FT
TIE ANGLE X=	30	DEGREE
Pb FACTOR =	22	
ALLOWABLE STRESS =	33	KSI

COS X=	0.8660	
Pb =	517	PSF
Pq =	99	PSF
LENGTH OFupper triangle	4.7	FT
LENGTH OF rectangle	14.1	FT

FORCE

7573.5
10327.075
18896.35
12201.75
43066.1
10327.075
21736.125
6467.67
956.3895

try diff.value of D

PILE PENETR. D=	6.95	FT
PAS.PRESSUp1=	6255	PSF
ACT.PRESSUp1=	930.6	PSF
ACT.PRESSUp2=	275.22	PSF
DIST.FROM DRE- DGE LINE TO TIE	14.5	FT

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

ARM

4.5
5.851
2.15
7.25
4.9
11.351
19.087
17.975
19.087

MOMENT

+	34080.75
+	60423.71583
+	40627.1525
-	88462.6875
-	211023.89
-	117222.6283
+	414877.4179
-	116256.3683
-	18254.60639

TOTAL MOMENT@TIE LOCATION= -1211.1
(THIS VALUE SHOULD APPROX.=0)

the (D)should be= 8.34 FT
used
USE D= 9.00 FT
F.S= 1.29 HENCE OK

TOTAL LENGTH=D+H= 32.50 FT

TO FIND TIEBACK FORCE
MAKE SUM. FOR Fx=0

F1=	19775.25	+
F2=	20654.15	+
F3=	61962.45	+
F4=	6467.67	+
F5=	21736.125	-
F6=	956.3895	+

Fx/ONE BAY=	88.08	KIP
Fx/ONE FT=	10.36	KIP/FT

Fx/ BAY =	88.08	KIP
TIE FORCE/1 BAY=	101.71	KIP
PROOFTEST LOAD=	122.05	KIP

FIND MAX. MOMENT

1 @TIEBACK LOCATION = -9 FT

FORCE
7573.5
10327.075
18896.35

ARM
4.5
5.851
2.15

MOMENT
34080.75
60423.72
40627.15

MAX. MOMENT= 135.13 KIP.FT

2 @ X-DISTANCE FROM
THE TOP OF THE PILE

FIND X WHEN SUM.OF Fx =0

(ZERO SHEAR POINT)

X= 18.794282 FT

FORCE
15815.39
88079.78
10327.08
61937.32

ARM
9.3971409
9.7942818
15.645282
7.0471409

MOMENT
148619.43
862678.23
161570.00
436481.03

MAX. MOMENT= 116.01 KIP.FT

USE MAX. MOMENT= 135.13 KIP.FT
MOD. SECTION = 49.14 CUB. IN (Gr-50)

CASE 1

CASE 2

DEPTH OF CUT H	32	FT
SOIL UNIT WEIGHT	115	#/C.FT.
SOLD. PILE SPACE S	8.5	FT
SURCHARGE LOAD	300	#/S.FT
Ka	0.33	
Kp	3	
LOCATION OF 1ST TIE	9	FT
LOCATION OF 2ND TIE	23	FT
TIE ANGLE UPPER=	30	DEGREE
TIE ANGLE LOWER=	30	DEGREE
Pb FACTOR ***	22	

COS X1= 0.8660
COS X2= 0.8660
Pb 704 PSF
Pq 99 PSF
LENGTH OF UPP.TRIA. 6.4 FT
LENGTH OF RECTANG 19.2 FT

TO FIND THE VALUE OF THE Fx1
MAKE SUM. OF MOM. @ Fx2 LOCATION

FORCE	ARM	MOMENT
19354.50	11.50	+ 222576.75
19148.80	18.73	+ 358720.85
99334.40	8.30	+ 824475.52
Fx1= 90059.20	14.00	- 1260828.80
7573.50	4.50	- 34080.75
19148.80	4.73	- 90637.65
15558.40	1.30	- 20225.92
		=====

SUM OF MOMENT SHOULD=0 -0.000000

Fx1= 90059.2

Fx1/ONE BAY= 90.0592 KIP
Fx1/ BAY = 90.0592 KIP
1stTIE FORCE/ BAY= 103.98 KIP
PROOF TEST LOAD= 124.79 KIP
FT

TO FIND THE VALUE OF Fx2
MAKE SUM. OF FX=0

Fx2= 90059.2

Fx2/ONE BAY= 90.0592 KIP
Fx2/ BAY = 90.0592 KIP
TIE FORCE /BAY= 103.98 KIP
PROOF TSEST LOAD= 124.79 KIP

TO FIND MAX. MOM.
1-@FIRST TIE LOCATION

FORCE	ARM	MOMENT
7573.5	4.5	34080.75
19148.8	4.7333333	90637.6533
15558.4	1.3	20225.92
		=====

MAX.MOM= 144944.323

1-MXMOM.@1st tie= 144.94 KIP.FT

2-@ X-DISTANCE FROM THE TOP OF THE PILE
FIND X-DISTANCE

X= 16 FT

MAX.MOMENT @ X-DISTANCE

FORCE	ARM	MOMENT
13464	8	+ 107712
19148.8	11.733333	+ 224679.253
57446.4	4.8	+ 275742.72
90059.2	7	- 630414.4
		=====

MAX MOM= -22280.427

MAX.MOM@1stTIE LOCAT.= 144.94 KIP.FT
MAX.MOMENT@x-distance= -22.26 KIP.FT

USE MAX.MOM.= 144.94 KIP.FT

DESIGN SOLD. PILE
MOD.SEC. FOR THE PILE = 52.71 CU.IN Gr-50

USE HP 12x53 Gr-50 Sx=66.8

CASE 3

DEPTH OF EXCA	15	FT
FRICTION- PHEE	30	DEGREE
SOIL UNIT WEIGHT	125	#/C.F
SURCHARGE S_{cu}	100	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	0.83	FT
PILE EFF. FACTOR	2.5	
K_a	0.33	
K_p	3	
TIE LOCATION	6	FT
TIE ANGLE $X=$	30	DEGREE
P_b FACTOR =	22	
ALLOWABLE STRESS =	33	KSI

$\cos X = 0.8660$
 $P_b = 330$ PSF
 $P_q = 33$ PSF
 LENGTH OF upper triangle 3 FT
 LENGTH OF rectangle 9 FT

FORCE
 1683
 4207.5
 8415
 2524.5
 16830
 4207.5
 7193.785625
 2208.31875
 316.5256875

PILE PENETR. $D = 4.3$ FT
 try diff. value of D
 PAS PRESSUp1= 3345.9375 PSF
 ACT. PRESSUp1= 513.5625 PSF
 ACT. PRESSUp2= 147.22125 PSF
 DIST. FROM DRE-
 DGE LINE TO TIE 9 FT

TO FIND PILE Penetration D
 MOMENT @TIEBACK LOCATION

ARM		MOMENT
3	+	5049
3.99	+	16787.925
1.5	+	12922.5
4.5	-	11390.25
3	-	50490
6.99	-	29410.425
11.838	+	85159.79747
11.15	-	24622.75406
11.838	-	3747.031089

TOTAL MOMENT@TIE LOCATION= -11.2 (THIS VALUE SHOULD APPROX.=0)

the (D) should be= 5.16 FT
 used
 USE $D = 6.00$ FT
 $F.S = 1.40$ HENCE OK
 TOTAL LENGTH $H = D + H = 21.00$ FT

CASE 3

DEPTH OF EXCA	15	FT
FRICTION- PHEE	30	DEGREE
SOIL UNIT WEIGHT	125	#/C.F
SURCHARGE Scu	100	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	0.83	FT
PILE EFF. FACTOR	2.5	
Ka	0.33	
Kp	3	
TIE LOCATION	6	FT
TIE ANGLE X=	30	DEGREE
Pb FACTOR =	22	
ALLOWABLE STRESS =	33	KSI

COS X=	0.8660	
Pb =	330	PSF
Pq =	33	PSF
LENGTH OFupper triangle	3	FT
LENGTH OF rectangle	9	FT

FORCE
1683
4207.5
8415
2524.5
16830
4207.5
7193.765625
2208.31875
316.5256875

try diff.value of D

PILE PENETR. D=	4.3	FT
-----------------	-----	----

PAS.PRESSUp1=	3345.9375	PSF
ACT.PRESSUp1=	513.5625	PSF
ACT.PRESSUp2=	147.22125	PSF
DIST.FROM DRE-	9	FT
DGE LINE TO TIE		

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

ARM		MOMENT
3	+	5049
3.99	+	16787.925
1.5	+	12622.5
4.5	-	11360.25
3	-	50490
6.99	-	29410.425
11.838	+	85159.79747
11.15	-	24622.75406
11.838	-	3747.031089
		=====

TOTAL MOMENT@TIE LOCATION= -11.2
(THIS VALUE SHOULD APPROX.=0)

the (D)should be=	5.16	FT
used		
USE D=	6.00	FT
F.S=	1.40	HENCE OK

TOTAL LENGTH=D+H= 21.00 FT

12/05/201009:45:29 PM1case3.123\

TO FIND TIEBACK FORCE
MAKE SUM. FOR Fx=0

F1=	4207.5	+
F2=	8415	+
F3=	25245	+
F4=	2208.31875	+
F5=	7193.765625	-
F6=	316.5256875	+

Fx/ONE BAY=	33.20	KIP
Fx/ONE FT=	3.91	KIP/FT

Fx/ BAY =	33.20	KIP
TIE FORCE/2 BAYS=	76.67	KIP
PROOFTEST LOAD=	92.00	KIP

FIND MAX. MOMENT

1 @TIEBACK LOCATION = -6 FT

FORCE
1683
4207.5
8415

ARM
3
3.99
1.5

MOMENT
5049.00
16787.93
12622.50

MAX. MOMENT= 34.46 KIP.FT

2 @ X-DISTANCE FROM
THE TOP OF THE PILE

FIND X WHEN SUM.OF Fx=0

(ZERO SHEAR POINT)

X= 12.123182 FT

FORCE
3400.55
33198.58
4207.50
25590.53

ARM
6.0615911
6.1231822
10.113182
4.5615911

MOMENT
20612.76
203280.95
42551.21
116733.52

MAX. MOMENT= 23.38 KIP.FT

USE MAX. MOMENT= 34.46 KIP.FT
MOD. SECTION = 12.53 CUB. IN (Gr-50)

CASE 3

SHEAR DIAGRAM

MOMENT DIAGRAM

CASE 4 PART A

DEPTH OF CUT H	28	FT
SOIL UNIT WEIGHT	125	#/C.FT.
SOLD. PILE SPACE S	8.5	FT
SURCHARGE LOAD	250	#/S.FT
Ka	0.33	
Kp	3	
LOCATION OF 1ST TIE	8	FT
LOCATION OF 2ND TIE	20	FT
TIE ANGLE UPPER=	30	DEGREE
TIE ANGLE LOWER=	30	DEGREE
Pb FACTOR ***	22	

COS X1=	0.8660	
COS X2=	0.8660	
Pb	616	PSF
Pq	82.5	PSF
LENGTH OF UPP. TRIA	5.6	FT
LENGTH OF RECTANG	16.8	FT

TO FIND THE VALUE OF THE Fx1
MAKE SUM. OF MOM. @Fx2 LOCATION

FORCE	ARM	MOMENT
14025.00	10.00	+ 140250.00
14660.80	16.27	+ 238482.35
75398.40	7.20	+ 542868.48
Fx1= 68460.70	12.00	- 821528.40
5610.00	4.00	- 22440.00
14660.80	4.27	- 62552.75
12566.40	1.20	- 15079.68
		=====

SUM OF MOMENT SHOULD=0 0.000000

Fx1/ONE BAY=	68.4607	KIP
Fx1/ BAY =	68.4607	KIP
1stTIE FORCE/ BAY=	79.05	KIP
PROOF TEST LOAD=	94.86	KIP
3 bays for 2 ties=	118.58	

TO FIND THE VALUE OF Fx2
MAKE SUM. OF FX=0

Fx2=	68460.7	
Fx2/ONE BAY=	68.4607	KIP
Fx2/ BAY =	68.4607	KIP
TIE FORCE /BAY=	79.05	KIP
PROOF TSEST LOAD=	94.86	KIP
3 bays for 2 ties=	118.58	KIP

TO FIND MAX. MOM.
1-@FIRST TIE LOCATION

FORCE	ARM	MOMENT
5610	4	22440
14660.8	4.266667	62552.7467
12566.4	1.2	15079.68
		=====

MAX.MOM= 100072.427

1-MX.MOM.@1st tie= 100.07 KIP.FT

2-@ X-DISTANCE FROM THE TOP OF THE PILE
FIND X-DISTANCE

X= 14 FT

MAX.MOMENT @ X-DISTANCE

FORCE	ARM	MOMENT
9817.5	7	+ 68722.5
14660.8	10.266667	+ 150517.547
43982.4	4.2	+ 184726.08
68460.7	6	- 410764.2
		=====

MAX.MOM= -6798.0733

MAX.MOM@1stTIE LOCAT.= 100.07 KIP.FT
MAX.MOMENT@x-distance= -6.80 KIP.FT

USE MAX.MOM. = 100.07 KIP.FT

DESIGN SOLD. PILE

MOD.SEC. FOR THE PILE = 36.39 CU.IN Gr-50

92.10810811

USE HP 12x53 Gr-50 Sx=66.8

CASE 4 PART B

H=	35	FT
S=	8.5	FT
SCH	250	PSF
TIE 1@	7.5	FT
TIE 2@	17.5	FT
TIE 3@	27.5	FT
Ka=	0.33	FT
TIE @	30	DEGREE
Pb FACTOR =	22	
upper triangle =	7	ft
lower triangle =	7	ft

W= 7.24625 K/1 VERTICAL FT

Fx1= 67.67 KIPS
Fx2= 72.46 KIPS
Fx3= 67.67 KIPS

COS X= 0.8660255

1.5x DL=

DL1= 78.14 KIP 117.21
DL2= 83.67 KIP 125.51
DL3= 78.14 KIP 117.21

PROOF LOAD 1.2 X DL

PROOF TIE 1 = 93.77 KIPS 140.65
PROOF TIE 2 = 100.41 KIPS 150.61
PROOF TIE 3 = 93.77 KIPS 140.65

MAX MOMENT 1 @ TIE 1 LOCATION

FORCE
5.26
26.30

ARM		MOMENT
3.75	+	19.72
2.50	+	65.74
=====		
85.46		

X IS BELOW 0.2 H X1= 5.50 FT

FORCE
4.91
22.91
39.85
67.67

ARM		MOMENT
9.00	+	44.18
7.83	+	179.44
2.75	+	109.60
5.00	-	338.35
=====		
-5.13		

MAX MOMENT 2 @ TIE 2 LOCATION

FORCE
4.91
22.91
76.09
67.67

ARM		MOMENT
14	+	68.72
12.83	+	293.98
5.25	+	399.45
10	-	676.71
=====		
85.45		

MAX MOMENT 3 BETWEEN 2nd AND 3rd TIE

X2 IS BELOW 0.2 H X2= 15.50

FORCE
4.91
22.91
112.32
67.67
72.46

ARM		MOMENT
19.00	+	93.27
17.83	+	408.52
7.75	+	870.46
15.00	-	1015.06
5.00	-	362.31
=====		
-5.13		

USE MAX MOM. = 85.46 FT-KIP
Sx= 31.08 IN^3 Gr-50
42.73 IN^3 A-36

CASE 5 PART A

H=	37.5	FT
S=	8.5	FT
SCH	300	PSF
TIE 1 @	8	FT
TIE 2 @	16.75	FT
TIE 3 @	29.5	FT
Ka=	0.33	FT
TIE @	30	DEGREE
Pb FACTOR =	22	
upper triangle =	7.5	ft
lower triangle =	7.5	ft

W= 7.854 K/1 VERTICAL FT

Fx1= 78.75 KIPS
Fx2= 84.43 KIPS
Fx3= 78.75 KIPS

COS X= 0.8660255

DL1= 90.93 KIP
DL2= 97.49 KIP
DL3= 90.93 KIP

PROOF LOAD 1.2 X DL

PROOF TIE 1 = 109.12 KIPS
PROOF TIE 2 = 116.99 KIPS
PROOF TIE 3 = 109.12 KIPS

MAX MOMENT 1 @ TIE 1 LOCATION

FORCE
6.73
29.92

ARM
4.00
2.67

MOMENT
+ 26.93
+ 79.79
=====

106.71

X IS BELOW 0.2 H X1= 5.88 FT

FORCE
6.31
26.30
46.14
78.75

ARM
9.63
8.38
2.94
5.38

MOMENT
+ 60.75
+ 220.24
+ 135.54
- 423.28
=====

-6.76

MAX MOMENT 2 @ TIE 2 LOCATION

FORCE
6.31
26.30
88.36
78.75

ARM
15
13.75
5.625
10.75

MOMENT
+ 94.67
+ 361.58
+ 497.01
- 846.57
=====

106.70

MAX MOMENT 3 BETWEEN 2nd AND 3rd TIE

X2 IS BELOW 0.2 H X2= 16.63

FORCE
6.31
26.30
130.57
78.75
84.43

ARM
20.38
19.13
8.31
16.13
5.38

MOMENT
+ 128.59
+ 502.93
+ 1085.39
- 1269.85
- 453.81
=====

-6.76

USE MAX MOM. = 106.71 FT-KIP
Sx= 38.81 IN^3 Gr-50
53.36 IN^3 A-36

use 12x53 to count for through beam penetration

DEPTH OF CUT H	26	FT
SOIL UNIT WEIGHT	120	#/C.FT.
SOLD. PILE SPACE S	8.5	FT
SURCHARGE LOAD	300	#/S.FT
Ka	0.33	
Kp	3	
LOCATION OF 1ST TIE	7	FT
LOCATION OF 2ND TIE	19	FT
TIE ANGLE UPPER=	30	DEGREE
TIE ANGLE LOWER=	30	DEGREE
Pb FACTOR ***	22	

COS X1=	0.8660	
COS X2=	0.8660	
Pb	572	PSF
Pq	99	PSF
LENGTH OF UPP.TRIA.	5.2	FT
LENGTH OF RECTANG	15.6	FT

TO FIND THE VALUE OF THE F_{x1}
MAKE SUM. OF MOM. @ F_{x2} LOCATION

	FOFCE	ARM	MOMENT
	15988.50	9.50	+ 151890.75
	12641.20	15.53	+ 196359.97
	67095.60	6.50	+ 462959.64
Fx1=	61504.30	12.00	- 738051.60
	5890.50	3.50	- 20616.75
	12641.20	3.53	- 44665.57
	8751.60	0.90	- 7876.44

SUM.OF MOMENT SHOULD=0 -0.000000

Fx1/ONE BAY=	61.5043	KIP		
Fx1/ BAY =	61.5043	KIP		
1stTIE FORCE/ BAY=	71.02	KIP	for 1.5 bay	106.53
PROOF TEST LOAD=	85.22	KIP	for 1.5 bay	127.83
		FT		

TO FIND THE VALUE OF F_{x2}
MAKE SUM. OF $F_x=0$

Fx2= 61504.3

Fx2/ONE BAY=	61.5043	KIP		
Fx2/ BAY =	61.5043	KIP		
TIE FORCE /BAY=	71.02	KIP	for 1.5 bay	106.53
PROOF TSEST LOAD=	85.22	KIP	for 1.5 bay	127.83

TO FIND MAX. MOM.
1-@FIRST TIE LOCATION

<u>FORCE</u>	<u>ARM</u>	<u>MOMENT</u>
5890.5	3.5	20616.75
12641.2	3.53333333	44665.5733
8751.6	0.9	7876.44

MAX.MOM= 73158.7633

1-MXMOM,@1st tie= 73.16 KIP FT

2-@ X-DISTANCE FROM THE TOP OF THE PILE
FIND X-DISTANCE

X= 13 FT

MAX.MOMENT @ X-DISTANCE

<u>FORCE</u>	<u>ARM</u>	<u>MOMENT</u>
10939.5	6.5	+ 71106.75
12641.2	9.5333333	+ 120512.773
37923.6	3.9	+ 147902.04
61504.3	6	- 369025.8

MAX.MOM= -29504.237

MAX.MOM@1stTIE LOCAT.=	73.16	KIP.FT
MAX.MOMENT@x-distance=	-29.50	KIP.FT

USE MAX.MOM. = 73.16 KIP.FT

DESIGN SOLD. PILE
MOD.SEC. FOR THE PILE = 26.60 CU.IN Gr-50

USE HP 12x53 Gr-50 Sx=66.8

CASE 6 PART A

H=	35.5	FT
S=	8.5	FT
SCH	250	PSF
TIE 1@	8	FT
TIE 2@	17.75	FT
TIE 3@	27.5	FT
Ka=	0.33	FT
TIE @	30	DEGREE
Pb FACTOR =	22	
upper triangle =	7.1	ft
lower triangle =	7.1	ft

W= 7.33975 K/1 VERTICAL FT

Fx1= 70.93 KIPS
Fx2= 71.56 KIPS
Fx3= 70.93 KIPS

COS X= 0.8660255

DL1= 81.91 KIP 122.86
DL2= 82.63 KIP 123.95
DL3= 81.91 KIP 122.86

PROOF LOAD 1.2 X DL

PROOF TIE 1 = 98.29 KIPS 147.43
PROOF TIE 2 = 99.16 KIPS 148.74
PROOF TIE 3 = 98.29 KIPS 147.43

MAX MOMENT 1 @ TIE 1 LOCATION

FORCE
5.61
29.92

ARM		MOMENT
4.00	+	22.44
2.67	+	79.79
=====		
102.23		

X IS BELOW 0.2 H X1= 5.78 FT

FORCE
4.98
23.57
42.39
70.93

ARM		MOMENT
9.33	+	46.43
8.14	+	191.87
2.89	+	122.39
4.88	-	345.80
=====		
14.90		

MAX MOMENT 2 @ TIE 2 LOCATION

FORCE
4.98
23.57
78.17
70.93

ARM		MOMENT
14.2	+	70.70
13.02	+	306.76
5.325	+	416.25
9.75	-	691.59
=====		
102.11		

MAX MOMENT 3 BETWEEN 2nd AND 3rd TIE

X2 IS BELOW 0.2 H X2= 15.53

FORCE
4.98
23.57
113.95
70.93
71.56

ARM		MOMENT
19.08	+	94.97
17.89	+	421.65
7.76	+	884.53
14.63	-	1037.39
4.88	-	348.87
=====		
14.90		

USE MAX MOM. = 102.23 FT-KIP
Sx= 37.17 IN³ Gr-50
51.11 IN³ A-36

CASE 6 PART B

DEPTH OF CUT H	26.5	FT
SOIL UNIT WEIGHT	125	#/C.FT.
SOLD PILE SPACE S	8.5	FT
SURCHARGE LOAD	250	#/S.FT
Ka	0.33	
Kp	3	
LOCATION OF 1ST TIE	7	FT
LOCATION OF 2ND TIE	19.5	FT
TIE ANGLE UPPER=	30	DEGREE
TIE ANGLE LOWER=	30	DEGREE
Pb FACTOR ***	22	

COS X1= 0.8660
COS X2= 0.8660
Pb 583 PSF
Pq 82.5 PSF
LENGTH OF UPP.TRIA. 5.3 FT
LENGTH OF RECTANG 15.9 FT

TO FIND THE VALUE OF THE Fx1
MAKE SUM. OF MOM. @Fx2 LOCATION

FORCE	ARM	MOMENT
13674.38	9.75	+ 133325.16
13132.08	15.97	+ 209675.46
70368.10	7.10	+ 499613.51
Fx1= 61819.86	12.50	- 772748.28
4908.75	3.50	- 17180.63
13132.08	3.47	- 45524.53
8424.35	0.85	- 7160.70
		=====

SUM.OF MOMENT SHOULD=0 -0.000000

Fx1= 61819.8625

Fx1/ONE BAY= 61.819863 KIP
Fx1/ BAY = 61.819863 KIP
1stTIE FORCE/ BAY= 71.38 KIP for 1.5 bay 107.08
PROOF TEST LOAD= 85.66 KIP for 1.5 bay 128.49

TO FIND THE VALUE OF Fx2
MAKE SUM. OF FX=0

Fx2= 61819.863

Fx2/ONE BAY= 61.819863 KIP
Fx2/ BAY = 61.819863 KIP
TIE FORCE /BAY= 71.38 KIP for 1.5 bay 107.08
PROOF TSEST LOAD= 85.66 KIP for 1.5 bay 128.49

TO FIND MAX. MOM.
1-@FIRST TIE LOCATION

FORCE	ARM	MOMENT
4908.75	3.5	17180.625
13132.075	3.4666667	45524.5267
8424.35	0.85	7160.6975
		=====

MAX.MOM= 69865.8492

1-MXMOM.@1st tie= 69.87 KIP.FT

2-@ X-DISTANCE FROM THE TOP OF THE PILE
FIND X-DISTANCE

X= 13.25 FT

MAX.MOMENT @ X-DISTANCE

FORCE	ARM	MOMENT
9291.5625	6.625	+ 61556.6016
13132.075	9.7166667	+ 127599.995
39396.225	3.975	+ 156599.894
61819.863	6.25	- 386374.141
		=====

MAX.MOM= -40617.549

MAX.MOM@1stTIE LOCAT.= 69.87 KIP.FT
MAX.MOMENT@x-distance= -40.62 KIP.FT

USE MAX.MOM.= 69.87 KIP.FT

DESIGN SOLD. PILE
MOD.SEC. FOR THE PILE = 25.41 CU.IN Gr-50

USE HP 12x53 Gr-50 Sx=66.8

CASE 6 PART C

DEPTH OF EXCA	22	FT
FRICTION- PHEE	25	DEGREE
SOIL UNIT WEIGHT	120	#/C.F
SURCHARGE S _{cu}	250	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	1	FT
PILE EFF. FACTOR	2.5	
K _a	0.33	
K _p	3	
TIE LOCATION	8.5	FT
TIE ANGLE X=	30	DEGREE
P _b FACTOR =	22	
ALLOWABLE STRESS =	33	KSI

COS X= 0.8660
 P_b = 484 PSF
 P_q = 82.5 PSF
 LENGTH OF upper triangle 4.4 FT
 LENGTH OF rectangle 13.2 FT

FORCE

5980.825
 9050.8
 18967.4
 9486.875
 37437.4
 9050.8
 18432
 5575.88
 811.008

PILE PENETR. D= 8.4 FT

PAS.PRESSUp1= 5780 PSF
 ACT.PRESSUp1= 871.2 PSF
 ACT.PRESSUp2= 253.44 PSF
 DIST.FROM DRE-
 DGE LINE TO TIE

try diff.value of D

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

ARM		MOMENT
4.25	+	25332.85625
5.552	+	50250.0418
2.05	+	34578.17
6.75	-	63901.40625
4.55	-	170340.17
10.552	-	95504.0418
17.724	+	328688.798
16.7	-	93113.858
17.724	-	14374.30579

TOTAL MOMENT@TIE LOCATION= -384.1 (THIS VALUE SHOULD APPROX.=0)

the (D) should be= 7.68 FT
used

USE D= 8.00 FT
 F.S= 1.25 HENCE OK

TOTAL LENGTH=(D)+H= 30.00 FT

CASE 6 PART C

DEPTH OF EXCA	22	FT
FRICTION- PHEE	25	DEGREE
SOIL UNIT WEIGHT	120	#/C.F
SURCHARGE Scu	250	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	1	FT
PILE EFF. FACTOR	2.5	
Ka	0.33	
Kp	3	
TIE LOCATION	8.5	FT
TIE ANGLE X=	30	DEGREE
Pb FACTOR =	22	
ALLOWABLE STRESS =	33	KSI

COS X=	0.8660	
Pb =	484	PSF
Pq =	82.5	PSF
LENGTH OFupper triangle	4.4	FT
LENGTH OF rectangle	13.2	FT

FORCE

5960.625
9050.8
16867.4
9466.875
37437.4
9050.8
18432
5575.68
811.008

try diff.value of D

PILE PENETR. D=	6.4	FT
PAS.PRESSUp1=	5760	PSF
ACT.PRESSUp1=	871.2	PSF
ACT.PRESSUp2=	253.44	PSF
DIST.FROM DRE-	13.5	FT
DGE LINE TO TIE		

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

ARM

4.25
5.552
2.05
6.75
4.55
10.552
17.724
16.7
17.724

MOMENT

+	25332.65625
+	50250.0416
+	34578.17
-	63901.40625
-	170340.17
-	95504.0416
+	326688.768
-	93113.856
-	14374.30579
=====	

TOTAL MOMENT@TIE LOCATION= -384.1
(THIS VALUE SHOULD APPROX.=0)

the (D)should be= 7.68 FT
used
USE D= 8.00 FT
F.S= 1.25 HENCE OK

TOTAL LENGTH=D+H= 30.00 FT

12/05/201009:50:14 PM1CASE6PARTC.123\

TO FIND TIEBACK FORCE
MAKE SUM. FOR Fx=0

F1=	15427.5	+
F2=	18101.6	+
F3=	54304.8	+
F4=	5575.68	+
F5=	18432	-
F6=	811.008	+

Fx/ONE BAY=	75.79	KIP
Fx/ONE FT=	8.92	KIP/FT

Fx/ BAY =	75.79	KIP
TIE FORCE/1 BAY=	87.51	KIP
PROOFTEST LOAD=	105.02	KIP

FIND MAX. MOMENT

1 @TIEBACK LOCATION = -8.5 FT

FORCE
5960.825
9050.8
18867.4

ARM
4.25
5.552
2.05

MOMENT
25332.66
50250.04
34578.17

MAX. MOMENT= 110.16 KIP.FT

2 @ X-DISTANCE FROM
THE TOP OF THE PILE

FIND X WHEN SUM.OF Fx =0

(ZERO SHEAR POINT)

X= 17.818896 FT

FORCE
12355.25
75788.59
9050.80
54382.54

ARM
8.8094479
9.1188958
14.670896
6.6094479

MOMENT
108842.94
691108.24
132783.34
359438.55

MAX. MOMENT=

90.04 KIP.FT

USE MAX. MOMENT=

110.16 KIP.FT
40.06 CUB. IN (Gr-50)

CASE 6 PART C

CASE 7 PART A

DEPTH OF EXCA	15	FT
FRICTION- PHEE	25	DEGREE
SOIL UNIT WEIGHT	110	#/C.F
SURCHARGE S _{cu}	300	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	0.83	FT
PILE EFF. FACTOR	2.5	
K _a	0.4	
K _p	3	
TIE LOCATION	6	FT
TIE ANGLE X=	30	DEGREE
Pb FACTOR =	22	
ALLOWABLE STRESS =	33	KSI

COS X= 0.8660
Pb = 330 PSF
Pq = 120 PSF
LENGTH OF Upper triangle 3 FT
LENGTH OF rectangle 9 FT

FORCE
6120
4207.5
8415
9180
16830
4207.5
9257.82
2848.56
493.7504

PILE PENETR. D= 5.2 FT
PAS.PRESSUp1= 3560.7 PSF
ACT.PRESSUp1= 547.8 PSF
ACT.PRESSUp2= 169.904 PSF
DIST.FROM DREDGE LINE TO TIE 9 FT

try diff.value of D

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

ARM		MOMENT
3	+	16390
3.99	+	16787.925
1.5	+	12622.5
4.5	-	41310
3	-	50490
6.99	-	29410.425
12.432	+	115093.2182
11.6	-	33043.296
12.432	-	6138.304973

TOTAL MOMENT@TIE LOCATION= 2471.6 (THIS VALUE SHOULD APPROX =0)

the (D)should be= 6.24 FT
used
USE D= 7.00 FT
F.S= 1.35 HENCE OK

TOTAL LENGTH=D+H= 22.00 FT

SHEAR DIAGRAM

MOMENT DIAGRAM

CASE 7 PART A

DEPTH OF EXCA	15	FT
FRICTION- PHEE	25	DEGREE
SOIL UNIT WEIGHT	110	#/C.F
SURCHARGE Scu	300	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	0.83	FT
PILE EFF. FACTOR	2.5	
Ka	0.4	
Kp	3	
TIE LOCATION	6	FT
TIE ANGLE X=	30	DEGREE
Pb FACTOR =	22	
ALLOWABLE STRESS =	33	KSI

COS X=	0.8660	
Pb =	330	PSF
Pq =	120	PSF
LENGTH OFupper triangle	3	FT
LENGTH OF rectangle	9	FT

FORCE

6120
4207.5
8415
9180
16830
4207.5
9257.82
2848.56
493.7504

try diff.value of D

PILE PENETR. D=	5.2	FT
PAS.PRESSUp1=	3560.7	PSF
ACT.PRESSUp1=	547.8	PSF
ACT.PRESSUp2=	189.904	PSF
DIST.FROM DRE- DGE LINE TO TIE	9	FT

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

ARM

3
3.99
1.5
4.5
3
6.99
12.432
11.6
12.432

MOMENT

+	18360
+	16787.925
+	12622.5
-	41310
-	50490
-	29410.425
+	115093.2182
-	33043.296
-	6138.304973

=====

TOTAL MOMENT@TIE LOCATION= 2471.6
(THIS VALUE SHOULD APPROX.=0)

the (D)should be= 6.24 FT
used

USE D= 7.00 FT
F.S= 1.35 HENCE OK

TOTAL LENGTH=D+H= 22.00 FT

TO FIND TIEBACK FORCE
MAKE SUM. FOR $F_x=0$

F1=	15300	+
F2=	8415	+
F3=	25245	+
F4=	2848.56	+
F5=	9257.82	-
F6=	493.7504	+

Fx/ONE BAY=	43.04	KIP
Fx/ONE FT=	5.06	KIP/FT

Fx/ BAY =	43.04	KIP
TIE FORCE/1 BAY=	49.70	KIP
PROOFTEST LOAD=	59.64	KIP

TIE FORCE/2 BAY=	99.41
PROOFTEST LOAD=	119.29

FIND MAX. MOMENT

1 @TIEBACK LOCATION = -6 FT

FORCE
6120
4207.5
8415

ARM
3
3.99
1.5

MOMENT
18360.00
16787.93
12622.50

MAX. MOMENT= 47.77 KIP.FT

2 @ X-DISTANCE FROM
THE TOP OF THE PILE

FIND X WHEN SUM.OF $F_x=0$

(ZERO SHEAR POINT)

X= 12.353462 FT

FORCE
12800.53
43044.49
4207.50
26236.46

ARM
6.1767308
6.3534615
10.343462
4.6767308

MOMENT
77830.09
273481.51
43520.11
122700.86

MAX. MOMENT= 29.43 KIP.FT

USE MAX. MOMENT= 47.77 KIP.FT
MOD. SECTION = 17.37 CUB. IN (Gr-60)

CASE 7 PART A

CASE 7 PART B

DEPTH OF EXCA	22.5	FT
FRICTION- PHEE	25	DEGREE
SOIL UNIT WEIGHT	115	#/C.F
SURCHARGE S_{cu}	300	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	1	FT
PILE EFF. FACTOR	2.5	
K_a	0.4	
K_p	3	
TIE LOCATION	9	FT
TIE ANGLE $X=$	30	DEGREE
P_b FACTOR $=$	22	
ALLOWABLE STRESS $=$	33	KSI

$\cos X = 0.8660$
 $P_b = 495$ PSF
 $P_q = 120$ PSF
 LENGTH OF Upper triangle 4.5 FT
 LENGTH OF rectangle 13.5 FT

FORCE

9180
 9466.875
 18933.75
 13770
 37867.5
 9466.875
 20235.328125
 7089.75
 1079.2175

PILE PENETR. $D =$	8.65	FT
PAS PRESSUP1 $=$	5908.125	PSF
ACT PRESSUP1 $=$	1035	PSF
ACT PRESSUP2 $=$	315.1	PSF
DIST. FROM DREDGE LINE TO TIE	13.5	FT

try diff. value of D

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

ARM		MOMENT
4.5	+	41310
5.985	+	56659.24688
2.25	+	42800.9375
6.75	-	92947.5
4.5	-	170403.75
10.485	-	99260.18438
18.021	+	364860.8481
16.925	-	119994.0188
18.021	-	19448.57857

TOTAL MOMENT@TIE LOCATION= 3177.0 (THIS VALUE SHOULD APPROX.=0)

the (D) should be= 8.22 FT
 used
 USE D= 9.00 FT
 F.S= 1.31 HENCE OK

TOTAL LENGTH= D+H= 31.50 FT

CASE 7 PART B

DEPTH OF EXCA	22.5	FT
FRICTION- PHEE	25	DEGREE
SOIL UNIT WEIGHT	115	#/C.F
SURCHARGE Scu	300	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	1	FT
PILE EFF. FACTOR	2.5	
Ka	0.4	
Kp	3	
TIE LOCATION	9	FT
TIE ANGLE X=	30	DEGREE
Pb FACTOR =	22	
ALLOWABLE STRESS =	33	KSI

COS X=	0.8660	
Pb =	495	PSF
Pq =	120	PSF
LENGTH OFupper triangle	4.5	FT
LENGTH OF rectangle	13.5	FT

FORCE
9180
9466.875
18933.75
13770
37867.5
9466.875
20235.328125
7089.75
1079.2175

try diff.value of D

PILE PENETR. D=	6.85	FT
PAS.PRESSUp1=	5908.125	PSF
ACT.PRESSUp1=	1035	PSF
ACT.PRESSUp2=	315.1	PSF
DIST.FROM DRE-	13.5	FT
DGE LINE TO TIE		

TO FIND PILE PEnetration D
MOMENT @TIEBACK LOCATION

ARM		MOMENT
4.5	+	41310
5.985	+	56659.24688
2.25	+	42600.9375
6.75	-	92947.5
4.5	-	170403.75
10.485	-	99260.18438
18.021	+	364660.8481
16.925	-	119994.0188
18.021	-	19448.57857
		=====

TOTAL MOMENT@TIE LOCATION= 3177.0
(THIS VALUE SHOULD APPROX.=0)

the (D)should be= 8.22 FT
used
USE D= 9.00 FT
F.S= 1.31 HENCE OK

TOTAL LENGTH=D+H= 31.50 FT

TO FIND TIEBACK FORCE
MAKE SUM. FOR $F_x=0$

F1=	22950	+
F2=	18933.75	+
F3=	56801.25	+
F4=	7089.75	+
F5=	20235.328125	-
F6=	1079.2175	+

Fx/ONE BAY=	88.62	KIP
Fx/ONE FT=	10.19	KIP/FT

Fx/ BAY =	88.62	KIP
TIE FORCE/1 BAY=	100.02	KIP
PROOFTEST LOAD=	120.02	KIP

FIND MAX. MOMENT

1 @TIEBACK LOCATION = -9 FT

FORCE
9180
9466.875
18933.75

ARM
4.5
5.965
2.25

MOMENT
41310.00
56659.25
42600.84

MAX. MOMENT= 140.57 KIP.FT

2 @ X-DISTANCE FROM
THE TOP OF THE PILE

FIND X WHEN SUM.OF $F_x=0$

(ZERO SHEAR POINT)

X= 18.380777 FT

FORCE
18748.39
86618.64
9466.88
58403.37

ARM
9.1903887
9.3807775
15.365777
6.9403887

MOMENT
172305.02
812550.18
145465.89
405342.10

MAX. MOMENT= 89.44 KIP.FT

USE MAX. MOMENT= 140.57 KIP.FT

MOD. SECTION = 51.12 CUB. IN (Gr-50)

CASE 7 PART B

SHEAR DIAGRAM

MOMENT DIAGRAM

CASE 7 PART C

DEPTH OF EXCA	15	FT
FRICTION- PHEE	30	DEGREE
SOIL UNIT WEIGHT	115	#/C.F
SURCHARGE S_{cu}	200	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	0.83	FT
PILE EFF. FACTOR	2.5	
K_a	0.33	
K_p	3	
TIE LOCATION	6	FT
TIE ANGLE $X=$	30	DEGREE
P_b FACTOR $=$	22	
ALLOWABLE STRESS $=$	33	KSI

$\cos X = 0.8660$
 $P_b = 330$ PSF
 $P_q = 65$ PSF
 LENGTH OF upper triangle 3 FT
 LENGTH OF rectangle 9 FT

FORCE
 3366
 4207.5
 8415
 5049
 16830
 4207.5
 7906.839375
 2220.84425
 347.9009325

PILE PENETR. $D = 4.7$ FT

PAS.PRESSUp1= 3364.6125 PSF
 ACT.PRESSUp1= 472.4775 PSF
 ACT.PRESSUp2= 148.04295 PSF
 DIST.FROM DRE- 9 FT
 DGE LINE TO TIE

try diff.value of D

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

ARM		MOMENT
3	+	10098
3.99	+	16787.825
1.5	+	12622.5
4.5	-	22720.5
3	-	50490
6.99	-	29410.425
12.102	+	95688.57012
11.35	-	25204.31224
12.102	-	4210.297085

TOTAL MOMENT@TIE LOCATION= 3161.5 (THIS VALUE SHOULD APPROX.=0)

the (D)should be= 5.64 FT
used

USE $D = 6.00$ FT
 $F.S = 1.28$ HENCE OK

TOTAL LENGTH= $D+H = 21.00$ FT

CASE 7 PART C

DEPTH OF EXCA	15	FT
FRICTION- PHEE	30	DEGREE
SOIL UNIT WEIGHT	115	#/C.F
SURCHARGE Scu	200	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	0.83	FT
PILE EFF. FACTOR	2.5	
Ka	0.33	
Kp	3	
TIE LOCATION	6	FT
TIE ANGLE X=	30	DEGREE
Pb FACTOR =	22	
ALLOWABLE STRESS =	33	KSI

COS X=	0.8660	
Pb =	330	PSF
Pq =	66	PSF
LENGTH OFupper triangle	3	FT
LENGTH OF rectangle	9	FT

FORCE
3366
4207.5
8415
5049
16830
4207.5
7906.839375
2220.64425
347.9009325

try diff.value of D

PILE PENETR. D=	4.7	FT
PAS.PRESSUp1=	3364.6125	PSF
ACT.PRESSUp1=	472.4775	PSF
ACT.PRESSUp2=	148.04295	PSF
DIST.FROM DRE-	9	FT
DGE LINE TO TIE		

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

ARM		MOMENT
3	+	10098
3.99	+	16787.925
1.5	+	12622.5
4.5	-	22720.5
3	-	50490
6.99	-	29410.425
12.102	+	95688.57012
11.35	-	25204.31224
12.102	-	4210.297085
		=====

TOTAL MOMENT@TIE LOCATION= 3161.5
(THIS VALUE SHOULD APPROX.=0)

the (D)should be= 5.64 FT
used
USE D= 6.00 FT
F.S= 1.28 HENCE OK

TOTAL LENGTH=D+H= 21.00 FT

TO FIND TIEBACK FORCE
MAKE SUM. FOR Fx=0

F1=	8415	+
F2=	8415	+
F3=	25245	+
F4=	2220.64425	+
F5=	7906.839375	-
F6=	347.9009325	+

Fx/ONE BAY=	36.74	KIP
Fx/ONE FT=	4.32	KIP/FT

Fx/ BAY =	36.74	KIP
TIE FORCE/1 BAY=	42.42	KIP
PROOFTEST LOAD=	50.90	KIP

FIND MAX. MOMENT

1 @TIEBACK LOCATION = -6 FT

FORCE	ARM	MOMENT
3368	3	10098.00
4207.5	3.99	16787.93
8415	1.5	12622.50

MAX. MOMENT= 39.51 KIP.FT

2 @ X-DISTANCE FROM THE TOP OF THE PILE

FIND X WHEN SUM.OF Fx =0 (ZERO SHEAR POINT)

X= 12.164054 FT

FORCE	ARM	MOMENT
6824.03	6.082027	41503.96
36736.71	6.164054	226447.04
4207.50	10.154054	42723.18
25705.17	4.582027	117781.79

MAX. MOMENT= 24.44 KIP.FT

USE MAX. MOMENT= 39.51 KIP.FT

MOD. SECTION = 14.37 CUB. IN (Gr-50)

CASE 7 PART C

CASE 8 PART A

DEPTH OF CUT H	28	FT
SOIL UNIT WEIGHT	115	#/C.FT.
SOLD. PILE SPACE S	8.5	FT
SURCHARGE LOAD	200	#/S.FT
Ka	0.33	
Kp	3	
LOCATION OF 1ST TIE	7	FT
LOCATION OF 2ND TIE	21	FT
TIE ANGLE UPPER=	30	DEGREE
TIE ANGLE LOWER=	30	DEGREE
Pb FACTOR ***	22	

COS X1= 0.8660
COS X2= 0.8660
Pb 616 PSF
Pq 66 PSF
LENGTH OF UPP.TRIA. 5.6 FT
LENGTH OF RECTANG 16.8 FT

TO FIND THE VALUE OF THE Fx1
MAKE SUM. OF MOM. @ Fx2 LOCATION

FORCE	ARM	MOMENT
11781.00	10.50	+ 123700.50
14660.80	17.27	+ 253143.15
80634.40	7.70	+ 620884.88
Fx1= 66497.20	14.00	- 930960.80
3927.00	3.50	- 13744.50
14660.80	3.27	- 47891.95
7330.40	0.70	- 5131.28
		=====
		SUM. OF MOMENT SHOULD=0 0.000000

Fx1/ONE BAY= 66.4972 KIP
Fx1/ BAY = 66.4972 KIP
1stTIE FORCE/ BAY= 76.78 KIP
PROOF TEST LOAD= 92.14 KIP
FT

Fx1= 66497.2

TO FIND THE VALUE OF Fx2
MAKE SUM. OF FX=0

Fx2= 66497.2

Fx2/ONE BAY= 66.4972 KIP
Fx2/ BAY = 66.4972 KIP
TIE FORCE /BAY= 76.78 KIP
PROOF TSEST LOAD= 92.14 KIP

TO FIND MAX. MOM.
1-@FIRST TIE LOCATION

FORCE	ARM	MOMENT
3927	3.5	13744.5
14660.8	3.265667	47891.9467
7330.4	0.7	5131.28
		=====
		MAX.MOM= 66767.7267

1-MXMOM.@1st tie= 66.77 KIP.FT

2-@ X-DISTANCE FROM THE TOP OF THE PILE
FIND X-DISTANCE

X= 14 FT

MAX.MOMENT @ X-DISTANCE

FORCE	ARM	MOMENT
7854	7	+ 54978
14660.8	10.266667	+ 150517.547
43982.4	4.2	+ 184726.08
66497.2	7	- 465480.4
		=====
		MAX.MOM= -75258.773

MAX.MOM@1stTIE LOCAT.= 66.77 KIP.FT
MAX.MOMENT@x-distance= -75.26 KIP.FT

USE MAX.MOM.= 75.26 KIP.FT

DESIGN SOLD. PILE
MOD.SEC. FOR THE PILE = 27.37 CU.IN Gr-50

CASE 8 PART B

DEPTH OF CUT H	25	FT
SOIL UNIT WEIGHT	115	#/C.FT.
SOLD. PILE SPACE S	8.5	FT
SURCHARGE LOAD	200	#/S.FT
Ka	0.33	
Kp	3	
LOCATION OF 1ST TIE	6	FT
LOCATION OF 2ND TIE	19	FT
TIE ANGLE UPPER=	30	DEGREE
TIE ANGLE LOWER=	30	DEGREE
Pb FACTOR ***	22	

COS X1=	0.8660	
COS X2=	0.8660	
Pb	550	PSF
Pq	66	PSF
LENGTH OF UPP. TRIA.	5	FT
LENGTH OF RECTANG	15	FT

TO FIND THE VALUE OF THE Fx1
MAKE SUM. OF MOM. @ Fx2 LOCATION

FORCE	ARM	MOMENT
10659.00	9.50	+ 101260.50
11687.50	15.67	+ 183104.17
65450.00	7.00	+ 458150.00
Fx1= 53762.50	13.00	- 698912.50
3366.00	3.00	- 10098.00
11687.50	2.67	- 31166.67
4675.00	0.50	- 2337.50
		=====

SUM. OF MOMENT SHOULD=0

-0.000000

Fx1/ONE BAY=	53.7625	KIP	
Fx1/ BAY =	53.7625	KIP	
1stTIE FORCE/ BAY=	62.08	KIP	for 1.5 bay
PROOF TEST LOAD=	74.50	KIP	for 1.5 bay
		FT	

TO FIND THE VALUE OF Fx2
MAKE SUM. OF FX=0

Fx2=	53762.5	
Fx2/ONE BAY=	53.7625	KIP
Fx2/ BAY =	53.7625	KIP
TIE FORCE /BAY=	62.08	KIP
PROOF TEST LOAD=	74.50	KIP
		for 1.5 bay
		111.74

TO FIND MAX. MOM.
1-@FIRST TIE LOCATION

FORCE	ARM	MOMENT
3366	3	10098
11687.5	2.666667	31166.667
4675	0.5	2337.5
		=====

MAX.MOM= 43602.1667

1-MXMOM.@1st tie= 43.60 KIP.FT

2-@ X-DISTANCE FROM THE TOP OF THE PILE
FIND X-DISTANCE

X= 12.5 FT

MAX.MOMENT @ X-DISTANCE

FORCE	ARM	MOMENT
7012.5	6.25	+ 43828.125
11687.5	9.166667	+ 107135.417
35062.5	3.75	+ 131484.375
53762.5	6.5	- 349456.25
		=====

MAX.MOM= -67008.333

MAX.MOM@1stTIE LOCAT.= 43.60 KIP.FT
MAX.MOMENT@x-distance= -67.01 KIP.FT

USE MAX.MOM. = 67.01 KIP.FT

DESIGN SOLD. PILE

MOD.SEC. FOR THE PILE = 24.37 CU.IN Gr-50

USE HP 12x53 Gr-50 Sx=66.8

CASE 8 PART C

DEPTH OF EXCA	15	FT
FRICTION- PHEE	30	DEGREE
SOIL UNIT WEIGHT	115	#/C.F
SURCHARGE S_{ou}	200	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	0.83	FT
PILE EFF. FACTOR	2.5	
K_a	0.33	
K_p	3	
TIE LOCATION	6	FT
TIE ANGLE $X=$	30	DEGREE
P_b FACTOR $=$	22	
ALLOWABLE STRESS $=$	33	KSI

$\cos X = 0.8660$
 $P_b = 330$ PSF
 $P_q = 66$ PSF
 LENGTH OF Upper triangle 3 FT
 LENGTH OF rectangle 9 FT

FORCE
 3366
 4207.5
 8415
 5049
 16830
 4207.5
 7573.9575
 2173.3965
 333.25413

PILE PENETR. $D = 4.6$ FT

PAS.PRESSUp1= 3293.025 PSF
 ACT.PRESSUp1= 472.4775 PSF
 ACT.PRESSUp2= 144.8931 PSF
 DIST.FROM DRE-
 DGE LINE TO TIE 9 FT

try diff.value of D

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

ARM		MOMENT
3	+	10098
3.99	+	16787.925
1.5	+	12622.5
4.5	-	22720.5
3	-	50490
6.99	-	29410.425
12.036	+	91160.15247
11.3	-	24559.38045
12.036	-	4011.046709

TOTAL MOMENT@TIE LOCATION= -522.8 (THIS VALUE SHOULD APPROX.=0)

the (D)should be= 5.52 FT

used

USE $D = 6.00$ FT

F.S= 1.30 HENCE OK

TOTAL LENGTH= $D+H = 21.00$ FT

SHEAR DIAGRAM

MOMENT DIAGRAM

CASE 8 PART C

DEPTH OF EXCA	15	FT
FRICTION- PHEE	30	DEGREE
SOIL UNIT WEIGHT	115	#/C.F
SURCHARGE Scu	200	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	0.83	FT
PILE EFF. FACTOR	2.5	
Ka	0.33	
Kp	3	
TIE LOCATION	6	FT
TIE ANGLE X=	30	DEGREE
Pb FACTOR =	22	
ALLOWABLE STRESS =	33	KSI

COS X=	0.8660	
Pb =	330	PSF
Pq =	66	PSF
LENGTH OFupper triangle	3	FT
LENGTH OF rectangle	9	FT

<u>FORCE</u>
3366
4207.5
8415
5049
16830
4207.5
7573.9575
2173.3965
333.25413

try diff.value of D

PILE PENETR. D=	4.6	FT
PAS.PRESSUp1=	3293.025	PSF
ACT.PRESSUp1=	472.4775	PSF
ACT.PRESSUp2=	144.8931	PSF
DIST.FROM DRE-	9	FT
DGE LINE TO TIE		

TO FIND PILE PEnetration D
MOMENT @TIEBACK LOCATION

<u>ARM</u>		<u>MOMENT</u>
3	+	10098
3.99	+	16787.925
1.5	+	12622.5
4.5	-	22720.5
3	-	50490
6.99	-	29410.425
12.036	+	91160.15247
11.3	-	24559.38045
12.036	-	4011.046709
		=====

TOTAL MOMENT@TIE LOCATION= -522.8
(THIS VALUE SHOULD APPROX.=0)

the (D)should be= 5.52 FT
used
USE D= 6.00 FT
F.S= 1.30 HENCE OK

TOTAL LENGTH=D+H= 21.00 FT

12/05/201009:57:43 PM1case8 part c.123\

TO FIND TIEBACK FORCE
MAKE SUM. FOR $F_x=0$

F1=	8415	+
F2=	8415	+
F3=	25245	+
F4=	2173.3965	+
F5=	7573.9575	-
F6=	333.25413	+

Fx/ONE BAY=	37.01	KIP
Fx/ONE FT=	4.35	KIP/FT

			<u>2 bays</u>	
Fx/ BAY =	37.01	KIP	74.02	KIP
TIE FORCE/1 BAY=	42.73	KIP	85.47	KIP
PROOFTEST LOAD=	51.28	KIP	102.56	KIP

FIND MAX. MOMENT

1 @TIEBACK LOCATION = -6 FT

FORCE
3366
4207.5
8415

ARM
3
3.99
1.5

MOMENT
10098.00
16787.93
12622.50

MAX. MOMENT= 39.51 KIP.FT

2 @ X-DISTANCE FROM
THE TOP OF THE PILE

FIND X WHEN SUM.OF $F_x=0$

(ZERO SHEAR POINT)

X= 12.244561 FT

FORCE
6869.20
37007.69
4207.50
25930.99

ARM
6.1222806
6.2445612
10.234561
4.6222806

MOMENT
42055.16
231096.81
43061.92
119860.33

MAX. MOMENT= 26.12 KIP.FT

USE MAX. MOMENT= 39.51 KIP.FT
MOD. SECTION = 14.37 CUB. IN (Gr-50)

CASE 8 PART C

SHEAR DIAGRAM

MOMENT DIAGRAM

CASE 9

DEPTH OF CUT H	33	FT
SOIL UNIT WEIGHT	120	#/C.FT.
SOLD. PILE SPACE S	8.5	FT
SURCHARGE LOAD	100	#/S.FT
Ka	0.33	
Kp	3	
LOCATION OF 1ST TIE	9	FT
LOCATION OF 2ND TIE	24	FT
TIE ANGLE UPPER=	30	DEGREE
TIE ANGLE LOWER=	30	DEGREE
Pb FACTOR ***	22	

COS X1= 0.8660
COS X2= 0.8660
Pb 726 PSF
Pq 33 PSF
LENGTH OF UPP. TRIA. 6.6 FT
LENGTH OF RECTANG 19.8 FT

TO FIND THE VALUE OF THE Fx1
MAKE SUM. OF MOM. @ Fx2 LOCATION

FORCE	ARM	MOMENT
6732.00	12.00	+ 80784.00
20364.30	19.60	+ 399140.28
107375.40	8.70	+ 934165.98
Fx1= 86085.45	15.00	- 1291281.75
2524.50	4.50	- 11360.25
20364.30	4.60	- 93675.78
14810.40	1.20	- 17772.48
		=====

SUM OF MOMENT SHOULD=0 0.000000

Fx1= 86085.45

Fx1/ONE BAY= 86.08545 KIP
Fx1/ BAY = 86.08545 KIP
1stTIE FORCE/ BAY= 99.40 KIP
PROOF TEST LOAD= 119.28 KIP
FT

TO FIND THE VALUE OF Fx2
MAKE SUM. OF FX=0

Fx2= 86085.45

Fx2/ONE BAY= 86.08545 KIP
Fx2/ BAY = 86.08545 KIP
TIE FORCE /BAY= 99.40 KIP
PROOF TSEST LOAD= 119.28 KIP

TO FIND MAX. MOM.
1-@FIRST TIE LOCATION

FORCE	ARM	MOMENT
2524.5	4.5	11360.25
20364.3	4.6	93675.78
14810.4	1.2	17772.48
		=====

MAX.MOM= 122808.51

1-MXMOM.@1st tie= 122.81 KIP.FT

2-@ X-DISTANCE FROM THE TOP OF THE PILE
FIND X-DISTANCE

X= 16.5 FT

MAX.MOMENT @ X-DISTANCE

FORCE	ARM	MOMENT
4628.25	8.25	+ 38183.0625
20364.3	12.1	+ 246408.03
61092.9	4.95	+ 302409.855
86085.45	7.5	- 645640.875
		=====

MAX MOM= -58639.928

MAX.MOM@1stTIE LOCAT.= 122.81 KIP.FT
MAX.MOMENT@x-distance= -58.64 KIP.FT

USE MAX.MOM. = 122.81 KIP.FT

DESIGN SOLD. PILE
MOD.SEC. FOR THE PILE = 44.66 CU.IN Gr-50

CASE 9 DURING CONSTRUCTION

DEPTH OF EXCA	25	FT
FRICTION- PHEE	30	DEGREE
SOIL UNIT WEIGHT	120	#/C.F
SURCHARGE S_{cu}	100	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	1	FT
PILE EFF. FACTOR	2.5	
K_a	0.33	
K_p	3	
TIE LOCATION	9	FT
TIE ANGLE α	30	DEGREE
P_b FACTOR	22	
ALLOWABLE STRESS	33	KSI

$COS \alpha$	0.8660	
P_b	550	PSF
P_q	33	PSF
LENGTH OF upper triangle	5	FT
LENGTH OF rectangle	15	FT

FORCE

2524.5
11667.5
18700
4488
51425
11687.5
24976.125
7375.5
1098.9495

PILE PENETR. $D=$ 7.45 FT

PAS.PRESSUp1=	6705	PSF
ACT.PRESSUp1=	990	PSF
ACT.PRESSUp2=	265.02	PSF
DIST.FROM DRE- DGE LINE TO TIE	16	FT

try diff.value of D

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

ARM		MOMENT
4.5	+	11360.25
5.65	+	66034.375
2	+	37400
8	-	35604
5.5	-	282937.5
12.65	-	147646.875
20.917	+	522425.9065
19.725	+	145481.7375
20.917	-	22886.72669

TOTAL MOMENT@TIE LOCATION= 2163.4 (THIS VALUE SHOULD APPROX.=0)

the (D)should be= 8.94 FT
used

USE $D=$ 9.00 FT
 $F.S.=$ 1.21 HENCE OK

TOTAL LENGTH= $D+H=$ 34.00 FT

CASE 9 DURING CONSTRUCTION

DEPTH OF EXCA	25	FT
FRICTION- PHEE	30	DEGREE
SOIL UNIT WEIGHT	120	#/C.F
SURCHARGE Scu	100	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	1	FT
PILE EFF. FACTOR	2.5	
Ka	0.33	
Kp	3	
TIE LOCATION	9	FT
TIE ANGLE X=	30	DEGREE
Pb FACTOR =	22	
ALLOWABLE STRESS =	33	KSI

COS X=	0.8660	
Pb =	550	PSF
Pq =	33	PSF
LENGTH OFupper triangle	5	FT
LENGTH OF rectangle	15	FT

FORCE
2524.5
11687.5
18700
4488
51425
11687.5
24976.125
7375.5
1098.9495

try diff.value of D

PILE PENETR. D=	7.45	FT
PAS.PRESSUp1=	6705	PSF
ACT.PRESSUp1=	990	PSF
ACT.PRESSUp2=	295.02	PSF
DIST.FROM DRE- DGE LINE TO TIE	16	FT

TO FIND PILE PEnetration D
MOMENT @TIEBACK LOCATION

ARM		MOMENT
4.5	+	11360.25
5.65	+	66034.375
2	+	37400
8	-	35904
5.5	-	282837.5
12.65	-	147846.875
20.917	+	522425.6066
19.725	-	145481.7375
20.917	-	22986.72669
		=====

TOTAL MOMENT@TIE LOCATION= 2163.4
(THIS VALUE SHOULD APPROX.=0)

the (D)should be= 8.94 FT
used
USE D= 9.00 FT
F.S= 1.21 HENCE OK

TOTAL LENGTH=D+H= 34.00 FT

TO FIND TIEBACK FORCE
MAKE SUM. FOR $F_x=0$

F1=	7012.5	+
F2=	23375	+
F3=	70125	+
F4=	7375.5	+
F5=	24976.125	-
F6=	1098.9495	+

Fx/ONE BAY=	84.01	KIP
Fx/ONE FT=	9.88	KIP/FT

Fx/ BAY =	84.01	KIP
TIE FORCE/1 BAY=	97.01	KIP
PROOFTEST LOAD=	116.41	KIP

FIND MAX. MOMENT

1 @TIEBACK LOCATION = -9 FT

FORCE
2524.5
11687.5
18700

ARM
4.5
5.65
2

MOMENT
11360.25
66034.38
37400.00

MAX. MOMENT= 114.79 KIP.FT

2 @ X-DISTANCE FROM
THE TOP OF THE PILE

FIND X WHEN SUM.OF $F_x=0$

(ZERO SHEAR POINT)

X= 19.311538 FT

FORCE
5416.89
84010.82
11687.50
66906.44

ARM
9.6557688
10.311538
15.961538
7.1557688

MOMENT
52304.20
866280.77
186550.47
478767.00

MAX. MOMENT= 148.66 KIP.FT

USE MAX. MOMENT= 148.66 KIP.FT
MOD. SECTION = 54.06 CUB. IN (Gr-50)

CASE 9 DURING CONSTRUCTION

CASE 10

DEPTH OF CUT H	30	FT
SOIL UNIT WEIGHT	115	#/C.FT.
SOLD. PILE SPACE S	8.5	FT
SURCHARGE LOAD	450	#/S.FT
Ka	0.33	
Kp	3	
LOCATION OF 1ST TIE	8	FT
LOCATION OF 2ND TIE	22	FT
TIE ANGLE UPPER=	30	DEGREE
TIE ANGLE LOWER=	30	DEGREE
Pb FACTOR ***	22	

COS X1= 0.8660
COS X2= 0.8660
Pb 660 PSF
Pq 148.5 PSF
LENGTH OF UPP TRIA. 6 FT
LENGTH OF RECTANG 18 FT

TO FIND THE VALUE OF THE Fx1
MAKE SUM. OF MOM. @Fx2 LOCATION

FOFCE	ARM	MOMENT
27769.50	11.00	+ 305464.50
16830.00	18.00	+ 302940.00
89760.00	8.00	+ 718080.00
Fx1= 86253.75	14.00	- 1207552.50
10098.00	4.00	- 40392.00
16830.00	4.00	- 67320.00
11220.00	1.00	- 11220.00
		=====
		SUM.OF MOMENT SHOULD=0 0.000000

Fx1/ONE BAY= 86.25375 KIP
Fx1/ BAY = 86.25375 KIP
1stTIE FORCE/ BAY= 99.60 KIP
PROOF TEST LOAD= 119.52 KIP
FT

TO FIND THE VALUE OF Fx2
MAKE SUM. OF FX=0

Fx2= 66253.75
Fx2/ONE BAY= 86.25375 KIP
Fx2/ BAY = 86.25375 KIP
TIE FORCE /BAY= 99.60 KIP
PROOF TSEST LOAD= 119.52 KIP

TO FIND MAX. MOM.
1-@FIRST TIE LOCATION

FORCE	ARM	MOMENT
10098	4	40392
16830	4	67320
11220	1	11220
		=====
		MAX.MOM= 118932

1-MXMOM.@1st tie= 118.93 KIP.FT

2-@ X-DISTANCE FROM THE TOP OF THE PILE
FIND X-DISTANCE

X= 15 FT

MAX MOMENT @ X-DISTANCE

FORCE	ARM	MOMENT
18933.75	7.5	+ 142003.125
16830	11	+ 185130
50490	4.5	+ 227205
86253.75	7	- 603776.25
		=====
		MAX.MOM= -49438.125

MAX.MOM@1stTIE LOCAT.= 118.93 KIP.FT
MAX.MOMENT@x-distance= -49.44 KIP.FT

USE MAX.MOM.= 118.93 KIP.FT

DESIGN SOLD. PILE
MOD.SEC. FOR THE PILE = 43.25 CU.IN Gr-50

CASE 10 DURING CONSTRUCTION

DEPTH OF EXCA	23.5	FT
FRICTION- PHEE	30	DEGREE
SOIL UNIT WEIGHT	120	#/C.F
SURCHARGE Scu	450	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	1	FT
PILE EFF. FACTOR	2.5	
Ka	0.33	
Kp	3	
TIE LOCATION	8	FT
TIE ANGLE X=	30	DEGREE
Pb FACTOR =	22	
ALLOWABLE STRESS =	33	KSI

COS X=	0.8660	
Pb =	517	PSF
Pq =	148.5	PSF
LENGTH OFupper triangle	4.7	FT
LENGTH OF rectangle	14.1	FT

FORCE

10098
10327.075
14501.85
19564.875
47460.6
10327.075
28084.5
7351.74
1235.718

try diff.value of D

PILE PENETR. D=	7.9	FT
PAS.PRESSUp1=	7110	PSF
ACT.PRESSUp1=	930.6	PSF
ACT.PRESSUp2=	312.84	PSF
DIST.FROM DRE- DGE LINE TO TIE	15.5	FT

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

ARM

4
4.851
1.65
7.75
5.4
12.351
20.714
19.45
20.714

MOMENT

+	40392
+	50096.64083
+	23928.0525
-	151627.7813
-	256287.24
-	127549.7033
+	581742.333
-	142991.343
-	25596.66265
=====	

TOTAL MOMENT@TIE LOCATION= -7893.7
(THIS VALUE SHOULD APPROX.=0)

the (D)should be= 9.48 FT
used
USE D= 9.50 FT
F.S= 1.20 HENCE OK

TOTAL LENGTH=D+H= 33.00 FT

TO FIND TIEBACK FORCE
MAKE SUM. FOR $F_x=0$

F1=	29662.875	+
F2=	20854.15	+
F3=	81982.45	+
F4=	7351.74	+
F5=	28084.5	-
F6=	1235.718	+

Fx/ONE BAY=	92.78	KIP
Fx/ONE FT=	10.92	KIP/FT

Fx/ BAY =	92.78	KIP
TIE FORCE/1 BAY=	107.14	KIP
PROOFTEST LOAD=	128.56	KIP

FIND MAX. MOMENT

1 @TIEBACK LOCATION = -8 FT

FORCE
10098
10327.075
14501.85

ARM
4
4.851
1.65

MOMENT
40392.00
50096.64
23928.05

MAX. MOMENT= 114.42 KIP.FT

2 @ X-DISTANCE FROM
THE TOP OF THE PILE

FIND X WHEN SUM.OF $F_x=0$

(ZERO SHEAR POINT)

X= 18.227694 FT

FORCE
23007.91
92782.43
10327.08
59447.45

ARM
9.113847
10.227694
15.078694
6.763847

MOMENT
209690.54
948950.33
155718.80
402093.46

MAX. MOMENT=	181.45	KIP.FT
USE MAX. MOMENT=	181.45	KIP.FT
MOD. SECTION =	65.98	CUB. IN (Gr-50)

CASE 10 DURING CONSTRUCTION

SHEAR DIAGRAM

MOMENT DIAGRAM

CASE 11 PART A

DEPTH OF EXCA	18	FT
FRICTION- PHEE	30	DEGREE
SOIL UNIT WEIGHT	125	#/C.F
SURCHARGE Squ	300	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	0.83	FT
PILE EFF. FACTOR	2.5	
Ka	0.33	
Kp	3	
TIE LOCATION	6.5	FT
TIE ANGLE X=	30	DEGREE
Pb FACTOR =	22	
ALLOWABLE STRESS =	33	KSI

COS X= 0.8660
Pb = 352 PSF
Pq = 99 PSF
LENGTH OF upper triangle 3.2 FT
LENGTH OF rectangle 9.6 FT

FORCE
5469.75
4787.2
9873.6
7994.25
18849.6
4787.2
9001.3889063
2634.918
396.08111188

PILE PENETR. D= 4.81 FT

PAS.PRESSUP1= 3742.7813 PSF
ACT.PRESSUP1= 547.8 PSF
ACT.PRESSUP2= 164.88238 PSF
DIST.FROM DRE-
DGE LINE TO TIE

try diff.value of D

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

ARM		MOMENT
3.25	+	17776.6875
4.356	+	20853.0432
1.65	+	16291.44
4.75	-	37972.6875
3.15	-	59376.24
7.356	-	35214.6432
12.8746	+	114069.0036
11.905	-	31368.69879
12.8746	-	5019.916169

TOTAL MOMENT@TIE LOCATION= 58.0 (THIS VALUE SHOULD APPROX.=0)

the (D)should be= 5.77 FT
used

USE D= 6.00 FT
F.S= 1.25 HENCE OK

TOTAL LENGTH= D+H= 22.00 FT

CASE 11 PART A

DEPTH OF EXCA	16	FT
FRICTION- PHEE	30	DEGREE
SOIL UNIT WEIGHT	125	#/C.F
SURCHARGE S_{cu}	300	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	0.83	FT
PILE EFF. FACTOR	2.5	
K_a	0.33	
K_p	3	
TIE LOCATION	6.5	FT
TIE ANGLE $X=$	30	DEGREE
P_b FACTOR =	22	
ALLOWABLE STRESS =	33	KSI

COS $X=$	0.8660	
$P_b =$	352	PSF
$P_q =$	99	PSF
LENGTH OF upper triangle	3.2	FT
LENGTH OF rectangle	9.6	FT

<u>FORCE</u>
5469.75
4787.2
9873.6
7994.25
18849.6
4787.2
9001.3889063
2634.918
396.06111188

try diff.value of D

PILE PENETR. $D=$	4.81	FT
PAS.PRESSUp1=	3742.7813	PSF
ACT.PRESSUp1=	547.8	PSF
ACT.PRESSUp2=	164.68238	PSF
DIST.FROM DRE-	9.5	FT
DGE LINE TO TIE		

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

<u>ARM</u>		<u>MOMENT</u>
3.25	+	17776.6875
4.356	+	20853.0432
1.65	+	16291.44
4.75	-	37972.6875
3.15	-	59376.24
7.356	-	35214.6432
12.6746	+	114089.0038
11.905	-	31368.69879
12.6746	-	5019.916169
		=====

TOTAL MOMENT@TIE LOCATION= 58.0
(THIS VALUE SHOULD APPROX.=0)

the (D)should be= 5.77 FT
used
USE $D=$ 6.00 FT
 $F.S=$ 1.25 HENCE OK

TOTAL LENGTH $t=D+H=$ 22.00 FT

TO FIND TIEBACK FORCE
MAKE SUM. FOR Fx=0

F1=	13464	+
F2=	9574.4	+
F3=	28723.2	+
F4=	2634.918	+
F5=	9001.3889063	-
F6=	396.06111188	+

Fx/ONE BAY=	45.79	KIP
Fx/ONE FT=	5.39	KIP/FT

Fx/ BAY =	45.79	KIP	1.5 bay	68.69
TIE FORCE/1 BAY=	52.88	KIP		79.31
PROOFTEST LOAD=	63.45	KIP		95.18

FIND MAX. MOMENT

1 @TIEBACK LOCATION = -6.5 FT

FORCE
5469.75
4787.2
9873.6

ARM
3.25
4.356
1.65

MOMENT
17776.69
20853.04
16291.44

MAX. MOMENT= 54.92 KIP.FT

2 @ X-DISTANCE FROM
THE TOP OF THE PILE

FIND X WHEN SUM.OF Fx =0

(ZERO SHEAR POINT)

X= 13.193789 FT

FORCE
11102.57
45791.19
4787.20
29901.42

ARM
6.5968945
6.693789
11.049789
4.9968945

MOMENT
73242.51
306516.57
52897.55
149414.23

MAX. MOMENT= 30.96 KIP.FT

USE MAX. MOMENT= 54.92 KIP.FT
MOD. SECTION = 19.97 CUB. IN (Gr-50)

CASE 11 PART A

SHEAR DIAGRAM

MOMENT DIAGRAM

CASE 11 PART B

DEPTH OF EXCA	14	FT
FRICTION- PHEE	30	DEGREE
SOIL UNIT WEIGHT	125	#/C.F
SURCHARGE S_{cu}	300	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	0.83	FT
PILE EFF. FACTOR	2.5	
K_a	0.33	
K_p	3	
TIE LOCATION	5.5	FT
TIE ANGLE $X=$	30	DEGREE
P_b FACTOR $=$	22	
ALLOWABLE STRESS $=$	33	KSI

$\cos X = 0.8660$
 $P_b = 308$ PSF
 $P_q = 99$ PSF
 LENGTH OF upper triangle 2.8 FT
 LENGTH OF rectangle 8.4 FT

FORCE

4628.25
 3665.2
 7068.6
 7152.75
 14922.6
 3665.2
 6863.0625
 2013.165
 301.97475

PILE PENETR. $D=$	4.2	FT
PAS.PRESSUp1 $=$	3268.125	PSF
ACT.PRESSUp1 $=$	479.325	PSF
ACT.PRESSUp2 $=$	143.7975	PSF
DIST.FROM DRE- DGE LINE TO TIE	8.5	FT

try diff.value of D

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

ARM		MOMENT
2.75	+	12727.6875
3.624	+	13282.6848
1.35	+	9542.61
4.25	-	30399.1875
2.85	-	42529.41
6.624	-	24278.2848
11.272	+	77360.4405
10.6	-	21339.549
11.272	-	3403.659382

TOTAL MOMENT@TIE LOCATION= -9030.9 (THIS VALUE SHOULD APPROX.=0)

the (D)should be= 5.04 FT
 used
 USE D= 5.50 FT
 F.S= 1.31 HENCE OK

TOTAL LENGTH=D+H= 19.50 FT

CASE 11 PART B

DEPTH OF EXCA	14	FT
FRICTION- PHEE	30	DEGREE
SOIL UNIT WEIGHT	125	#/C.F
SURCHARGE Scu	300	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	0.83	FT
PILE EFF. FACTOR	2.5	
Ka	0.33	
Kp	3	
TIE LOCATION	5.5	FT
TIE ANGLE X=	30	DEGREE
Pb FACTOR =	22	
ALLOWABLE STRESS =	33	KSI

COS X=	0.8660	
Pb =	308	PSF
Pq =	99	PSF
LENGTH OFupper triangle	2.8	FT
LENGTH OF rectangle	8.4	FT

FORCE
4628.25
3665.2
7068.6
7152.75
14922.6
3665.2
6863.0625
2013.165
301.97475

try diff.value of D

PILE PENETR. D=	4.2	FT
PAS.PRESSUp1=	3268.125	PSF
ACT.PRESSUp1=	479.325	PSF
ACT.PRESSUp2=	143.7975	PSF
DIST.FROM DRE-	8.5	FT
DGE LINE TO TIE		

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

ARM		MOMENT
2.75	+	12727.6875
3.624	+	13282.6848
1.35	+	9542.61
4.25	-	30399.1875
2.85	-	42529.41
6.624	-	24278.2848
11.272	+	77360.4405
10.6	-	21339.549
11.272	-	3403.859382
		=====

TOTAL MOMENT@TIE LOCATION= -9036.9
(THIS VALUE SHOULD APPROX.=0)

the (D)should be= 5.04 FT
used
USE D= 5.50 FT
F.S= 1.31 HENCE OK

TOTAL LENGTH=D+H= 19.50 FT

TO FIND TIEBACK FORCE
MAKE SUM. FOR $F_x=0$

F1=	11781	+
F2=	7330.4	+
F3=	21991.2	+
F4=	2013.165	+
F5=	6863.0625	-
F6=	301.97475	+

Fx/ONE BAY=	36.55	KIP
Fx/ONE FT=	4.30	KIP/FT

Fx/ BAY =	36.55	KIP	2 bayS	73.11
TIE FORCE/1 BAY=	42.21	KIP		84.42
PROOFTEST LOAD=	50.65	KIP		101.30

FIND MAX. MOMENT

1 @TIEBACK LOCATION = -5.5 FT

FORCE
4628.25
3665.2
7068.6

ARM
2.75
3.624
1.35

MOMENT
12727.69
13282.68
9542.61

MAX. MOMENT= 35.55 KIP.FT

2 @ X-DISTANCE FROM
THE TOP OF THE PILE

FIND X WHEN SUM.OF $F_x=0$

(ZERO SHEAR POINT)

X= 11.625922 FT

FORCE
9783.21
36554.68
3665.20
23106.26

ARM
5.812961
6.125922
9.749922
4.412961

MOMENT
56869.44
223931.10
35735.41
101967.04

MAX. MOMENT= 29.36 KIP.FT

USE MAX. MOMENT= 35.55 KIP.FT

MOD. SECTION = 12.93 CUB. IN (Gr-50)

CASE 11 PART B

CASE 11 PART C

DEPTH OF EXCA	12	FT
FRICTION- PHEE	30	DEGREE
SOIL UNIT WEIGHT	125	#/C F
SURCHARGE S_{cu}	300	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	1	FT
PILE EFF. FACTOR	2.5	
K_a	0.33	
K_p	3	
TIE LOCATION	5	FT
TIE ANGLE $X=$	30	DEGREE
P_b FACTOR $=$	22	
ALLOWABLE STRESS $=$	33	KSI

$\cos X = 0.8660$
 $P_b = 264$ PSF
 $P_q = 99$ PSF
 LENGTH OF upper triangle = 2.4 FT
 LENGTH OF rectangle = 7.2 FT

FORCE

4207.5
 2692.8
 5834.4
 5890.5
 10322.4
 2692.8
 5104.6875
 1633.5
 224.60625

PILE PENETR. $D = 3.3$ FT

PAS PRESSUP1 = 3093.75 PSF
 ACT PRESSUP1 = 495 PSF
 ACT PRESSUP2 = 136.125 PSF
 DIST FROM DRE-
 DGE LINE TO TIE 7 FT

try diff. value of D

TO FIND PILE Penetration D
 MOMENT @TIEBACK LOCATION

ARM		MOMENT
2.5	+	10518.75
3.392	+	9133.9776
1.3	+	7584.72
3.5	-	20616.75
2.3	-	23741.52
5.392	-	14519.5776
9.178	+	46850.82188
8.85	-	14129.775
9.178	-	2081.436183

TOTAL MOMENT@TIE LOCATION = -880.8 (THIS VALUE SHOULD APPROX.=0)

the (D) should be = 3.95 FT
 used
 USE $D = 4.00$ FT
 F.S. = 1.21 HENCE OK

TOTAL LENGTH = $D + H = 16.00$ FT

SHEAR DIAGRAM

MOMENT DIAGRAM

CASE 11 PART C

DEPTH OF EXCA	12	FT
FRICTION- PHEE	30	DEGREE
SOIL UNIT WEIGHT	125	#/C.F
SURCHARGE Scu	300	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	1	FT
PILE EFF. FACTOR	2.5	
Ka	0.33	
Kp	3	
TIE LOCATION	5	FT
TIE ANGLE X=	30	DEGREE
Pb FACTOR =	22	
ALLOWABLE STRESS =	33	KSI

COS X=	0.8660	
Pb =	264	PSF
Pq =	99	PSF
LENGTH OFupper triangle	2.4	FT
LENGTH OF rectangle	7.2	FT

FORCE

4207.5
2692.8
5834.4
5890.5
10322.4
2692.8
5104.6875
1633.5
224.60625

try diff.value of D

PILE PENETR. D=	3.3	FT
-----------------	-----	----

PAS.PRESSUp1=	3093.75	PSF
ACT.PRESSUp1=	495	PSF
ACT.PRESSUp2=	136.125	PSF
DIST.FROM DRE- DGE LINE TO TIE	7	FT

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

ARM

2.5
3.392
1.3
3.5
2.3
5.392
9.178
8.65
9.178

MOMENT

+	10518.75
+	9133.9776
+	7584.72
-	20616.75
-	23741.52
-	14519.5776
+	46850.82188
-	14129.775
-	2061.436163
=====	

TOTAL MOMENT@TIE LOCATION= -980.8
(THIS VALUE SHOULD APPROX.=0)

the (D)should be= 3.96 FT
used

USE D= 4.00 FT
F.S= 1.21 HENCE OK

TOTAL LENGTH=D+H= 16.00 FT

TO FIND TIEBACK FORCE
MAKE SUM. FOR Fx=0

F1=	10098	+
F2=	5385.6	+
F3=	16156.8	+
F4=	1633.5	+
F5=	5104.6875	-
F6=	224.60625	+

Fx/ONE BAY=	28.39	KIP
Fx/ONE FT=	3.34	KIP/FT

Fx/ BAY =	28.39	KIP	2 bayS	56.79
TIE FORCE/1 BAY=	32.79	KIP		65.57
PROOFTEST LOAD=	39.34	KIP		78.69

FIND MAX. MOMENT

1 @TIEBACK LOCATION = -5 FT

FORCE
4207.5
2692.8
5834.4

ARM
2.5
3.392
1.3

MOMENT
10518.75
9133.98
7584.72

MAX. MOMENT= 27.24 KIP.FT

2 @ X-DISTANCE FROM
THE TOP OF THE PILE

FIND X WHEN SUM.OF Fx =0

(ZERO SHEAR POINT)

X= 10.075067 FT

FORCE
8478.17
28393.82
2692.80
17222.85

ARM
5.0375334
5.0750668
8.4670668
3.8375334

MOMENT
42709.06
144100.53
22800.12
66093.26

MAX. MOMENT= 12.50 KIP.FT

USE MAX. MOMENT= 27.24 KIP.FT
MOD. SECTION = 9.90 CUB. IN (Gr-50)

CASE 11 PART C

CASE 12 PART A 30 FT WALL

DEPTH OF CUT H	30	FT
SOIL UNIT WEIGHT	120	#/C.FT.
SOLD. PILE SPACE S	8.5	FT
SURCHARGE LOAD	300	#/S.FT
Ka	0.33	
Kp	3	
LOCATION OF 1ST TIE	8	FT
LOCATION OF 2ND TIE	22	FT
TIE ANGLE UPPER=	30	DEGREE
TIE ANGLE LOWER=	30	DEGREE
Pb FACTOR ***	22	

COS X1= 0.8660
COS X2= 0.8660
Pb 660 PSF
Pq 99 PSF
LENGTH OF UPP. TRIA 6 FT
LENGTH OF RECTANG 18 FT

TO FIND THE VALUE OF THE Fx1
MAKE SUM. OF MOM. @Fx2 LOCATION

FORCE	ARM	MOMENT
18513.00	11.00	+ 203643.00
16830.00	18.00	+ 302940.00
89760.00	8.00	+ 718080.00
Fx1= 79942.50	14.00	- 1119195.00
6732.00	4.00	- 26928.00
16830.00	4.00	- 67320.00
11220.00	1.00	- 11220.00
		=====
		SUM.OF MOMENT SHOULD=0 0.000000

Fx1/ONE BAY= 79.9425 KIP
Fx1/ BAY = 79.9425 KIP
1stTIE FORCE/ BAY= 92.31 KIP
PROOF TEST LOAD= 110.77 KIP

Fx1= 79942.5

TO FIND THE VALUE OF Fx2
MAKE SUM. OF FX=0

Fx2= 79942.5
Fx2/ONE BAY= 79.9425 KIP
Fx2/ BAY = 79.9425 KIP
TIE FORCE /BAY= 92.31 KIP
PROOF TSEST LOAD= 110.77 KIP

TO FIND MAX. MOM.
1-@FIRST TIE LOCATION

FORCE	ARM	MOMENT
6732	4	26928
16830	4	67320
11220	1	11220
		=====
		MAX.MOM= 105468

1-MXMOM.@1st tie= 105.47 KIP.FT

2-@ X-DISTANCE FROM THE TOP OF THE PILE
FIND X-DISTANCE

X= 15 FT

MAX.MOMENT @ X-DISTANCE

FORCE	ARM	MOMENT
12622.5	7.5	+ 94668.75
16830	11	+ 185130
50490	4.5	+ 227205
79942.5	7	- 559597.5
		=====
		MAX.MOM= -52593.75

MAX.MOM@1stTIE LOCAT.= 105.47 KIP.FT
MAX.MOMENT@x-distance= -52.59 KIP.FT

USE MAX.MOM. = 105.47 KIP.FT

DESIGN SOLD. PILE
MOD.SEC. FOR THE PILE = 38.35 CU.IN Gr-50

USE HP 12x53 Gr-50

CASE 12 PART B

DEPTH OF EXCA	25	FT
FRICTION- PHEE	30	DEGREE
SOIL UNIT WEIGHT	120	#/C.F
SURCHARGE q_{cu}	300	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	1	FT
PILE EFF. FACTOR	2.5	
K_a	0.33	
K_p	3	
TIE LOCATION	10	FT
TIE ANGLE $X=$	30	DEGREE
P_b FACTOR $=$	22	
ALLOWABLE STRESS $=$	33	KSI

$\cos X = 0.8660$
 $P_b = 550$ PSF
 $P_q = 99$ PSF
 LENGTH OF Upper triangle 5 FT
 LENGTH OF rectangle 15 FT

FORCE

8415
 11687.5
 23375
 12822.5
 46750
 11687.5
 22050
 6930
 970.2

PILE PENETR. $D = 7$ FT

PAS.PRESSUp1= 6300 PSF
 ACT.PRESSUp1= 990 PSF
 ACT.PRESSUp2= 277.2 PSF
 DIST.FROM DREDGE LINE TO TIE 15 FT

try diff.value of D

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

ARM		MOMENT
5	+	42075
6.85	+	77721.875
2.5	+	58437.5
7.5	-	94658.75
5	-	233750
11.85	-	138159.375
19.82	+	432621
18.5	-	128205
19.82	-	19035.324

TOTAL MOMENT@TIE LOCATION= -993.1 (THIS VALUE SHOULD APPROX.=0)

the (D) should be= 8.40 FT
used

USE $D = 9.00$ FT
 $F.S. = 1.29$ HENCE OK

TOTAL LENGTH $= D + H = 34.00$ FT

CASE 12 PART B

DEPTH OF EXCA	25	FT
FRICTION- PHEE	30	DEGREE
SOIL UNIT WEIGHT	120	#/C.F
SURCHARGE Scu	300	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	1	FT
PILE EFF. FACTOR	2.5	
Ka	0.33	
Kp	3	
TIE LOCATION	10	FT
TIE ANGLE X=	30	DEGREE
Pb FACTOR =	22	
ALLOWABLE STRESS =	33	KSI

COS X=	0.8660	
Pb =	550	PSF
Pq =	99	PSF
LENGTH OFupper triangle	5	FT
LENGTH OF rectangle	15	FT

FORCE

8415
11687.5
23375
12622.5
46750
11687.5
22050
6930
970.2

try diff.value of D

PILE PENETR. D=	7	FT
-----------------	---	----

PAS.PRESSUp1=	6300	PSF
ACT.PRESSUp1=	990	PSF
ACT.PRESSUp2=	277.2	PSF
DIST.FROM DRE-	15	FT
DGE LINE TO TIE		

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

ARM

5
6.65
2.5
7.5
5
11.65
19.62
18.5
19.62

MOMENT

+	42075
+	77721.875
+	58437.5
-	94668.75
-	233750
-	136159.375
+	432621
-	128205
-	19035.324

=====

TOTAL MOMENT@TIE LOCATION= -963.1
(THIS VALUE SHOULD APPROX.=0)

the (D)should be= 8.40 FT
used

USE D= 9.00 FT
F.S= 1.29 HENCE OK

TOTAL LENGTH=D+H= 34.00 FT

TO FIND TIEBACK FORCE
MAKE SUM. FOR $F_x=0$

F1=	21037.5	+
F2=	23375	+
F3=	70125	+
F4=	6930	+
F5=	22050	-
F6=	970.2	+

Fx/ONE BAY=	100.39	KIP
Fx/ONE FT=	11.81	KIP/FT

Fx/ BAY =	100.39	KIP
TIE FORCE/1 BAY=	115.92	KIP
PROOFTEST LOAD=	139.10	KIP

FIND MAX. MOMENT

1 @TIEBACK LOCATION = -10 FT

FORCE
8415
11687.5
23375

ARM
5
6.65
2.5

MOMENT
42075.00
77721.88
58437.50

MAX. MOMENT= 178.23 KIP.FT

2 @ X-DISTANCE FROM
THE TOP OF THE PILE

FIND X WHEN SUM.OF $F_x=0$

(ZERO SHEAR POINT)

X= 20.31636 FT

FORCE
17096.22
100387.70
11687.50
71603.98

ARM
10.15818
10.31636
16.96636
7.65818

MOMENT
173666.45
1035635.65
198294.33
548356.19

MAX. MOMENT= 115.32 KIP.FT

USE MAX. MOMENT= 178.23 KIP.FT
MOD. SECTION = 64.81 CUB. IN (Gr-50)

CASE 12 PART B

SHEAR DIAGRAM

MOMENT DIAGRAM

CASE 12 PART C

DEPTH OF EXCA	21	FT
FRICTION- PHEE	30	DEGREE
SOIL UNIT WEIGHT	120	#/C.F
SURCHARGE S _{cu}	300	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	1	FT
PILE EFF. FACTOR	2.5	
K _a	0.33	
K _p	3	
TIE LOCATION	8.5	FT
TIE ANGLE X=	30	DEGREE
Pb FACTOR =	22	
ALLOWABLE STRESS =	33	KSI

COS X= 0.8660
Pb = 462 PSF
Pq = 99 PSF
LENGTH OF upper triangle 4.2 FT
LENGTH OF rectangle 12.6 FT

FORCE

7152.75
8246.7
16888.1
10518.75
32594.1
8246.7
15400.125
4364.88
677.6055

PILE PENETR D= 5.85 FT

PAS.PRESSUp1= 5265 PSF
ACT.PRESSUp1= 831.6 PSF
ACT.PRESSUp2= 231.66 PSF
DIST FROM DRE-
DGE LINE TO TIE 12.5 FT

try diff.value of D

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

ARM		MOMENT
4.25	+	30399.1875
5.685	+	48890.7352
2.15	+	36305.115
6.25	-	65742.1875
4.15	-	135265.515
9.688	-	78877.5382
18.381	+	251981.4451
15.425	-	75040.4855
18.381	-	11088.30359

TOTAL MOMENT@TIE LOCATION= -1455.5 (THIS VALUE SHOULD APPROX.=0)

the (D)should be= 7.02 FT

used
USE D= 7.50 FT
F.S= 1.28 HENCE OK

TOTAL LENGTH=D+H= 28.50 FT

CASE 12 PART C

DEPTH OF EXCA	21	FT
FRICTION- PHEE	30	DEGREE
SOIL UNIT WEIGHT	120	#/C.F
SURCHARGE Scu	300	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	1	FT
PILE EFF. FACTOR	2.5	
Ka	0.33	
Kp	3	
TIE LOCATION	8.5	FT
TIE ANGLE X=	30	DEGREE
Pb FACTOR =	22	
ALLOWABLE STRESS =	33	KSI

COS X=	0.8660	
Pb =	462	PSF
Pq =	99	PSF
LENGTH OFupper triangle	4.2	FT
LENGTH OF rectangle	12.6	FT

<u>FORCE</u>
7152.75
8246.7
16886.1
10518.75
32594.1
8246.7
15400.125
4864.86
677.6055

try diff.value of D

PILE PENETR. D=	5.85	FT
PAS.PRESSUp1=	5265	PSF
ACT.PRESSUp1=	831.6	PSF
ACT.PRESSUp2=	231.66	PSF
DIST.FROM DRE-	12.5	FT
DGE LINE TO TIE		

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

<u>ARM</u>		<u>MOMENT</u>
4.25	+	30399.1875
5.686	+	46890.7362
2.15	+	36305.115
6.25	-	65742.1875
4.15	-	135265.515
9.686	-	79877.5362
16.361	+	251961.4451
15.425	-	75040.4655
16.361	-	11086.30359
		=====

TOTAL MOMENT@TIE LOCATION= -1455.5
(THIS VALUE SHOULD APPROX.=0)

the (D)should be= 7.02 FT
used
USE D= 7.50 FT
F.S= 1.28 HENCE OK

TOTAL LENGTH=D+H= 28.50 FT

TO FIND TIEBACK FORCE
MAKE SUM. FOR $F_x=0$

F1=	17671.5	+
F2=	16493.4	+
F3=	49480.2	+
F4=	4864.86	+
F5=	15400.125	-
F6=	677.6055	+

Fx/ONE BAY=	73.79	KIP
Fx/ONE FT=	8.68	KIP/FT

Fx/ BAY =	73.79	KIP
TIE FORCE/1 BAY=	85.20	KIP
PROOFTEST LOAD=	102.24	KIP

FIND MAX. MOMENT

1 @TIEBACK LOCATION = -8.5 FT

FORCE
7152.75
8246.7
16886.1

ARM
4.25
5.686
2.15

MOMENT
30399.19
46890.74
36305.12

MAX. MOMENT= 113.60 KIP.FT

2 @ X-DISTANCE FROM
THE TOP OF THE PILE

FIND X WHEN SUM.OF $F_x=0$

(ZERO SHEAR POINT)

X= 17.203343 FT

FORCE
14476.61
73787.44
8246.70
51064.13

ARM
8.6016714
8.7033429
14.389343
6.5016714

MOMENT
124523.07
642197.39
118664.59
332002.18

MAX. MOMENT= 67.01 KIP.FT

USE MAX. MOMENT= 113.60 KIP.FT
MOD. SECTION = 41.31 CUB. IN (Gr-50)

CASE 12 PART C

CASE 12 PART A DURING CONSTRUCTION

DEPTH OF EXCA	23	FT
FRICTION- PHEE	30	DEGREE
SOIL UNIT WEIGHT	120	#/C.F
SURCHARGE Sou	300	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	1	FT
PILE EFF. FACTOR	2.5	
Ka	0.33	
Kp	3	
TIE LOCATION	8	FT
TIE ANGLE X=	30	DEGREE
Pb FACTOR =	22	
ALLOWABLE STRESS =	33	KSI

COS X= 0.8660
Pb = 506 PSF
Pa = 99 PSF
LENGTH OF upper triangle 4.8 FT
LENGTH OF rectangle 13.8 FT

FORCE

6732
9892.3
14623.4
12822.5
44730.4
9892.3
24976.125
6785.46
1098.9465

PILE PENETR. D= 7.45 FT

PAS.PRESSUp1= 6705 PSF
ACT.PRESSUp1= 910.8 PSF
ACT.PRESSUp2= 295.02 PSF
DIST.FROM DREDGE LINE TO TIE 15 FT

try diff.value of D

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

ARM		MOMENT
4	+	26928
4.918	+	48650.3314
1.7	+	24859.78
7.5	-	94668.75
5.2	-	232598.08
11.918	-	117896.4314
19.917	+	497449.4816
18.725	-	127057.7385
19.917	-	21887.77719

TOTAL MOMENT@TIE LOCATION= 3778.8 (THIS VALUE SHOULD APPROX.=0)

the (D)should be= 8.94 FT

used

USE D= 9.00 FT
F.S= 1.21 HENCE OK

TOTAL LENGTH= D+H= 32.00 FT

CASE 12 PART A DURING CONSTRUCTION

DEPTH OF EXCA	23	FT
FRICTION- PHEE	30	DEGREE
SOIL UNIT WEIGHT	120	#/C.F
SURCHARGE Scu	300	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	1	FT
PILE EFF. FACTOR	2.5	
Ka	0.33	
Kp	3	
TIE LOCATION	8	FT
TIE ANGLE X=	30	DEGREE
Pb FACTOR =	22	
ALLOWABLE STRESS =	33	KSI

COS X=	0.8660	
Pb =	506	PSF
Pq =	99	PSF
LENGTH OF upper triangle	4.6	FT
LENGTH OF rectangle	13.8	FT

<u>FORCE</u>
6732
9892.3
14623.4
12622.5
44730.4
9892.3
24976.125
6785.46
1098.9495

try diff.value of D

PILE PENETR. D=	7.45	FT
PAS.PRESSUp1=	6705	PSF
ACT.PRESSUp1=	910.8	PSF
ACT.PRESSUp2=	295.02	PSF
DIST.FROM DRE- DGE LINE TO TIE	15	FT

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

<u>ARM</u>		<u>MOMENT</u>
4	+	26928
4.918	+	48650.3314
1.7	+	24859.78
7.5	-	94668.75
5.2	-	232598.08
11.918	-	117896.4314
19.917	+	497449.4816
18.725	-	127057.7385
19.917	-	21887.77719
		=====

TOTAL MOMENT@TIE LOCATION= 3778.8
(THIS VALUE SHOULD APPROX.=0)

the (D)should be= 8.94 FT
used
USE D= 9.00 FT
F.S= 1.21 HENCE OK

TOTAL LENGTH=D+H= 32.00 FT

TO FIND TIEBACK FORCE
MAKE SUM. FOR $F_x=0$

F1=	19354.5	+
F2=	19784.6	+
F3=	59353.8	+
F4=	6785.46	+
F5=	24976.125	-
F6=	1098.9495	+

Fx/ONE BAY=	81.40	KIP
Fx/ONE FT=	9.58	KIP/FT

Fx/ BAY =	81.40	KIP
TIE FORCE/1 BAY=	93.99	KIP
PROOFTEST LOAD=	112.79	KIP

FIND MAX. MOMENT

1 @TIEBACK LOCATION = -8 FT

FORCE
6732
9892.3
14623.4

ARM
4
4.918
1.7

MOMENT
26928.00
48650.33
24859.78

MAX. MOMENT= 100.44 KIP.FT

2 @ X-DISTANCE FROM
THE TOP OF THE PILE

FIND X WHEN SUM.OF $F_x=0$

(ZERO SHEAR POINT)

X= 17.752744 FT

FORCE
14938.93
81401.18
9892.30
56569.95

ARM
8.8763719
9.7527437
14.670744
6.5763719

MOMENT
132603.53
793884.89
145127.40
372025.03

MAX. MOMENT=	144.13	KIP.FT
USE MAX. MOMENT=	144.13	KIP.FT
MOD. SECTION =	52.41	CUB. IN (Gr-50)

CASE 12 PART A DURING CONSTRUCTION

CASE 13

DEPTH OF EXCA	13.5	FT
FRICTION- PHEE	20	DEGREE
SOIL UNIT WEIGHT	115	#/C.F
SURCHARGE S _{cu}	200	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	0.83	FT
PILE EFF. FACTOR	2	
K _a	0.37	
K _p	2.7	
TIE LOCATION	5.5	FT
TIE ANGLE X=	30	DEGREE
Pb FACTOR =	22	
ALLOWABLE STRESS =	33	KSI

COS X= 0.8660
 Pb = 297 PSF
 Pq = 74 PSF
 LENGTH OF upper triangle 2.7 FT
 LENGTH OF rectangle 8.1 FT

FORCE
 3459.5
 3408.075
 7088.6
 5032
 13379.85
 3408.075
 8703.18715
 2431.541025
 459.2910825

PILE PENETR. D= 5.1 FT
 try diff. value of D
 PAS.PRESSUp1= 2628.693 PSF
 ACT.PRESSUp1= 476.77275 PSF
 ACT.PRESSUp2= 180.11415 PSF
 DIST FROM DRE: 8 FT
 DGE LINE TO TIE

TO FIND PILE Penetration D
 MOMENT @TIEBACK LOCATION

ARM		MOMENT
2.75	+	9513.625
3.691	+	12579.20463
1.4	+	9896.04
4	-	20128
2.85	-	35456.6025
6.191	-	21099.39233
11.366	+	78188.19783
10.55	-	25852.75781
11.366	-	5220.302444

TOTAL MOMENT@TIE LOCATION= 620.0 (THIS VALUE SHOULD APPROX.=0)

the (D) should be= 6.12 FT
 used
 USE D= 6.50 FT
 F.S= 1.27 HENCE OK

TOTAL LENGTH=D+H= 20.00 FT

CASE 13

DEPTH OF EXCA	13.5	FT
FRICTION- PHEE	20	DEGREE
SOIL UNIT WEIGHT	115	#/C.F
SURCHARGE Scu	200	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	0.83	FT
PILE EFF. FACTOR	2	
Ka	0.37	
Kp	2.7	
TIE LOCATION	5.5	FT
TIE ANGLE X=	30	DEGREE
Pb FACTOR =	22	
ALLOWABLE STRESS =	33	KSI

COS X=	0.8660	
Pb =	297	PSF
Pq =	74	PSF
LENGTH OFupper triangle	2.7	FT
LENGTH OF rectangle	8.1	FT

<u>FORCE</u>
3459.5
3408.075
7088.6
5032
13379.85
3408.075
6703.16715
2431.541025
459.2910825

try diff.value of D

PILE PENETR. D=	5.1	FT
PAS.PRESSUp1=	2628.693	PSF
ACT.PRESSUp1=	476.77275	PSF
ACT.PRESSUp2=	180.11415	PSF
DIST.FROM DRE-	8	FT
DGE LINE TO TIE		

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

<u>ARM</u>		<u>MOMENT</u>
2.75	+	9513.625
3.691	+	12579.20483
1.4	+	9896.04
4	-	20128
2.65	-	35456.6025
6.191	-	21099.39233
11.366	+	76188.19783
10.55	-	25652.75781
11.366	-	5220.302444
		=====

TOTAL MOMENT@TIE LOCATION= 620.0
(THIS VALUE SHOULD APPROX.=0)

the (D)should be= 6.12 FT
used
USE D= 6.50 FT
F.S= 1.27 HENCE OK

TOTAL LENGTH=D+H= 20.00 FT

12/05/201010:12:38 PM1case 13.123\

TO FIND TIEBACK FORCE
MAKE SUM. FOR $F_x=0$

F1=	8491.5	+
F2=	6816.15	+
F3=	20448.45	+
F4=	2431.541025	+
F5=	6703.16715	-
F6=	459.2910825	+

Fx/ONE BAY=	31.94	KIP
Fx/ONE FT=	3.76	KIP/FT

Fx/ BAY =	31.94	KIP	2.bays
TIE FORCE/1 BAY=	36.89	KIP	73.77
PROOFTEST LOAD=	44.26	KIP	88.53

FIND MAX. MOMENT

1 @TIEBACK LOCATION = -5.5 FT

FORCE
3459.5
3408.075
7068.6

ARM
2.75
3.691
1.4

MOMENT
9513.63
12579.20
9896.04

MAX. MOMENT= 31.99 KIP.FT

2 @ X-DISTANCE FROM
THE TOP OF THE PILE

FIND X WHEN SUM.OF $F_x=0$

(ZERO SHEAR POINT)

X= 11.21035 FT

FORCE
7051.31
31943.76
3408.08
21484.38

ARM
5.6051752
5.7103504
9.4013504
4.2551752

MOMENT
39523.83
182410.09
32040.51
91419.80

MAX. MOMENT= 19.43 KIP.FT

USE MAX. MOMENT= 31.99 KIP.FT
MOD. SECTION = 11.63 CUB. IN (Gr-50)

CASE 13

CASE 14 PART A

DEPTH OF EXCA	22	FT
FRICTION- PHEE	28	DEGREE
SOIL UNIT WEIGHT	115	#/C.F
SURCHARGE S_{cu}	300	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	1	FT
PILE EFF. FACTOR	2.5	
K_a	0.34	
K_p	2.9	
TIE LOCATION	9	FT
TIE ANGLE $X=$	30	DEGREE
P_b FACTOR $=$	22	
ALLOWABLE STRESS $=$	33	KSI

$COS X=$ 0.8690
 $P_b =$ 484 PSF
 $P_q =$ 102 PSF
 LENGTH OF upper triangle 4.4 FT
 LENGTH OF rectangle 13.2 FT

FORCE

7803
 9050.8
 18924.4
 11271
 35380.4
 9050.8
 16545.76875
 5419.26
 775.8395

PILE PENETR. $D=$ 8.3 FT

try diff. value of D

PAS.PRESSUp1= 5252.625 PSF
 ACT.PRESSUp1= 860.2 PSF
 ACT.PRESSUp2= 246.33 PSF
 DIST.FROM DRE-
 DGE LINE TO TIE 13 FT

TO FIND PILE PENetration D
 MOMENT @TIEBACK LOCATION

ARM		MOMENT
4.5	+	35113.5
6.052	+	54775.4416
2.3	+	43526.12
6.5	-	73281.5
4.3	-	152135.72
10.052	-	90978.6416
17.158	+	283892.3002
16.15	-	87521.049
17.158	-	13313.56994

TOTAL MOMENT@TIE LOCATION= 96.9 (THIS VALUE SHOULD APPROX.=0)

the (D) should be= 7.58 FT
used

USE $D=$ 8.00 FT

$F.S.=$ 1.27 HENCE OK

TOTAL LENGTH= $H+D=$ 30.00 FT

SHEAR DIAGRAM

MOMENT DIAGRAM

CASE 14 PART A

DEPTH OF EXCA	22	FT
FRICTION- PHEE	28	DEGREE
SOIL UNIT WEIGHT	115	#/C.F
SURCHARGE Scu	300	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	1	FT
PILE EFF. FACTOR	2.5	
Ka	0.34	
Kp	2.9	
TIE LOCATION	9	FT
TIE ANGLE X=	30	DEGREE
Pb FACTOR =	22	
ALLOWABLE STRESS =	33	KSI

COS X=	0.8660	
Pb =	484	PSF
Pq =	102	PSF
LENGTH OFupper triangle	4.4	FT
LENGTH OF rectangle	13.2	FT

FORCE
7803
9050.8
18924.4
11271
35380.4
9050.8
16545.76875
5419.26
775.9395

try diff.value of D

PILE PENETR. D=	6.3	FT
PAS.PRESSUp1=	5252.625	PSF
ACT.PRESSUp1=	860.2	PSF
ACT.PRESSUp2=	246.33	PSF
DIST.FROM DRE- DGE LINE TO TIE	13	FT

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

ARM		MOMENT
4.5	+	35113.5
6.052	+	54775.4416
2.3	+	43526.12
6.5	-	73261.5
4.3	-	152135.72
10.052	-	90978.6416
17.158	+	283892.3002
16.15	-	87521.049
17.158	-	13313.56994
		=====

TOTAL MOMENT@TIE LOCATION= 96.9
(THIS VALUE SHOULD APPROX.=0)

the (D)should be= 7.56 FT
used
USE D= 8.00 FT
F.S= 1.27 HENCE OK

TOTAL LENGTH=D+H= 30.00 FT

TO FIND TIEBACK FORCE
MAKE SUM. FOR $F_x=0$

F1=	19074	+
F2=	18101.6	+
F3=	54304.8	+
F4=	5419.26	+
F5=	16545.76875	-
F6=	775.9395	+

Fx/ONE BAY=	81.13	KIP
Fx/ONE FT=	9.54	KIP/FT

Fx/ BAY =	81.13	KIP
TIE FORCE/1 BAY=	93.68	KIP
PROOFTEST LOAD=	112.42	KIP

FIND MAX. MOMENT

1 @TIEBACK LOCATION = -9 FT

FORCE
7803
9050.8
18924.4

ARM
4.5
6.052
2.3

MOMENT
35113.50
54775.44
43526.12

MAX. MOMENT= 133.42 KIP.FT

2 @ X-DISTANCE FROM
THE TOP OF THE PILE

FIND X WHEN SUM.OF $F_x=0$

(ZERO SHEAR POINT)

X= 18.104925 FT

FORCE
15696.97
81129.83
9050.80
56382.06

ARM
9.0524624
9.1049249
15.156925
6.8524624

MOMENT
142096.23
738681.01
137182.30
386355.95

MAX. MOMENT= 73.05 KIP.FT

USE MAX. MOMENT= 133.42 KIP.FT
MOD. SECTION = 48.51 CUB. IN (Gr-50)

CASE 14 PART A

SHEAR DIAGRAM

MOMENT DIAGRAM

CASE 14 PART B

DEPTH OF EXCA	13	FT
FRICTION- PHEE	28	DEGREE
SOIL UNIT WEIGHT	120	#/C.F
SURCHARGE S_{cu}	300	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	0.83	FT
PILE EFF. FACTOR	2.5	
K_a	0.35	
K_p	2.9	
TIE LOCATION	5	FT
TIE ANGLE $X=$	30	DEGREE
P_b FACTOR $=$	22	
ALLOWABLE STRESS $=$	33	KSI

$\cos X=$	0.8660	
$P_b =$	286	PSF
$P_q =$	105	PSF
LENGTH OF upper triangle	2.8	FT
LENGTH OF rectangle	7.8	FT

FORCE

4482.5
3180.3
5834.4
7140
13127.4
3160.3
7311.2825
2039.31
352.9575

PILE PENETR. $D=$ 4.5 FT

PAS.PRESSUp1=	3249.45	PSF
ACT.PRESSUp1=	453.18	PSF
ACT.PRESSUp2=	156.87	PSF
DIST.FROM DRE-	8	FT
DGE LINE TO TIE		

try diff.value of D

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

ARM		MOMENT
2.5	+	11156.25
3.258	+	10296.2574
1.2	+	7001.28
4	-	28560
2.7	-	35443.98
6.258	-	19777.1574
10.97	+	80204.54993
10.25	-	20902.9275
10.97	-	3871.943775

TOTAL MOMENT@TIE LOCATION= 102.3 (THIS VALUE SHOULD APPROX.=0)

the (D)should be= 5.40 FT

used

USE D= 5.50 FT

F.S= 1.22 HENCE OK

TOTAL LENGTH=D+H= 18.50 FT

CASE 14 PART B

DEPTH OF EXCA	13	FT
FRICTION- PHEE	28	DEGREE
SOIL UNIT WEIGHT	120	#/C.F
SURCHARGE Scu	300	PSF
PILES SPACE s	8.5	FT
PILE FLANGE b	0.83	FT
PILE EFF. FACTOR	2.5	
Ka	0.35	
Kp	2.9	
TIE LOCATION	5	FT
TIE ANGLE X=	30	DEGREE
Pb FACTOR =	22	
ALLOWABLE STRESS =	33	KSI

COS X=	0.8660	
Pb =	286	PSF
Pq =	105	PSF
LENGTH OFupper triangle	2.6	FT
LENGTH OF rectangle	7.8	FT

<u>FORCE</u>
4462.5
3160.3
5834.4
7140
13127.4
3160.3
7311.2625
2039.31
352.9575

try diff.value of D

PILE PENETR. D=	4.5	FT
PAS.PRESSUp1=	3249.45	PSF
ACT.PRESSUp1=	453.18	PSF
ACT.PRESSUp2=	156.87	PSF
DIST.FROM DRE- DGE LINE TO TIE	8	FT

TO FIND PILE Penetration D
MOMENT @TIEBACK LOCATION

<u>ARM</u>		<u>MOMENT</u>
2.5	+	11156.25
3.258	+	10296.2574
1.2	+	7001.28
4	-	28560
2.7	-	35443.98
6.258	-	19777.1574
10.97	+	80204.54963
10.25	-	20902.9275
10.97	-	3871.943775
		=====

TOTAL MOMENT@TIE LOCATION= 102.3
(THIS VALUE SHOULD APPROX.=0)

the (D)should be= 5.40 FT
used
USE D= 5.50 FT
F.S= 1.22 HENCE OK

TOTAL LENGTH=D+H= 18.50 FT

TO FIND TIEBACK FORCE

MAKE SUM. FOR Fx=0

F1=	11602.5	+
F2=	6320.6	+
F3=	18961.8	+
F4=	2039.31	+
F5=	7311.2625	-
F6=	352.9575	+

Fx/ONE BAY=	31.97	KIP
Fx/ONE FT=	3.76	KIP/FT

			2 BAYS
Fx/ BAY =	31.97	KIP	63.93
TIE FORCE/1 BAY=	36.91	KIP	73.82
PROOFTTEST LOAD=	44.29	KIP	88.59

FIND MAX. MOMENT

1 @TIEBACK LOCATION = -5 FT

FORCE
4462.5
3160.3
5834.4

ARM
2.5
3.258
1.2

MOMENT
11156.25
10296.26
7001.28

MAX. MOMENT= 28.45 KIP.FT

2 @ X-DISTANCE FROM
THE TOP OF THE PILE

FIND X WHEN SUM.OF Fx =0

(ZERO SHEAR POINT)

X= 10.56904 FT

FORCE
9432.87
31965.91
3160.30
19372.74

ARM
5.2845201
5.5690402
8.8270402
3.9845201

MOMENT
49848.18
178019.41
27896.10
77191.06

MAX. MOMENT= 23.08 KIP.FT

USE MAX. MOMENT=	28.45	KIP.FT
MOD. SECTION =	10.35	CUB. IN (Gr-50)

CASE 14 PART B

CASE 15

H=	42	FT
S=	8.5	FT
SCH	300	PSF
TIE 1 @	8	FT
TIE 2 @	16.5	FT
TIE 3 @	25.5	FT
TIE 4 @	34	FT
TIE @	30	DEGREE
Ka=	0.33	FT
Pb FACTOR =	22	
upper triangle =	8.4	ft
lower triangle =	8.4	ft

W= 8.6955 K/1 VERTICAL FT

Fx1= 73.53 KIPS
Fx2= 76.09 KIPS
Fx3= 76.09 KIPS
Fx4= 73.53 KIPS

COS X= 0.8680255

DL1= 85 KIP
DL2= 88 KIP
DL3= 88 KIP
DL4= 85 KIP

PROOF LOAD 1.2 X DL

PROOF TIE 1 = 102 KIPS
PROOF TIE 2 = 105 KIPS
PROOF TIE 3 = 105 KIPS
PROOF TIE 4 = 102 KIPS

MAX MOMENT 1 @ TIE 1 LOCATION

FORCE	ARM	MOMENT
6.73	4	26.93
29.92	2.6666667	79.79
		106.71

X IS BELOW 0.2 H X1= 3.85 FT

FORCE	ARM	MOMENT
7.07	8.05	56.90
32.99	6.65	219.36
33.48	1.93	64.44
73.53	4.25	312.52
		28.19

MAX MOMENT 2 @ TIE 2 LOCATION

FORCE	ARM	MOMENT
7.07	12.3	86.94
32.99	10.90	359.56
70.43	4.05	285.26
73.53	8.5	625.03
		106.72

MAX MOMENT 3 BETWEEN 2nd AND 3rd TIE

X2 IS BELOW 0.2 H X2= 12.60

FORCE	ARM	MOMENT
7.07	16.80	118.75
32.99	15.40	508.00
109.56	6.30	690.25
73.53	13.00	955.83
76.09	4.50	342.39
		18.68

MAX MOMENT 4 @ TIE 3 LOCATION

FORCE	ARM	MOMENT
7.07	21.30	150.56
32.99	19.90	656.44
148.69	8.55	1271.33
73.53	17.50	1286.83
76.09	9.00	684.77
		106.72

MAX MOMENT 5 BETWEEN 3rd AND 4th TIE

X3 IS BELOW 0.2 H X3= 21.35

FORCE	ARM	MOMENT
7.07	25.55	180.60
32.99	24.15	796.63
185.65	10.68	1981.80
73.53	21.75	1599.34
76.09	13.25	1008.13
76.09	4.25	323.36
		28.19

USE MAX MOM. = 106.72 FT-KIP
Sx= 38.81 IN³
53.36 IN³ Gr-50 A-36

Appendix C: Soil Nailing Design

Case 1

< OUTPUT FILE > = casel.out

INPUT DATA

SOIL PROPERTIES

FOUNDATION	SOIL UNIT WEIGHT	=	125.000	pcf
	COHESION OF SOIL	=	1500.000	psf
	FRICTION ANGLE OF SOIL	=	30.000	deg
	UNDRAINED SU AT EXCAVATION LEVEL	=	3000.000	psf
	DEPTH TO HARDPAN FROM EXCAVATION	=	60.000	ft
	SU AT THE TOP OF HARDPAN	=	4500.000	psf

LAYER 1	HEIGHT TO TOP OF LAYER FROM TOE	=	15.000	ft
	SOIL UNIT WEIGHT	=	125.000	pcf
	COHESION OF SOIL	=	1500.000	psf
	FRICTION ANGLE OF SOIL	=	30.000	deg

LAYER 2	HEIGHT TO TOP OF LAYER FROM TOE	=	23.500	ft
	SOIL UNIT WEIGHT	=	120.000	pcf
	COHESION OF SOIL	=	700.000	psf
	FRICTION ANGLE OF SOIL	=	25.000	deg

WALL & LOAD CONDITION

WALL INCLINATION ANGLE	=	5.000	deg
SURCHARGE LOAD	=	300.000	psf
HOR. ACCELERATION COEFF.	=	.000	

NAIL PROPERTIES

CROSS SECTIONAL AREA OF NAIL	=	.790	si
GROUT DIAMETER	=	4.000	in
YIELD STRENGTH OF NAIL	=	60.000	ksi
NAIL INCLINATION ANGLE	=	20.000	deg
NAIL SKEW ANGLE AT SURFACE	=	.000	deg
DEPTH TO UPPERMOST NAIL	=	2.000	ft
VER. SPACING OF NAIL	=	5.000	ft
HOR. SPACING OF NAIL	=	5.000	ft
TOTAL LENGTH OF UPPERMOST NAIL	=	20.000	ft
TOTAL LENGTH OF LOWERMOST NAIL	=	15.000	ft
TOTAL UNBONDED LENGTH OF NAIL	=	2.000	ft

OUTPUT RESULTS

I= 2	A= .133	F.S.= 7.077	T= 5503.8660	lb/ft
I= 3	A= .200	F.S.= 4.378	T= 8471.4570	lb/ft
I= 4	A= .267	F.S.= 3.459	T= 10022.7700	lb/ft
I= 5	A= .333	F.S.= 2.999	T= 10816.7900	lb/ft
I= 6	A= .400	F.S.= 2.727	T= 11132.3400	lb/ft
I= 7	A= .467	F.S.= 2.556	T= 11135.6700	lb/ft
I= 8	A= .533	F.S.= 2.444	T= 10882.2900	lb/ft
I= 9	A= .600	F.S.= 2.370	T= 10488.4400	lb/ft
I= 10	A= .667	F.S.= 2.324	T= 9991.4200	lb/ft

I= 11 A=	.733	F.S.=	2.298	T=	9426.2420 lb/ft
I= 12 A=	.800	F.S.=	2.289	T=	8819.0440 lb/ft
I= 13 A=	.867	F.S.=	2.290	T=	8195.4920 lb/ft
I= 14 A=	.933	F.S.=	2.304	T=	7555.6580 lb/ft
I= 15 A=	1.000	F.S.=	2.325	T=	6919.8540 lb/ft
I= 16 A=	1.067	F.S.=	2.352	T=	6281.2810 lb/ft
I= 17 A=	1.133	F.S.=	2.388	T=	5651.4060 lb/ft
I= 18 A=	1.200	F.S.=	2.441	T=	5147.3830 lb/ft
I= 19 A=	1.267	F.S.=	2.494	T=	4739.9440 lb/ft
I= 20 A=	1.333	F.S.=	2.550	T=	4439.7350 lb/ft
I= 21 A=	1.400	F.S.=	2.603	T=	4161.5110 lb/ft
I= 22 A=	1.467	F.S.=	2.655	T=	3902.4920 lb/ft
I= 23 A=	1.533	F.S.=	2.708	T=	3660.1650 lb/ft
I= 24 A=	1.600	F.S.=	2.763	T=	3528.2610 lb/ft
I= 25 A=	1.667	F.S.=	2.822	T=	3400.6970 lb/ft
I= 26 A=	1.733	F.S.=	2.879	T=	3282.1720 lb/ft
I= 27 A=	1.800	F.S.=	2.937	T=	3170.3480 lb/ft
I= 28 A=	1.867	F.S.=	2.995	T=	3064.6080 lb/ft
I= 29 A=	1.933	F.S.=	3.053	T=	2964.4420 lb/ft
I= 30 A=	2.000	F.S.=	3.113	T=	2869.4180 lb/ft

H/LT= 1.175 H/LB= 1.567

NO. OF NAILS (INCLUDING THE LOWERMOST) = 5

**F.S. AGAINST SLIDING THROUGH TOE = 2.29

**F.S. AGAINST DEEP SEATED ROTATION = 6.29

1 TH NAIL FORCE =	613.4692 lb/ft
2 TH NAIL FORCE =	1173.2160 lb/ft
3 TH NAIL FORCE =	1560.6300 lb/ft
4 TH NAIL FORCE =	2163.4070 lb/ft
5 TH NAIL FORCE =	3308.3210 lb/ft

CASE 2

< DATA FILE > = CASE2.IN

< OUTPUT FILE > = CASE2.OUT

INPUT DATA

SOIL PROPERTIES

FOUNDATION	SOIL UNIT WEIGHT	=	120.000 pcf
	COHESION OF SOIL	=	1000.000 psf
	FRICTION ANGLE OF SOIL	=	30.000 deg
	UNDRAINED SU AT EXCAVATION LEVEL	=	2000.000 psf
	DEPTH TO HARDPAN FROM EXCAVATION	=	50.000 ft
	SU AT THE TOP OF HARDPAN	=	4000.000 psf

LAYER 1	HEIGHT TO TOP OF LAYER FROM TOE	=	17.000 ft
	SOIL UNIT WEIGHT	=	120.000 pcf
	COHESION OF SOIL	=	1000.000 psf
	FRICTION ANGLE OF SOIL	=	30.000 deg

LAYER 2	HEIGHT TO TOP OF LAYER FROM TOE	=	32.000 ft
	SOIL UNIT WEIGHT	=	110.000 pcf
	COHESION OF SOIL	=	500.000 psf
	FRICTION ANGLE OF SOIL	=	20.000 deg

WALL & LOAD CONDITION

WALL INCLINATION ANGLE	=	5.000 deg
SURCHARGE LOAD	=	300.000 psf
HOR. ACCELERATION COEFF.	=	.000

NAIL PROPERTIES

CROSS SECTIONAL AREA OF NAIL	=	.790 si
GROUT DIAMETER	=	4.000 in
YIELD STRENGTH OF NAIL	=	60.000 ksi
NAIL INCLINATION ANGLE	=	20.000 deg
NAIL SKEW ANGLE AT SURFACE	=	.000 deg
DEPTH TO UPPERMOST NAIL	=	2.000 ft
VER. SPACING OF NAIL	=	5.000 ft
HOR. SPACING OF NAIL	=	5.000 ft
TOTAL LENGTH OF UPPERMOST NAIL	=	28.000 ft
TOTAL LENGTH OF LOWERMOST NAIL	=	22.000 ft
TOTAL UNBONDED LENGTH OF NAIL	=	2.000 ft

OUTPUT RESULTS

I=	2	A=	.133	F.S.=	3.622	T=	15909.9900 lb/ft
I=	3	A=	.200	F.S.=	2.559	T=	20901.1700 lb/ft
I=	4	A=	.267	F.S.=	2.142	T=	23113.2700 lb/ft
I=	5	A=	.333	F.S.=	1.920	T=	24033.1300 lb/ft
I=	6	A=	.400	F.S.=	1.783	T=	24222.1900 lb/ft
I=	7	A=	.467	F.S.=	1.696	T=	23813.9900 lb/ft
I=	8	A=	.533	F.S.=	1.639	T=	23022.3000 lb/ft
I=	9	A=	.600	F.S.=	1.604	T=	21982.9300 lb/ft

I= 10	A= .667	F.S.= 1.582	T= 20800.5700	lb/ft
I= 11	A= .733	F.S.= 1.573	T= 19512.8600	lb/ft
I= 12	A= .800	F.S.= 1.573	T= 18164.1300	lb/ft
I= 13	A= .867	F.S.= 1.580	T= 16787.8700	lb/ft
I= 14	A= .933	F.S.= 1.593	T= 15417.0900	lb/ft
I= 15	A= 1.000	F.S.= 1.611	T= 14050.8300	lb/ft
I= 16	A= 1.067	F.S.= 1.633	T= 12673.3600	lb/ft
I= 17	A= 1.133	F.S.= 1.658	T= 11289.5100	lb/ft
I= 18	A= 1.200	F.S.= 1.708	T= 10016.9300	lb/ft
I= 19	A= 1.267	F.S.= 1.755	T= 8997.1090	lb/ft
I= 20	A= 1.333	F.S.= 1.796	T= 8165.0090	lb/ft
I= 21	A= 1.400	F.S.= 1.836	T= 7427.8940	lb/ft
I= 22	A= 1.467	F.S.= 1.881	T= 6902.8730	lb/ft
I= 23	A= 1.533	F.S.= 1.921	T= 6424.9310	lb/ft
I= 24	A= 1.600	F.S.= 1.960	T= 5978.8790	lb/ft
I= 25	A= 1.667	F.S.= 2.001	T= 5556.2960	lb/ft
I= 26	A= 1.733	F.S.= 2.041	T= 5177.5890	lb/ft
I= 27	A= 1.800	F.S.= 2.085	T= 4944.5800	lb/ft
I= 28	A= 1.867	F.S.= 2.129	T= 4726.2550	lb/ft
I= 29	A= 1.933	F.S.= 2.173	T= 4521.2840	lb/ft
I= 30	A= 2.000	F.S.= 2.217	T= 4328.5330	lb/ft

H/LT= 1.143 H/LB= 1.455

NO. OF NAILS (EXCLUDING THE LOWERMOST) = 6

**F.S. AGAINST SLIDING THROUGH TOE = 1.57

**F.S. AGAINST DEEP SEATED ROTATION = 4.18

1 TH NAIL FORCE =	896.1140	lb/ft
2 TH NAIL FORCE =	1498.1950	lb/ft
3 TH NAIL FORCE =	2470.0860	lb/ft
4 TH NAIL FORCE =	3202.5090	lb/ft
5 TH NAIL FORCE =	4186.5750	lb/ft
6 TH NAIL FORCE =	5910.6460	lb/ft

CASE 3

< DATA FILE > = CASE3.IN
< OUTPUT FILE > = CASE3.OUT

INPUT DATA

SOIL PROPERTIES

FOUNDATION	SOIL UNIT WEIGHT	=	125.000 pcf
	COHESION OF SOIL	=	700.000 psf
	FRICTION ANGLE OF SOIL	=	30.000 deg
	UNDRAINED SU AT EXCAVATION LEVEL	=	1500.000 psf
	DEPTH TO HARDPAN FROM EXCAVATION	=	35.000 ft
	SU AT THE TOP OF HARDPAN	=	2500.000 psf
LAYER 1	HEIGHT TO TOP OF LAYER FROM TOE	=	15.000 ft
	SOIL UNIT WEIGHT	=	125.000 pcf
	COHESION OF SOIL	=	550.000 psf
	FRICTION ANGLE OF SOIL	=	30.000 deg

WALL & LOAD CONDITION

WALL INCLINATION ANGLE	=	5.000 deg
SURCHARGE LOAD	=	100.000 psf
HOR. ACCELERATION COEFF.	=	.000

NAIL PROPERTIES

CROSS SECTIONAL AREA OF NAIL	=	.440 si
GROUT DIAMETER	=	4.000 in
YIELD STRENGTH OF NAIL	=	60.000 ksi
NAIL INCLINATION ANGLE	=	20.000 deg
NAIL SKEW ANGLE AT SURFACE	=	.000 deg
DEPTH TO UPPERMOST NAIL	=	2.000 ft
VER. SPACING OF NAIL	=	5.000 ft
HOR. SPACING OF NAIL	=	6.000 ft
TOTAL LENGTH OF UPPERMOST NAIL	=	12.000 ft
TOTAL LENGTH OF LOWERMOST NAIL	=	10.000 ft
TOTAL UNBONDED LENGTH OF NAIL	=	2.000 ft

OUTPUT RESULTS

I= 2	A= .133	F.S.= 5.346	T= 1018.3570 lb/ft
I= 3	A= .200	F.S.= 3.125	T= 1644.8320 lb/ft
I= 4	A= .267	F.S.= 2.392	T= 2019.7430 lb/ft
I= 5	A= .333	F.S.= 2.037	T= 2199.8860 lb/ft
I= 6	A= .400	F.S.= 1.833	T= 2257.6530 lb/ft
I= 7	A= .467	F.S.= 1.710	T= 2224.8860 lb/ft
I= 8	A= .533	F.S.= 1.635	T= 2141.1770 lb/ft
I= 9	A= .600	F.S.= 1.592	T= 2023.4140 lb/ft
I= 10	A= .667	F.S.= 1.571	T= 1884.8360 lb/ft
I= 11	A= .733	F.S.= 1.567	T= 1732.0180 lb/ft
I= 12	A= .800	F.S.= 1.575	T= 1573.0860 lb/ft
I= 13	A= .867	F.S.= 1.593	T= 1412.6720 lb/ft
I= 14	A= .933	F.S.= 1.617	T= 1254.0750 lb/ft

I=	15	A=	1.000	F.S.=	1.648	T=	1099.5830	lb/ft
I=	16	A=	1.067	F.S.=	1.684	T=	950.7256	lb/ft
I=	17	A=	1.133	F.S.=	1.736	T=	850.3796	lb/ft
I=	18	A=	1.200	F.S.=	1.788	T=	763.5302	lb/ft
I=	19	A=	1.267	F.S.=	1.838	T=	683.5601	lb/ft
I=	20	A=	1.333	F.S.=	1.888	T=	609.1956	lb/ft
I=	21	A=	1.400	F.S.=	1.936	T=	549.6973	lb/ft
I=	22	A=	1.467	F.S.=	1.986	T=	522.0972	lb/ft
I=	23	A=	1.533	F.S.=	2.036	T=	496.5759	lb/ft
I=	24	A=	1.600	F.S.=	2.086	T=	472.8983	lb/ft
I=	25	A=	1.667	F.S.=	2.136	T=	450.8690	lb/ft
I=	26	A=	1.733	F.S.=	2.186	T=	430.3256	lb/ft
I=	27	A=	1.800	F.S.=	2.236	T=	411.1305	lb/ft
I=	28	A=	1.867	F.S.=	2.286	T=	393.1662	lb/ft
I=	29	A=	1.933	F.S.=	2.337	T=	376.3316	lb/ft
I=	30	A=	2.000	F.S.=	2.388	T=	360.5376	lb/ft

H/LT= 1.250 H/LB= 1.500

NO. OF NAILS (INCLUDING THE LOWERMOST) = 3

**F.S. AGAINST SLIDING THROUGH TOE = 1.57

**F.S. AGAINST DEEP SEATED ROTATION = 5.16

1 TH NAIL FORCE =	302.1914	lb/ft
2 TH NAIL FORCE =	510.6113	lb/ft
3 TH NAIL FORCE =	919.2153	lb/ft

CASE 4 Part A
 < DATA FILE > = CASE4.IN
 < OUTPUT FILE > = CASE4.OUT

INPUT DATA

SOIL PROPERTIES

FOUNDATION	SOIL UNIT WEIGHT	=	130.000 pcf
	COHESION OF SOIL	=	1600.000 psf
	FRICTION ANGLE OF SOIL	=	35.000 deg
	UNDRAINED SU AT EXCAVATION LEVEL	=	3200.000 psf
	DEPTH TO HARDPAN FROM EXCAVATION	=	40.000 ft
	SU AT THE TOP OF HARDPAN	=	4000.000 psf
LAYER 1	HEIGHT TO TOP OF LAYER FROM TOE	=	10.000 ft
	SOIL UNIT WEIGHT	=	130.000 pcf
	COHESION OF SOIL	=	1100.000 psf
	FRICTION ANGLE OF SOIL	=	30.000 deg
LAYER 2	HEIGHT TO TOP OF LAYER FROM TOE	=	20.000 ft
	SOIL UNIT WEIGHT	=	125.000 pcf
	COHESION OF SOIL	=	1100.000 psf
	FRICTION ANGLE OF SOIL	=	30.000 deg
LAYER 3	HEIGHT TO TOP OF LAYER FROM TOE	=	28.000 ft
	SOIL UNIT WEIGHT	=	120.000 pcf
	COHESION OF SOIL	=	650.000 psf
	FRICTION ANGLE OF SOIL	=	25.000 deg

WALL & LOAD CONDITION

WALL INCLINATION ANGLE	=	5.000 deg
SURCHARGE LOAD	=	250.000 psf
HOR. ACCELERATION COEFF.	=	.000

NAIL PROPERTIES

CROSS SECTIONAL AREA OF NAIL	=	.790 si
GROUT DIAMETER	=	4.000 in
YIELD STRENGTH OF NAIL	=	60.000 ksi
NAIL INCLINATION ANGLE	=	20.000 deg
NAIL SKEW ANGLE AT SURFACE	=	.000 deg
DEPTH TO UPPERMOST NAIL	=	2.000 ft
VER. SPACING OF NAIL	=	5.000 ft
HOR. SPACING OF NAIL	=	5.000 ft
TOTAL LENGTH OF UPPERMOST NAIL	=	25.000 ft
TOTAL LENGTH OF LOWERMOST NAIL	=	20.000 ft
TOTAL UNBONDED LENGTH OF NAIL	=	2.000 ft

O U T P U T R E S U L T S

I=	2	A=	.133	F.S.=	4.393	T=	13252.5400	lb/ft
I=	3	A=	.200	F.S.=	3.096	T=	17732.9800	lb/ft
I=	4	A=	.267	F.S.=	2.595	T=	19779.9400	lb/ft
I=	5	A=	.333	F.S.=	2.331	T=	20723.3800	lb/ft
I=	6	A=	.400	F.S.=	2.169	T=	21035.1600	lb/ft
I=	7	A=	.467	F.S.=	2.066	T=	20918.5900	lb/ft
I=	8	A=	.533	F.S.=	2.001	T=	20487.0400	lb/ft
I=	9	A=	.600	F.S.=	1.961	T=	19850.1600	lb/ft
I=	10	A=	.667	F.S.=	1.940	T=	19060.2700	lb/ft
I=	11	A=	.733	F.S.=	1.930	T=	18169.0500	lb/ft
I=	12	A=	.800	F.S.=	1.931	T=	17212.1900	lb/ft
I=	13	A=	.867	F.S.=	1.936	T=	16178.2300	lb/ft
I=	14	A=	.933	F.S.=	1.949	T=	15131.6900	lb/ft
I=	15	A=	1.000	F.S.=	1.965	T=	14097.2600	lb/ft
I=	16	A=	1.067	F.S.=	1.989	T=	13064.1500	lb/ft
I=	17	A=	1.133	F.S.=	2.015	T=	12049.2100	lb/ft
I=	18	A=	1.200	F.S.=	2.058	T=	11109.4800	lb/ft
I=	19	A=	1.267	F.S.=	2.110	T=	10280.1500	lb/ft
I=	20	A=	1.333	F.S.=	2.162	T=	9622.8340	lb/ft
I=	21	A=	1.400	F.S.=	2.212	T=	9068.8740	lb/ft
I=	22	A=	1.467	F.S.=	2.260	T=	8557.4080	lb/ft
I=	23	A=	1.533	F.S.=	2.306	T=	8098.5090	lb/ft
I=	24	A=	1.600	F.S.=	2.355	T=	7782.3040	lb/ft
I=	25	A=	1.667	F.S.=	2.404	T=	7485.4040	lb/ft
I=	26	A=	1.733	F.S.=	2.452	T=	7208.2800	lb/ft
I=	27	A=	1.800	F.S.=	2.500	T=	6947.5890	lb/ft
I=	28	A=	1.867	F.S.=	2.547	T=	6701.5480	lb/ft
I=	29	A=	1.933	F.S.=	2.594	T=	6468.7310	lb/ft
I=	30	A=	2.000	F.S.=	2.642	T=	6247.9790	lb/ft

H/LT= 1.120 H/LB= 1.400

NO. OF NAILS (INCLUDING THE LOWERMOST) = 6

**F.S. AGAINST SLIDING THROUGH TOE = 1.93

**F.S. AGAINST DEEP SEATED ROTATION = 4.39

1	TH NAIL FORCE =	1324.2500	lb/ft
2	TH NAIL FORCE =	1788.2400	lb/ft
3	TH NAIL FORCE =	2329.6610	lb/ft
4	TH NAIL FORCE =	3044.7810	lb/ft
5	TH NAIL FORCE =	3997.9020	lb/ft
6	TH NAIL FORCE =	5684.2140	lb/ft

CASE 4 PART B 35 FT WALL
 < OUTPUT FILE > = CASE4B.OUT

INPUT DATA

SOIL PROPERTIES

FOUNDATION	SOIL UNIT WEIGHT	=	130.000	pcf
	COHESION OF SOIL	=	1600.000	psf
	FRICTION ANGLE OF SOIL	=	35.000	deg
	UNDRAINED SU AT EXCAVATION LEVEL	=	3200.000	psf
	DEPTH TO HARDPAN FROM EXCAVATION	=	33.000	ft
	SU AT THE TOP OF HARDPAN	=	4000.000	psf
LAYER 1	HEIGHT TO TOP OF LAYER FROM TOE	=	17.000	ft
	SOIL UNIT WEIGHT	=	130.000	pcf
	COHESION OF SOIL	=	1100.000	psf
	FRICTION ANGLE OF SOIL	=	30.000	deg
LAYER 2	HEIGHT TO TOP OF LAYER FROM TOE	=	27.000	ft
	SOIL UNIT WEIGHT	=	125.000	pcf
	COHESION OF SOIL	=	1100.000	psf
	FRICTION ANGLE OF SOIL	=	30.000	deg
LAYER 3	HEIGHT TO TOP OF LAYER FROM TOE	=	35.000	ft
	SOIL UNIT WEIGHT	=	120.000	pcf
	COHESION OF SOIL	=	650.000	psf
	FRICTION ANGLE OF SOIL	=	25.000	deg

WALL & LOAD CONDITION

WALL INCLINATION ANGLE	=	5.000	deg
SURCHARGE LOAD	=	250.000	psf
HOR. ACCELERATION COEFF.	=	.000	

NAIL PROPERTIES

CROSS SECTIONAL AREA OF NAIL	=	.790	si
GROUT DIAMETER	=	4.000	in
YIELD STRENGTH OF NAIL	=	60.000	ksi
NAIL INCLINATION ANGLE	=	20.000	deg
NAIL SKEW ANGLE AT SURFACE	=	.000	deg
DEPTH TO UPPERMOST NAIL	=	2.000	ft
VER. SPACING OF NAIL	=	5.000	ft
HOR. SPACING OF NAIL	=	5.000	ft
TOTAL LENGTH OF UPPERMOST NAIL	=	30.000	ft
TOTAL LENGTH OF LOWERMOST NAIL	=	25.000	ft
TOTAL UNBONDED LENGTH OF NAIL	=	2.000	ft

OUTPUT RESULTS

I= 2	A= .133	F.S.= 3.779	T= 24687.3000	lb/ft
I= 3	A= .200	F.S.= 2.790	T= 31042.4100	lb/ft
I= 4	A= .267	F.S.= 2.385	T= 33798.9800	lb/ft
I= 5	A= .333	F.S.= 2.157	T= 34891.5900	lb/ft

I=	6	A=	.400	F.S.=	2.017	T=	34958.5200	lb/ft
I=	7	A=	.467	F.S.=	1.922	T=	34407.2600	lb/ft
I=	8	A=	.533	F.S.=	1.861	T=	33370.4000	lb/ft
I=	9	A=	.600	F.S.=	1.823	T=	32000.9800	lb/ft
I=	10	A=	.667	F.S.=	1.796	T=	30422.1600	lb/ft
I=	11	A=	.733	F.S.=	1.779	T=	28665.8600	lb/ft
I=	12	A=	.800	F.S.=	1.772	T=	26810.5800	lb/ft
I=	13	A=	.867	F.S.=	1.771	T=	24921.7400	lb/ft
I=	14	A=	.933	F.S.=	1.778	T=	22987.8000	lb/ft
I=	15	A=	1.000	F.S.=	1.791	T=	21067.0200	lb/ft
I=	16	A=	1.067	F.S.=	1.807	T=	19176.4600	lb/ft
I=	17	A=	1.133	F.S.=	1.840	T=	17375.2100	lb/ft
I=	18	A=	1.200	F.S.=	1.884	T=	15789.1100	lb/ft
I=	19	A=	1.267	F.S.=	1.930	T=	14535.2400	lb/ft
I=	20	A=	1.333	F.S.=	1.972	T=	13404.5900	lb/ft
I=	21	A=	1.400	F.S.=	2.017	T=	12569.0500	lb/ft
I=	22	A=	1.467	F.S.=	2.059	T=	11796.5000	lb/ft
I=	23	A=	1.533	F.S.=	2.100	T=	11078.0200	lb/ft
I=	24	A=	1.600	F.S.=	2.142	T=	10459.8200	lb/ft
I=	25	A=	1.667	F.S.=	2.186	T=	10008.5700	lb/ft
I=	26	A=	1.733	F.S.=	2.230	T=	9585.9750	lb/ft
I=	27	A=	1.800	F.S.=	2.274	T=	9188.9340	lb/ft
I=	28	A=	1.867	F.S.=	2.317	T=	8814.9540	lb/ft
I=	29	A=	1.933	F.S.=	2.361	T=	8462.0050	lb/ft
I=	30	A=	2.000	F.S.=	2.406	T=	8128.3920	lb/ft

H/LT= 1.167 H/LB= 1.400

NO. OF NAILS (INCLUDING THE LOWERMOST) = 7

**F.S. AGAINST SLIDING THROUGH TOE = 1.77

**F.S. AGAINST DEEP SEATED ROTATION = 3.54

1 TH NAIL FORCE =	1101.7300	lb/ft
2 TH NAIL FORCE =	1552.0660	lb/ft
3 TH NAIL FORCE =	2217.8890	lb/ft
4 TH NAIL FORCE =	3001.5250	lb/ft
5 TH NAIL FORCE =	4033.2320	lb/ft
6 TH NAIL FORCE =	5443.0500	lb/ft
7 TH NAIL FORCE =	7572.2500	lb/ft

CASE 5 PART A 37.5 FT WALL
 < DATA FILE > = CASE5A.IN
 < OUTPUT FILE > = CASE5A.OUT

INPUT DATA

SOIL PROPERTIES

FOUNDATION	SOIL UNIT WEIGHT	=	125.000 pcf
	COHESION OF SOIL	=	1500.000 psf
	FRICTION ANGLE OF SOIL	=	25.000 deg
	UNDRAINED SU AT EXCAVATION LEVEL	=	3000.000 psf
	DEPTH TO HARDPAN FROM EXCAVATION	=	40.000 ft
	SU AT THE TOP OF HARDPAN	=	5000.000 psf
LAYER 1	HEIGHT TO TOP OF LAYER FROM TOE	=	16.000 ft
	SOIL UNIT WEIGHT	=	125.000 pcf
	COHESION OF SOIL	=	1500.000 psf
	FRICTION ANGLE OF SOIL	=	25.000 deg
LAYER 2	HEIGHT TO TOP OF LAYER FROM TOE	=	26.000 ft
	SOIL UNIT WEIGHT	=	120.000 pcf
	COHESION OF SOIL	=	1000.000 psf
	FRICTION ANGLE OF SOIL	=	30.000 deg
LAYER 3	HEIGHT TO TOP OF LAYER FROM TOE	=	37.500 ft
	SOIL UNIT WEIGHT	=	115.000 pcf
	COHESION OF SOIL	=	500.000 psf
	FRICTION ANGLE OF SOIL	=	30.000 deg

WALL & LOAD CONDITION

WALL INCLINATION ANGLE	=	5.000 deg
SURCHARGE LOAD	=	300.000 psf
HOR. ACCELERATION COEFF.	=	.000

NAIL PROPERTIES

CROSS SECTIONAL AREA OF NAIL	=	.790 si
GROUT DIAMETER	=	4.000 in
YIELD STRENGTH OF NAIL	=	60.000 ksi
NAIL INCLINATION ANGLE	=	20.000 deg
NAIL SKEW ANGLE AT SURFACE	=	.000 deg
DEPTH TO UPPERMOST NAIL	=	2.000 ft
VER. SPACING OF NAIL	=	5.500 ft
HOR. SPACING OF NAIL	=	5.000 ft
TOTAL LENGTH OF UPPERMOST NAIL	=	30.000 ft
TOTAL LENGTH OF LOWERMOST NAIL	=	25.000 ft
TOTAL UNBONDED LENGTH OF NAIL	=	2.000 ft

OUTPUT RESULTS

I= 2	A= .133	F.S.= 4.102	T= 22879.6900 lb/ft
I= 3	A= .200	F.S.= 2.896	T= 29925.6400 lb/ft
I= 4	A= .267	F.S.= 2.421	T= 33272.0000 lb/ft

I=	5	A=	.333	F.S.=	2.166	T=	34650.8500	lb/ft
I=	6	A=	.400	F.S.=	2.007	T=	34955.6000	lb/ft
I=	7	A=	.467	F.S.=	1.903	T=	34417.8600	lb/ft
I=	8	A=	.533	F.S.=	1.835	T=	33306.4300	lb/ft
I=	9	A=	.600	F.S.=	1.787	T=	31881.5700	lb/ft
I=	10	A=	.667	F.S.=	1.756	T=	30204.3800	lb/ft
I=	11	A=	.733	F.S.=	1.738	T=	28361.3900	lb/ft
I=	12	A=	.800	F.S.=	1.729	T=	26433.6400	lb/ft
I=	13	A=	.867	F.S.=	1.729	T=	24424.6300	lb/ft
I=	14	A=	.933	F.S.=	1.736	T=	22403.3800	lb/ft
I=	15	A=	1.000	F.S.=	1.746	T=	20322.9200	lb/ft
I=	16	A=	1.067	F.S.=	1.777	T=	18370.5800	lb/ft
I=	17	A=	1.133	F.S.=	1.815	T=	16731.5800	lb/ft
I=	18	A=	1.200	F.S.=	1.851	T=	15343.4200	lb/ft
I=	19	A=	1.267	F.S.=	1.889	T=	14258.7000	lb/ft
I=	20	A=	1.333	F.S.=	1.929	T=	13332.1200	lb/ft
I=	21	A=	1.400	F.S.=	1.967	T=	12469.8500	lb/ft
I=	22	A=	1.467	F.S.=	2.007	T=	11722.2600	lb/ft
I=	23	A=	1.533	F.S.=	2.049	T=	11189.4900	lb/ft
I=	24	A=	1.600	F.S.=	2.092	T=	10690.9700	lb/ft
I=	25	A=	1.667	F.S.=	2.134	T=	10223.0400	lb/ft
I=	26	A=	1.733	F.S.=	2.177	T=	9782.7750	lb/ft
I=	27	A=	1.800	F.S.=	2.220	T=	9367.7940	lb/ft
I=	28	A=	1.867	F.S.=	2.264	T=	8976.1250	lb/ft
I=	29	A=	1.933	F.S.=	2.308	T=	8606.0970	lb/ft
I=	30	A=	2.000	F.S.=	2.353	T=	8256.2400	lb/ft

H/LT= 1.250 H/LB= 1.500

NO. OF NAILS (INCLUDING THE LOWERMOST) = 7

**F.S. AGAINST SLIDING THROUGH TOE = 1.73

**F.S. AGAINST DEEP SEATED ROTATION = 4.32

1 TH NAIL FORCE =	958.8588	lb/ft
2 TH NAIL FORCE =	1616.2710	lb/ft
3 TH NAIL FORCE =	2323.0820	lb/ft
4 TH NAIL FORCE =	3266.6650	lb/ft
5 TH NAIL FORCE =	4331.3090	lb/ft
6 TH NAIL FORCE =	5805.6800	lb/ft
7 TH NAIL FORCE =	8131.7790	lb/ft

CASE 5 PART B 26 FT WALL
 < OUTPUT FILE > = CASE5B.OUT

INPUT DATA

SOIL PROPERTIES

FOUNDATION	SOIL UNIT WEIGHT	=	125.000 pcf
	COHESION OF SOIL	=	1500.000 psf
	FRICTION ANGLE OF SOIL	=	25.000 deg
	UNDRAINED SU AT EXCAVATION LEVEL	=	3000.000 psf
	DEPTH TO HARDPAN FROM EXCAVATION	=	51.000 ft
	SU AT THE TOP OF HARDPAN	=	5000.000 psf
LAYER 1	HEIGHT TO TOP OF LAYER FROM TOE	=	4.500 ft
	SOIL UNIT WEIGHT	=	125.000 pcf
	COHESION OF SOIL	=	1500.000 psf
	FRICTION ANGLE OF SOIL	=	25.000 deg
LAYER 2	HEIGHT TO TOP OF LAYER FROM TOE	=	14.500 ft
	SOIL UNIT WEIGHT	=	120.000 pcf
	COHESION OF SOIL	=	1000.000 psf
	FRICTION ANGLE OF SOIL	=	30.000 deg
LAYER 3	HEIGHT TO TOP OF LAYER FROM TOE	=	26.000 ft
	SOIL UNIT WEIGHT	=	115.000 pcf
	COHESION OF SOIL	=	500.000 psf
	FRICTION ANGLE OF SOIL	=	30.000 deg

WALL & LOAD CONDITION

WALL INCLINATION ANGLE	=	5.000 deg
SURCHARGE LOAD	=	300.000 psf
HOR. ACCELERATION COEFF.	=	.000

NAIL PROPERTIES

CROSS SECTIONAL AREA OF NAIL	=	.790 si
GROUT DIAMETER	=	4.000 in
YIELD STRENGTH OF NAIL	=	60.000 ksi
NAIL INCLINATION ANGLE	=	20.000 deg
NAIL SKEW ANGLE AT SURFACE	=	.000 deg
DEPTH TO UPPERMOST NAIL	=	2.000 ft
VER. SPACING OF NAIL	=	5.000 ft
HOR. SPACING OF NAIL	=	5.000 ft
TOTAL LENGTH OF UPPERMOST NAIL	=	20.000 ft
TOTAL LENGTH OF LOWERMOST NAIL	=	15.000 ft
TOTAL UNBONDED LENGTH OF NAIL	=	2.000 ft

OUTPUT RESULTS

I= 2	A= .133	F.S.= 5.109	T= 6058.9010 lb/ft
I= 3	A= .200	F.S.= 3.223	T= 8946.1340 lb/ft
I= 4	A= .267	F.S.= 2.564	T= 10276.8900 lb/ft
I= 5	A= .333	F.S.= 2.225	T= 10806.8400 lb/ft

I=	6	A=	.400	F.S.=	2.026	T=	10860.9100	lb/ft
I=	7	A=	.467	F.S.=	1.902	T=	10596.5600	lb/ft
I=	8	A=	.533	F.S.=	1.820	T=	10129.9500	lb/ft
I=	9	A=	.600	F.S.=	1.769	T=	9517.5800	lb/ft
I=	10	A=	.667	F.S.=	1.739	T=	8810.1890	lb/ft
I=	11	A=	.733	F.S.=	1.724	T=	8037.0160	lb/ft
I=	12	A=	.800	F.S.=	1.719	T=	7196.7130	lb/ft
I=	13	A=	.867	F.S.=	1.724	T=	6341.6580	lb/ft
I=	14	A=	.933	F.S.=	1.737	T=	5493.4940	lb/ft
I=	15	A=	1.000	F.S.=	1.760	T=	4679.7290	lb/ft
I=	16	A=	1.067	F.S.=	1.801	T=	3995.1290	lb/ft
I=	17	A=	1.133	F.S.=	1.845	T=	3538.9230	lb/ft
I=	18	A=	1.200	F.S.=	1.888	T=	3117.1950	lb/ft
I=	19	A=	1.267	F.S.=	1.933	T=	2812.2840	lb/ft
I=	20	A=	1.333	F.S.=	1.979	T=	2579.6100	lb/ft
I=	21	A=	1.400	F.S.=	2.026	T=	2362.6640	lb/ft
I=	22	A=	1.467	F.S.=	2.071	T=	2164.3820	lb/ft
I=	23	A=	1.533	F.S.=	2.116	T=	1980.0310	lb/ft
I=	24	A=	1.600	F.S.=	2.161	T=	1808.4740	lb/ft
I=	25	A=	1.667	F.S.=	2.207	T=	1648.7360	lb/ft
I=	26	A=	1.733	F.S.=	2.256	T=	1544.2970	lb/ft
I=	27	A=	1.800	F.S.=	2.308	T=	1483.2490	lb/ft
I=	28	A=	1.867	F.S.=	2.360	T=	1425.9990	lb/ft
I=	29	A=	1.933	F.S.=	2.412	T=	1372.2310	lb/ft
I=	30	A=	2.000	F.S.=	2.469	T=	1320.0930	lb/ft

H/LT= 1.300 H/LB= 1.733

NO. OF NAILS (INCLUDING THE LOWERMOST) = 5

**F.S. AGAINST SLIDING THROUGH TOE = 1.72

**F.S. AGAINST DEEP SEATED ROTATION = 6.16

1 TH NAIL FORCE =	437.9256	lb/ft
2 TH NAIL FORCE =	847.6405	lb/ft
3 TH NAIL FORCE =	1202.5260	lb/ft
4 TH NAIL FORCE =	1827.5260	lb/ft
5 TH NAIL FORCE =	2881.0940	lb/ft

CASE 6 PART A 35.5 FT WALL
 < OUTPUT FILE > = CASE6A.OUT

INPUT DATA

SOIL PROPERTIES

FOUNDATION	SOIL UNIT WEIGHT	=	130.000	pcf
	COHESION OF SOIL	=	1700.000	psf
	FRICTION ANGLE OF SOIL	=	30.000	deg
	UNDRAINED SU AT EXCAVATION LEVEL	=	3000.000	psf
	DEPTH TO HARDPAN FROM EXCAVATION	=	30.000	ft
	SU AT THE TOP OF HARDPAN	=	4500.000	psf
LAYER 1	HEIGHT TO TOP OF LAYER FROM TOE	=	8.500	ft
	SOIL UNIT WEIGHT	=	130.000	pcf
	COHESION OF SOIL	=	1600.000	psf
	FRICTION ANGLE OF SOIL	=	30.000	deg
LAYER 2	HEIGHT TO TOP OF LAYER FROM TOE	=	24.500	ft
	SOIL UNIT WEIGHT	=	125.000	pcf
	COHESION OF SOIL	=	1500.000	psf
	FRICTION ANGLE OF SOIL	=	25.000	deg
LAYER 3	HEIGHT TO TOP OF LAYER FROM TOE	=	35.500	ft
	SOIL UNIT WEIGHT	=	120.000	pcf
	COHESION OF SOIL	=	600.000	psf
	FRICTION ANGLE OF SOIL	=	20.000	deg

WALL & LOAD CONDITION

WALL INCLINATION ANGLE	=	5.000	deg
SURCHARGE LOAD	=	250.000	psf
HOR. ACCELERATION COEFF.	=	.000	

NAIL PROPERTIES

CROSS SECTIONAL AREA OF NAIL	=	.790	si
GROUT DIAMETER	=	4.000	in
YIELD STRENGTH OF NAIL	=	60.000	ksi
NAIL INCLINATION ANGLE	=	20.000	deg
NAIL SKEW ANGLE AT SURFACE	=	.000	deg
DEPTH TO UPPERMOST NAIL	=	2.000	ft
VER. SPACING OF NAIL	=	5.000	ft
HOR. SPACING OF NAIL	=	5.000	ft
TOTAL LENGTH OF UPPERMOST NAIL	=	30.000	ft
TOTAL LENGTH OF LOWERMOST NAIL	=	25.000	ft
TOTAL UNBONDED LENGTH OF NAIL	=	2.000	ft

OUTPUT RESULTS

I= 2	A= .133	F.S.= 4.441	T= 22194.0400	lb/ft
I= 3	A= .200	F.S.= 3.124	T= 29345.1600	lb/ft
I= 4	A= .267	F.S.= 2.622	T= 32600.2200	lb/ft
I= 5	A= .333	F.S.= 2.352	T= 34013.5000	lb/ft

I=	6	A=	.400	F.S.=	2.185	T=	34375.8100	lb/ft
I=	7	A=	.467	F.S.=	2.078	T=	33967.5200	lb/ft
I=	8	A=	.533	F.S.=	2.006	T=	32925.8000	lb/ft
I=	9	A=	.600	F.S.=	1.956	T=	31588.1000	lb/ft
I=	10	A=	.667	F.S.=	1.924	T=	30012.9500	lb/ft
I=	11	A=	.733	F.S.=	1.906	T=	28282.1600	lb/ft
I=	12	A=	.800	F.S.=	1.899	T=	26456.4300	lb/ft
I=	13	A=	.867	F.S.=	1.899	T=	24590.5600	lb/ft
I=	14	A=	.933	F.S.=	1.907	T=	22670.9700	lb/ft
I=	15	A=	1.000	F.S.=	1.916	T=	20643.6200	lb/ft
I=	16	A=	1.067	F.S.=	1.931	T=	18628.8800	lb/ft
I=	17	A=	1.133	F.S.=	1.968	T=	16798.2700	lb/ft
I=	18	A=	1.200	F.S.=	2.012	T=	15272.2500	lb/ft
I=	19	A=	1.267	F.S.=	2.055	T=	13998.5600	lb/ft
I=	20	A=	1.333	F.S.=	2.099	T=	12960.3100	lb/ft
I=	21	A=	1.400	F.S.=	2.144	T=	12101.6200	lb/ft
I=	22	A=	1.467	F.S.=	2.186	T=	11304.1000	lb/ft
I=	23	A=	1.533	F.S.=	2.228	T=	10574.6700	lb/ft
I=	24	A=	1.600	F.S.=	2.275	T=	10064.6400	lb/ft
I=	25	A=	1.667	F.S.=	2.321	T=	9587.8830	lb/ft
I=	26	A=	1.733	F.S.=	2.366	T=	9140.9280	lb/ft
I=	27	A=	1.800	F.S.=	2.412	T=	8721.6540	lb/ft
I=	28	A=	1.867	F.S.=	2.456	T=	8331.6190	lb/ft
I=	29	A=	1.933	F.S.=	2.500	T=	7963.6640	lb/ft
I=	30	A=	2.000	F.S.=	2.545	T=	7616.1090	lb/ft

H/LT= 1.183 H/LB= 1.420

NO. OF NAILS (INCLUDING THE LOWERMOST) = 7

**F.S. AGAINST SLIDING THROUGH TOE = 1.90

**F.S. AGAINST DEEP SEATED ROTATION = 4.15

1 TH NAIL FORCE =	978.6779	lb/ft
2 TH NAIL FORCE =	1589.4520	lb/ft
3 TH NAIL FORCE =	2125.5350	lb/ft
4 TH NAIL FORCE =	2814.0790	lb/ft
5 TH NAIL FORCE =	4077.0120	lb/ft
6 TH NAIL FORCE =	5468.3210	lb/ft
7 TH NAIL FORCE =	7537.4830	lb/ft

CASE 6 PART B FOR 26.5 AND 22 FT WALL
 < OUTPUT FILE > = CASE6B.OUT

I N P U T D A T A

S O I L P R O P E R T I E S

FOUNDATION	SOIL UNIT WEIGHT	=	130.000	pcf
	COHESION OF SOIL	=	1500.000	psf
	FRICTION ANGLE OF SOIL	=	30.000	deg
	UNDRAINED SU AT EXCAVATION LEVEL	=	3000.000	psf
	DEPTH TO HARDPAN FROM EXCAVATION	=	39.000	ft
	SU AT THE TOP OF HARDPAN	=	4500.000	psf
LAYER 1	HEIGHT TO TOP OF LAYER FROM TOE	=	15.500	ft
	SOIL UNIT WEIGHT	=	125.000	pcf
	COHESION OF SOIL	=	1500.000	psf
	FRICTION ANGLE OF SOIL	=	25.000	deg
LAYER 2	HEIGHT TO TOP OF LAYER FROM TOE	=	26.500	ft
	SOIL UNIT WEIGHT	=	120.000	pcf
	COHESION OF SOIL	=	600.000	psf
	FRICTION ANGLE OF SOIL	=	20.000	deg

W A L L & L O A D C O N D I T I O N

WALL INCLINATION ANGLE	=	5.000	deg
SURCHARGE LOAD	=	250.000	psf
HOR. ACCELERATION COEFF.	=	.000	

N A I L P R O P E R T I E S

CROSS SECTIONAL AREA OF NAIL	=	.790	si
GROUT DIAMETER	=	4.000	in
YIELD STRENGTH OF NAIL	=	60.000	ksi
NAIL INCLINATION ANGLE	=	20.000	deg
NAIL SKEW ANGLE AT SURFACE	=	.000	deg
DEPTH TO UPPERMOST NAIL	=	2.000	ft
VER. SPACING OF NAIL	=	5.000	ft
HOR. SPACING OF NAIL	=	5.000	ft
TOTAL LENGTH OF UPPERMOST NAIL	=	25.000	ft
TOTAL LENGTH OF LOWERMOST NAIL	=	20.000	ft
TOTAL UNBONDED LENGTH OF NAIL	=	2.000	ft

O U T P U T R E S U L T S

I= 2	A= .133	F.S.= 5.622	T= 7978.3750	lb/ft
I= 3	A= .200	F.S.= 3.578	T= 11870.6100	lb/ft
I= 4	A= .267	F.S.= 2.867	T= 13847.7800	lb/ft
I= 5	A= .333	F.S.= 2.507	T= 14834.7800	lb/ft
I= 6	A= .400	F.S.= 2.295	T= 15236.6400	lb/ft
I= 7	A= .467	F.S.= 2.160	T= 15244.9300	lb/ft
I= 8	A= .533	F.S.= 2.075	T= 14973.5200	lb/ft
I= 9	A= .600	F.S.= 2.019	T= 14521.7400	lb/ft
I= 10	A= .667	F.S.= 1.984	T= 13876.1700	lb/ft

I= 11	A= .733	F.S.= 1.963	T= 13102.2000	lb/ft
I= 12	A= .800	F.S.= 1.955	T= 12273.3100	lb/ft
I= 13	A= .867	F.S.= 1.956	T= 11421.9500	lb/ft
I= 14	A= .933	F.S.= 1.966	T= 10550.0800	lb/ft
I= 15	A= 1.000	F.S.= 1.983	T= 9683.0740	lb/ft
I= 16	A= 1.067	F.S.= 2.005	T= 8831.0580	lb/ft
I= 17	A= 1.133	F.S.= 2.031	T= 8001.4690	lb/ft
I= 18	A= 1.200	F.S.= 2.062	T= 7199.6000	lb/ft
I= 19	A= 1.267	F.S.= 2.102	T= 6445.7430	lb/ft
I= 20	A= 1.333	F.S.= 2.148	T= 5792.4090	lb/ft
I= 21	A= 1.400	F.S.= 2.193	T= 5226.7410	lb/ft
I= 22	A= 1.467	F.S.= 2.243	T= 4819.9720	lb/ft
I= 23	A= 1.533	F.S.= 2.288	T= 4448.7380	lb/ft
I= 24	A= 1.600	F.S.= 2.333	T= 4101.7750	lb/ft
I= 25	A= 1.667	F.S.= 2.380	T= 3772.3880	lb/ft
I= 26	A= 1.733	F.S.= 2.430	T= 3578.4970	lb/ft
I= 27	A= 1.800	F.S.= 2.480	T= 3401.2160	lb/ft
I= 28	A= 1.867	F.S.= 2.530	T= 3235.2220	lb/ft
I= 29	A= 1.933	F.S.= 2.580	T= 3079.5230	lb/ft
I= 30	A= 2.000	F.S.= 2.631	T= 2933.2690	lb/ft

H/LT= 1.060 H/LB= 1.325

NO. OF NAILS (INCLUDING THE LOWERMOST) = 5

**F.S. AGAINST SLIDING THROUGH TOE = 1.96

**F.S. AGAINST DEEP SEATED ROTATION = 5.52

1 TH NAIL FORCE =	779.7014	lb/ft
2 TH NAIL FORCE =	1942.9830	lb/ft
3 TH NAIL FORCE =	2390.6170	lb/ft
4 TH NAIL FORCE =	3011.1190	lb/ft
5 TH NAIL FORCE =	4148.8860	lb/ft

CASE 7 22.5 FT WALL
 < OUTPUT FILE > = CASE7B.OUT

INPUT DATA

SOIL PROPERTIES

FOUNDATION	SOIL UNIT WEIGHT	=	125.000	pcf
	COHESION OF SOIL	=	1100.000	psf
	FRICTION ANGLE OF SOIL	=	30.000	deg
	UNDRAINED SU AT EXCAVATION LEVEL	=	2200.000	psf
	DEPTH TO HARDPAN FROM EXCAVATION	=	8.000	ft
	SU AT THE TOP OF HARDPAN	=	3500.000	psf
LAYER 1	HEIGHT TO TOP OF LAYER FROM TOE	=	9.500	ft
	SOIL UNIT WEIGHT	=	125.000	pcf
	COHESION OF SOIL	=	1000.000	psf
	FRICTION ANGLE OF SOIL	=	30.000	deg
LAYER 2	HEIGHT TO TOP OF LAYER FROM TOE	=	22.500	ft
	SOIL UNIT WEIGHT	=	110.000	pcf
	COHESION OF SOIL	=	750.000	psf
	FRICTION ANGLE OF SOIL	=	20.000	deg

WALL & LOAD CONDITION

WALL INCLINATION ANGLE	=	5.000	deg
SURCHARGE LOAD	=	300.000	psf
HOR. ACCELERATION COEFF.	=	.000	

NAIL PROPERTIES

CROSS SECTIONAL AREA OF NAIL	=	.790	si
GROUT DIAMETER	=	4.000	in
YIELD STRENGTH OF NAIL	=	60.000	ksi
NAIL INCLINATION ANGLE	=	20.000	deg
NAIL SKEW ANGLE AT SURFACE	=	.000	deg
DEPTH TO UPPERMOST NAIL	=	2.000	ft
VER. SPACING OF NAIL	=	5.500	ft
HOR. SPACING OF NAIL	=	6.000	ft
TOTAL LENGTH OF UPPERMOST NAIL	=	20.000	ft
TOTAL LENGTH OF LOWERMOST NAIL	=	15.000	ft
TOTAL UNBONDED LENGTH OF NAIL	=	2.000	ft

OUTPUT RESULTS

I= 2	A= .133	F.S.= 5.845	T= 3134.1630	lb/ft
I= 3	A= .200	F.S.= 3.443	T= 5025.1370	lb/ft
I= 4	A= .267	F.S.= 2.646	T= 6042.6960	lb/ft
I= 5	A= .333	F.S.= 2.251	T= 6534.1570	lb/ft
I= 6	A= .400	F.S.= 2.024	T= 6711.1930	lb/ft
I= 7	A= .467	F.S.= 1.883	T= 6679.2290	lb/ft
I= 8	A= .533	F.S.= 1.794	T= 6497.4730	lb/ft
I= 9	A= .600	F.S.= 1.739	T= 6212.4800	lb/ft
I= 10	A= .667	F.S.= 1.708	T= 5858.4100	lb/ft

I=	11	A=	.733	F.S.=	1.694	T=	5463.8070	lb/ft
I=	12	A=	.800	F.S.=	1.694	T=	5040.0630	lb/ft
I=	13	A=	.867	F.S.=	1.705	T=	4606.2330	lb/ft
I=	14	A=	.933	F.S.=	1.724	T=	4172.4250	lb/ft
I=	15	A=	1.000	F.S.=	1.750	T=	3746.1910	lb/ft
I=	16	A=	1.067	F.S.=	1.781	T=	3332.2150	lb/ft
I=	17	A=	1.133	F.S.=	1.817	T=	2929.3270	lb/ft
I=	18	A=	1.200	F.S.=	1.862	T=	2550.4220	lb/ft
I=	19	A=	1.267	F.S.=	1.917	T=	2243.7440	lb/ft
I=	20	A=	1.333	F.S.=	1.971	T=	1976.4510	lb/ft
I=	21	A=	1.400	F.S.=	2.029	T=	1813.7950	lb/ft
I=	22	A=	1.467	F.S.=	2.084	T=	1665.5530	lb/ft
I=	23	A=	1.533	F.S.=	2.134	T=	1530.4360	lb/ft
I=	24	A=	1.600	F.S.=	2.183	T=	1405.0670	lb/ft
I=	25	A=	1.667	F.S.=	2.232	T=	1288.4800	lb/ft
I=	26	A=	1.733	F.S.=	2.281	T=	1179.8700	lb/ft
I=	27	A=	1.800	F.S.=	2.331	T=	1092.1270	lb/ft
I=	28	A=	1.867	F.S.=	2.383	T=	1050.7940	lb/ft
I=	29	A=	1.933	F.S.=	2.435	T=	1012.0890	lb/ft
I=	30	A=	2.000	F.S.=	2.490	T=	974.9437	lb/ft

H/LT= 1.125 H/LB= 1.500

NO. OF NAILS (INCLUDING THE LOWERMOST) = 4

**F.S. AGAINST SLIDING THROUGH TOE = 1.69

NO F.S. OF DEEP SEATED FAILURE IS CALCULATED

**DMAX IS TOO SMALL

1 TH NAIL FORCE =	731.8884	lb/ft
2 TH NAIL FORCE =	989.3829	lb/ft
3 TH NAIL FORCE =	1558.7860	lb/ft
4 TH NAIL FORCE =	2183.7500	lb/ft

CASE 8 PART A 25 AND 28 FT WALL
 < OUTPUT FILE > = CASE8A.OUT

INPUT DATA

SOIL PROPERTIES

FOUNDATION	SOIL UNIT WEIGHT	=	120.000 pcf
	COHESION OF SOIL	=	1500.000 psf
	FRICTION ANGLE OF SOIL	=	28.000 deg
	UNDRAINED SU AT EXCAVATION LEVEL	=	3000.000 psf
	DEPTH TO HARDPAN FROM EXCAVATION	=	28.000 ft
	SU AT THE TOP OF HARDPAN	=	4500.000 psf
LAYER 1	HEIGHT TO TOP OF LAYER FROM TOE	=	13.000 ft
	SOIL UNIT WEIGHT	=	120.000 pcf
	COHESION OF SOIL	=	1500.000 psf
	FRICTION ANGLE OF SOIL	=	28.000 deg
LAYER 2	HEIGHT TO TOP OF LAYER FROM TOE	=	28.000 ft
	SOIL UNIT WEIGHT	=	115.000 pcf
	COHESION OF SOIL	=	800.000 psf
	FRICTION ANGLE OF SOIL	=	30.000 deg

WALL & LOAD CONDITION

WALL INCLINATION ANGLE	=	5.000 deg
SURCHARGE LOAD	=	200.000 psf
HOR. ACCELERATION COEFF.	=	.000

NAIL PROPERTIES

CROSS SECTIONAL AREA OF NAIL	=	.790 si
GROUT DIAMETER	=	4.000 in
YIELD STRENGTH OF NAIL	=	60.000 ksi
NAIL INCLINATION ANGLE	=	20.000 deg
NAIL SKEW ANGLE AT SURFACE	=	.000 deg
DEPTH TO UPPERMOST NAIL	=	2.000 ft
VER. SPACING OF NAIL	=	5.500 ft
HOR. SPACING OF NAIL	=	5.000 ft
TOTAL LENGTH OF UPPERMOST NAIL	=	25.000 ft
TOTAL LENGTH OF LOWERMOST NAIL	=	20.000 ft
TOTAL UNBONDED LENGTH OF NAIL	=	2.000 ft

OUTPUT RESULTS

I= 2	A= .133	F.S.= 6.636	T= 7602.4290 lb/ft
I= 3	A= .200	F.S.= 4.189	T= 11316.5800 lb/ft
I= 4	A= .267	F.S.= 3.335	T= 13205.7900 lb/ft
I= 5	A= .333	F.S.= 2.896	T= 14147.2300 lb/ft
I= 6	A= .400	F.S.= 2.636	T= 14478.7600 lb/ft
I= 7	A= .467	F.S.= 2.469	T= 14406.4500 lb/ft
I= 8	A= .533	F.S.= 2.361	T= 14052.1600 lb/ft
I= 9	A= .600	F.S.= 2.287	T= 13516.9100 lb/ft
I= 10	A= .667	F.S.= 2.241	T= 12847.0000 lb/ft

I=	11	A=	.733	F.S.=	2.214	T=	12087.2600	lb/ft
I=	12	A=	.800	F.S.=	2.202	T=	11271.2800	lb/ft
I=	13	A=	.867	F.S.=	2.199	T=	10431.9400	lb/ft
I=	14	A=	.933	F.S.=	2.209	T=	9570.1800	lb/ft
I=	15	A=	1.000	F.S.=	2.226	T=	8712.2710	lb/ft
I=	16	A=	1.067	F.S.=	2.249	T=	7868.3760	lb/ft
I=	17	A=	1.133	F.S.=	2.278	T=	7046.0410	lb/ft
I=	18	A=	1.200	F.S.=	2.323	T=	6298.4610	lb/ft
I=	19	A=	1.267	F.S.=	2.377	T=	5637.6350	lb/ft
I=	20	A=	1.333	F.S.=	2.432	T=	5123.6650	lb/ft
I=	21	A=	1.400	F.S.=	2.488	T=	4719.4480	lb/ft
I=	22	A=	1.467	F.S.=	2.541	T=	4345.1570	lb/ft
I=	23	A=	1.533	F.S.=	2.594	T=	3995.0380	lb/ft
I=	24	A=	1.600	F.S.=	2.646	T=	3727.5750	lb/ft
I=	25	A=	1.667	F.S.=	2.701	T=	3543.3810	lb/ft
I=	26	A=	1.733	F.S.=	2.759	T=	3368.0560	lb/ft
I=	27	A=	1.800	F.S.=	2.816	T=	3205.8140	lb/ft
I=	28	A=	1.867	F.S.=	2.872	T=	3053.7030	lb/ft
I=	29	A=	1.933	F.S.=	2.929	T=	2910.8550	lb/ft
I=	30	A=	2.000	F.S.=	2.986	T=	2776.5290	lb/ft

H/LT= 1.120 H/LB= 1.400

NO. OF NAILS (INCLUDING THE LOWERMOST) = 5

**F.S. AGAINST SLIDING THROUGH TOE = 2.20

**F.S. AGAINST DEEP SEATED ROTATION = 5.58

1 TH NAIL FORCE =	830.2226	lb/ft
2 TH NAIL FORCE =	1226.3060	lb/ft
3 TH NAIL FORCE =	1907.8990	lb/ft
4 TH NAIL FORCE =	2642.8250	lb/ft
5 TH NAIL FORCE =	3824.6880	lb/ft

CASE 8 PART C 15 FT WALL
 < OUTPUT FILE > = CASE8C.OUT

INPUT DATA

SOIL PROPERTIES

FOUNDATION	SOIL UNIT WEIGHT	=	120.000 pcf
	COHESION OF SOIL	=	1500.000 psf
	FRICTION ANGLE OF SOIL	=	28.000 deg
	UNDRAINED SU AT EXCAVATION LEVEL	=	3000.000 psf
	DEPTH TO HARDPAN FROM EXCAVATION	=	41.000 ft
	SU AT THE TOP OF HARDPAN	=	4500.000 psf
LAYER 1	HEIGHT TO TOP OF LAYER FROM TOE	=	15.000 ft
	SOIL UNIT WEIGHT	=	115.000 pcf
	COHESION OF SOIL	=	800.000 psf
	FRICTION ANGLE OF SOIL	=	30.000 deg

WALL & LOAD CONDITION

WALL INCLINATION ANGLE	=	5.000 deg
SURCHARGE LOAD	=	200.000 psf
HOR. ACCELERATION COEFF.	=	.000

NAIL PROPERTIES

CROSS SECTIONAL AREA OF NAIL	=	.790 si
GROUT DIAMETER	=	4.000 in
YIELD STRENGTH OF NAIL	=	60.000 ksi
NAIL INCLINATION ANGLE	=	20.000 deg
NAIL SKEW ANGLE AT SURFACE	=	.000 deg
DEPTH TO UPPERMOST NAIL	=	2.000 ft
VER. SPACING OF NAIL	=	5.500 ft
HOR. SPACING OF NAIL	=	5.000 ft
TOTAL LENGTH OF UPPERMOST NAIL	=	12.000 ft
TOTAL LENGTH OF LOWERMOST NAIL	=	10.000 ft
TOTAL UNBONDED LENGTH OF NAIL	=	2.000 ft

OUTPUT RESULTS

I=	2	A=	.133	F.S.=	10.486	T=	955.5569 lb/ft
I=	3	A=	.200	F.S.=	5.853	T=	1655.0820 lb/ft
I=	4	A=	.267	F.S.=	4.361	T=	2118.4950 lb/ft
I=	5	A=	.333	F.S.=	3.637	T=	2372.9560 lb/ft
I=	6	A=	.400	F.S.=	3.223	T=	2509.2840 lb/ft
I=	7	A=	.467	F.S.=	2.967	T=	2557.8770 lb/ft
I=	8	A=	.533	F.S.=	2.799	T=	2517.0350 lb/ft
I=	9	A=	.600	F.S.=	2.690	T=	2430.8000 lb/ft
I=	10	A=	.667	F.S.=	2.621	T=	2312.9400 lb/ft
I=	11	A=	.733	F.S.=	2.579	T=	2175.2560 lb/ft
I=	12	A=	.800	F.S.=	2.562	T=	2022.3720 lb/ft
I=	13	A=	.867	F.S.=	2.561	T=	1862.3860 lb/ft
I=	14	A=	.933	F.S.=	2.573	T=	1699.6880 lb/ft
I=	15	A=	1.000	F.S.=	2.596	T=	1537.5160 lb/ft

I=	16	A=	1.067	F.S.=	2.627	T=	1378.2240	lb/ft
I=	17	A=	1.133	F.S.=	2.681	T=	1276.2540	lb/ft
I=	18	A=	1.200	F.S.=	2.736	T=	1187.9290	lb/ft
I=	19	A=	1.267	F.S.=	2.790	T=	1105.5080	lb/ft
I=	20	A=	1.333	F.S.=	2.844	T=	1028.4000	lb/ft
I=	21	A=	1.400	F.S.=	2.898	T=	956.0954	lb/ft
I=	22	A=	1.467	F.S.=	2.954	T=	887.4468	lb/ft
I=	23	A=	1.533	F.S.=	3.014	T=	860.8624	lb/ft
I=	24	A=	1.600	F.S.=	3.074	T=	836.4722	lb/ft
I=	25	A=	1.667	F.S.=	3.135	T=	813.3091	lb/ft
I=	26	A=	1.733	F.S.=	3.198	T=	791.2628	lb/ft
I=	27	A=	1.800	F.S.=	3.265	T=	769.4709	lb/ft
I=	28	A=	1.867	F.S.=	3.330	T=	749.2644	lb/ft
I=	29	A=	1.933	F.S.=	3.396	T=	729.9737	lb/ft
I=	30	A=	2.000	F.S.=	3.464	T=	711.5373	lb/ft

H/LT= 1.250 H/LB= 1.500

NO. OF NAILS (INCLUDING THE LOWERMOST) = 3

**F.S. AGAINST SLIDING THROUGH TOE = 2.56

**F.S. AGAINST DEEP SEATED ROTATION =10.23

1 TH NAIL FORCE =	224.6582	lb/ft
2 TH NAIL FORCE =	480.1544	lb/ft
3 TH NAIL FORCE =	1157.5740	lb/ft

CASE 9 33 FT WALL

< OUTPUT FILE > = CASE9.OUT

I N P U T D A T A

S O I L P R O P E R T I E S

FOUNDATION	SOIL UNIT WEIGHT	=	130.000 pcf
	COHESION OF SOIL	=	1200.000 psf
	FRICTION ANGLE OF SOIL	=	33.000 deg
	UNDRAINED SU AT EXCAVATION LEVEL	=	2000.000 psf
	DEPTH TO HARDPAN FROM EXCAVATION	=	23.000 ft
	SU AT THE TOP OF HARDPAN	=	3500.000 psf
LAYER 1	HEIGHT TO TOP OF LAYER FROM TOE	=	16.000 ft
	SOIL UNIT WEIGHT	=	130.000 pcf
	COHESION OF SOIL	=	1100.000 psf
	FRICTION ANGLE OF SOIL	=	33.000 deg
LAYER 2	HEIGHT TO TOP OF LAYER FROM TOE	=	33.000 ft
	SOIL UNIT WEIGHT	=	115.000 pcf
	COHESION OF SOIL	=	550.000 psf
	FRICTION ANGLE OF SOIL	=	28.000 deg

W A L L & L O A D C O N D I T I O N

WALL INCLINATION ANGLE	=	5.000 deg
SURCHARGE LOAD	=	100.000 psf
HOR. ACCELERATION COEFF.	=	.000

N A I L P R O P E R T I E S

CROSS SECTIONAL AREA OF NAIL	=	.790 si
GROUT DIAMETER	=	4.000 in
YIELD STRENGTH OF NAIL	=	60.000 ksi
NAIL INCLINATION ANGLE	=	20.000 deg
NAIL SKEW ANGLE AT SURFACE	=	.000 deg
DEPTH TO UPPERMOST NAIL	=	2.000 ft
VER. SPACING OF NAIL	=	5.000 ft
HOR. SPACING OF NAIL	=	5.000 ft
TOTAL LENGTH OF UPPERMOST NAIL	=	28.000 ft
TOTAL LENGTH OF LOWERMOST NAIL	=	25.000 ft
TOTAL UNBONDED LENGTH OF NAIL	=	2.000 ft

O U T P U T R E S U L T S

I=	2 A=	.133	F.S.=	3.564	T=	23037.3800 lb/ft
I=	3 A=	.200	F.S.=	2.740	T=	28099.2000 lb/ft
I=	4 A=	.267	F.S.=	2.389	T=	30303.7000 lb/ft
I=	5 A=	.333	F.S.=	2.193	T=	31235.0200 lb/ft
I=	6 A=	.400	F.S.=	2.075	T=	31353.9000 lb/ft
I=	7 A=	.467	F.S.=	1.999	T=	30990.5000 lb/ft
I=	8 A=	.533	F.S.=	1.951	T=	30277.1000 lb/ft
I=	9 A=	.600	F.S.=	1.925	T=	29302.0800 lb/ft
I=	10 A=	.667	F.S.=	1.914	T=	28129.2800 lb/ft

I=	11	A=	.733	F.S.=	1.910	T=	26816.0600	lb/ft
I=	12	A=	.800	F.S.=	1.915	T=	25425.0700	lb/ft
I=	13	A=	.867	F.S.=	1.926	T=	24013.1500	lb/ft
I=	14	A=	.933	F.S.=	1.946	T=	22569.2000	lb/ft
I=	15	A=	1.000	F.S.=	1.970	T=	21139.7700	lb/ft
I=	16	A=	1.067	F.S.=	1.997	T=	19736.7000	lb/ft
I=	17	A=	1.133	F.S.=	2.053	T=	18303.9300	lb/ft
I=	18	A=	1.200	F.S.=	2.116	T=	16965.4700	lb/ft
I=	19	A=	1.267	F.S.=	2.183	T=	15860.3600	lb/ft
I=	20	A=	1.333	F.S.=	2.238	T=	14890.5800	lb/ft
I=	21	A=	1.400	F.S.=	2.293	T=	14044.2500	lb/ft
I=	22	A=	1.467	F.S.=	2.351	T=	13378.3800	lb/ft
I=	23	A=	1.533	F.S.=	2.409	T=	12750.9000	lb/ft
I=	24	A=	1.600	F.S.=	2.461	T=	12182.4300	lb/ft
I=	25	A=	1.667	F.S.=	2.511	T=	11658.2000	lb/ft
I=	26	A=	1.733	F.S.=	2.561	T=	11165.1900	lb/ft
I=	27	A=	1.800	F.S.=	2.612	T=	10818.2600	lb/ft
I=	28	A=	1.867	F.S.=	2.662	T=	10507.9200	lb/ft
I=	29	A=	1.933	F.S.=	2.707	T=	10223.6800	lb/ft
I=	30	A=	2.000	F.S.=	2.750	T=	9964.0190	lb/ft

H/LT= 1.179 H/LB= 1.320

NO. OF NAILS (INCLUDING THE LOWERMOST) = 7

**F.S. AGAINST SLIDING THROUGH TOE = 1.91

**F.S. AGAINST DEEP SEATED ROTATION = 3.40

1	TH NAIL FORCE =	945.6563	lb/ft
2	TH NAIL FORCE =	1336.6560	lb/ft
3	TH NAIL FORCE =	2548.4430	lb/ft
4	TH NAIL FORCE =	3379.1010	lb/ft
5	TH NAIL FORCE =	4447.3010	lb/ft
6	TH NAIL FORCE =	5898.1320	lb/ft
7	TH NAIL FORCE =	8260.7730	lb/ft

CASE 10 30 FT WALL WITH 15 FT BENCH 2:1 SLOPE
 < DATA FILE > = CASE10.IN
 < OUTPUT FILE > = CASE10.OUT

INPUT DATA

SOIL PROPERTIES

FOUNDATION	SOIL UNIT WEIGHT	=	125.000 pcf
	COHESION OF SOIL	=	1600.000 psf
	FRICTION ANGLE OF SOIL	=	33.000 deg
	UNDRAINED SU AT EXCAVATION LEVEL	=	3000.000 psf
	DEPTH TO HARDPAN FROM EXCAVATION	=	10.000 ft
	SU AT THE TOP OF HARDPAN	=	4500.000 psf
LAYER 1	HEIGHT TO TOP OF LAYER FROM TOE	=	8.000 ft
	SOIL UNIT WEIGHT	=	125.000 pcf
	COHESION OF SOIL	=	1600.000 psf
	FRICTION ANGLE OF SOIL	=	33.000 deg
LAYER 2	HEIGHT TO TOP OF LAYER FROM TOE	=	30.000 ft
	SOIL UNIT WEIGHT	=	120.000 pcf
	COHESION OF SOIL	=	1000.000 psf
	FRICTION ANGLE OF SOIL	=	30.000 deg

GROUND SURFACE CONDITION

COORDINATES OF LINES

LINE NUMBER	X-COORD. (ft)	Y-COORD. (ft)
1	10.000	30.000
2	40.000	45.000

SOIL PROPERTIES

UNIT WEIGHT OF SOIL	=	110.000 pcf
COHESION OF SOIL	=	500.000 psf
FRICTION ANGLE OF SOIL	=	30.000 deg

WALL & LOAD CONDITION

WALL INCLINATION ANGLE	=	5.000 deg
SURCHARGE LOAD	=	100.000 psf
HOR. ACCELERATION COEFF.	=	.000

NAIL PROPERTIES

CROSS SECTIONAL AREA OF NAIL	=	.790 si
GROUT DIAMETER	=	4.000 in
YIELD STRENGTH OF NAIL	=	60.000 ksi
NAIL INCLINATION ANGLE	=	20.000 deg
NAIL SKEW ANGLE AT SURFACE	=	.000 deg
DEPTH TO UPPERMOST NAIL	=	2.000 ft
VER. SPACING OF NAIL	=	5.000 ft
HOR. SPACING OF NAIL	=	5.000 ft
TOTAL LENGTH OF UPPERMOST NAIL	=	28.000 ft

TOTAL LENGTH OF LOWERMOST NAIL = 23.000 ft
TOTAL UNBONDED LENGTH OF NAIL = 2.000 ft

O U T P U T R E S U L T S

I=	2	A=	.133	F.S.=	4.725	T=	14812.8800	lb/ft
I=	3	A=	.200	F.S.=	3.504	T=	18412.1200	lb/ft
I=	4	A=	.267	F.S.=	3.007	T=	19865.9100	lb/ft
I=	5	A=	.333	F.S.=	2.742	T=	20329.8900	lb/ft
I=	6	A=	.400	F.S.=	2.566	T=	20419.8500	lb/ft
I=	7	A=	.467	F.S.=	2.439	T=	20249.1400	lb/ft
I=	8	A=	.533	F.S.=	2.345	T=	19875.8100	lb/ft
I=	9	A=	.600	F.S.=	2.269	T=	19369.0900	lb/ft
I=	10	A=	.667	F.S.=	2.208	T=	18749.8600	lb/ft
I=	11	A=	.733	F.S.=	2.157	T=	18045.3600	lb/ft
I=	12	A=	.800	F.S.=	2.111	T=	17271.9400	lb/ft
I=	13	A=	.867	F.S.=	2.070	T=	16398.6300	lb/ft
I=	14	A=	.933	F.S.=	2.031	T=	15477.8500	lb/ft
I=	15	A=	1.000	F.S.=	2.008	T=	14397.1300	lb/ft
I=	16	A=	1.067	F.S.=	2.019	T=	13412.3200	lb/ft
I=	17	A=	1.133	F.S.=	2.039	T=	12530.3800	lb/ft
I=	18	A=	1.200	F.S.=	2.066	T=	11781.6500	lb/ft
I=	19	A=	1.267	F.S.=	2.099	T=	11036.9000	lb/ft
I=	20	A=	1.333	F.S.=	2.137	T=	10468.5300	lb/ft
I=	21	A=	1.400	F.S.=	2.182	T=	9966.4360	lb/ft
I=	22	A=	1.467	F.S.=	2.226	T=	9493.1150	lb/ft
I=	23	A=	1.533	F.S.=	2.272	T=	9036.6480	lb/ft
I=	24	A=	1.600	F.S.=	2.316	T=	8615.8000	lb/ft
I=	25	A=	1.667	F.S.=	2.359	T=	8218.3610	lb/ft
I=	26	A=	1.733	F.S.=	2.396	T=	7858.2590	lb/ft
I=	27	A=	1.800	F.S.=	2.440	T=	7638.1380	lb/ft
I=	28	A=	1.867	F.S.=	2.480	T=	7454.2070	lb/ft
I=	29	A=	1.933	F.S.=	2.515	T=	7293.0430	lb/ft
I=	30	A=	2.000	F.S.=	2.541	T=	7157.7830	lb/ft

H/LT= 1.071 H/LB= 1.304

NO. OF NAILS (INCLUDING THE LOWERMOST) = 6

**F.S. AGAINST SLIDING THROUGH TOE = 2.01

NO F.S. OF DEEP SEATED FAILURE IS CALCULATED
**DMAX IS TOO SMALL

1 TH NAIL FORCE =	.0000	lb/ft
2 TH NAIL FORCE =	442.7396	lb/ft
3 TH NAIL FORCE =	1082.9810	lb/ft
4 TH NAIL FORCE =	2427.8700	lb/ft
5 TH NAIL FORCE =	4006.4210	lb/ft
6 TH NAIL FORCE =	6437.1180	lb/ft

CASE 11 PART A 16 FT WALL
 < OUTPUT FILE > = CASE11A.OUT

INPUT DATA

SOIL PROPERTIES

FOUNDATION	SOIL UNIT WEIGHT	=	125.000 pcf
	COHESION OF SOIL	=	1100.000 psf
	FRICTION ANGLE OF SOIL	=	30.000 deg
	UNDRAINED SU AT EXCAVATION LEVEL	=	2000.000 psf
	DEPTH TO HARDPAN FROM EXCAVATION	=	30.000 ft
	SU AT THE TOP OF HARDPAN	=	3500.000 psf
LAYER 1	HEIGHT TO TOP OF LAYER FROM TOE	=	16.000 ft
	SOIL UNIT WEIGHT	=	125.000 pcf
	COHESION OF SOIL	=	1100.000 psf
	FRICTION ANGLE OF SOIL	=	30.000 deg

WALL & LOAD CONDITION

WALL INCLINATION ANGLE	=	5.000 deg
SURCHARGE LOAD	=	300.000 psf
HOR. ACCELERATION COEFF.	=	.000

NAIL PROPERTIES

CROSS SECTIONAL AREA OF NAIL	=	.790 si
GROUT DIAMETER	=	4.000 in
YIELD STRENGTH OF NAIL	=	60.000 ksi
NAIL INCLINATION ANGLE	=	20.000 deg
NAIL SKEW ANGLE AT SURFACE	=	.000 deg
DEPTH TO UPPERMOST NAIL	=	2.000 ft
VER. SPACING OF NAIL	=	6.000 ft
HOR. SPACING OF NAIL	=	5.000 ft
TOTAL LENGTH OF UPPERMOST NAIL	=	15.000 ft
TOTAL LENGTH OF LOWERMOST NAIL	=	12.000 ft
TOTAL UNBONDED LENGTH OF NAIL	=	2.000 ft

OUTPUT RESULTS

I=	2	A=	.133	F.S.=	8.666	T=	1554.5450 lb/ft
I=	3	A=	.200	F.S.=	4.945	T=	2620.1590 lb/ft
I=	4	A=	.267	F.S.=	3.744	T=	3280.5980 lb/ft
I=	5	A=	.333	F.S.=	3.160	T=	3653.1700 lb/ft
I=	6	A=	.400	F.S.=	2.827	T=	3853.6260 lb/ft
I=	7	A=	.467	F.S.=	2.619	T=	3926.3700 lb/ft
I=	8	A=	.533	F.S.=	2.484	T=	3886.6280 lb/ft
I=	9	A=	.600	F.S.=	2.397	T=	3784.2450 lb/ft
I=	10	A=	.667	F.S.=	2.345	T=	3638.0920 lb/ft
I=	11	A=	.733	F.S.=	2.316	T=	3462.3070 lb/ft
I=	12	A=	.800	F.S.=	2.305	T=	3269.5390 lb/ft
I=	13	A=	.867	F.S.=	2.309	T=	3063.0140 lb/ft
I=	14	A=	.933	F.S.=	2.324	T=	2851.7980 lb/ft
I=	15	A=	1.000	F.S.=	2.348	T=	2640.2120 lb/ft

I=	16	A=	1.067	F.S.=	2.379	T=	2431.4120	lb/ft
I=	17	A=	1.133	F.S.=	2.416	T=	2227.6570	lb/ft
I=	18	A=	1.200	F.S.=	2.458	T=	2030.5170	lb/ft
I=	19	A=	1.267	F.S.=	2.505	T=	1841.0300	lb/ft
I=	20	A=	1.333	F.S.=	2.565	T=	1708.3350	lb/ft
I=	21	A=	1.400	F.S.=	2.627	T=	1604.1140	lb/ft
I=	22	A=	1.467	F.S.=	2.688	T=	1507.5620	lb/ft
I=	23	A=	1.533	F.S.=	2.750	T=	1416.6720	lb/ft
I=	24	A=	1.600	F.S.=	2.807	T=	1333.7210	lb/ft
I=	25	A=	1.667	F.S.=	2.863	T=	1255.8850	lb/ft
I=	26	A=	1.733	F.S.=	2.920	T=	1182.7090	lb/ft
I=	27	A=	1.800	F.S.=	2.980	T=	1112.7150	lb/ft
I=	28	A=	1.867	F.S.=	3.042	T=	1073.6880	lb/ft
I=	29	A=	1.933	F.S.=	3.106	T=	1047.2190	lb/ft
I=	30	A=	2.000	F.S.=	3.170	T=	1022.0380	lb/ft

H/LT= 1.067 H/LB= 1.333

NO. OF NAILS (INCLUDING THE LOWERMOST) = 3

**F.S. AGAINST SLIDING THROUGH TOE = 2.30

**F.S. AGAINST DEEP SEATED ROTATION = 6.91

1 TH NAIL FORCE =	650.2002	lb/ft
2 TH NAIL FORCE =	973.7742	lb/ft
3 TH NAIL FORCE =	1645.5640	lb/ft

CASE 12 PART A 30 FT WALL

< DATA FILE > = CASE12A.IN
 < OUTPUT FILE > = CASE12A.OUT

INPUT DATA

SOIL PROPERTIES

FOUNDATION	SOIL UNIT WEIGHT	=	125.000	pcf
	COHESION OF SOIL	=	1300.000	psf
	FRICTION ANGLE OF SOIL	=	32.000	deg
	UNDRAINED SU AT EXCAVATION LEVEL	=	2000.000	psf
	DEPTH TO HARDPAN FROM EXCAVATION	=	23.000	ft
	SU AT THE TOP OF HARDPAN	=	4000.000	psf
LAYER 1	HEIGHT TO TOP OF LAYER FROM TOE	=	10.000	ft
	SOIL UNIT WEIGHT	=	125.000	pcf
	COHESION OF SOIL	=	1100.000	psf
	FRICTION ANGLE OF SOIL	=	32.000	deg
LAYER 2	HEIGHT TO TOP OF LAYER FROM TOE	=	30.000	ft
	SOIL UNIT WEIGHT	=	120.000	pcf
	COHESION OF SOIL	=	550.000	psf
	FRICTION ANGLE OF SOIL	=	28.000	deg

WALL & LOAD CONDITION

WALL INCLINATION ANGLE	=	5.000	deg
SURCHARGE LOAD	=	300.000	psf
HOR. ACCELERATION COEFF.	=	.000	

NAIL PROPERTIES

CROSS SECTIONAL AREA OF NAIL	=	.790	si
GROUT DIAMETER	=	4.000	in
YIELD STRENGTH OF NAIL	=	60.000	ksi
NAIL INCLINATION ANGLE	=	20.000	deg
NAIL SKEW ANGLE AT SURFACE	=	.000	deg
DEPTH TO UPPERMOST NAIL	=	2.000	ft
VER. SPACING OF NAIL	=	5.000	ft
HOR. SPACING OF NAIL	=	5.000	ft
TOTAL LENGTH OF UPPERMOST NAIL	=	25.000	ft
TOTAL LENGTH OF LOWERMOST NAIL	=	20.000	ft
TOTAL UNBONDED LENGTH OF NAIL	=	2.000	ft

OUTPUT RESULTS

I= 2	A= .133	F.S.= 3.445	T= 14818.0600	lb/ft
I= 3	A= .200	F.S.= 2.471	T= 19169.1700	lb/ft
I= 4	A= .267	F.S.= 2.083	T= 21087.7200	lb/ft
I= 5	A= .333	F.S.= 1.874	T= 21774.4200	lb/ft
I= 6	A= .400	F.S.= 1.745	T= 21819.6800	lb/ft
I= 7	A= .467	F.S.= 1.663	T= 21422.8400	lb/ft
I= 8	A= .533	F.S.= 1.612	T= 20707.2100	lb/ft
I= 9	A= .600	F.S.= 1.581	T= 19727.0200	lb/ft

I=	10	A=	.667	F.S.=	1.563	T=	18622.0100	lb/ft
I=	11	A=	.733	F.S.=	1.557	T=	17428.8400	lb/ft
I=	12	A=	.800	F.S.=	1.560	T=	16187.4000	lb/ft
I=	13	A=	.867	F.S.=	1.570	T=	14927.3200	lb/ft
I=	14	A=	.933	F.S.=	1.585	T=	13677.6800	lb/ft
I=	15	A=	1.000	F.S.=	1.607	T=	12437.4400	lb/ft
I=	16	A=	1.067	F.S.=	1.632	T=	11229.8000	lb/ft
I=	17	A=	1.133	F.S.=	1.681	T=	10137.7200	lb/ft
I=	18	A=	1.200	F.S.=	1.731	T=	9231.2700	lb/ft
I=	19	A=	1.267	F.S.=	1.780	T=	8481.4030	lb/ft
I=	20	A=	1.333	F.S.=	1.826	T=	7800.4670	lb/ft
I=	21	A=	1.400	F.S.=	1.874	T=	7307.5520	lb/ft
I=	22	A=	1.467	F.S.=	1.923	T=	6848.1220	lb/ft
I=	23	A=	1.533	F.S.=	1.965	T=	6434.7920	lb/ft
I=	24	A=	1.600	F.S.=	2.006	T=	6051.6010	lb/ft
I=	25	A=	1.667	F.S.=	2.049	T=	5687.0930	lb/ft
I=	26	A=	1.733	F.S.=	2.093	T=	5407.5280	lb/ft
I=	27	A=	1.800	F.S.=	2.139	T=	5208.9350	lb/ft
I=	28	A=	1.867	F.S.=	2.186	T=	5022.1770	lb/ft
I=	29	A=	1.933	F.S.=	2.233	T=	4846.1730	lb/ft
I=	30	A=	2.000	F.S.=	2.280	T=	4680.0120	lb/ft

H/LT= 1.200 H/LB= 1.500

NO. OF NAILS (INCLUDING THE LOWERMOST) = 6

**F.S. AGAINST SLIDING THROUGH TOE = 1.56

**F.S. AGAINST DEEP SEATED ROTATION = 4.06

1 TH NAIL FORCE =	1028.6790	lb/ft
2 TH NAIL FORCE =	1386.0530	lb/ft
3 TH NAIL FORCE =	1832.5430	lb/ft
4 TH NAIL FORCE =	3189.0050	lb/ft
5 TH NAIL FORCE =	4200.8570	lb/ft
6 TH NAIL FORCE =	5791.7080	lb/ft

CASE 12 PART B 25 FT WALL

< DATA FILE > = CASE12B.IN
 < OUTPUT FILE > = CASE12B.OUT

INPUT DATA

SOIL PROPERTIES

FOUNDATION	SOIL UNIT WEIGHT	=	125.000	pcf
	COHESION OF SOIL	=	1200.000	psf
	FRICTION ANGLE OF SOIL	=	32.000	deg
	UNDRAINED SU AT EXCAVATION LEVEL	=	2000.000	psf
	DEPTH TO HARDPAN FROM EXCAVATION	=	28.000	ft
	SU AT THE TOP OF HARDPAN	=	4000.000	psf
LAYER 1	HEIGHT TO TOP OF LAYER FROM TOE	=	5.000	ft
	SOIL UNIT WEIGHT	=	125.000	pcf
	COHESION OF SOIL	=	1100.000	psf
	FRICTION ANGLE OF SOIL	=	32.000	deg
LAYER 2	HEIGHT TO TOP OF LAYER FROM TOE	=	25.000	ft
	SOIL UNIT WEIGHT	=	120.000	pcf
	COHESION OF SOIL	=	550.000	psf
	FRICTION ANGLE OF SOIL	=	28.000	deg

WALL & LOAD CONDITION

WALL INCLINATION ANGLE	=	5.000	deg
SURCHARGE LOAD	=	300.000	psf
HOR. ACCELERATION COEFF.	=	.000	

NAIL PROPERTIES

CROSS SECTIONAL AREA OF NAIL	=	.790	si
GROUT DIAMETER	=	4.000	in
YIELD STRENGTH OF NAIL	=	60.000	ksi
NAIL INCLINATION ANGLE	=	20.000	deg
NAIL SKEW ANGLE AT SURFACE	=	.000	deg
DEPTH TO UPPERMOST NAIL	=	2.000	ft
VER. SPACING OF NAIL	=	5.000	ft
HOR. SPACING OF NAIL	=	5.000	ft
TOTAL LENGTH OF UPPERMOST NAIL	=	20.000	ft
TOTAL LENGTH OF LOWERMOST NAIL	=	18.000	ft
TOTAL UNBONDED LENGTH OF NAIL	=	2.000	ft

OUTPUT RESULTS

I=	2	A=	.133	F.S.=	3.666	T=	8465.2760	lb/ft
I=	3	A=	.200	F.S.=	2.517	T=	11523.5400	lb/ft
I=	4	A=	.267	F.S.=	2.084	T=	12923.1200	lb/ft
I=	5	A=	.333	F.S.=	1.858	T=	13487.9000	lb/ft
I=	6	A=	.400	F.S.=	1.721	T=	13586.9800	lb/ft
I=	7	A=	.467	F.S.=	1.637	T=	13367.7500	lb/ft
I=	8	A=	.533	F.S.=	1.587	T=	12935.2000	lb/ft
I=	9	A=	.600	F.S.=	1.556	T=	12379.0600	lb/ft

I=	10	A=	.667	F.S.=	1.542	T=	11728.8600	lb/ft
I=	11	A=	.733	F.S.=	1.540	T=	11020.6300	lb/ft
I=	12	A=	.800	F.S.=	1.547	T=	10280.7100	lb/ft
I=	13	A=	.867	F.S.=	1.559	T=	9499.9340	lb/ft
I=	14	A=	.933	F.S.=	1.578	T=	8714.3180	lb/ft
I=	15	A=	1.000	F.S.=	1.603	T=	7943.5140	lb/ft
I=	16	A=	1.067	F.S.=	1.642	T=	7228.9170	lb/ft
I=	17	A=	1.133	F.S.=	1.691	T=	6603.8290	lb/ft
I=	18	A=	1.200	F.S.=	1.737	T=	6063.8700	lb/ft
I=	19	A=	1.267	F.S.=	1.785	T=	5655.7730	lb/ft
I=	20	A=	1.333	F.S.=	1.830	T=	5280.3920	lb/ft
I=	21	A=	1.400	F.S.=	1.876	T=	4929.3220	lb/ft
I=	22	A=	1.467	F.S.=	1.917	T=	4609.9880	lb/ft
I=	23	A=	1.533	F.S.=	1.961	T=	4370.7580	lb/ft
I=	24	A=	1.600	F.S.=	2.008	T=	4173.4990	lb/ft
I=	25	A=	1.667	F.S.=	2.054	T=	3991.0590	lb/ft
I=	26	A=	1.733	F.S.=	2.100	T=	3820.0300	lb/ft
I=	27	A=	1.800	F.S.=	2.147	T=	3659.3500	lb/ft
I=	28	A=	1.867	F.S.=	2.193	T=	3508.2790	lb/ft
I=	29	A=	1.933	F.S.=	2.239	T=	3367.2820	lb/ft
I=	30	A=	2.000	F.S.=	2.285	T=	3234.1200	lb/ft

H/LT= 1.250 H/LB= 1.389

NO. OF NAILS (INCLUDING THE LOWERMOST) = 5

**F.S. AGAINST SLIDING THROUGH TOE = 1.54

**F.S. AGAINST DEEP SEATED ROTATION = 4.82

1 TH NAIL FORCE =	709.6445	lb/ft
2 TH NAIL FORCE =	1088.8630	lb/ft
3 TH NAIL FORCE =	1607.4020	lb/ft
4 TH NAIL FORCE =	3057.3800	lb/ft
5 TH NAIL FORCE =	4557.3410	lb/ft

* CASE 13 13.5 FT WALL
 < OUTPUT FILE > = CASE13.OUT

INPUT DATA

SOIL PROPERTIES

FOUNDATION	SOIL UNIT WEIGHT	=	115.000 pcf
	COHESION OF SOIL	=	1100.000 psf
	FRICTION ANGLE OF SOIL	=	20.000 deg
	UNDRAINED SU AT EXCAVATION LEVEL	=	2200.000 psf
	DEPTH TO HARDPAN FROM EXCAVATION	=	40.000 ft
	SU AT THE TOP OF HARDPAN	=	4500.000 psf
LAYER 1	HEIGHT TO TOP OF LAYER FROM TOE	=	13.500 ft
	SOIL UNIT WEIGHT	=	115.000 pcf
	COHESION OF SOIL	=	1100.000 psf
	FRICTION ANGLE OF SOIL	=	20.000 deg

WALL & LOAD CONDITION

WALL INCLINATION ANGLE	=	5.000 deg
SURCHARGE LOAD	=	200.000 psf
HOR. ACCELERATION COEFF.	=	.000

NAIL PROPERTIES

CROSS SECTIONAL AREA OF NAIL	=	.440 si
GROUT DIAMETER	=	4.000 in
YIELD STRENGTH OF NAIL	=	60.000 ksi
NAIL INCLINATION ANGLE	=	20.000 deg
NAIL SKEW ANGLE AT SURFACE	=	.000 deg
DEPTH TO UPPERMOST NAIL	=	2.000 ft
VER. SPACING OF NAIL	=	4.000 ft
HOR. SPACING OF NAIL	=	6.000 ft
TOTAL LENGTH OF UPPERMOST NAIL	=	12.000 ft
TOTAL LENGTH OF LOWERMOST NAIL	=	10.000 ft
TOTAL UNBONDED LENGTH OF NAIL	=	2.000 ft

OUTPUT RESULTS

I=	2 A=	.133	F.S.=	11.544	T=	567.2046 lb/ft
I=	3 A=	.200	F.S.=	6.214	T=	1030.8820 lb/ft
I=	4 A=	.267	F.S.=	4.511	T=	1379.7120 lb/ft
I=	5 A=	.333	F.S.=	3.691	T=	1582.9620 lb/ft
I=	6 A=	.400	F.S.=	3.220	T=	1686.3220 lb/ft
I=	7 A=	.467	F.S.=	2.924	T=	1716.3920 lb/ft
I=	8 A=	.533	F.S.=	2.726	T=	1698.9190 lb/ft
I=	9 A=	.600	F.S.=	2.596	T=	1643.8350 lb/ft
I=	10 A=	.667	F.S.=	2.510	T=	1563.0680 lb/ft
I=	11 A=	.733	F.S.=	2.454	T=	1464.4730 lb/ft
I=	12 A=	.800	F.S.=	2.421	T=	1354.1770 lb/ft
I=	13 A=	.867	F.S.=	2.406	T=	1236.9160 lb/ft
I=	14 A=	.933	F.S.=	2.404	T=	1116.3190 lb/ft
I=	15 A=	1.000	F.S.=	2.413	T=	995.1240 lb/ft

I=	16	A=	1.067	F.S.=	2.431	T=	875.3663	lb/ft
I=	17	A=	1.133	F.S.=	2.457	T=	758.5211	lb/ft
I=	18	A=	1.200	F.S.=	2.489	T=	645.6303	lb/ft
I=	19	A=	1.267	F.S.=	2.529	T=	561.9138	lb/ft
I=	20	A=	1.333	F.S.=	2.574	T=	501.3703	lb/ft
I=	21	A=	1.400	F.S.=	2.622	T=	444.1593	lb/ft
I=	22	A=	1.467	F.S.=	2.667	T=	390.7994	lb/ft
I=	23	A=	1.533	F.S.=	2.714	T=	340.6147	lb/ft
I=	24	A=	1.600	F.S.=	2.767	T=	314.9128	lb/ft
I=	25	A=	1.667	F.S.=	2.821	T=	296.4200	lb/ft
I=	26	A=	1.733	F.S.=	2.877	T=	279.0861	lb/ft
I=	27	A=	1.800	F.S.=	2.934	T=	262.8392	lb/ft
I=	28	A=	1.867	F.S.=	2.992	T=	247.6110	lb/ft
I=	29	A=	1.933	F.S.=	3.050	T=	233.3381	lb/ft
I=	30	A=	2.000	F.S.=	3.110	T=	219.9597	lb/ft

H/LT= 1.125 H/LB= 1.350

NO. OF NAILS (INCLUDING THE LOWERMOST) = 3

**F.S. AGAINST SLIDING THROUGH TOE = 2.40

**F.S. AGAINST DEEP SEATED ROTATION =11.27

1 TH NAIL FORCE =	219.7532	lb/ft
2 TH NAIL FORCE =	340.4219	lb/ft
3 TH NAIL FORCE =	556.1438	lb/ft

CASE 14 PART A 22 FT WALL
 < OUTPUT FILE > = CASE14A.OUT

INPUT DATA

SOIL PROPERTIES

FOUNDATION	SOIL UNIT WEIGHT	=	120.000 pcf
	COHESION OF SOIL	=	1100.000 psf
	FRICTION ANGLE OF SOIL	=	30.000 deg
	UNDRAINED SU AT EXCAVATION LEVEL	=	2000.000 psf
	DEPTH TO HARDPAN FROM EXCAVATION	=	25.000 ft
	SU AT THE TOP OF HARDPAN	=	4500.000 psf
LAYER 1	HEIGHT TO TOP OF LAYER FROM TOE	=	9.000 ft
	SOIL UNIT WEIGHT	=	120.000 pcf
	COHESION OF SOIL	=	1100.000 psf
	FRICTION ANGLE OF SOIL	=	30.000 deg
LAYER 2	HEIGHT TO TOP OF LAYER FROM TOE	=	22.000 ft
	SOIL UNIT WEIGHT	=	110.000 pcf
	COHESION OF SOIL	=	500.000 psf
	FRICTION ANGLE OF SOIL	=	28.000 deg

WALL & LOAD CONDITION

WALL INCLINATION ANGLE	=	5.000 deg
SURCHARGE LOAD	=	300.000 psf
HOR. ACCELERATION COEFF.	=	.000

NAIL PROPERTIES

CROSS SECTIONAL AREA OF NAIL	=	.790 si
GROUT DIAMETER	=	4.000 in
YIELD STRENGTH OF NAIL	=	60.000 ksi
NAIL INCLINATION ANGLE	=	20.000 deg
NAIL SKEW ANGLE AT SURFACE	=	.000 deg
DEPTH TO UPPERMOST NAIL	=	2.000 ft
VER. SPACING OF NAIL	=	4.500 ft
HOR. SPACING OF NAIL	=	6.000 ft
TOTAL LENGTH OF UPPERMOST NAIL	=	18.000 ft
TOTAL LENGTH OF LOWERMOST NAIL	=	15.000 ft
TOTAL UNBONDED LENGTH OF NAIL	=	2.000 ft

OUTPUT RESULTS

I= 2	A= .133	F.S.= 4.979	T= 4341.9030 lb/ft
I= 3	A= .200	F.S.= 3.142	T= 6495.7700 lb/ft
I= 4	A= .267	F.S.= 2.506	T= 7572.0930 lb/ft
I= 5	A= .333	F.S.= 2.185	T= 8118.3150 lb/ft
I= 6	A= .400	F.S.= 2.000	T= 8319.2480 lb/ft
I= 7	A= .467	F.S.= 1.885	T= 8264.1870 lb/ft
I= 8	A= .533	F.S.= 1.813	T= 8028.0540 lb/ft
I= 9	A= .600	F.S.= 1.769	T= 7694.2030 lb/ft
I= 10	A= .667	F.S.= 1.745	T= 7291.1250 lb/ft

I=	11	A=	.733	F.S.=	1.737	T=	6844.6630	lb/ft
I=	12	A=	.800	F.S.=	1.739	T=	6378.7210	lb/ft
I=	13	A=	.867	F.S.=	1.752	T=	5895.6130	lb/ft
I=	14	A=	.933	F.S.=	1.773	T=	5413.2820	lb/ft
I=	15	A=	1.000	F.S.=	1.800	T=	4938.8370	lb/ft
I=	16	A=	1.067	F.S.=	1.831	T=	4477.3240	lb/ft
I=	17	A=	1.133	F.S.=	1.884	T=	4086.9140	lb/ft
I=	18	A=	1.200	F.S.=	1.934	T=	3730.3320	lb/ft
I=	19	A=	1.267	F.S.=	1.986	T=	3464.5930	lb/ft
I=	20	A=	1.333	F.S.=	2.037	T=	3229.0440	lb/ft
I=	21	A=	1.400	F.S.=	2.084	T=	3012.9620	lb/ft
I=	22	A=	1.467	F.S.=	2.132	T=	2810.4290	lb/ft
I=	23	A=	1.533	F.S.=	2.177	T=	2641.7670	lb/ft
I=	24	A=	1.600	F.S.=	2.226	T=	2535.5150	lb/ft
I=	25	A=	1.667	F.S.=	2.278	T=	2433.6170	lb/ft
I=	26	A=	1.733	F.S.=	2.328	T=	2339.2010	lb/ft
I=	27	A=	1.800	F.S.=	2.379	T=	2250.3590	lb/ft
I=	28	A=	1.867	F.S.=	2.430	T=	2166.6070	lb/ft
I=	29	A=	1.933	F.S.=	2.481	T=	2087.5350	lb/ft
I=	30	A=	2.000	F.S.=	2.533	T=	2012.7900	lb/ft

H/LT= 1.222 H/LB= 1.467

NO. OF NAILS (INCLUDING THE LOWERMOST) = 5

**F.S. AGAINST SLIDING THROUGH TOE = 1.74

**F.S. AGAINST DEEP SEATED ROTATION = 6.34

1 TH NAIL FORCE =	482.6588	lb/ft
2 TH NAIL FORCE =	681.4896	lb/ft
3 TH NAIL FORCE =	1287.9620	lb/ft
4 TH NAIL FORCE =	1781.7990	lb/ft
5 TH NAIL FORCE =	2610.7540	lb/ft

CASE 14 B 13 FT WALL
 < OUTPUT FILE > = CASE14B.OUT

INPUT DATA

SOIL PROPERTIES

FOUNDATION	SOIL UNIT WEIGHT	=	120.000	pcf
	COHESION OF SOIL	=	1100.000	psf
	FRICTION ANGLE OF SOIL	=	30.000	deg
	UNDRAINED SU AT EXCAVATION LEVEL	=	2000.000	psf
	DEPTH TO HARDPAN FROM EXCAVATION	=	33.000	ft
	SU AT THE TOP OF HARDPAN	=	4000.000	psf
LAYER 1	HEIGHT TO TOP OF LAYER FROM TOE	=	13.000	ft
	SOIL UNIT WEIGHT	=	110.000	pcf
	COHESION OF SOIL	=	500.000	psf
	FRICTION ANGLE OF SOIL	=	28.000	deg

WALL & LOAD CONDITION

	WALL INCLINATION ANGLE	=	5.000	deg
	SURCHARGE LOAD	=	300.000	psf
	HOR. ACCELERATION COEFF.	=	.000	

NAIL PROPERTIES

	CROSS SECTIONAL AREA OF NAIL	=	.790	si
	GROUT DIAMETER	=	4.000	in
	YIELD STRENGTH OF NAIL	=	60.000	ksi
	NAIL INCLINATION ANGLE	=	20.000	deg
	NAIL SKEW ANGLE AT SURFACE	=	.000	deg
	DEPTH TO UPPERMOST NAIL	=	2.000	ft
	VER. SPACING OF NAIL	=	4.500	ft
	HOR. SPACING OF NAIL	=	6.000	ft
	TOTAL LENGTH OF UPPERMOST NAIL	=	11.000	ft
	TOTAL LENGTH OF LOWERMOST NAIL	=	9.000	ft
	TOTAL UNBONDED LENGTH OF NAIL	=	2.000	ft

OUTPUT RESULTS

I=	2	A=	.133	F.S.=	5.851	T=	838.3091	lb/ft
I=	3	A=	.200	F.S.=	3.329	T=	1397.3720	lb/ft
I=	4	A=	.267	F.S.=	2.524	T=	1762.8160	lb/ft
I=	5	A=	.333	F.S.=	2.136	T=	1949.2420	lb/ft
I=	6	A=	.400	F.S.=	1.915	T=	2037.6140	lb/ft
I=	7	A=	.467	F.S.=	1.781	T=	2061.4430	lb/ft
I=	8	A=	.533	F.S.=	1.698	T=	2027.2590	lb/ft
I=	9	A=	.600	F.S.=	1.647	T=	1951.0920	lb/ft
I=	10	A=	.667	F.S.=	1.618	T=	1854.8580	lb/ft
I=	11	A=	.733	F.S.=	1.607	T=	1743.3730	lb/ft
I=	12	A=	.800	F.S.=	1.609	T=	1624.3700	lb/ft
I=	13	A=	.867	F.S.=	1.620	T=	1501.8930	lb/ft
I=	14	A=	.933	F.S.=	1.639	T=	1378.9450	lb/ft
I=	15	A=	1.000	F.S.=	1.664	T=	1257.6740	lb/ft

I=	16	A=	1.067	F.S.=	1.694	T=	1139.5780	lb/ft
I=	17	A=	1.133	F.S.=	1.728	T=	1025.6660	lb/ft
I=	18	A=	1.200	F.S.=	1.769	T=	937.7746	lb/ft
I=	19	A=	1.267	F.S.=	1.815	T=	873.8356	lb/ft
I=	20	A=	1.333	F.S.=	1.859	T=	814.3790	lb/ft
I=	21	A=	1.400	F.S.=	1.904	T=	758.5441	lb/ft
I=	22	A=	1.467	F.S.=	1.947	T=	707.0550	lb/ft
I=	23	A=	1.533	F.S.=	1.991	T=	658.7742	lb/ft
I=	24	A=	1.600	F.S.=	2.036	T=	622.6808	lb/ft
I=	25	A=	1.667	F.S.=	2.083	T=	603.6078	lb/ft
I=	26	A=	1.733	F.S.=	2.134	T=	585.0589	lb/ft
I=	27	A=	1.800	F.S.=	2.183	T=	567.9261	lb/ft
I=	28	A=	1.867	F.S.=	2.233	T=	551.6949	lb/ft
I=	29	A=	1.933	F.S.=	2.283	T=	536.2932	lb/ft
I=	30	A=	2.000	F.S.=	2.333	T=	521.6588	lb/ft

H/LT= 1.182 H/LB= 1.444

NO. OF NAILS (INCLUDING THE LOWERMOST) = 3

**F.S. AGAINST SLIDING THROUGH TOE = 1.61

**F.S. AGAINST DEEP SEATED ROTATION = 9.81

1 TH NAIL FORCE =	320.4303	lb/ft
2 TH NAIL FORCE =	480.8821	lb/ft
3 TH NAIL FORCE =	942.0610	lb/ft

CASE 15 42 FT WALL

< DATA FILE > = CASE15.IN
 < OUTPUT FILE > = CASE15FINAL.OUT

INPUT DATA

SOIL PROPERTIES

FOUNDATION	SOIL UNIT WEIGHT	=	125.000 pcf
	COHESION OF SOIL	=	1500.000 psf
	FRICTION ANGLE OF SOIL	=	30.000 deg
	UNDRAINED SU AT EXCAVATION LEVEL	=	3000.000 psf
	DEPTH TO HARDPAN FROM EXCAVATION	=	25.000 ft
	SU AT THE TOP OF HARDPAN	=	4500.000 psf
LAYER 1	HEIGHT TO TOP OF LAYER FROM TOE	=	19.000 ft
	SOIL UNIT WEIGHT	=	125.000 pcf
	COHESION OF SOIL	=	1500.000 psf
	FRICTION ANGLE OF SOIL	=	30.000 deg
LAYER 2	HEIGHT TO TOP OF LAYER FROM TOE	=	32.000 ft
	SOIL UNIT WEIGHT	=	120.000 pcf
	COHESION OF SOIL	=	1000.000 psf
	FRICTION ANGLE OF SOIL	=	30.000 deg
LAYER 3	HEIGHT TO TOP OF LAYER FROM TOE	=	42.000 ft
	SOIL UNIT WEIGHT	=	110.000 pcf
	COHESION OF SOIL	=	550.000 psf
	FRICTION ANGLE OF SOIL	=	20.000 deg

WALL & LOAD CONDITION

WALL INCLINATION ANGLE	=	5.000 deg
SURCHARGE LOAD	=	300.000 psf
HOR. ACCELERATION COEFF.	=	.000

NAIL PROPERTIES

CROSS SECTIONAL AREA OF NAIL	=	1.000 si
GROUT DIAMETER	=	4.000 in
YIELD STRENGTH OF NAIL	=	60.000 ksi
NAIL INCLINATION ANGLE	=	20.000 deg
NAIL SKEW ANGLE AT SURFACE	=	.000 deg
DEPTH TO UPPERMOST NAIL	=	2.000 ft
VER. SPACING OF NAIL	=	4.500 ft
HOR. SPACING OF NAIL	=	6.000 ft
TOTAL LENGTH OF UPPERMOST NAIL	=	35.000 ft
TOTAL LENGTH OF LOWERMOST NAIL	=	30.000 ft
TOTAL UNBONDED LENGTH OF NAIL	=	2.000 ft

OUTPUT RESULTS

I= 2	A= .133	F.S.= 3.803	T= 35928.9100 lb/ft
I= 3	A= .200	F.S.= 2.838	T= 44459.9400 lb/ft
I= 4	A= .267	F.S.= 2.438	T= 48168.2600 lb/ft

I=	5	A=	.333	F.S.=	2.214	T=	49646.9800	lb/ft
I=	6	A=	.400	F.S.=	2.075	T=	49596.8400	lb/ft
I=	7	A=	.467	F.S.=	1.981	T=	48649.5800	lb/ft
I=	8	A=	.533	F.S.=	1.920	T=	47039.8400	lb/ft
I=	9	A=	.600	F.S.=	1.881	T=	44978.7200	lb/ft
I=	10	A=	.667	F.S.=	1.855	T=	42670.7300	lb/ft
I=	11	A=	.733	F.S.=	1.839	T=	40129.9800	lb/ft
I=	12	A=	.800	F.S.=	1.831	T=	37441.2000	lb/ft
I=	13	A=	.867	F.S.=	1.829	T=	34669.3300	lb/ft
I=	14	A=	.933	F.S.=	1.834	T=	31837.8400	lb/ft
I=	15	A=	1.000	F.S.=	1.845	T=	29025.7900	lb/ft
I=	16	A=	1.067	F.S.=	1.863	T=	26256.1000	lb/ft
I=	17	A=	1.133	F.S.=	1.907	T=	23667.3700	lb/ft
I=	18	A=	1.200	F.S.=	1.950	T=	21504.7000	lb/ft
I=	19	A=	1.267	F.S.=	1.993	T=	19752.2600	lb/ft
I=	20	A=	1.333	F.S.=	2.032	T=	18188.5800	lb/ft
I=	21	A=	1.400	F.S.=	2.074	T=	16978.8400	lb/ft
I=	22	A=	1.467	F.S.=	2.114	T=	15855.3600	lb/ft
I=	23	A=	1.533	F.S.=	2.153	T=	14858.2500	lb/ft
I=	24	A=	1.600	F.S.=	2.195	T=	14099.5700	lb/ft
I=	25	A=	1.667	F.S.=	2.237	T=	13389.5000	lb/ft
I=	26	A=	1.733	F.S.=	2.279	T=	12722.8800	lb/ft
I=	27	A=	1.800	F.S.=	2.321	T=	12095.5700	lb/ft
I=	28	A=	1.867	F.S.=	2.363	T=	11504.2500	lb/ft
I=	29	A=	1.933	F.S.=	2.408	T=	11040.9100	lb/ft
I=	30	A=	2.000	F.S.=	2.455	T=	10656.6700	lb/ft

H/LT= 1.200 H/LB= 1.400

NO. OF NAILS (INCLUDING THE LOWERMOST) = 9

**F.S. AGAINST SLIDING THROUGH TOE = 1.83

**F.S. AGAINST DEEP SEATED ROTATION = 3.65

1	TH NAIL FORCE =	938.5042	lb/ft
2	TH NAIL FORCE =	1283.4570	lb/ft
3	TH NAIL FORCE =	1708.7710	lb/ft
4	TH NAIL FORCE =	2606.9420	lb/ft
5	TH NAIL FORCE =	3320.7850	lb/ft
6	TH NAIL FORCE =	4197.1860	lb/ft
7	TH NAIL FORCE =	5289.5090	lb/ft
8	TH NAIL FORCE =	6693.6060	lb/ft
9	TH NAIL FORCE =	8630.5700	lb/ft

Appendix D: Materials Take offs

CASE 1

CONVENTIONAL									
AREA=	12,560	Sq ft			FT	Each	Total, FT	Total steel #	
				HP 12X53	32.5	63	2047.5		
TIE BACK	63	Each	1.25 " rebar	HP 12X53	15	1	15		
WALES	30	Each	10 ft	HP 12X53	22	1	22		
WALES	1	Each	20 ft	pound			=====		
				53			2,084.50	110,478.50	

SOILNAILING				
		Each	Length	Total Length
#8 rebar	1 st	111	20.00	2,220
	2 nd	109	19.00	2,071
	3 rd	107	17.50	1,873
	4 th	105	16.50	1,733
	5 th	103	15.00	1,545
		=====		=====
		535		9,441
#4 wales	10,700	Ft		
6x6 wire mesh	12,560	Sqft		
2 2" gunnite	12,560	Sqft	153.51	Cy
no. of nails	535			

SOIL NAILING				
total linear ft of nails drill	9,441	ft		
# 8 total linear ft of nails steel	9,441	ft		
no. of soil nails	535	each		
no. of soil nails plates & nuts	535	each		
#4 wales	10,700	lft		
6x6 wire mesh	12,560	sqft		
shotcrete or gunnite 2 2"	12,560	sqft		
OTA		ls		

CASE 2

CONVENTIONAL									
AREA=	22,400	Sqft				FT	Each	Total, FT	Total steel #
					HP 12X53	34	84	2,856	
TIE BACK	164	Each	1.25 " rebar						
WALES	80	Each	10 FT						
WALES	2	Each	20' FT		pound			=====	
					53			2856	151368

SOIL NAILING					
		Each	Length	Total Length	
#8 rebar	1 st	146	28.00	4,088	
	2 nd	144	27.00	3,888	
	3 rd	142	25.50	3,621	
	4 th	140	24.50	3,430	
	5 th	138	23.00	3,174	
	6 th	136	22.00	2,992	
		=====		=====	
		846		21,193.00	
#4 wales	16,800	ft			
6x6 wire mesh	22,400	sqft			
2 2" gunnite	22,400	sqft	273.78	cy	
no. of nails	846				

SOIL NAILING					
total linear ft of nails drill	21,193	ft			
# 8 total linear ft of nails steel	21,193	ft			
no. of soil nails	846	each			
no. of soil nails plates & nuts	846	each			
#4 wales	16,800	lft			
6x6 wire mesh	22400	sqft			
shotcrete or gunnite 2 2"	22400	sqft			
layout	40	man/day			

CASE 3

CONVENTIONAL								
AREA=	1,875	Sq ft			FT	Each	Total, FT	Total steel #
				HP 10X42	21	14	294	
TIE BACK	7	Each	1" rebar	HP 10X42	15	2	30	
WALES	7	Each	10 FT					
				pound			=====	
				42			324	13,608.00

SOIL NAILING				
		Each	Length	Total length
#8 rebar	1 st	23	12.00	276
	2 nd	21	11.00	231
	3 rd	20	10.00	200
		=====		=====
		64		707.00
#4 wales	1,440	ft		
6x6 wire mesh	1,875	sqft		
2 2" gunnite	1,875	sqft	22.92	cy
no. of nail plate plates	64			

SOIL NAILING				
total linear ft of nails drill	707	ft		
# 8 total linear ft of nails steel	707	ft		
no. of soil nails	64	each		
no. of soil nails plates & nuts	64	each		
#4 wales	1,440	lft		
6x6 wire mesh	1,875.00	sqft		
shotcrete or gunnite 2 2"	1,875.00	sqft		
layout	40	man/day		

Case 4

CONVENTIONAL									
	AREA=	8,940	Sq ft			FT	Each	Total, Ft	Total steel #
					HP 10X42	30	24	720	
	TIE BACK	7	Each	1" rebar	HP 10X42	25	1	25	
	TIE BACK	46	Each	1.25" rebar	HP 10X42	37	10	370	
					HP 10X42	27	2	54	
	WALES	2	Each	10 FT	pound		=====	=====	
	WALES	25	Each	20 FT	42		37	1,169.00	49,098.00

Soil Nailing									
		Each	Length	Total length					
#8 rebar	1 st	23	25.00	575					
	2 nd	22	24.00	528					
	3 rd	21	23.00	483					
part A	4 th	20	22.00	440	A=	5,656	Sqft	Avg length=	22.5
	5 th	19	21.00	399					
	6 th	18	20.00	360					
		=====		=====					
		123		2,785.00					
		Each	Length	Total length					
#8 rebar	1 st	22	30.00	660					
	2 nd	21	29.00	609					
	3 rd	20	28.50	570					
part B	4 th	19	27.50	523	A=	3,284	Sqft	Avg length=	27.5
	5 th	18	26.50	477					
	6 th	17	26.00	442					
	7 th	15	25	375					
		=====		=====					
		132		3,655.50					
#4 wales	7,200	ft							
6x6 wire mesh	8,940	sqft							
2 2" gunnite	8,940	sqft	109.27	cy					
no. of nails	255								

SOIL NAILING					
total linear ft of nails drill	6,441	ft			
# 8 total linear ft of nails steel	6,441	ft			
no. of soil nails	255	each			
no. of soil nails plates & nuts	255	each			
#4 wales	7,200	lft			
6x6 wire mesh	8,940.00	sqft			
shotcrete or gunnite 2 2"	8,940.00	sqft			
design		ls			
layout	40	man/day			

CASE 5

CONVENTIONAL								
AREA=	24,318	Sq ft			FT	Each	Total ft	Total steel #
				HP 12X53	30	42	1,260.0	
TIE BACK	190	Each	1.25" rebar					
				pound		===	=====	=====
WALES	3	Each	10 ft	53		42	1,260.0	66,780.0
WALES	30	Each	20 ft					
12x53 Beam Penetration	130	Each		HP 10x42	10	1	10	
				HP 10x42	25	2	50	
				HP 10x42	33	1	33	
				HP 10x42	28	46	1288	
				HP 10x42	20	1	20	
				HP 10x42	6.5	1	6.5	
				pound		=====	=====	
				42		52.00	1,407.5	59,115.0
								=====
								125,895.00

CASE 5

SOIL NAILING							
		Each	Length	Total length			
#8 rebar	1 st	79	30.00	2,370			
	2 nd	78	29.50	2,301			
	3 rd	77	28.50	2,195			
part A	4 th	76	27.50	2,090			
	5 th	75	26.50	1,988			
	6 th	73	26.00	1,898			
	7 th	71	25.00	1,775			
		=====		=====			
		529		14,616.00			
		Each	Length	Total length			
#8 rebar	1 st	80	20.00	1,600			
	2 nd	79	19.00	1,501			
	3 rd	78	17.50	1,365			
part B	4 th	76	16.50	1,254			
	5 th	75	15.00	1,125			
		=====		=====			
		388		6,845.00			
#4 wales	18,412	ft					
6x6 wire mesh	24,318	sqft					
2 2" gunnite	24,318	sqft	297.22	cy			
no. of nails	917						

SOIL NAILING							
total linear ft of nails drill	21,461	ft					
# 8 total linear ft of nails steel	21,461	ft					
no. of soil nails	917	each					
no. of soil nails plates & nuts	917	each					
#4 wales	18,412	lft					
6x6 wire mesh	24,318.00	sqft					
shotcrete or gunnite 2 2"	24,318.00	sqft					
layout	40	man/day					
mob/demob		ls					

CASE 6

CONVENTIONAL								
AREA=	24,933	Sq ft			FT	Each	Total, ft	Total steel #
				HP 12X53	25	1	25.00	
TIE BACK	148	Each	1" rebar		37.5	37	1,387.50	
TIE BACK	40	Each	1.25" rebar		30	68	2,040.00	
					15	1	15.00	
Beam Penetration	148	Each		pound		====	=====	=====
				53		107	3,467.50	183,777.50
WALES	20	Each	20 ft					

SOIL NAILING					
total linear ft of nails drill	23,886	ft			
# 8 total linear ft of nails steel	23,886	ft			
no. of soil nails	951	each			
no. of soil nails plates & nuts	951	each			
#4 wales	19,080	lft			
6x6 wire mesh	24,933.00	sqft			
shotcrete or gunnite 2 2"	24,933.00	sqft			
design		ls			
layout	40	man/day			

CASE 6

SOIL NAILING									
		<u>Each</u>	<u>Length</u>	<u>Total length</u>					
#8 rebar	1 st	68	30.00	2,040					
	2 nd	67	29.50	1,977					
	3 rd	66	28.50	1,881					
part A	4 th	65	27.50	1,788	A=	11,382	Sqft	Avg length=	27.5
	5 th	64	26.50	1,696					
	6 th	62	26.00	1,612					
	7 th	60	25.00	1,500					
		====		=====					
		452		12,493.00					
		<u>Each</u>	<u>Length</u>	<u>Total length</u>					
#8 rebar	1 st	49	25.00	1,225					
	2 nd	49	24.00	1,176					
	3 rd	49	22.50	1,103	A=	6,522	Sqft	Avg length=	22.5
part B	4 th	49	21.00	1,029					
	5 th	49	20.00	980					
		====		=====					
		245		5,512.50					
		<u>Each</u>	<u>Length</u>	<u>Total length</u>					
part C	1 st	65	25.00	1,625					
	2 nd	64	24.00	1,536	A=	7,029	Sqft	Avg length=	23.50
	3 rd	63	22.50	1,418					
	4 th	62	21.00	1,302					
		====		=====					
		254		5,880.50					
#4 wales	19,080	ft							
6x6 wire mesh	24,933	sqft							
2 2" gunnite	24,933	sqft	304.74	cy					
no. of nails	951								

CASE 7

CONVENTIONAL								
AREA=	3,020	Sq ft			FT	EACH	LFT	Total steel #
				HP 10X42	22	10	220.00	
TIE BACK	0	Each	1" rebar	HP 10X42	31.5	10	315.00	
TIE BACK	14	Each	1.25" rebar					
Beam penetration	10	Each		pound		=====	=====	=====
				42		20.00	535.00	22,470.00
WALES	4	Each	10 FT					

SOIL NAILING									
		<u>Each</u>	<u>Length</u>	<u>Total length</u>					
#8 rebar	1 st	12	12.00	144				Avg length=	
	2 nd	11	11.00	121	A=	1,005	Sqft		11
part A	3 rd	10	10.00	100					
		=====		=====					
		33		365.00					
		<u>Each</u>	<u>Length</u>	<u>Total length</u>					
	1 st	17	20.00	340				Avg length=	
part B	2 nd	16	18.50	296	A=	2,015	Sqft		17
	3 rd	15	16.50	248					
	4 th	13	15.00	195					
		=====		=====					
		61		1,078.50					
#4 wales	2,272	ft							
6x6 wire mesh	3,020	sqft							
2 2" gunnite	3,020	sqft	36.91	cy					
no. of nails	94								

CASE 7

SOIL NAILING				
total linear ft of nails drill	1,444	ft		
# 8 total linear ft of nails steel	1,444	ft		
no. of soil nails	94	each		
no. of soil nails plates & nuts	94	each		
#4 wales	2,272	lft		
6x6 wire mesh	3,020.00	sqft		
shotcrete or gunnite 2 2"	3,020.00	sqft		
layout	40	man/day		
mob/demob		ls		

CASE 8

CONVENTIONAL								
AREA=	13,234	Sq ft			FT	Each	LFT	Total steel #
				HP 10X42	11	1	11.00	
TIE BACK	86	Each	1 " rebar	HP 10X42	20	1	20.00	
				HP 10X42	30	10	300.00	
				HP 10X42	28	39	1,092.0	
				HP 10X42	21	11	231.00	
				pound		=====	=====	=====
				42		62.00	1,654.0	69,468.00
WALES	20	Each	10 ft					
WALES	26	Each	20 ft					

SOIL NAILING				
total linear ft of nails drill	10,931	ft		
# 8 total linear ft of nails steel	10,931	ft		
no. of soil nails	514	each		
no. of soil nails plates & nuts	514	each		
#4 wales	10,264	lft		
6x6 wire mesh	13,234.00	sqft		
shotcrete or gunnite 2 2"	13,234.00	sqft		
design		ls		
layout	40	man/day		

CASE 8

SOIL NAILING				
		Each	Length	Total length
#8 rebar	1 st	31	25.00	775
	2 nd	30	24.00	720
	3 rd	29	22.50	653
part A	4 th	28	21.00	588
	5 th	27	20.00	540
		=====		=====
		145		3,275.50
		Each	Length	Total length
#8 rebar	1 st	63	25.00	1,575
	2 nd	63	24.00	1,512
	3 rd	63	22.50	1,418
part B	4 th	62	21.00	1,302
	5 th	61	20.00	1,220
		=====		=====
		312		7,026.50
		Each	Length	Total length
part C	1 st	20	12.00	240
	2 nd	19	11.00	209
	3 rd	18	10.00	180
		=====		=====
		57		629.00
#4 wales	10,264	ft		
6x6 wire mesh	13,234	sqft		
2 2" gunnite	13,234	sqft	161.75	cy
no. of nails	514			

CASE 9

CONVENTIONAL								
AREA=	8,514	Sq ft			FT	Each	Total, ft	Total steel #
				HP 12X53	35	28	980.00	
				HP 12X53	30	2	60.00	
TIE BACK	57	Each	1.25" rebar					
beam penetration=	57	Each						
				pound		=====	=====	=====
				53		30.00	1,040.0	55,120.00

SOIL NAILING									
		Each	Length	Total length					
#8 rebar	1 st	51	28.00	1,428					
	2 nd	50	27.50	1,375					
	3 rd	49	27.00	1,323	A=	8,514	Sqft	Avg length=	26.50
	4 th	48	26.00	1,248					
	5 th	47	25.50	1,199					
	6 th	46	25.00	1,150					
		=====		=====					
		291.00		7,722.50					
#4 walers	5,808	ft							
6x6 wire mesh	8,514	sqft							
2 2" gunnite	8,514	sqft	104.06	cy					
no. of nails	291.00								

CASE 9

SOIL NAILING				
total linear ft of nails drill	7,723	ft		
# 8 total linear ft of nails steel	7,723	ft		
no. of soil nails	291.00	each		
no. of soil nails plates & nuts	291	each		
#4 wales	5,808	lft		
6x6 wire mesh	8,514.00	sqft		
shotcrete or gunnite 2 2"	8,514.00	sqft		
layout	40	man/day		
mob/demob		ls		
OTA		ls		

CASE 10

CONVENTIONAL								
AREA=	11,940	Sq ft			FT	Each	LFT	Total steel #
				HP 12X53	15	1	15.00	
					20	1	20.00	
TIE BACK	46	Each	1.25 "	rebar	25	2	50.00	
					32	46	1,472.0	
				pound		====	=====	=====
				53		50.0	1,557.0	82,521.00
WALES	46	Each	10 FT					

SOIL NAILING									
		Each	Length	Total length					
#8 rebar	1 st	84	28.00	2,352					
	2 nd	82	27.50	2,255					
	3 rd	80	27.00	2,160	A=	11,940	Sqft	Avg length=	26.50
	4 th	78	26.00	2,028					
	5 th	76	25.50	1,938					
	6 th	74	25.00	1,850					
		=====		=====					
		474.00		12,583.00					
#4 walers	9,552	ft							
6x6 wire mesh	11,940	sqft							
2, 2" gunnite	11,940	sqft	145.93	cy					
no. of nails	474.00								

CASE 10

SOIL NAILING			
total linear ft of nails drill	12,583	ft	
# 8 total linear ft of nails steel	12,583	ft	
no. of soil nails	474.00	each	
no. of soil nails plates & nuts	474	each	
#4 wales	9,552	lft	
6x6 wire mesh	11,940.00	sqft	
shotcrete or gunnite 2 2"	11,940.00	sqft	
design		ls	
layout	40	man/day	
mob/demob		ls	
OTA		ls	

CASE 11

CONVENTIONAL								
AREA=	6,218	Sq ft			FT	Each	LFT	total steel #
				HP 10X42	24	21	504.00	
TIE BACK	35	Each	1" rebar	HP 10X42	19.5	16	312.00	
				HP 10X42	16	12	192.00	
WALES	25	Each	10 FT					
				pound		=====	=====	=====
				42		49.00	1,008.0	42,336.00

SOIL NAILING									
		<u>Each</u>	<u>Length</u>	<u>Total length</u>					
#8 rebar	1 st	74	15.00	1,110					
	2 nd	72	14.00	1,008	A=	6,218	Sqft	Avg length=	13.50
	3 rd	71	13.00	923					
	4 th	33	12.00	396					
		=====		=====					
		250.00		3,437.00					
#4 wales	3,368	ft							
6x6 wire mesh	6,218	sqft							
2, 2" gunnite	6,218	sqft	76.00	cy					
no. of nails	250.00								

CASE 11

SOIL NAILING			
total linear ft of nails drill	3,437	ft	
# 8 total linear ft of nails steel	3,437	ft	
no. of soil nails	250.00	each	
no. of soil nails plates & nuts	250	each	
#4 wales	3,368	lft	
6x6 wire mesh	6,218.00	sqft	
shotcrete or gunnite 2 2"	6,218.00	sqft	
design		ls	
layout	40	man/day	
mob/demob		ls	
OTA		ls	

CASE 12

CONVENTIONAL									
AREA=	14,710	Sq ft				FT	Each	Total ft	Total steel #
				HP 10X42	42	15	1	15.00	630.00
TIE BACK	24	Each	1 " rebar	HP 10X42	42	20	2	40.00	1,680.00
TIE BACK	42	Each	1.25 " rebar	HP 12X53	53	32	21	672.00	35,616.00
TIE BACK	24	Each	1.25 " rebar	HP 12X53	53	34	25	850.00	45,050.00
				HP 12X53	53	28.5	23	655.50	34,741.50
							=====	=====	=====
WALES	45	Each	10 FT				72.00	2,232.5	117,717.50

SOIL NAILING				
total linear ft of nails drill	12,389	ft		
# 8 total linear ft of nails steel	12,389	ft		
no. of soil nails	579.0	each		
no. of soil nails plates & nuts	579	each		
#4 wales	11,592	lft		
6x6 wire mesh	14,710.00	sqft		
shotcrete or gunnite 2 2"	14,710.00	sqft		
design		ls		
layout	40	man/day		
mob/demob		ls		
OTA		ls		

CASE 12

SOIL NAILING									
		<u>Each</u>	<u>Length</u>	<u>Total length</u>					
#8 rebar	1 st	37	25.00	925					
	2 nd	36	24.00	864					
	3 rd	35	23.00	805	A=	5,250	Sqft	Avg length=	22.50
<u>part A</u>	4 th	34	22.00	748					
	5 th	33	21.00	693					
	6 th	32	20.00	640					
		=====		=====					
		207.0		4,675.0					
		<u>Each</u>	<u>Length</u>	<u>Total length</u>					
#8 rebar	1 st	42	25.00	1,050					
	2 nd	42	24.00	1,008					
	3 rd	42	23.00	966	A=	5,250	Sqft	Avg length=	23.00
<u>part B</u>	4 th	42	22.00	924					
	5 th	42	21.00	882					
		=====		=====					
		210.0		4,830.0					
		<u>Each</u>	<u>Length</u>	<u>Total length</u>					
#8 rebar	1 st	42	20.00	840					
	2 nd	41	19.00	779	A=	4,210	Sqft	Avg length=	17.75
	3 rd	40	17.00	680					
<u>part C</u>	4 th	39	15.00	585					
		=====		=====					
		162.0		2,884.0					
#4 wales	11,592	ft							
6x6 wire mesh	14,710	sqft							
2 2" gunnite	14,710	sqft	179.79	cy					
no. of nails	579.0								

CASE 13

CONVENTIONAL									
AREA=	2,765	Sq ft							
				HP	42	20	25	500.00	21,000.00
				10X42					
TIE BACK	12	Each	1" rebar						
							=====	=====	=====
							25.00	500.00	21,000.00
WALES	12	Each	10 FT						

SOIL NAILING					
		<u>each</u>	<u>length</u>	<u>total length</u>	
#8 rebar	1 st	35	12.00	420	
	2 nd	35	11.00	385	
	3 rd	34	10.00	340	
		=====		=====	
		104		1145	
#4 wales	2,460	ft			
6x6 wire mesh	2,765	sqft			
2 2" gunnite	2,765	sqft	33.79	cy	
no. of nails	104				

CASE 13

SOIL NAILING			
total linear ft of nails drill	1,145	ft	
# 8 total linear ft of nails steel	1,145	ft	
no. of soil nails	104	each	
no. of soil nails plates & nuts	104	each	
#4 wales	2,460	lft	
6x6 wire mesh	2,765.0 0	sqft	
shotcrete or gunnite 2 2"	2,765.0 0	sqft	
design		ls	
layout	40	man/day	
mob/demob		ls	
OTA		ls	

CASE 14

CONVENTIONAL									
AREA=	4,100	Sq ft				FT	Each	Total ft	Total steel #
				HP 10X42	42	15	1	15.00	630.00
TIE BACK	5	Each	1" rebar	HP 10X42	42	20	2	40.00	1,680.00
TIE BACK	16	Each	1.25" rebar	HP 10X42	42	30	16	480.00	20,160.00
TIE BACK	0	Each	1.25" rebar	HP 10X42	42	25	1	25.00	1,050.00
				HP 10X42	42	18.5	9	166.50	6,993.00
							=====	=====	=====
							29.00	726.50	30,513.00
WALES	13	EACH	10 FT						

SOIL NAILING									
		<u>Each</u>	<u>Length</u>	<u>Total length</u>					
#8 rebar	1 st	25	18.00	450					
	2 nd	24	17.00	408					
	3 rd	24	16.50	396	A=	3,105	Sqft	Avg length=	16.50
part A	4 th	23	16.00	368					
	5 th	22	15.00	330					
		=====		=====					
		118		1952					
		<u>Each</u>	<u>Length</u>	<u>Total length</u>					
#8 rebar	1 st	14	11.00	154					
	2 nd	13	10.00	130	A=	995	Sqft	Avg length=	10.00
part B	3 rd	12	9.00	108					
		=====		=====					
		39		392					
#4 wales	3,728	ft							
6x6 wire mesh	4,100	sqft							
2 2" gunnite	4,100	sqft	50.11	cy					
no. of nails	157								

CASE 14

SOIL NAILING				
total linear ft of nails drill	2,344	ft		
# 8 total linear ft of nails steel	2,344	ft		
no. of soil nails	157	each		
no. of soil nails plates & nuts	157	each		
#4 wales	3,728	lft		
6x6 wire mesh	4,100.00	sqft		
shotcrete or gunnite 2 2"	4,100.00	sqft		
design		ls		
layout	40	man/day		
mob/demob		ls		
OTA		ls		

CASE 15

CONVENTIONAL									
AREA=	71,820	Sq ft				FT	Each	Total ft	Total steel #
				HP 12X53	53	44	202	8,888.00	471,064.00
TIE BACK	808	Each	1 " rebar						
beam penetration=	808	Each							

SOIL NAILING									
		<u>Each</u>	<u>Length</u>	<u>Total length</u>					
#8 rebar	1 st	286	35.00	10,010					
	2 nd	286	34.50	9,867					
	3 rd	286	34.00	9,724					
	4 th	286	33.00	9,438	A=	71,820	Sqft	Avg length=	32.50
	5 th	286	32.50	9,295					
	6 th	286	32.00	9,152					
	7 th	286	31.50	9,009					
	8 th	286	30.50	8,723					
	9 th	286	30.00	8,580					
		=====		=====					
		1,716.0		57,486.0					
#4 wales	61,596	ft							
6x6 wire mesh	71,820	sqft							
2 2" gunnite	71,820	sqft	877.80	cy					
no. of nails	1,716.0								

CASE 15

SOIL NAILING		
total linear ft of nails drill	57,486	ft
# 8 total linear ft of nails steel	57,486	ft
no. of soil nails	1,716.0	each
no. of soil nails plates & nuts	1716	each
#4 wales	61,596	lft
6x6 wire mesh	71,820.00	sqft
shotcrete or gunnite 2 2"	71,820.00	sqft
pins to hold mesh		
design		ls
layout	40	man/day
mob/demob		ls
OTA		ls

Appendix E: Cost for Conventional Systems

FOR TIEBACK Anchor

CASE 1

use proof load @ 120% of DL

1 inch	3.1
1 1/4 inch	4.5
1 3/8 inch	5.7

bonded zone

25 ft FOR 1" BAR	8	bag
30 ft FOR 1.25" BAR	10	bag

$$1 \text{ 1/4 inch bar} = (3.85 + .14) \times 1.06 = 4.23 \text{ \$/ft}$$

$$\text{nut + plate} = \longrightarrow 34.8 \text{ \$/each}$$

40' long		
tieback labor 6 mh/tie=	1.5x192.5=	288.75
cement=	10 bagsx10 =	100

for tieback length of	43	ft
labor for this length vs 40'=	0.075	21.66

bar length for tieback =	43	ft
tail =	3	ft

$$\text{total tie length} = \text{=====} 46 \text{ ft}$$

for 1" bar=	0.00	pick one of them
for 1 1/4" bar=	194.55	pick one of them
Nut and Plate+tax =	36.89	\\$/each
test / stress load=	100.00	\\$/each
Tieback Equip. and FOG=	225.00	\\$/each
Grout Equip. and Air comp.=	60.00	\\$/each
	=====	
tieback cost=	1,027.00	\\$/each
no. of tie backs=		

Beam at 8.5' c-c

driving cost=	10	\\$/ft
H beam length =	32.5	ft
H beam =	0.52	Lft/pound
for 53 beam=	895.70	pick one of them
for 42 beam=	709.80	pick one of them

Install lagging

top 8' =	4.1	\\$/sft
below 8' =	6.76	\\$/sft
Production rate=	1,200.00	sft/day
Job sft=	12,560.00	sft
How many days=	11.00	day
OTA= 6x100xdays=	6,600.00	\\$/job

Waler 10'	550	\\$/each	or beam penetration
Waler 20'	650	\\$/each	or beam penetration

beam penetration 10" beam=	100.00	pick one of them	or use Waler
or 12" beam=	125.00	pick one of them	or use Waler

Mob and DeMob=	3,500	\\$ more than soilnailig out of town
	1,000	\\$ more than soilnailig in town

CASE 1
SHEETING COST

	<u>rate \$</u>	<u>per</u>	<u>unit</u>	<u>rate x unit</u>	<u>\$/sft</u>
driving in earth	10.00	ft	2,084.50	20,845.00	1.66
steel cost beam	0.52	lb	110,478.5	57,448.82	4.57
wale 10' long	450.00	ea.	30	13,500.00	1.07
wale 20' long	550.00	ea.	1	550.00	0.04
wale 10' uninstall	200.00	ea.	30	6,000.00	0.48
wale 20' uninstall	250.00	ea.	1	250.00	0.02
tieback cost	1,027.00	ea.	63	64,701.00	5.15
pent. throu. 10" beam	100.00	ea.	0	0.00	0.00
pent. throu. 12" beam	125.00	ea.	0	0.00	0.00
wood lagging down to 1 st tie	4.10	sft	5,365.00	21,996.50	1.75
wood lagging below	6.76	sft	7,195.00	48,638.20	3.87
mob/demob + shop dwg	1.00	ea.	10,000.00	10,000.00	0.80
OTA	100.00	M / DAY	0	0.00	0.00
set pile in hole	5.00	\$/ft	2,084.50	10,422.50	0.83

=====

254,352.02 20.25

total sheeted area = 12,560.00 sft

cost = 20.25 \$/sft

CASE 2
SHEETING COST

	<u>rate \$</u>	<u>per</u>	<u>unit</u>	<u>rate x unit</u>	<u>\$/sft</u>
driving in earth	10.00	ft	2,856.00	28,560.00	1.28
steel cost beam	0.52	lb	151,368.0	78,711.36	3.51
wale 10' long	450.00	ea.	80	36,000.00	1.61
wale 20' long	550.00	ea.	2	1,100.00	0.05
wale 10' uninstall	200.00	ea.	80	16,000.00	0.71
wale 20' uninstall	250.00	ea.	2	500.00	0.02
tieback cost	1,200.00	ea.	164	196,800.00	8.79
pent. throu. 10" beam	100.00	ea.	0	0.00	0.00
pent. throu. 12" beam	125.00	ea.	0	0.00	0.00
wood lagging down to 1 st tie	4.10	sft	5,224.0	21,418.40	0.96
wood lagging below	6.76	sft	17,176.0	116,109.76	5.18
mob/demob + shop dwg	1.00	ea.	10,000.00	10,000.00	0.45
OTA	100.00	M / DAY	0	0.00	0.00
set pile in hole	5.00	\$/ft	2,856.00	14,280.00	0.64

=====

519,479.52	23.19
------------	-------

=====

area =	22,400	sft	cost =	23.19	\$/sft
--------	--------	-----	--------	-------	--------

CASE 3
SHEETING COST

	<u>rate \$</u>	<u>per</u>	<u>unit</u>	<u>rate x unit</u>	<u>\$/sft</u>
driving in earth	10.00	ft	324.00	3,240.00	1.73
steel cost beam	0.52	lb	13,608.0	7,076.16	3.77
wale 10' long	450.00	ea.	7	3,150.00	1.68
wale 20' long	550.00	ea.	0	0.00	0.00
wale 10' uninstall	200.00	ea.	7	1,400.00	0.75
wale 20' uninstall	250.00	ea.	0	0.00	0.00
tieback cost	916.00	ea.	7	6,412.00	3.42
pent. throu. 10" beam	100.00	ea.	0	0.00	0.00
pent. throu. 12" beam	125.00	ea.	0	0.00	0.00
wood lagging down to 1 st tie	4.10	sft	875	3,587.50	1.91
wood lagging below	6.76	sft	1,000	6,760.00	3.61
mob/demob + shop dwg	1.00	ea.	15,000	15,000.00	8.00
OTA	100.00	M / DAY	12	1,200.00	0.64
set pile in hole	5.00	\$/ft	324.00	1,620.00	0.86
				=====	=====
			total cost=	49,445.66	26.37
area =	1,875	sft	COST =	26.37	\$/sft

CASE 4
SHEETING COST

	<u>rate \$</u>	<u>per</u>	<u>unit</u>	<u>rate x unit</u>	<u>\$/sft</u>
driving in earth	10.00	ft	1,169.00	11,690.00	1.31
steel cost beam	0.52	lb	49,098.0	25,530.96	2.86
wale 10' long	450.00	ea.	2	900.00	0.10
wale 20' long	550.00	ea.	25	13,750.00	1.54
wale 10' uninstall	200.00	ea.	2	400.00	0.04
wale 20' uninstall	250.00	ea.	25	6,250.00	0.70
tieback cost	1,140.00	ea.	53	60,420.00	6.76
pent. throu. 10" beam	100.00	ea.	0	0.00	0.00
pent. throu. 12" beam	125.00	ea.	0	0.00	0.00
wood lagging down to 1 st tie	4.10	sft	2,640.0	10,824.00	1.21
wood lagging below	6.76	sft	6,300.0	42,588.00	4.76
mob/demob + shop dwg	1.00	ea.	10,000.00	10,000.00	1.12
OTA	100.00	M / DAY	0	0.00	0.00
set pile in hole	5.00	\$/ft	1,169.00	5,845.00	0.65
				=====	=====
				188,197.96	21.05
area =	8,940	sft	cost=	21.05	\$/sft

CASE 5
SHEETING COST

	<u>rate \$</u>	<u>per</u>	<u>unit</u>	<u>rate x unit</u>	<u>\$/sft</u>
driving in earth	10.00	ft	2,667.50	26,675.00	1.10
steel cost beam	0.52	lb	125,895.0	65,465.40	2.69
wale 10' long	450.00	ea.	3	1,350.00	0.06
wale 20' long	550.00	ea.	30	16,500.00	0.68
wale 10' uninstall	200.00	ea.	3	600.00	0.02
wale 20' uninstall	250.00	ea.	30	7,500.00	0.31
tieback cost	1,100.00	ea.	190	209,000.00	8.59
pent. throu. 10" beam	100.00	ea.	0	0.00	0.00
pent. throu. 12" beam	125.00	ea.	130	16,250.00	0.67
wood lagging down to 1 st tie	4.10	sft	6,309.0	25,866.90	1.06
wood lagging below	6.76	sft	18,009.0	121,740.84	5.01
mob/demob + shop dwg	1.00	ea.	10,000.00	10,000.00	0.41
OTA	100.00	M / DAY	0	0.00	0.00
set pile in hole	5.00	\$/ft	2,667.50	13,337.50	0.55

=====

514,285.64 21.15

area = 24,318 sft cost = 21.15 \$/sft

CASE 6
SHEETING COST

	<u>rate \$</u>	<u>per</u>	<u>unit</u>	<u>rate x unit</u>	<u>\$/sft</u>
driving in earth	10.00	ft	3,467.50	34,675.00	1.39
steel cost beam	0.52	lb	183,777.5	95,564.30	3.83
wale 10' long	450.00	ea.	0	0.00	0.00
wale 20' long	550.00	ea.	20	11,000.00	0.44
wale 10' uninstall	200.00	ea.	0	0.00	0.00
wale 20' uninstall	250.00	ea.	20	5,000.00	0.20
tieback cost	1,100.00	ea.	190	209,000.00	8.38
pent. throu. 10" beam	100.00	ea.	0	0.00	0.00
pent. throu. 12" beam	125.00	ea.	148	18,500.00	0.74
wood lagging down to 1 st tie	4.10	sft	7,465.0	30,606.50	1.23
wood lagging below	6.76	sft	17,468.0	118,083.68	4.74
mob/demob + shop dwg	1.00	ea.	15,000.00	15,000.00	0.60
OTA	100.00	M / DAY	126.00	12,600.00	0.51
set pile in hole	5.00	\$/ft	3,467.50	17,337.50	0.70
				=====	=====
				567,366.98	22.76

area = 24,933 SFT cost 22.76 \$/sft

CASE 7
SHEETING COST

	<u>rate \$</u>	<u>per</u>	<u>unit</u>	<u>rate x unit</u>	<u>\$/sft</u>
driving in earth	10.00	ft	535.00	5,350.00	1.77
steel cost beam	0.52	lb	22,470.0	11,684.40	3.87
wale 10' long	450.00	ea.	4	1,800.00	0.60
wale 20' long	550.00	ea.	0	0.00	0.00
wale 10' uninstall	200.00	ea.	4	800.00	0.26
wale 20' uninstall	250.00	ea.	0	0.00	0.00
tieback cost	1,027.00	ea.	14	14,378.00	4.76
pent. throu. 10" beam	100.00	ea.	10	1,000.00	0.33
pent. throu. 12" beam	125.00	ea.	0	0.00	0.00
wood lagging down to 1 st tie	4.10	sft	1,329.0	5,448.90	1.80
wood lagging below	6.76	sft	1,691.0	11,431.16	3.79
mob/demob + shop dwg	1.00	ea.	15,000.00	15,000.00	4.97
OTA	100.00	M / DAY	0	0.00	0.00
set pile in hole	5.00	\$/ft	535.00	2,675.00	0.89
				=====	=====
				69,567.46	23.04

area = 3,020 SFT cost= 23.04 \$/sft

CASE 8
SHEETING COST

	<u>rate \$</u>	<u>per</u>	<u>unit</u>	<u>rate x unit</u>	<u>\$/sft</u>
driving in earth	10.00	ft	1,654.00	16,540.00	1.25
steel cost beam	0.52	lb	69,468.0	36,123.36	2.73
wale 10' long	450.00	ea.	20	9,000.00	0.68
wale 20' long	550.00	ea.	26	14,300.00	1.08
wale 10' uninstall	200.00	ea.	20	4,000.00	0.30
wale 20' uninstall	250.00	ea.	26	6,500.00	0.49
tieback cost	1,150.00	ea.	86	98,900.00	7.47
pent. throu. 10" beam	100.00	ea.	0	0.00	0.00
pent. throu. 12" beam	125.00	ea.	0	0.00	0.00
wood lagging down to 1 st tie	4.10	sft	4,225.0	17,322.50	1.31
wood lagging below	6.76	sft	9,009.0	60,900.84	4.60
mob/demob + shop dwg	1.00	ea.	15,000.00	15,000.00	1.13
OTA	100.00	M / DAY	72	7,200.00	0.54
set pile in hole	5.00	\$/ft	1,654.00	8,270.00	0.62
				=====	=====
				294,056.70	22.22

area = 13,234 SFT cost= 22.22 \$/sft

CASE 9
SHEETING COST

	<u>rate \$</u>	<u>per</u>	<u>unit</u>	<u>rate x unit</u>	<u>\$/sft</u>
driving in earth	10.00	ft	1,040.00	10,400.00	1.22
steel cost beam	0.52	lb	55,120.0	28,662.40	3.37
wale 10' long	450.00	ea.	0	0.00	0.00
wale 20' long	550.00	ea.	0	0.00	0.00
wale 10' uninstall	200.00	ea.	0	0.00	0.00
wale 20' uninstall	250.00	ea.	0	0.00	0.00
tieback cost	1,150.00	ea.	57	65,550.00	7.70
pent. throu. 10" beam	100.00	ea.	57	5,700.00	0.67
pent. throu. 12" beam	125.00	ea.	0	0.00	0.00
wood lagging down to 1 st tie	4.10	sft	3,267.0	13,394.70	1.57
wood lagging below	6.76	sft	5,247.0	35,469.72	4.17
mob/demob + shop dwg	1.00	ea.	15,000.00	15,000.00	1.76
OTA	100.00	M / DAY	0	0.00	0.00
set pile in hole	5.00	\$/ft	1,040.00	5,200.00	0.61
				=====	=====
				179,376.82	21.07

area = 8,514 SFT cost= 21.07 \$/sft

CASE 10
SHEETING COST

	<u>rate \$</u>	<u>per</u>	<u>unit</u>	<u>rate x unit</u>	<u>\$/sft</u>
driving in earth	10.00	ft	1,557.00	15,570.00	1.30
steel cost beam	0.52	lb	82,521.0	42,910.92	3.59
wale 10' long	450.00	ea.	46	20,700.00	1.73
wale 20' long	550.00	ea.	0	0.00	0.00
wale 10' uninstall	200.00	ea.	46	9,200.00	0.77
wale 20' uninstall	250.00	ea.	0	0.00	0.00
tieback cost	1,150.00	ea.	46	52,900.00	4.43
pent. throu. 10" beam	100.00	ea.	0	0.00	0.00
pent. throu. 12" beam	125.00	ea.	0	0.00	0.00
wood lagging down to 1 st tie	4.10	sft	3,578.0	14,669.80	1.23
wood lagging below	6.76	sft	8,362.0	56,527.12	4.73
mob/demob + shop dwg	1.00	ea.	15,000.00	15,000.00	1.26
OTA	100.00	M / DAY	0	0.00	0.00
set pile in hole	5.00	\$/ft	1,557.00	7,785.00	0.65
				=====	=====
				235,262.84	19.70
area =	11,940	SFT	cost	19.70	\$/sft

CASE 11
SHEETING COST

	<u>rate \$</u>	<u>per</u>	<u>unit</u>	<u>rate x unit</u>	<u>\$/sft</u>
driving in earth	10.00	ft	1,008.00	10,080.00	1.62
steel cost beam	0.52	lb	42,336.0	22,014.72	3.54
wale 10' long	450.00	ea.	25	11,250.00	1.81
wale 20' long	550.00	ea.	0	0.00	0.00
wale 10' uninstall	200.00	ea.	25	5,000.00	0.80
wale 20' uninstall	250.00	ea.	0	0.00	0.00
tieback cost	1,000.00	ea.	35	35,000.00	5.63
pent. throu. 10" beam	100.00	ea.	0	0.00	0.00
pent. throu. 12" beam	125.00	ea.	0	0.00	0.00
wood lagging down to 1 st tie	4.10	sft	2,741.0	11,238.10	1.81
wood lagging below	6.76	sft	3,477.0	23,504.52	3.78
mob/demob + shop dwg	1.00	ea.	15,000.00	15,000.00	2.41
OTA	100.00	M / DAY	0	0.00	0.00
set pile in hole	5.00	\$/ft	1,008.00	5,040.00	0.81
				=====	=====
				138,127.34	22.21

area = 6,218 SFT cost= 22.21 \$/sft

CASE 12
SHEETING COST

	<u>rate \$</u>	<u>per</u>	<u>unit</u>	<u>rate x unit</u>	<u>\$/sft</u>
driving in earth	10.00	ft	2,232.50	22,325.00	1.52
steel cost beam	0.52	lb	117,717.5	61,213.10	4.16
wale 10' long	450.00	ea.	45	20,250.00	1.38
wale 20' long	550.00	ea.	0	0.00	0.00
wale 10' uninstall	200.00	ea.	45	9,000.00	0.61
wale 20' uninstall	250.00	ea.	0	0.00	0.00
tieback cost	1,050.00	ea.	90	94,500.00	6.42
pent. throu. 10" beam	100.00	ea.	0	0.00	0.00
pent. throu. 12" beam	125.00	ea.	0	0.00	0.00
wood lagging down to 1 st tie	4.10	sft	5,754.0	23,591.40	1.60
wood lagging below	6.76	sft	8,956.0	60,542.56	4.12
mob/demob + shop dwg	1.00	ea.	15,000.00	15,000.00	1.02
OTA	100.00	M / DAY	78	7,800.00	0.53
set pile in hole	5.00	\$/ft	2,232.50	11,162.50	0.76
				=====	=====
				325,384.56	22.12

area = 14,710 SFT

cost 22.12 \$/sft

CASE 13
SHEETING COST

	<u>rate \$</u>	<u>per</u>	<u>unit</u>	<u>rate x unit</u>	<u>\$/sft</u>
driving in earth	10.00	ft	500.00	5,000.00	1.81
steel cost beam	0.52	lb	21,000.0	10,920.00	3.95
wale 10' long	450.00	ea.	12	5,400.00	1.95
wale 20' long	550.00	ea.	0	0.00	0.00
wale 10' uninstall	200.00	ea.	12	2,400.00	0.87
wale 20' uninstall	250.00	ea.	0	0.00	0.00
tieback cost	1,000.00	ea.	12	12,000.00	4.34
pent. throu. 10" beam	100.00	ea.	0	0.00	0.00
pent. throu. 12" beam	125.00	ea.	0	0.00	0.00
wood lagging down to 1 st tie	4.10	sft	1,258.0	5,157.80	1.87
wood lagging below	6.76	sft	1,507.0	10,187.32	3.68
mob/demob + shop dwg	1.00	ea.	15,000.00	15,000.00	5.42
OTA	100.00	M / DAY	0	0.00	0.00
set pile in hole	5.00	\$/ft	500.00	2,500.00	0.90
				=====	=====
				68,565.12	24.80

area = 2,765 SFT

cost= 24.80 \$/sft

CASE 14
SHEETING COST

	<u>rate \$</u>	<u>per</u>	<u>unit</u>	<u>rate x unit</u>	<u>\$/sft</u>
driving in earth	10.00	ft	726.50	7,265.00	1.77
steel cost beam	0.52	lb	30,513.0	15,866.76	3.87
wale 10' long	450.00	ea.	13	5,850.00	1.43
wale 20' long	550.00	ea.	0	0.00	0.00
wale 10' uninstall	200.00	ea.	13	2,600.00	0.63
wale 20' uninstall	250.00	ea.	0	0.00	0.00
tieback cost	1,050.00	ea.	21	22,050.00	5.38
pent. throu. 10" beam	100.00	ea.	0	0.00	0.00
pent. throu. 12" beam	125.00	ea.	0	0.00	0.00
wood lagging down to 1 st tie	4.10	sft	1,742.0	7,142.20	1.74
wood lagging below	6.76	sft	2,358.0	15,940.08	3.89
mob/demob + shop dwg	1.00	ea.	15,000.00	15,000.00	3.66
OTA	100.00	M / DAY	0	0.00	0.00
set pile in hole	5.00	\$/ft	726.50	3,632.50	0.89

=====
95,346.54 23.26

area = 4,100 SFT cost= 23.26 \$/sft

CASE 15
SHEETING COST

	<u>rate \$</u>	<u>per</u>	<u>unit</u>	<u>rate x unit</u>	<u>\$/sft</u>
driving in earth	10.00	ft	8,888.00	88,880.00	1.24
steel cost beam	0.52	lb	471,064.0	244,953.28	3.41
wale 10' long	450.00	ea.	0	0.00	0.00
wale 20' long	550.00	ea.	0	0.00	0.00
wale 10' uninstall	200.00	ea.	0	0.00	0.00
wale 20' uninstall	250.00	ea.	0	0.00	0.00
tieback cost	1,100.00	ea.	808	888,800.00	12.38
pent. throu. 10" beam	100.00	ea.	0	0.00	0.00
pent. throu. 12" beam	125.00	ea.	808	101,000.00	1.41
wood lagging down to 1 st tie	4.10	sft	13,000.0	53,300.00	0.74
wood lagging below	6.76	sft	58,820.0	397,623.20	5.54
mob/demob + shop dwg	1.00	ea.	15,000.00	15,000.00	0.21
OTA	100.00	M / DAY	0	0.00	0.00
set pile in hole	5.00	\$/ft	8,888.00	44,440.00	0.62

area =

71,820

SFT

cost=

1,833,996.48

25.54

\$/sft

Appendix F: Cost for Soil Nailing System

Case 1 Soil Nailing Cost

1- Drill hole
drill rate 6 each x 20' each/hr 6 nail length 20 used length 17.5

Equip. 24000/160 = 150
fog / hr = 40
drill string wear = 15
labor 42.17+34.37 = 80

=====

285	→	total	total\$/each	ratio	total\$/each
300		50	0.875	43.75	

spacing = 25 sft

1- Drill hole total/each / spacing = 1.75 \$/sft

2- Furnish and install nails rate/hr = 4

Equip
grout mixer w/360 air = 6400
small excavator = 3600

=====

10,000.00	/ 160 =	62.50	/ rate / hr =	15.63
-----------	---------	-------	---------------	-------

Labor 4 men crew

50+42+69 = 161 → 175 / rate / hr = 43.75

F O G = 35 → 35 / rate / hr = 8.75

Matl - Bar = 2.75 → 3+CONT. 0.15 3.15 x nail length = 55.125

grout / ft = 0.83 → 0.83 x nail length = 14.525

=====

sub. tot. = 137.78

Plate and nuts → 10

=====

147.78	/ spacing =	5.91	\$/sft
--------	-------------	------	--------

3- Place 6x6 mesh w/ miradrain

mesh matl = 20.45x1.06 / 8x15 = 0.18 \$/sft

miradrain @ 4' c-c = 210/150 = 1.4 / 4 = 0.35 \$/sft

chairs for mesh @ 4sft/chair = 0.1 \$/sft

labor to place 4 @ 34.37 = 140/(6x4x15) 0.39 \$/sft

=====

1.02	→	1.1	\$/sft
------	---	-----	--------

contin. walers @ dbl #4 and 4'

nail spacing = 5 ft
nail spacing = 0.416 x 0.5 mat and 0.5 lab → 0.54 \$/sft

4- Shotcrete

82.5x1.06+75/cy sub/place

162.45/cy @ 4" = 81 sft=2/sft* 1.1 = 2.2 /sft → 1.72 \$/sft

=====

11.02 \$/sft

5- over-all supervision @ 750 sft/day 75x8/750 = 0.8

0.8 \$/sft

6- OTA 6 men x 100/day /750 = 0.8

0 go to zero if job is in town

=====

11.82

7- MOB AND DEMOB+ SHOP DRAWING (2500+2500+5000)/27000

0.37

=====

12.19 \$/sft

shaving excavation face 0.25 \$/sft

=====

12.44	\$/sft	sft	total cost
		12,560	156,259

Case 2 Soil Nailing Cost

1- Drill hole		<u>each/hr</u>	<u>nail length</u>	<u>used length</u>				
drill rate	6 each x 20'	6	20	25				
Equip. 24000/160 =		150						
fog / hr =		40						
drill string wear =		15						
labor 42.17+34.37=		80						
=====								
285		→	<u>total</u>	<u>total\$/each</u>	<u>ratio</u>	<u>total\$/each</u>		
spacing = 25		sft	300	50	1.25	62.5		
1- Drill hole						total/each / spacing=	2.50 \$/sft	
2- Furnish and install nails		<u>rate/ hr =</u>	4					
Equip								
grout mixer w/360 air=		6400						
small excavator=		3600						
=====								
10,000.00		/ 160 =	62.50	/ rate / hr =	15.63			
Labor 4 men crew								
50+42+69=		161	→	175	/ rate / hr =	43.75		
F O G =		35		35	/ rate / hr =	8.75		
Matl - Bar = 2.75		→	3+CONT. 0.15	3.15	x nail length	78.75		
grout / ft =		0.83		0.83	x nail length	20.75		
				=====				
				sub. tot. =		167.63		
Plate and nuts						10		
						=====		
						177.63	/ spacing= 7.11 \$/sft	
3- Place 6x6 mesh w/ miradrain								
mesh mat= 20.45x1.06 / 8x15 =				0.18	\$/sft			
miradrain @ 4' c-c = 210/150 =		1.4	/ 4 =	0.35	\$/sft			
chairs for mesh @ 4sft/chair =				0.1	\$/sft			
labor to place 4 @ 34.37 = 140/(6x4x15)				0.39	\$/sft			
				=====				
				1.02	→	1.1	\$/sft	
contin. walers @ dbl #4 and 4'								
nail spacing= 5		ft						
nail spacing= 0.416		x 0.5 mat and 0.5 lab	→	0.54	\$/sft			
4- Shotcrete								
82.5x1.06+75/cy sub/place								
162.45/cy @ 4" = 81 sft=2/sft* 1.1= 2.2 /sft				→	1.72	\$/sft		
				=====		12.97	\$/sft	
5- over-all supervision @ 750 sft/day 75x8/750= 0.8				→	0.8	\$/sft		
6- OTA 6 men x 100/day /750: 0.8				→	0	go to zero if job is in town		
				=====		13.77		
7- MOB AND DEMOB+ SHOP DRAWING (2500+2500+5000)/27000				→	0.37			
				=====		14.14	\$/sft	
shaving excavation face						0.25	\$/sft	
				=====		14.39	\$/sft	
						<u>sft</u>	<u>total cost</u>	
						22,400	322,224	

Case 3 Soil Nailing Cost

1- Drill hole									
drill rate	6 each x 20'	each/hr 6	nail length 20	used length 11					
Equip.	24000/160 =	150							
fog / hr =	40								
drill string wear =	15								
labor 42.17+34.37=	80								
	=====								
	285	→	total	total\$/each	ratio	total\$/each			
	300		50	0.55	27.5				
spacing =	30	sft							
1- Drill hole					total/each / spacing=	0.92	\$/sft		
2- Furnish and install nails									
Equip	rate/ hr =	4							
grout mixer w/360 air=	6400								
small excavator=	3600								
	=====								
	10,000.00	/ 160 =	62.50	/ rate / hr =	15.63				
Labor 4 men crew									
	50+42+69=	161	→	175	/ rate / hr =	43.75			
FOG =	35			35	/ rate / hr =	8.75			
Matl - Bar= 2.75			3+CONT. 0.15	3.15	x nail length	34.65			
grout / ft =	0.83			0.83	x nail length	9.13			
					=====				
					sub. tot. =	111.91			
Plate and nuts						10			
					=====				
						121.91	/ spacing=	4.06	\$/sft
3- Place 6x6 mesh w/ miradrain									
mesh matl=	20.45x1.06 / 8x15	=	0.18	\$/sft					
miradrain @ 4' c-c =	210/150 =	1.4	/ 4 =	0.35	\$/sft				
chairs for mesh @ 4sft/chair =			0.1	\$/sft					
labor to place 4 @ 34.37 =	140/(6x4x15)	0.39	\$/sft						
			=====						
			1.02	→	1.1	\$/sft			
contin. walers @ dbl #4 and 4'									
nail spacing=	5	ft							
nail spacing=	0.416	x 0.5 mat and 0.5 lab	→	0.54	\$/sft				
4- Shotcrete									
82.5x1.06+75/cy sub/place									
162.45/cy @ 4" = 81 sft=2/sft* 1.1= 2.2 /sft			→	1.72	\$/sft				
					=====				
					8.34	\$/sft			
5- over-all supervision @ 750 sft/day 75x8/750= 0.8									
			→	1.00	→	1	\$/sft		
6- OTA 6 men x 100/day /750 : 0.8									
					→	0.8	go to zero if job is in town		
					=====				
					10.14				
7- MOB AND DEMOB+ SHOP DRAWING (2500+2500+10,000)/2' 0.56									
					→	0.56			
					=====				
					10.70	\$/sft			
shaving excavation face					0.25	\$/sft			
					=====				
					10.95	\$/sft	<u>sft</u>	<u>total cost</u>	
							1,875	20,532	

Case 4 Part A Soil Nailing Cost

1- Drill hole		each/hr	nail length	used length				
drill rate	6 each x 20'	6	20	23				
Equip. 24000/160 =		150						
fog / hr =		40						
drill string wear =		15						
labor 42.17+34.37 =		80						
		=====						
		285			total	total\$/each	ratio	total\$/each
spacing =	25	sft			300	50	1.15	57.5
1- Drill hole							total/each / spacing =	2.30 \$/sft
2- Furnish and install nails		rate/ hr =	4					
Equip								
grout mixer w/360 air=		6400						
small excavator=		3600						
		=====						
		10,000.00	/ 160 =	62.50	/ rate / hr =	15.63		
Labor 4 men crew								
50+42+69=	161			175	/ rate / hr =	43.75		
F O G =	35			35	/ rate / hr =	8.75		
Matl - Bar= 2.75				3+CONT. 0.15	x nail length	72.45		
grout / ft =	0.83			0.83	x nail length	19.09		
					=====			
					sub. tot. =	159.67		
Plate and nuts						10		
					=====			
						169.67	/ spacing =	6.79 \$/sft
3- Place 6x6 mesh w/ miradrain								
mesh matl=	20.45x1.06 / 8x15	=		0.18	\$/sft			
miradrain @ 4' c-c =	210/150 =	1.4	/ 4 =	0.35	\$/sft			
chairs for mesh @ 4sft/chair =				0.1	\$/sft			
labor to place 4 @ 34.37 =	140/(6x4x15)			0.39	\$/sft			
				=====				
				1.02				1.1 \$/sft
contin. walers @ dbl #4 and 4'								
nail spacing=	5	ft						
nail spacing=	0.416	x 0.5 mat and 0.5 lab						0.54 \$/sft
4- Shotcrete								
82.5x1.06+75/cy sub/place								
162.45/cy @ 4" = 81 sft=2/sft* 1.1= 2.2 /sft								1.72 \$/sft
						=====		
						12.45	\$/sft	
5- over-all supervision @ 750 sft/day 75x8/750= 0.8								0.8 \$/sft
6- OTA 6 men x 100/day /750 : 0.8								0 go to zero if job is in town
						=====		
						13.25		
7- MOB AND DEMOB+ SHOP DRAWING (2500+2500+5000)/27000								0.37
						=====		
						13.62	\$/sft	
shaving excavation face						0.25	\$/sft	
						=====		
						13.87	\$/sft	
							<u>sft</u>	<u>total cost</u>
							5,657	78,443

Case 4 Part B Soil Nailing Cost

1- Drill hole									
drill rate	6 each x 20'	each/hr 6	nail length 20	used length 27.5					
	Equip. 24000/160 =	150							
	fog / hr =	40							
	drill string wear =	15							
	labor 42.17+34.37=	80							
		=====							
		285		total	total\$/each	ratio	total\$/each		
	spacing =	25	sft	300	50	1.375	68.75		
1- Drill hole								total/each / spacing=	2.75 \$/sft
2- Furnish and install nails		rate/ hr =	4						
Equip									
grout mixer w/360 air=		6400							
small excavator=		3600							
		=====							
		10,000.00	/ 160 =	62.50	/ rate / hr =	15.63			
Labor 4 men crew									
	50+42+69=	161		175	/ rate / hr =	43.75			
FOG =	35			35	/ rate / hr =	8.75			
Matl - Bar= 2.75			3+CONT. 0.15	3.15	x nail length	86.625			
grout / ft =	0.83			0.83	x nail length	22.825			
					=====				
				sub. tot. =	177.58				
Plate and nuts					10				
					=====				
					187.58	/ spacing=	7.50		\$/sft
3- Place 6x6 mesh w/ miradrain									
	mesh mat= 20.45x1.06 / 8x15 =			0.18					\$/sft
	miradrain @ 4' c-c = 210/150 =	1.4	/ 4 =	0.35					\$/sft
	chairs for mesh @ 4sft/chair =			0.1					\$/sft
	labor to place 4 @ 34.37 = 140/(6x4x15)			0.39					\$/sft
				=====					
				1.02					1.1 \$/sft
contin. walers @ dbl #4 and 4'									
	nail spacing=	5	ft						
	nail spacing=	0.416	x 0.5 mat and 0.5 lab						0.54 \$/sft
4- Shotcrete									
	82.5x1.06+75/cy sub/place								
	162.45/cy @ 4" = 81 sft=2/sft* 1.1= 2.2 /sft								1.72 \$/sft
									=====
									13.61 \$/sft
5- over-all supervision @ 750 sft/day 75x8/750= 0.8									0.8 \$/sft
6- OTA 6 men x 100/day /750 sft 0.8									0 go to zero if job is in town
									=====
									14.41
7- MOB AND DEMOB+ SHOP DRAWING (2500+2500+5000)/27000									0.37
									=====
									14.78 \$/sft
	shaving excavation face				0.25				\$/sft
					=====				
					15.03				\$/sft
									=====
									sft total cost
									3,283 49,353

1- Drill hole	<u>each/hr</u>	<u>nail length</u>	<u>used length</u>				
drill rate 6 each x 20'	6	20	27.5				
Equip. 24000/160 =	150						
fog / hr =	40						
drill string wear =	15						
labor 42.17+34.37=	80						
	=====						
	285	→	<u>total</u>	<u>total\$/each</u>	<u>ratio</u>	<u>total\$/each</u>	
spacing = 27.5 sft			300	50	1.375	68.75	
1- Drlll hole					total/each / spacing=	2.50	\$/sft

2- Furnish and install nails		rate/ hr =	4		
Equip					
grout mixer w/360 air=	6400				
small excavator=	3600				
	=====				
	10,000.00	/ 160 =	62.50	/ rate / hr =	15.63
Labor 4 men crew					
	50+42+69=	161	→	175	/ rate / hr = 43.75
FOG =	35			35	/ rate / hr = 8.75
Matl - Bar = 2.75		→	3+CONT. 0.15	3.15	x nail length 86.625
grout / ft =	0.83			0.83	x nail length 22.825
					=====
				sub. tot. =	177.58
Plate and nuts				→	10
					=====
					187.58 / spacing= 6.82 \$/sf

3- Place 6x6 mesh w/ miradrain

mesh mat=	20.45x1.06 / 8x15	=	0.18	\$/sft
miradrain @ 4' c-c =	210/150	=	1.4	/ 4 = 0.35 \$/sft
chairs for mesh @4sft/chair =			0.1	\$/sft
labor to place 4 @ 34.37 =	140/(6x4x15)		0.39	\$/sft
			1.02	1.1 \$/sft

contin. walers @ dbl #4 and 4'

nail spacing=	5	ft		
nail spacing=	0.416	x 0.5 mat and 0.5 lab		0.54 \$/sft

4- Shotcrete
82.5x1.06+75/cy sub/place
162.45/cy @ 4" = 81 sft=2/sft* 1.1= 2.2 /sft → 1.72 \$/sft
===== 12.68 \$/sft

5- over-all supervision @ 750 sft/day $75 \times 8 / 750 = 0.8$ 0.8 \$/sft

[illegible]

7- MOB AND DEMOB+ SHOP DRAWING (2500+2500+5000)/27000	0.37			
	=====			
	13.85	\$/sft		
shaving excavation face	0.25	\$/sft		
	=====			
	14.10	\$/sft	<u>sft</u>	<u>total cost</u>
			14,270	201,220

Case 5 Part B Soil Nailing Cost

1- Drill hole	drill rate	6 each x 20'	each/hr 6	nail length 20	used length 17.5				
		Equip. 24000/160 = fog / hr = drill string wear = labor 42.17+34.37=	150 40 15 80						
		=====	285		<u>total</u> 300	<u>total\$/each</u> 50	<u>ratio</u> 0.875	<u>total\$/each</u> 43.75	
	spacing =	27.5	sft						
1- Drill hole							total/each / spacing=	1.59	\$/sft
2- Furnish and install nails	rate/ hr =	4							
	Equip								
	grout mixer w/360 air=	6400							
	small excavator=	3600							
		=====							
Labor 4 men crew		10,000.00	/ 160 =	62.50	/ rate / hr =	15.63			
F O G =	50+42+69=	161	→	175	/ rate / hr =	43.75			
Matl - Bar= 2.75	35		→	35	/ rate / hr =	8.75			
grout / ft =	0.83		→	3+CONT. 0.15	x nail length	55.125			
				0.83	x nail length	14.525			
					=====				
					sub. tot. =	137.78			
Plate and nuts					→	10			
					=====				
						147.78	/ spacing=	5.37	\$/sft
3- Place 6x6 mesh w/ miradrain									
	mesh mat=	20.45x1.06 / 8x15	=	0.18	\$/sft				
	miradrain @ 4' c-c =	210/150 =	1.4	/ 4 =	0.35	\$/sft			
	chairs for mesh @ 4sft/chair =			0.1	\$/sft				
	labor to place 4 @ 34.37 =	140/(6x4x15)		0.39	\$/sft				
				=====					
				1.02	→	1.1	\$/sft		
contin. walers @ dbl #4 and 4'									
	nail spacing=	5	ft.						
	nail spacing=	0.416	x 0.5 mat and 0.5 lab	→	0.54	\$/sft			
4- Shotcrete									
	82.5x1.06+75/cy sub/place								
	162.45/cy @ 4" = 81 sft=2/sft* 1.1= 2.2 /sft			→	1.72	\$/sft			
					=====				
					10.32	\$/sft			
5- over-all supervision @ 750 sft/day 75x8/750= 0.8				→	0.8	\$/sft			
6- OTA 6 men x 100/day /750 sft 0.8				→	0	go to zero if job is in town			
					=====				
					11.12				
7- MOB AND DEMOB+ SHOP DRAWING (2500+2500+5000)/27000				→	0.37				
					=====				
					11.49	\$/sft			
	shaving excavation face	0.25	\$/sft						
					=====				
					11.74	\$/sft			
							<u>sft</u>	<u>total cost</u>	
							10,048	118,009	

Case 6 Part A Soil Nailing Cost

1- Drill hole		each/hr	nail length	used length				
drill rate	6 each x 20'	6	20	27.5				
Equip. 24000/160 =		150						
fog / hr =		40						
drill string wear =		15						
labor 42.17+34.37=		80						
		=====			total	total\$/each	ratio	total\$/each
		285			300	50	1.375	68.75
spacing =	25	sft						
1- Drill hole							total/each / spacing=	2.75 \$/sft
2- Furnish and install nails		rate/ hr =	4					
Equip								
grout mixer w/360 air=		6400						
small excavator=		3600						
		=====						
		10,000.00	/ 160 =	62.50	/ rate / hr =	15.63		
Labor 4 men crew								
	50+42+69=	161		175	/ rate / hr =	43.75		
F O G =	35			35	/ rate / hr =	8.75		
Matl - Bar= 2.75			3+CONT. 0.15	3.15	x nail length	86.625		
grout / ft =	0.83			0.83	x nail length	22.825		
					sub. tot. =	177.58		
Plate and nuts						10		
					=====	187.58	/ spacing=	7.50 \$/sft
3- Place 6x6 mesh w/ miradrain								
mesh mat= 20.45x1.06 / 8x15 =					0.18	\$/sft		
miradrain @ 4' c-c =	210/150 =	1.4	/ 4 =		0.35	\$/sft		
chairs for mesh @ 4sft/chair =					0.1	\$/sft		
labor to place 4 @ 34.37 =	140/(6x4x15)				0.39	\$/sft		
					=====	1.02		
							1.1	\$/sft
contin. walers @ dbl #4 and 4'								
nail spacing=	5	ft						
nail spacing=	0.416	x 0.5 mat and 0.5 lab					0.54	\$/sft
4- Shotcrete								
82.5x1.06+75/cy sub/place								
	162.45/cy @ 4" = 81 sft=2/sft* 1.1= 2.2 /sft				1.72	\$/sft		
					=====	13.61	\$/sft	
5- over-all supervision @ 750 sft/day 75x8/750= 0.8					0.8	\$/sft		
6- OTA 6 men x 100/day /750: 0.8					0.8	go to zero if job is in town		
					=====	15.21		
7- MOB AND DEMOB+ SHOP DRAWING (2500+2500+10,000)/2 0.56					0.56			
					=====	15.77	\$/sft	
shaving excavation face					0.25	\$/sft		
					=====	16.02	\$/sft	
							sft	total cost
							11,382	182,374

Case 6 Part B Soil Nailing Cost

[illegible]

Case 6 Part C Soil Nailing Cost

[illegible]

[illegible]

Case 7 Part B Soil Nailing Cost

1- Drill hole		each/hr	nail length	used length				
drill rate	6 each x 20'	6	20	17				
Equip.	24000/160 =	150						
fog / hr =	40							
drill string wear =	15							
labor	42.17+34.37=	80						
	=====				total	total\$/each	ratio	total\$/each
	285				300	50	0.85	42.5
spacing =	33	sft						
1- Drill hole							total/each / spacing=	1.29 \$/sft
2- Furnish and install nail rate/ hr = 4								
Equip								
grout mixer	6400							
small excav	3600							
	=====							
	10,000.00	/ 160 =	62.50	/ rate / hr =	15.63			
Labor 4 men crew								
50+42+69=	161		175	/ rate / hr =	43.75			
F O G =	35		35	/ rate / hr =	8.75			
Matl - Bar=	2.75		3.15	x nail length	53.55			
grout / ft =	0.83		0.83	x nail length	14.11			
					=====			
				sub. tot. =	135.79			
Plate and nuts					10			
					=====			
					145.79	/ spacing=	4.42	\$/sft
3- Place 6x6 mesh w/ miradrain								
mesh matl=	20.45x1.06 / 8x15 =		0.18	\$/sft				
miradrain @ 4' c-c =	210/150 = 1.4 / 4 =		0.35	\$/sft				
chairs for mesh @ 4sft/chair =			0.1	\$/sft				
labor to place 4 @ 34.37 = 140/(6x4x15)			0.39	\$/sft				
			=====					
			1.02				1.1	\$/sft
contin. walers @ dbl #4 and 4'								
nail spacing=	5	ft						
nail spacing=	0.416	x 0.5 mat and 0.5 lab					0.54	\$/sft
4- Shotcrete								
82.5x1.06+75/cy sub/place								
162.45/cy @ 4" = 81 sft=2/sft* 1.1= 2.2 /sft							1.72	\$/sft
							=====	
							9.07	\$/sft
5- over-all supervision @ 750 sft/day 75x8/750= 0.8							0.8	\$/sft
6- OTA 6 men x 100/day /750 sft 0.8							0	go to zero if job is in town
							=====	
							9.87	
7- MOB AND DEMOB+ SHOP DRAWING (2500+2500+10,000). 0.56							0.56	
							=====	
							10.43	\$/sft
shaving excavation face							0.25	\$/sft
							=====	
							10.68	\$/sft
							sft	total cost
							2,015	21,511

Case 8 Part A Soil Nailing Cost

1- Drill hole		<u>each/hr</u>	<u>nail length</u>	<u>used length</u>								
drill rate	6 each x 20'	6	20	22.5								
Equip. 24000/160 =	150											
fog / hr =	40											
drill string wear =	15											
labor 42.17+34.37=	80											
	=====											
	285	→	<u>total</u>	<u>total\$/each</u>	<u>ratio</u>	<u>total\$/each</u>						
spacing =	27.5	sft	300	50	1.125	56.25						
1- Drill hole						total/each / spacing=		2.05	\$/sft			
2- Furnish and install nails		<u>rate/ hr =</u>	<u>4</u>									
Equip												
grout mixer w/360 air=	6400											
small excavator=	3600											
	=====											
	10,000.00	/ 160 =	62.50	/ rate / hr =	15.63							
Labor 4 men crew	50+42+69=	161	→	175	/ rate / hr =	43.75						
F O G =	35			35	/ rate / hr =	8.75						
Matl - Bar= 2.75		→	3+CONT. 0.15	3.15	x nail length	70.875						
grout / ft =	0.83			0.83	x nail length	18.675						
					=====							
				<u>sub. tot. =</u>	157.68							
Plate and nuts				→	10							
					=====							
					167.68	/ spacing=	6.10	\$/sft				
3- Place 6x6 mesh w/ miradrain												
mesh matl=	20.45x1.06 / 8x15	=	0.18	\$/sft								
miradrain @ 4' c-c =	210/150 =	1.4	/ 4 =	0.35	\$/sft							
chairs for mesh @4sft/chair =			0.1	\$/sft								
labor to place 4 @ 34.37 =	140/(6x4x15)		0.39	\$/sft								
			=====									
			1.02	→	1.1	\$/sft						
contin. walers @ dbl #4 and 4'												
nail spacing=	5	ft										
nail spacing=	0.416	x 0.5 mat and 0.5 lab	→	0.54	\$/sft							
4- Shotcrete												
82.5x1.06+75/cy sub/place	162.45/cy @ 4" = 81 sft=2/sft* 1.1= 2.2 /sft	→	1.72	\$/sft								
			=====									
			11.50	\$/sft								
5- over-all supervision @ 750 sft/day 75x8/750= 0.8												
6- OTA 6 men x 100/day /750 :		0.8	→	0.8	go to zero if job is in town							
				=====								
				13.10								
7- MOB AND DEMOB+ SHOP DRAWING (2500+2500+10,000)/2												
	0.56	→	0.56									
			=====									
			13.66	\$/sft								
shaving excavation face				0.25	\$/sft							
			=====									
			13.91	\$/sft	<u>sft</u>	<u>total cost</u>						
					3,972	55,261						

Case 8 Part B Soil Nailing Cost

1- Drill hole		<u>each/hr</u>	<u>nail length</u>	<u>used length</u>				
drill rate	6 each x 20'	6	20	22.5				
Equip. 24000/160 =		150						
fog / hr =		40						
drill string wear =		15						
labor 42.17+34.37=		80						
		=====						
spacing =		27.5	sft	total	total\$/each	ratio	total\$/each	
				300	50	1.125	56.25	
1- Drill hole								
		total/each / spacing= 2.05 \$/sft						
2- Furnish and install nails		<u>rate/ hr =</u>	<u>4</u>					
Equip								
grout mixer w/360 air=		6400						
small excavator=		3600						
		=====						
Labor 4 men crew		10,000.00	/ 160 =	62.50	/ rate / hr =	15.63		
50+42+69=		161	→	175	/ rate / hr =	43.75		
F O G =		35		35	/ rate / hr =	8.75		
Matl - Bar= 2.75		→	3+CONT. 0.15	3.15	x nail length	70.875		
grout / ft =		0.83		0.83	x nail length	18.675		
							=====	
							sub. tot. =	
							157.68	
Plate and nuts							→	
							10	
							=====	
							167.68 / spacing= 6.10 \$/sft	
3- Place 6x6 mesh w/ miradrain								
mesh matl= 20.45x1.06 / 8x15 =						0.18	\$/sft	
miradrain @ 4' c-c = 210/150 =		1.4	/ 4 =	0.35	\$/sft			
chairs for mesh @ 4sft/chair =						0.1	\$/sft	
labor to place 4 @ 34.37 = 140/(6x4x15)						0.39	\$/sft	
							=====	
							1.02 → 1.1 \$/sft	
contin. walers @ dbl #4 and 4'								
nail spacing= 5		ft						
nail spacing= 0.416		x 0.5 mat and 0.5 lab					→ 0.54 \$/sft	
4- Shotcrete								
82.5x1.06+75/cy sub/place								
162.45/cy @ 4" = 81 sft=2/sft* 1.1= 2.2 /sft						1.72	\$/sft	
							=====	
							11.50 \$/sft	
5- over-all supervision @ 750 sft/day 75x8/750= 0.8						0.8	\$/sft	
6- OTA 6 men x 100/day /750 sft 0.8						0.8	go to zero if job is in town	
							=====	
							13.10	
7- MOB AND DEMOB+ SHOP DRAWING (2500+2500+10,000)/27000						0.56		
							=====	
							13.66 \$/sft	
shaving excavation face							0.25 \$/sft	
							=====	
							13.91 \$/sft	
							<u>sft</u>	
							7,875	
							<u>total cost</u>	
							109,563	

Case 8 Part C Soil Nailing Cost

1- Drill hole									
drill rate	6 each x 20'	each/hr	nail length	used length					
		6	20	11					
Equip.	24000/160 =	150							
fog / hr =	40								
drill string wear =	15								
labor 42.17+34.37=	80								
	=====								
	285		total	total\$/each	ratio	total\$/each			
	27.5	sft	300	50	0.55	27.5			
spacing =	27.5	sft							
1- Drill hole				total/each / spacing=		1.00	\$/sft		
2- Furnish and install nail rate/ hr = 4									
Equip									
grout mixer	6400								
small excav	3600								
	=====								
	10,000.00	/ 160 =	62.50	/ rate / hr =	15.63				
Labor 4 men crew									
	50+42+69=	161	175	/ rate / hr =	43.75				
F O G =	35		35	/ rate / hr =	8.75				
Matl - Bar=	2.75	2+CONT. 0.1	3.15	x nail length	34.65				
grout / ft =	0.83		0.83	x nail length	9.13				
				=====					
			sub. tot =	111.91					
Plate and nuts				10					
				=====					
				121.91	/ spacing=	4.43	\$/sft		
3- Place 6x6 mesh w/ miradrain									
mesh matl=	20.45x1.06 / 8x15	=	0.18	\$/sft					
miradrain @ 4' c-c =	210/150 =	1.4	/ 4 =	0.35	\$/sft				
chairs for mesh @ 4sft/chair =			0.1	\$/sft					
labor to place 4 @ 34.37 =	140/(6x4x15)		0.39	\$/sft					
			=====						
			1.02			1.1	\$/sft		
contin. walers @ dbl #4 and 4'									
nail spacing=	5	ft							
nail spacing=	0.416	x 0.5 mat and 0.5 lab			0.54	\$/sft			
4- Shotcrete									
	82.5x1.06+75/cy sub/place								
	162.45/cy @ 4" = 81 sft=2/sft* 1.1= 2.2 /sft			1.72	\$/sft				
				=====					
				8.79	\$/sft				
5- over-all supervision @ 750 sft/day 75x8/750= 0.8									
				0.8	\$/sft				
6- OTA 6 men x 100/day /750 sft 0.8									
				0.8	go to zero if job is in town				
				=====					
				10.39					
7- MOB AND DEMOB+ SHOP DRAWING (2500+2500+10,000). 0.56									
				0.56					
				=====					
				10.95	\$/sft				
shaving excavation face				0.25	\$/sft				
				=====					
				11.20	\$/sft		sft	total cost	
							1,387	15,538	

Case 9 Soil Nailing Cost

1- Drill hole		<u>each/hr</u>	<u>nail length</u>	<u>used length</u>				
drill rate	6 each x 20'	6	20	26.5				
Equip. 24000/160 =		150						
fog / hr =		40						
drill string wear =		15						
labor 42.17+34.37=		80						
		=====						
285		→	<u>total</u>	<u>total\$/each</u>	<u>ratio</u>	<u>total\$/each</u>		
spacing = 25 sft			300	50	1.325	66.25		
1- Drill hole						total/each / spacing=	2.65	\$/sft
2- Furnish and install nails		<u>rate/ hr =</u>	4					
Equip								
grout mixer w/360 air=		6400						
small excavator=		3600						
		=====						
10,000.00 / 160 =		62.50	/ rate / hr =	15.63				
Labor 4 men crew								
50+42+69=		161	→	175	/ rate / hr =	43.75		
F O G =		35		35	/ rate / hr =	8.75		
Matl - Bar = 2.75			3+CONT. 0.15	3.15	x nail length	83.475		
grout / ft =		0.83		0.83	x nail length	21.995		
				=====				
				sub. tot. =	173.60			
Plate and nuts					10			
				=====	183.60	/ spacing=	7.34 \$/sft	
3- Place 6x6 mesh w/ miradrain								
mesh matl= 20.45x1.06 / 6x15 =		0.18	\$/sft					
miradrain @ 4' c-c = 210/150 = 1.4 / 4 =		0.35	\$/sft					
chairs for mesh @ 4sft/chair =		0.1	\$/sft					
labor to place 4 @ 34.37 = 140/(6x4x15)		0.39	\$/sft					
		=====						
1.02		→	1.1	\$/sft				
contin. walers @ dbl #4 and 4'								
nail spacing= 5 ft								
nail spacing= 0.416 x 0.5 mat and 0.5 lab		→	0.54	\$/sft				
4- Shotcrete								
82.5x1.06+75/cy sub/place								
162.45/cy @ 4" = 81 sft=2/sft* 1.1= 2.2 /sft		→	1.72	\$/sft				
				=====	13.35	\$/sft		
5- over-all supervision @ 750 sft/day 75x8/750= 0.8		→	0.8	\$/sft				
6- OTA 6 men x 100/day /750 : 0.8		→	0	go to zero if job is in town				
				=====				14.15
7- MOB AND DEMOB+ SHOP DRAWING (2500+2500+10,000)/2' 0.56		→	0.56					
								=====
shaving excavation face		0.25	\$/sft					
				=====	14.96	\$/sft		
				<u>sft</u>	<u>total cost</u>			
				8,514	127,402			

Case 10 Soil Nailing Cost

1- Drill hole	drill rate	6 each x 20'	<u>each/hr</u> 6	<u>nail length</u> 20	<u>used length</u> 26.5				
	Equip.	24000/160 =	150						
	fog / hr =	40							
	drill string wear =	15							
	labor 42.17+34.37=	80							
		=====				<u>total</u>	<u>total\$/each</u>	<u>ratio</u>	<u>total\$/each</u>
		285	→			300	50	1.325	66.25
	spacing =	25	sft						
1- Drill hole							total/each / spacing=	2.65	\$/sft
2- Furnish and install nails	rate/ hr =	4							
	Equip								
	grout mixer w/360 air=	6400							
	small excavator=	3600							
		=====							
		10,000.00	/ 160 =	62.50	/ rate / hr =	15.63			
Labor 4 men crew									
	FOG =	50+42+69=	161	→	175	/ rate / hr =	43.75		
	Mati - Bar = 2.75	35			35	/ rate / hr =	8.75		
	grout / ft =	0.83	→	3+CONT. 0.15	3.15	x nail length	83.475		
					0.83	x nail length	21.995		
						=====			
						sub. tot. =	173.60		
Plate and nuts							10		
						=====			
							183.60	/ spacing=	7.34 \$/sft
3- Place 6x6 mesh w/ miradrain									
	mesh mat=	20.45x1.06 / 8x15	=	0.18	\$/sft				
	miradrain @ 4' c-c =	210/150 =	1.4	/ 4 =	0.35	\$/sft			
	chairs for mesh @ 4sft/chair =			0.1	\$/sft				
	labor to place 4 @ 34.37 =	140/(6x4x15)		0.39	\$/sft				
				=====					
				1.02	→		1.1	\$/sft	
	contin. walers @ dbl #4 and 4'								
	nail spacing=	5	ft						
	nail spacing=	0.416	x 0.5 mat and 0.5 lab	→			0.54	\$/sft	
4- Shotcrete									
	82.5x1.06+75/cy sub/place								
		162.45/cy @ 4" = 81 sft=2/sft* 1.1= 2.2 /sft	→				1.72	\$/sft	
						=====			
							13.35	\$/sft	
5- over-all supervision @ 750 sft/day 75x8/750= 0.8							0.8	\$/sft	
6- OTA	6 men x 100/day /750 :	0.8					0	go to zero if job is in town	
						=====			
							14.15		
7- MOB AND DEMOB+ SHOP DRAWING (2500+2500+5,000)/271	0.37						0.37		
						=====			
							14.52	\$/sft	
	shaving excavation face						0.25	\$/sft	
						=====			
							14.77	\$/sft	
							<u>sft</u>	<u>total cost</u>	
							11,940	176.39	

Case 11 Soil Nailing Cost

1- Drill hole		<u>each/hr</u>	<u>nail length</u>	<u>used length</u>									
drill rate	6 each x 20'	6	20	14									
Equip. 24000/160 =		150											
fog / hr =		40											
drill string wear =		15											
labor 42.17+34.37=		80											
		=====											
		285	→	<u>total</u>	<u>total\$/each</u>	<u>ratio</u>	<u>total\$/each</u>						
spacing = 24 sft				300	50	0.7	35						
1- Drill hole						total/each / spacing= 1.46 \$/sft							
2- Furnish and install nails		<u>rate/ hr =</u>	<u>4</u>										
Equip													
grout mixer w/360 air=		6400											
small excavator=		3600											
		=====											
Labor 4 men crew		10,000.00	/ 160 =	62.50	/ rate / hr =	15.63							
FOG =		50+42+69=	161	→	175	/ rate / hr =	43.75						
Matl - Bar= 2.75		35			35	/ rate / hr =	8.75						
grout / ft =		0.83	→	3+CONT. 0.15	3.15	x nail length	44.1						
					0.83	x nail length	11.62						
						=====							
				<u>sub. tot. =</u>		123.85							
Plate and nuts						10							
						=====							
						133.85	/ spacing=	5.58 \$/sft					
3- Place 6x6 mesh w/ miradrain													
mesh matl= 20.45x1.06 / 8x15 =				0.18	\$/sft								
miradrain @ 4' c-c = 210/150 =		1.4	/ 4 =	0.35	\$/sft								
chairs for mesh @ 4sft/chair =				0.1	\$/sft								
labor to place 4 @ 34.37 = 140/(6x4x15)				0.39	\$/sft								
				=====									
				1.02	→	1.1	\$/sft						
contin. walers @ dbl #4 and 4'													
nail spacing=		5	ft										
nail spacing=		0.416	x 0.5 mat and 0.5 lab	→	0.54	\$/sft							
4- Shotcrete													
82.5x1.06+75/cy sub/place													
162.45/cy @ 4" = 81 sft=2/sft* 1.1= 2.2 /sft				→	1.72	\$/sft							
					=====								
					10.40	\$/sft							
5- over-all supervision @ 750 sft/day 75x8/750= 0.8				→	0.8	\$/sft							
6- OTA 6 men x 100/day /750: 0.8				→	0	go to zero if job is in town							
					=====								
					11.20								
7- MOB AND DEMOB+ SHOP DRAWING (2500+2500+10,000)/2' 0.56				→	0.56								
					=====								
					11.76	\$/sft							
shaving excavation face					0.25	\$/sft							
					=====								
					12.01	\$/sft							
					=====								
					<u>sft</u>	<u>total cost</u>							
					6,218	74,648							

[illegible]

Case 12 Part B Soil Nailing Cost

1- Drill hole
drill rate 6 each x 20' each/hr 6 nail length 20 used length 23

Equip. 24000/160 = 150
fog / hr = 40
drill string wear = 15
labor 42.17+34.37 = 80

=====
285 → total 300 total\$/each 50 ratio 1.15 total\$/each 57.5
spacing = 25 sft

1- Drill hole

total/each / spacing = 2.30 \$/sft

2- Furnish and install nails

rate/ hr = 4

Equip
grout mixer w/360 air = 6400
small excavator = 3600

=====
10,000.00 / 160 = 62.50 / rate / hr = 15.63

Labor 4 men crew

50+42+69 = 161

→ 175 / rate / hr = 43.75

F O G =

35

35 / rate / hr = 8.75

Mat - Bar = 2.75

→ 3+CONT. 0.15

3.15 x nail length 72.45

grout / ft =

0.83

0.83 x nail length 19.09

sub. tot. = 159.67

Plate and nuts

→ 10

=====
169.67 / spacing = 6.79 \$/sft

3- Place 6x6 mesh w/ miradrain

mesh mat = 20.45x1.06 / 8x15 = 0.18 \$/sft

miradrain @ 4' c-c = 210/150 = 1.4 / 4 = 0.35 \$/sft

chains for mesh @ 4sft/chair = 0.1 \$/sft

labor to place 4 @ 34.37 = 140/(6x4x15) 0.39 \$/sft

=====
1.02 → 1.1 \$/sft

contin. walers @ dbl #4 and 4'

nail spacing = 5 ft
nail spacing = 0.416 x 0.5 mat and 0.5 lab

→ 0.54 \$/sft

4- Shotcrete

82.5x1.06+75/cy sub/place

162.45/cy @ 4" = 81 sft=2/sft* 1.1= 2.2 /sft

→ 1.72 \$/sft

=====
12.45 \$/sft

5- over-all supervision @ 750 sft/day 75x8/750 = 0.8

→ 0.8 \$/sft

6- OTA 6 men x 100/day /750 sft 0.8

→ 0.8 go to zero if job is in town

=====
14.05

7- MOB AND DEMOB+ SHOP DRAWING (2500+2500+10,000)/27000

0.56

→ 0.56

=====

14.61 \$/sft

shaving excavation face 0.25 \$/sft

=====

14.86 \$/sft

sft total cost

5,250 77,997

Case 12 Part C Soil Nailing Cost

[illegible]

Case 13 Soil Nailing Cost

[illegible]

Case 14 Part A Soil Nailing Cost

1- Drill hole		<u>each/hr</u>	<u>nail length</u>	<u>used length</u>				
drill rate	6 each x 20'	6	20	16.5				
Equip. 24000/160 =		150						
fog / hr =		40						
drill string wear =		15						
labor 42.17+34.37=		80						
		=====						
spacing =		285	→	<u>total</u>	<u>total\$/each</u>	<u>ratio</u>	<u>total\$/each</u>	
		27	sft	300	50	0.825	41.25	
1- Drill hole				total/each / spacing= 1.53 \$/sft				
2- Furnish and install nails		<u>rate/ hr =</u>	<u>4</u>					
Equip								
grout mixer w/360 air=		6400						
small excavator=		3600						
		=====						
		10,000.00	/ 160 =	62.50	/ rate / hr =	15.63		
Labor 4 men crew								
50+42+69=		161	→	175	/ rate / hr =	43.75		
F O G =		35		35	/ rate / hr =	8.75		
Matl - Bar= 2.75			→	3+CONT. 0.15	x nail length	51.975		
grout / ft =		0.83		0.83	x nail length	13.695		
				=====				
				sub. tot. =		133.80		
Plate and nuts						10		
						=====		
				143.80		/ spacing=	5.33 \$/sft	
3- Place 6x6 mesh w/ miradrain								
mesh matl= 20.45x1.06 / 8x15 =				0.18	\$/sft			
miradrain @ 4' c-c = 210/150 =		1.4	/ 4 =	0.35	\$/sft			
chairs for mesh @ 4sft/chair =				0.1	\$/sft			
labor to place 4 @ 34.37 = 140/(6x4x15)				0.39	\$/sft			
				=====				
				1.02	→	1.1	\$/sft	
contin. walers @ dbl #4 and 4'								
nail spacing=		5	ft					
nail spacing=		0.416	x 0.5 mat and 0.5 lab	→	0.54	\$/sft		
4- Shotcrete								
82.5x1.06+75/cy sub/place								
162.45/cy @ 4" = 81 sft=2/sft* 1.1= 2.2 /sft				1.72	\$/sft			
				=====				
				10.21	\$/sft			
5- over-all supervision @ 750 sft/day 75x8/750= 0.8				0.8	\$/sft			
6- OTA 6 men x 100/day /750: 0.8				0				
				=====				
				11.01				
7- MOB AND DEMOB+ SHOP DRAWING (2500+2500+10,000)/2' 0.56				0.56				
				=====				
				11.57	\$/sft			
shaving excavation face				0.25	\$/sft			
				=====				
				11.82	\$/sft	<u>sft</u>	<u>total cost</u>	
						3,105	36,712	

Case 14 Part B Soil Nailing Cost

1- Drill hole									
drill rate	6 each x 20'	each/hr	nail length	used length					
		6	20	10					
	Equip. 24000/160 =	150							
	fog / hr =	40							
	drill string wear =	15							
	labor 42.17+34.37=	80							
		=====							
		285		total	total\$/each	ratio	total\$/each		
	spacing =	27	sft	300	50	0.5	25		
1- Drill hole								total/each / spacing=	0.93 \$/sft
2- Furnish and install nails		rate/ hr =	4						
Equip									
grout mixer w/360 air=	6400								
small excavator=	3600								
	=====								
	10,000.00	/ 160 =	62.50	/ rate / hr =	15.63				
Labor 4 men crew									
	50+42+69=	161		175	/ rate / hr =	43.75			
F O G =	35			35	/ rate / hr =	8.75			
Matl - Bar= 2.75			3+CONT. 0.15	3.15	x nail length	31.5			
grout / ft =	0.83			0.83	x nail length	8.3			
					=====				
				sub. tot. =	107.93				
Plate and nuts					10				
					=====				
					117.93	/ spacing=	4.37	\$/sft	
3- Place 6x6 mesh w/ miradrain									
	mesh matl= 20.45x1.06 / 8x15 =			0.18	\$/sft				
	miradrain @ 4' c-c = 210/150 =	1.4	/ 4 =	0.35	\$/sft				
	chairs for mesh @4sft/chair =			0.1	\$/sft				
	labor to place 4 @ 34.37 = 140/(6x4x15)			0.39	\$/sft				
				=====					
				1.02			1.1	\$/sft	
contin. walers @ dbl #4 and 4'									
	nail spacing=	5	ft						
	nail spacing=	0.416	x 0.5 mat and 0.5 lab				0.54	\$/sft	
4- Shotcrete									
	82.5x1.06+75/cy sub/place						1.72	\$/sft	
	162.45/cy @ 4" = 81 sft=2/sft* 1.1= 2.2 /sft						=====		
							8.65	\$/sft	
5- over-all supervision @ 750 sft/day 75x3/750= 0.8							0.8	\$/sft	
6- OTA 6 men x 100/day /750 sft 0.8							0	go to zero if job is in town	
							=====		
							9.45		
7- MOB AND DEMOB+ SHOP DRAWING (2500+2500+10,000)/27000				0.56			0.56		
							=====		
							10.01	\$/sft	
							shaving excavation face	0.25	\$/sft
							=====		
							10.26	\$/sft	
							sft	total cost	
							995	10,212	

Case 15 Soil Nailing Cost

1- Drill hole									
drill rate	6 each x 20'	each/hr	nail length	used length					
		6	20	32.5					
	Equip. 24000/160 =	150							
	fog / hr =	40							
	drill string wear =	15							
	labor 42.17+34.37=	80							
		=====							
		285	→	total	total\$/each	ratio	total\$/each		
	spacing =	27	sft	300	50	1.625	81.25		
1- Drill hole								total/each / spacing=	3.01 \$/sft
2- Furnish and install nails		rate/ hr =	4						
Equip									
grout mixer w/380 air=	6400								
small excavator=	3600								
	=====								
	10,000.00	/ 160 =	62.50	/ rate / hr =	15.63				
Labor 4 men crew									
	50+42+89=	161	→	175	/ rate / hr =	43.75			
F O G =	35			35	/ rate / hr =	8.75			
Matl - Bar= 2.75		→	3+CONT. 0.15	3.15	x nail length	102.375			
grout / ft =	0.83			0.83	x nail length	26.975			
					=====				
				sub. tot. =	197.48				
Plate and nuts					10				
					=====				
					207.48	/ spacing=	7.68	\$/sft	
3- Place 6x6 mesh w/ miradrain									
	mesh matl= 20.45x1.06 / 8x15 =			0.18	\$/sft				
	miradrain @ 4' c-c = 210/150 =	1.4	/ 4 =	0.35	\$/sft				
	chairs for mesh @ 4sft/chair =			0.1	\$/sft				
	labor to place 4 @ 34.37 = 140/(6x4x15)			0.39	\$/sft				
				=====					
				1.02	→	1.1	\$/sft		
contin. walers @ dbl #4 and 4'									
	nail spacing= 5	ft							
	nail spacing= 0.416	x 0.5 mat and 0.5 lab	→	0.54	\$/sft				
4- Shotcrete									
	82.5x1.06+75/cy sub/place								
	162.45/cy @ 4" = 81 sft=2/sft* 1.1= 2.2 /sft		→	1.72	\$/sft				
				=====					
				14.05	\$/sft				
5- over-all supervision @ 750 sft/day 75x8/750= 0.8			→	0.8	\$/sft				
6- OTA 6 men x 100/day /750: 0.8			→	0	go to zero if job is in town				
				=====					
				14.85					
7- MOB AND DEMOB+ SHOP DRAWING (2500+2500+10,000)/2' 0.56			→	0.56					
				=====					
				15.41	\$/sft				
				shaving excavation face	0.25	\$/sft			
				=====					
				15.66	\$/sft				
						sft	total cost		
						71,820	1,124,954		

CURRICULUM VITA

NAME: Abdul Amir Al-Helu

ADDRESS: 9304 Minute Men Ct
Louisville, KY 40214

DOB: July, 1st 1966

EDUCATION

& TRAINING: B.S. Building and Construction Engineering
Technology University, Baghdad, Iraq
1983-1987

M.Eng., Civil Engineering
University of Louisville, Louisville, Kentucky
1995-1997

Ph.D., Civil Engineering
University of Louisville, Louisville, Kentucky
2004-2011

PROFESSIONAL SOCIETIES:

The author is a member of the Chi Epsilon honor society and the American Society of Civil Engineers.