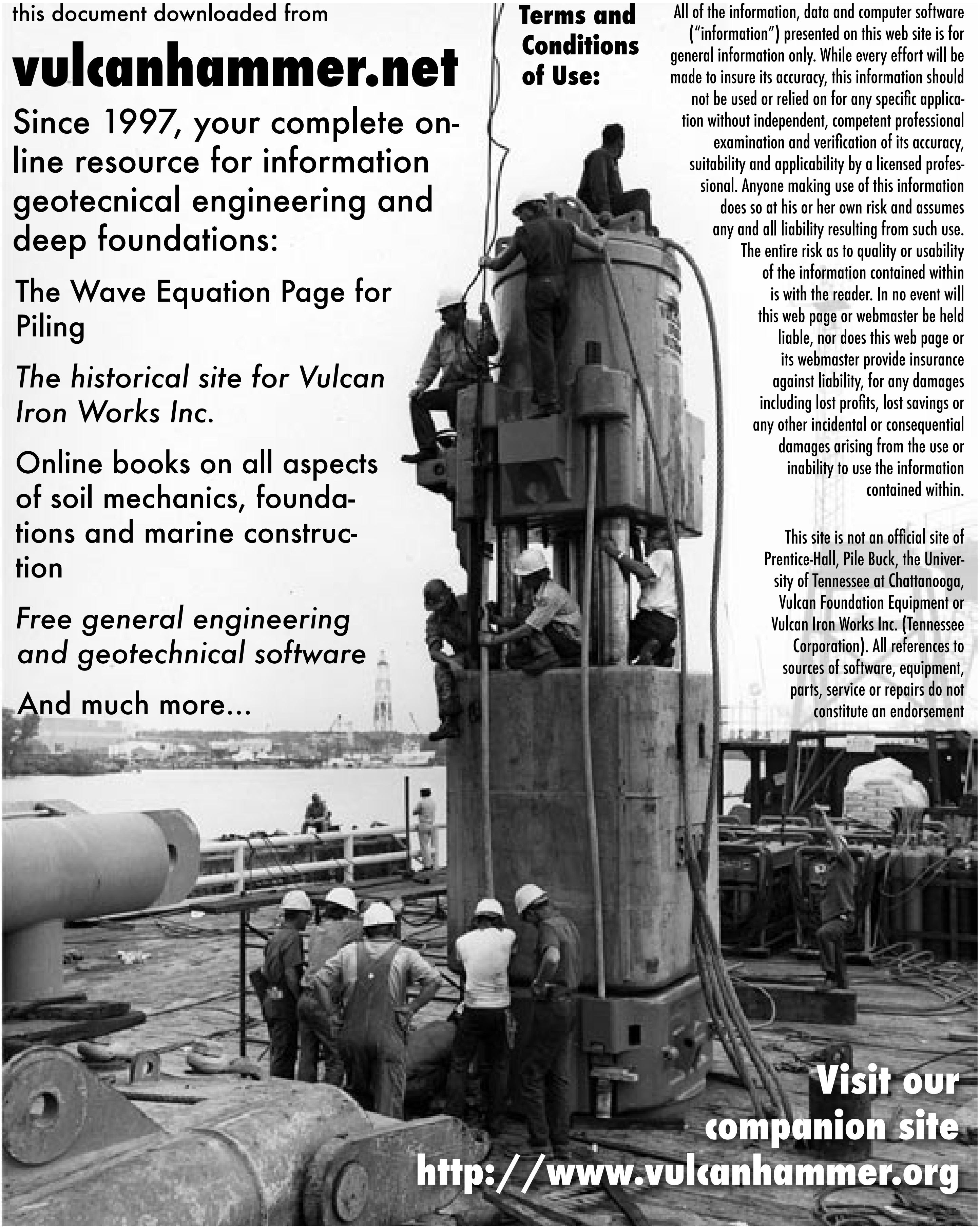

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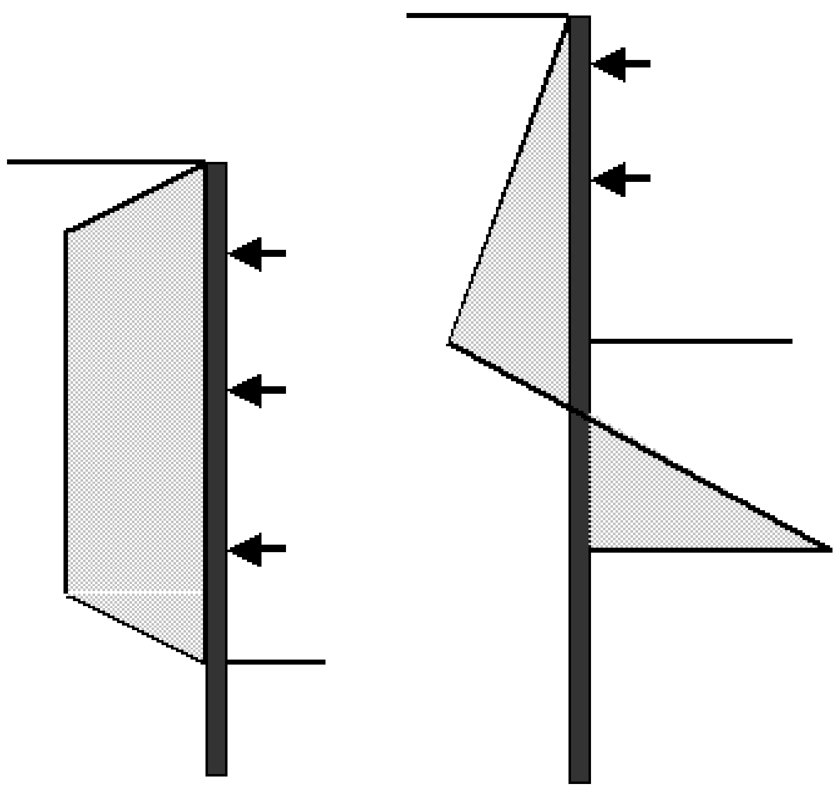
Innovations for Navigation Projects Research Program

Simplified Procedures for the Design of Tall, Stiff Tieback Walls

Ralph W. Strom and Robert M. Ebeling

November 2002



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Simplified Procedures for the Design of Tall, Stiff Tieback Walls

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Final report

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ABSTRACT: Methods used in the design of flexible and stiff tieback walls are described. Methods applicable to the design of stiff tieback wall systems are illustrated by example. Important in the design of stiff tieback wall systems is the consideration of construction sequencing effects. Illustrated by example are the equivalent beam on rigid supports method and the equivalent beam on inelastic supports method.

Both the equivalent beam on rigid supports and the equivalent beam on inelastic supports analysis methods consider construction sequencing effects. The equivalent beam on rigid supports method uses soil pressure distributions based on classical methods. The equivalent beam on inelastic supports method uses soil springs (nonlinear) to determine earth-pressure loadings and preloaded concentrated springs (nonlinear) to determine tieback forces. Soil springs are in accordance with the reference deflection method proposed in the Federal Highway Administration's "Summary report of research on permanent ground anchor walls; Vol II, Full-scale tests and soil structure interaction model" (FHWA-RD-98-066).

Soil springs are shifted after each excavation stage to account for the plastic soil movements that occur during excavation. The software program CMULTIANC, newly developed to facilitate the equivalent beam on inelastic supports construction-sequencing analysis, is illustrated in the report.

The results from the equivalent beam on rigid supports and equivalent beam on inelastic supports analyses are compared with each other and to the results obtained from other tieback wall analyses. The results are also compared with those obtained from apparent pressure diagram analyses. The apparent pressure diagram approach is common to the design of flexible wall systems.

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Preface

The study described in this report was authorized by Headquarters, U.S. Army Corps of Engineers (HQUSACE), as part of the Innovations for Navigation Projects (INP) Research Program. The study was conducted under Work Unit (WU) 33272, "Soil-Structure Interaction Studies of Walls with Multiple Rows of Anchors."

Dr. Tony C. Liu was the INP Coordinator at the Directorate of Research and Development, HQUSACE; Research Area Manager was Mr. Barry Holliday, HQUSACE; and Program Monitors were Mr. Mike Kidby and Ms. Anjana Chudgar, HQUSACE. Mr. William H. McAnally of the ERDC Coastal and Hydraulics Laboratory was the Lead Technical Director for Navigation Systems; Dr. Stanley C. Woodson, ERDC Geotechnical and Structures Laboratory (GSL), was the INP Program Manager.

This report was prepared by Mr. Ralph W. Strom, Portland, OR, and Dr. Robert M. Ebeling, U.S. Army Engineer Research and Development Center (ERDC), Information Technology Laboratory (ITL). The research was monitored by Dr. Ebeling, Principal Investigator for WU 33272, under the supervision of Mr. H. Wayne Jones, Chief, Computer-Aided Engineering Division, ITL; Dr. Jeffery P. Holland, Director, ITL; and Dr. David R. Pittman, Acting Director, GSL.

At the time of publication of this report, Dr. James R. Houston was Director of ERDC, and COL John W. Morris III, EN, was Commander and Executive Director.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
degrees (angle)	0.01745329	radians
feet	0.3048	meters
inches	25.4	millimeters
kips (force) per square foot	47.88026	kilopascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic meter
pounds (force) per square foot	47.88026	pascals
pounds (force) per square inch	6.894757	kilopascals

1 Introduction to Example Problems

1.1 Design of Flexible Tieback Wall Systems

The equivalent beam on rigid support method of analysis using apparent earth-pressure envelopes is most often the design method of choice, primarily because of its expediency in the practical design of tieback wall systems. This method provides the most reliable solution for ***flexible*** wall systems, i.e., soldier beam-lagging systems and sheet-pile wall systems, since for these types of systems a significant redistribution of earth pressures occurs behind the wall. Soil arching, stressing of ground anchors, construction-sequencing effects, and lagging flexibility all cause the earth pressures behind flexible walls to redistribute to, and concentrate at, anchor support locations (FHWA-RD-98-066). This redistribution effect in flexible wall systems cannot be captured by equivalent beam on rigid support methods or by beam on inelastic foundation analysis methods where the active and passive limit states are defined in terms of Rankine or Coulomb coefficients. Full-scale wall tests on flexible wall systems (FHWA-RD-98-066) indicated the active earth pressure used to define the minimum load associated with the soil springs behind the wall had to be reduced by 50 percent to match measured behavior. Since the apparent earth-pressure diagrams used in equivalent beam on rigid support analyses were developed from measured loads, and thus include the effects of soil aching, stressing of ground anchors, construction-sequencing effects, and lagging flexibility, they provide a better indication of the strength performance of flexible tieback wall systems. This is not the case for ***stiff*** wall systems, however, and in fact is applicable only to those flexible wall systems in which

a. Overexcavation to facilitate ground anchor installation does not occur.

b. Ground anchor preloading is compatible with active limit state conditions.

c. The water table is below the base of the wall.

The design of flexible wall systems is illustrated in Ebeling, Azene, and Strom (2002).

1.2 Design of Stiff Tieback Wall Systems

Construction-sequencing analyses are important in the evaluation of stiff tieback wall systems, since for such systems the temporary construction stages are often more demanding than the final permanent loading condition (Kerr and Tamaro 1990). This may also be true for flexible wall systems where significant overexcavation occurs and for flexible wall systems subject to anchor prestress loads producing soil pressures in excess of active limit state conditions. The purpose of the example problems contained herein is to illustrate the use of construction-sequencing analysis for the design of stiff tieback wall systems. Although many types of construction-sequencing analyses have been used in the design of tieback wall systems, only three types of construction-sequencing analyses are demonstrated in the example problems. The three construction-sequencing analyses chosen for the example problems are ones considered to be the most promising for the design and evaluation of Corps tieback wall systems. The analyses are

a. Equivalent beam on rigid supports by classical methods (identified herein as the RIGID 2 method).

b. Beam on inelastic foundation methods using elastoplastic soil-pressure deformation curves (R-y curves) that account for plastic (nonrecoverable) movements (identified herein as the WINKLER 1 method).

c. Beam on inelastic foundation methods using elastoplastic soil-pressure deformation curves (R-y curves) for the resisting side only with classical soil pressures applied on the driving side (identified herein as the WINKLER 2 method).

The results from these three construction-sequencing methods are compared with the results obtained from the equivalent beam on rigid support method using apparent pressure loading (identified herein as the RIGID 1 method). Recall that apparent earth pressures are an envelope of maximum past pressures encountered over all stages of excavation. The results are also compared with field measurements and finite element analyses when available.

1.2.1 Identifying stiff wall systems

Five focus wall systems were identified in Strom and Ebeling (2001):

a. Vertical sheet-pile system with wales and post-tensioned tieback anchors.

b. Soldier beam system with wood or reinforced concrete lagging and post-tensioned tieback anchors. For the wood lagging system, a permanent concrete facing system is required.

c. Secant cylinder pile system with post-tensioned tieback anchors.

d. Continuous reinforced concrete slurry wall system with post-tensioned tieback anchors.

e. Discrete concrete slurry wall system (soldier beams with concrete lagging) with post-tensioned tieback anchors

These systems are described in detail in Chapter 2 of the report (Strom and Ebeling 2001).

Deformations and wall movements in excavations are a function of soil strength and wall stiffness, with wall stiffness a function of structural rigidity (EI) of the wall and the vertical spacing of anchors (L). Soil stiffness correlates to soil strength; therefore, soil strength is often used in lieu of soil stiffness to characterize the influence of the soil on wall displacements. Steel sheet-pile and steel soldier beams with timber lagging systems are considered to be flexible tieback wall systems. Secant cylinder pile, continuous concrete slurry wall, and discrete concrete slurry wall systems are considered to be stiff tieback wall systems. The effect of wall stiffness on wall displacements and earth pressures is described in Xanthakos (1991) and in FHWA-RD-81-150. In the FHWA report, it is indicated that Clough and Tsui (1974) showed, by finite element analyses, that wall and soil movements could be reduced by increasing wall rigidity and tieback stiffness. None of the reductions in movements were proportional to the increased stiffness, however. For example, an increase in wall rigidity of 32 times reduced the movements by a factor of 2. Likewise, an increase in the tieback stiffness by a factor of 10 caused a 50-percent reduction in movements.

Other investigators have also studied the effect of support stiffness for clays (as reported in FHWA-RD-75-128). They defined system stiffness by EI/L^4, where EI is the stiffness of the wall, and L is the distance between supports (see Figure 1.1). The measure of wall stiffness is defined as a variation on the inverse of Rowe's flexibility number for walls, and is thus expressed by EI/L^4, where L is the vertical distance between two rows of anchors. Wall stiffness refers not only to the structural rigidity derived from the elastic modulus and the moment of inertia, but also to the vertical spacing of supports (in this case anchors). It is suggested by Figure 9-106 in FHWA-RD-75-128 that, for stiff clays with a stability number ($\gamma H/s_u$) equal to or less than 3, a system stiffness (EI/L^4) of 10 or more would keep soil displacement equal to or less than 1 in.[1,2] However, other factors, such as prestress level, overexcavation, and factors of safety, also influence displacement. Data in this figure clearly indicate that stiff wall systems in stiff clays will displace less than flexible wall systems in soft clays. Table 1.1 categorizes flexible and stiff wall systems with respect to the focus wall systems of the Strom and Ebeling (2001) report.

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[1] At this time, the authors of this report recommend that, when tieback wall system displacements are the quantity of interest (i.e., stringent displacement control design), they should be estimated by nonlinear finite element-soil structure interaction (NLFEM) analysis.

[2] A table of factors for converting non-SI units of measurement to SI units is presented on page vi.

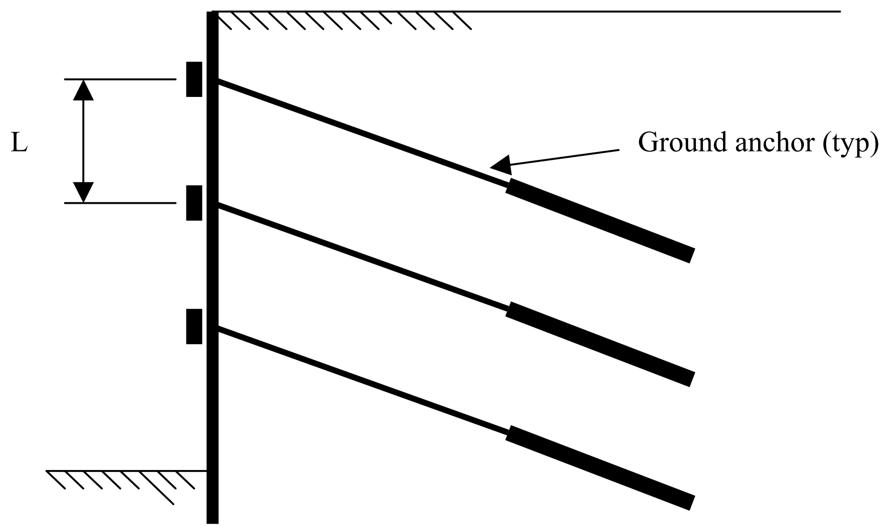


Figure 1.1. Definition of span length *L*

Table 1.1
Stiffness Categorization of Focus Wall Systems (from Strom and Ebeling 2001)

Focus Tieback Wall System Description	Wall Stiffness Category	
	Flexible	***Stiff***
Vertical sheet-pile system	√	
Soldier beam system	√	
Secant cylinder pile		√
Continuous reinforced concrete slurry wall system		√
Discrete concrete slurry wall system		√

Using the approach of FHWA-RD-75-128, the wall stiffness can be quantified in terms of the flexural stiffness (EI) per foot run of wall and in terms of the relative flexural stiffness (EI/L^4). This information is presented in Table 1.2 for the focus wall systems of the Strom and Ebeling (2001) report. The relative flexural stiffness in the table is based on a span length (L), i.e., a vertical anchor spacing of 10 ft.

It should be recognized from the above stiffness calculations that a secant pile system with L equal to 28.5 ft would produce a flexural stiffness value of EI/L^4, equal to that for the vertical sheet-pile wall system with L equal to 10 ft. Therefore, it is possible, by spacing anchors at close intervals, to obtain a stiff wall system using flexible sheetpiling or, vice versa, to obtain a flexible wall system using secant piles with widely spaced anchors.

Table 1.2 General Stiffness Quantification for Focus Wall Systems (from Strom and Ebeling 2001)			
Wall Stiffness	**Wall System**	**EI ($k\text{-}ft^2/ft \times 10^4$)**	**EI/L^4 (ksf/ft)**
Flexible	Vertical sheet-pile system	0.3 to 5.0	3.7 (1)
	Soldier beam system	0.1 to 4.0	1.5 (2)
Stiff	Secant cylinder pile	8.0 to 250.0	239.8 (3)
	Continuous reinforced concrete slurry wall	30.0 to 150.0	123.1 (4)
	Discrete concrete slurry wall	35.0 to 160.0	92.3 (5)

(1) Relative stiffness based on PZ 27 sheetpiling. Per Olmsted Prototype Wall.
(2) Relative stiffness based on HP12×53 soldier beams spaced at 8.0 ft on center (OC). Per FHWA-RD-97-130 design example.
(3) Relative stiffness based on 5.0-ft-diam caisson piles spaced at 7.0 feet OC. Per Monongahela River Locks and Dams 2 Project.
(4) Relative stiffness based on 3.0-ft-thick continuous slurry trench wall. Per Bonneville Navigation Lock Temporary Tieback Wall.
(5) Relative stiffness based on W36×393 soldier beams spaced at 6.0 ft OC with concrete lagging. Per Bonneville Navigation Lock Upstream Wall.

1.2.2 Tieback wall performance objectives

1.2.2.1 Safety with economy design. Common factors of safety used in practice for the design of anchored walls range between 1.1 and 1.5 applied to the shear strength of the soil and used in the calculation of the earth-pressure coefficient that characterizes the magnitude of the total force applied to the wall (FHWA-RD-98-065). Values adopted for a factor of safety vary with the importance of the wall, the consequences of failure, the performance objective (i.e., “safety with economy” or “stringent displacement control”), and economics.

Factors of safety ranging from 1.1 to 1.2 are generally considered unacceptable for the design of permanent walls. Walls constructed with factors of safety between 1.1 and 1.2 may be stable, but may also experience undesirable displacements near the wall (FHWA-RD-98-065). Therefore, factors of safety in this range should be used with caution and only for temporary walls where large displacements are considered to be acceptable. The design and construction of a temporary excavation tieback wall support system with a low factor of safety (i.e., large displacements were anticipated) is described in Cacoilo, Tamaro, and Edinger (1998). For permanent walls, in most situations some lateral movement of the tieback wall system can be tolerated, recognizing that with lateral wall movement settlements will occur in the retained soil immediately behind the wall. Tieback wall designs based on strength only, without special consideration of wall displacement, are termed safety with economy designs.

The Soletanche wall example (discussed in Chapter 2) is a safety with economy design. This means that, for flexible wall systems, the tieback anchors and wall system can be designed for soil- pressure conditions approaching active state conditions. As such, the apparent earth-pressure diagrams used in the design can be based on a total load approach using a factor of safety of 1.3 applied to the shear strength of the soil per the design recommendations of FHWA-RD-97-130. Trapezoidal earth-pressure distributions are used for this type of analysis. For stiff wall systems, active earth pressures in the retained soil can often be assumed and used in a construction-sequencing analysis to size anchors and determine wall properties. Earth-pressure distribution for this type of analysis would be in accordance with classical earth pressures theory, i.e. triangular with the absence of a water table.

The general practice for safety with economy design is to keep anchor prestress loads to a minimum consistent with active, or near-active, soil-pressure conditions (depending upon the value assigned to the factor of safety). This means the anchor size would be smaller, the anchor spacing larger, and the anchor prestress lower than those found in designs requiring stringent displacement control.

1.2.2.2 Stringent displacement control design. A performance objective for a tieback wall can be to restrict wall and soil movements during excavation to a tolerable level so that structures adjacent to the excavation will not experience distress (as for the Bonneville temporary tieback wall example). According to FHWA-RD-81-150, the tolerable ground surface settlement may be less than 0.5 in. if a settlement-sensitive structure is founded on the same soil used for supporting the anchors. Tieback wall designs that are required to meet specified displacement control performance objectives are termed stringent displacement control designs. Selection of the appropriate design pressure diagram for determining anchor prestress loading depends on the level of wall and soil movement that can be tolerated. Walls built with factors of safety between 1.3 and 1.5 applied to the shear strength of the soil may result in smaller displacements if stiff wall components are used (FHWA-RD-98-065).

To minimize the outward movement, the design would proceed using soil pressures at a magnitude approaching at-rest pressure conditions (i.e., a factor of safety of 1.5 applied to the shear strength of the soil). It should be recognized that even though the use of a factor of safety equal to 1.5 is consistent with an at-rest (i.e., zero soil-displacement condition) earth-pressure coefficient (as shown in Figure 3-6 of Engineer Manual 1110-2-2502 [Headquarters, Department of the Army 1989]), several types of lateral wall movement could still occur. These include cantilever movements associated with installation of the first anchor; elastic elongation of the tendon anchor associated with a load increase; anchor yielding, creep, and load redistribution in the anchor bond zone; and mass movements behind the ground anchors (FHWA-SA-99-015). It also should be recognized that a stiff rather than flexible wall system may be required to reduce bending displacements in the wall to levels consistent with the performance objectives established for the stringent displacement control design. A stringent displacement control design for a flexible wall system, however, would result in anchor spacings that are closer and anchor prestress levels that are higher than those for a comparable safety with economy design. If displacement control is a critical performance objective for the project being designed, the use of a stiff rather than flexible wall system should be considered.

1.2.3 Progressive design of tieback wall systems

As with most designs, a progressive analysis (starting with the simplest design tools and progressing to more comprehensive design tools when necessary) is highly recommended by the authors. With respect to flexible wall systems, some of the more comprehensive analysis tools used for stiff wall system analysis (construction-sequencing analysis based on classical earth-pressure distributions and beam on inelastic foundation analysis) are not generally considered appropriate for the analysis of flexible wall systems. This is because apparent pressure diagrams, since they are "envelopes" based on measurements made during construction, include the effects of soil arching, wall flexibility, preloading of supports, facial stiffness, and construction sequencing. However, with stiff wall systems, the aforementioned items will not affect earth-pressure redistribution to the same extent they affect flexible wall systems. Therefore, in practice, construction-sequencing analyses and beam on inelastic foundation analyses are considered valid tools for the investigation of stiff wall system behavior. The design and analysis tools typically used in the design and analysis of flexible and stiff wall systems are summarized in Tables 1.3 and 1.4, respectively, starting with the simplest design tool and progressing to the more comprehensive analytical tools. The most comprehensive design tools are linear elastic finite element (LEFEM) and nonlinear finite element (NLFEM) soil-structure interaction analyses. The NLFEM analysis is required when it becomes necessary to verify that the design meets stringent displacement control performance objectives. Both the LEFEM and NLFEM analyses can be used to verify safety with economy designs.

Table 1.3
Design and Analysis Tools for Flexible Wall Systems

Analysis	Objective	Description	Analysis Method
RIGID 1	Final design when performance goal is "safety with economy." Preliminary design when performance goal is "stringent displacement control."	Beam on rigid supports analysis using apparent pressure "envelope" diagram. Apparent pressure diagram based on a total load approach. Total load is based on a factor of safety of 1.3 applied to the shear strength of the soil when the performance goal is safety with economy. Total load is based on a factor of safety of 1.5 applied to the shear strength of the soil when the performance goal is stringent displacement control.	Hand calculations
NLFEM	Final design when performance goal is "stringent displacement control."	Nonlinear soil-structure finite element construction-sequencing analysis.	PC SOILSTRUCT-ALPHA

Table 1.4
Design and Analysis Tools for Stiff Wall Systems

Analysis	Objective	Description	Analysis Method
RIGID 1	Preliminary design tool to estimate upper anchor loads and bending moments in upper region of wall.	Beam on rigid supports analysis using apparent pressure "envelope" diagram. Apparent pressure diagram based on a total load approach. Total load is based on a factor of safety of 1.3 applied to the shear strength of the soil when the performance goal is "safety with economy." Total load is based on a factor of safety of 1.5 applied to the shear strength of the soil when the performance goal is "stringent displacement control."	Hand calculations
RIGID 2	Construction-sequencing analysis using classical soil pressures. Used to estimate lower anchor loads and bending moments in lower regions of wall.	Beam on rigid supports analysis. Soil-pressure distribution by classical methods, i.e. Rankine, Coulomb, etc. Active pressures used to determine anchor loads and wall bending moments based on a factor of safety of 1.0 applied to the shear strength of the soil when the performance goal is "safety with economy." At-rest earth pressures used to determine anchor loads and wall bending moments based on a factor of safety of 1.5 applied to the shear strength of the soil when the performance goal is "stringent displacement control." Passive pressures used to determine anchor loads and wall bending moments based on a factor of safety of 1.0 applied to the shear strength of the soil.	Hand calculations for determinate systems. CBEAMC equivalent beam analysis for indeterminate systems.
WINKLER 1	Construction-sequencing analysis to affirm results of RIGID 1 and RIGID 2 analyses.	Beam on inelastic supports analysis. Inelastic springs used to represent soil on both sides of wall. Inelastic springs used to represent anchors. R-y curves shifted to account for inelastic soil deformations.	CMULTIANC beam on inelastic supports analysis.
WINKLER 2	Construction-sequencing analysis to affirm results of RIGID 1 and RIGID 2 analyses.	Beam on inelastic supports analysis. Inelastic springs used to represent soil on excavated side of wall. Classical soil pressures applied to retained earth side of wall. Inelastic springs used to represent anchors.	CBEAMC beam on nonlinear supports analysis.

(Continued)

Table 1.4 (Concluded)			
Analysis	**Objective**	**Description**	**Analysis Method**
LEFEM	Construction-sequencing analysis to affirm results of RIGID 1 and RIGID 2 analyses and to evaluate 3-D effects and investigate loss of anchor effects. Used for cases where bending effects in the longitudinal direction are important.	Plate elements used to represent wall to capture redistribution effects in the longitudinal direction of the wall. Elastic springs used to represent soil on excavated side of wall. Classical soil pressures applied to retained earth side of wall. Elastic springs used to represent anchors.	Structural analysis software with plate element analysis capability.
NLFEM	Final design when performance goal is "stringent displacement control."	Nonlinear soil-structure finite element construction-sequencing analysis	PC SOILSTRUCT-ALPHA.

Descriptions of the analysis methods cited in the above tables and used in the example problems are provided in the following paragraphs. With respect to the WINKLER beam on inelastic spring analyses cited above, there are several methods for constructing the spring load-displacement (R-y) curves. These methods are summarized in Table 1.5 and described in the first example.

1.3 RIGID 1 Method

In the RIGID 1 method, a vertical strip of the tieback wall is treated as a multispan beam supported on rigid supports located at tieback points in the upper region of the wall. The lowermost rigid support is assumed to occur at finish grade. The wall is loaded on the driving side with an apparent pressure loading. In general practice, the use of soil-pressure envelopes as loadings for a beam on rigid support analysis provides an expedient method for the initial layout, and sometimes the final design of tieback wall systems. However, the soil-pressure envelopes, or apparent earth-pressure diagrams, were not intended to represent the real distribution of earth pressure, but instead constituted hypothetical pressures. These hypothetical pressures were a basis from which there could be calculated strut loads that might be approached but would not be exceeded during the entire construction process.

The apparent pressure loading used in the example problems is in accordance FHWA-RD-97-130. (See Figure 28 of the report for the apparent pressure diagram used for a wall supported by a single row of anchors and Figure 29 for the apparent pressure diagram used for a wall supported by multiple rows of anchors.) This information is also presented in Strom and Ebeling (2001, Figures 5.3 and 5.4).

Table 1.5
Summary of R-y Curve Construction Methods

Method	Description
Constant of Horizontal Subgrade Reaction/ Subgrade Constant	A constant of horizontal subgrade reaction method was developed by Terzaghi (1955) for use in the evaluation of discrete wall systems. A subgrade constant method was also developed for continuous walls. Interaction distances used in the analysis are per Haliburton (1971). Methods generally provide a reasonable estimate of wall moments and shears, but often overestimate displacements.
Soletanche	FHWA-RD-81-150 presents coefficients of subgrade reaction based on information obtained from pressure meter tests. Subgrade reaction values are a function of the shear parameters of the soil. Soletanche used beam on inelastic foundation analyses, based on the Pfister coefficient of subgrade reaction values, to verify that anchor loads and computed wall displacements met performance objectives.
Reference Deflection Method	Method reported in FHWA-RD-98-066 for use in beam on inelastic foundation analyses. Displacements representing the elastoplastic intersection point of the R-y curve were established for granular and clay soils. R-y curves are shifted to account for inelastic nonrecoverable displacements. These investigators indicated that the deflection response estimated by the reference deflection method generally underpredicted displacements because it does not account for mass movements in the soil.

RIGID 1 design procedures are illustrated in the example problems contained in this report and in the example problems in Section 10 of FHWA-RD-97-130. When tiebacks are prestressed to levels consistent with active pressure conditions (i.e., Example 1), the total load used to determine the apparent earth pressure is based on that approximately corresponding to a factor of safety of 1.3 on the shear strength of the soil. When tiebacks are prestressed to minimize wall displacement (Example 2), the total load used to determine the apparent earth pressure is based on at-rest conditions, or that approximately corresponding to a factor of safety of 1.5 applied to the shear strength of the soil. Empirical formulas are provided with the apparent pressure method for use in estimating anchor forces and wall bending moments.

1.4 RIGID 2 Method

As with the RIGID 1 method, a vertical strip of the tieback wall is treated as a multispan beam supported on rigid supports located at tieback points. The lowest support location is assumed to be below the bottom of the excavation at the point of zero net pressure (Ratay 1996). Two earth-pressure diagrams are used in each of the incremental excavation, anchor placement, and prestressing analyses. Active earth pressure (or at-rest earth pressure when wall displacements are critical) is applied to the driving side and extends from the top of the ground to the actual bottom of the wall. Passive earth pressure (based on a factor of safety of 1.0 applied to the shear strength of soil) is applied to the resisting side of the wall and extends from the bottom of the excavation to the actual bottom of the wall. The application of the RIGID 2 method is demonstrated in the two example problems. The RIGID 2 method is useful for determining if the wall and anchor capacities determined by the RIGID 1 analysis are adequate for stiff tieback wall systems, and permits redesign of both flexible and stiff tieback wall systems to ensure that strength is adequate for all stages of construction. No useful information can be obtained from the RIGID 2 analysis regarding displacement demands, however.

1.5 WINKLER 1 Method

The WINKLER 1 method uses idealized elastoplastic springs to represent soil load-deformation response and anchor springs to represent ground anchor load-deformation response. The elastoplastic curves (R-y curves) representing the soil springs for the example problems are based on the reference deflection method (FHWA-RD-98-066). Other methods are available for developing elastoplastic R-y curves for beam on inelastic foundation analyses. The reference deflection method (FHWA-RD-98-066), the Haliburton (1971) method, and the Pfister method (FHWA-RD-81-150) are described in the first example problem. Elastoplastic curves can be shifted with respect to the undeflected position of the tieback wall to capture nonrecoverable plastic movements that may occur in the soil during various construction stages (e.g., excavating, anchor placement, and prestressing of anchors). This R-y curve shifting was used in both example problems to consider the nonrecoverable active state yielding that occurs in the retained soil during the first-stage excavation (cantilever-stage excavation). The R-y curve shift following the first-stage excavation will help to capture the increase in earth pressure that occurs behind the wall as anchor prestress is applied, and as second-stage excavation takes place. In the two example problems, once the upper anchor is installed, the second-stage excavation causes the upper section of the tieback wall to deflect into the retained soil—soil that has previously experienced active state yielding during first-stage excavation. The WINKLER 1 method is useful for determining if the wall and anchor capacities determined by a RIGID 1 or RIGID 2 analysis are adequate, and permits redesign of stiff tieback wall systems to ensure that strength is adequate for all stages of construction. It also provides useful information on “relative” displacement demands and facilitates redesign of the wall system when it becomes necessary to meet displacement-based performance objectives.[3]

[3] At this time, the authors of this report do not propose to use WINKLER inelastic spring-based methods of analyses to predict wall displacements. However, the differences in the computed deformations of an altered wall system based on WINKLER analyses may be useful as a qualitative assessment of change in stiffness effects.

1.6 WINKLER 2 Method

The WINKLER 2 method is a simple beam on inelastic foundation method that uses soil loadings on the driving side of the wall and elastoplastic soil springs on the resisting side of the wall in an incremental excavation, anchor placement, and anchor prestressing analysis. As with the WINKLER 1 method, the elastoplastic curves representing the soil springs are based on the reference deflection method, and anchor springs are used to represent the ground anchor load-deformation response. However, the WINKLER 2 method is unable to capture the effects of nonrecoverable plastic movements that may occur in the soil during various construction stages. Although not considered to be as reliable as the WINKLER 1 method, the WINKLER 2 method is useful for determining if the wall and anchor capacities determined by a RIGID 1 or RIGID 2 analysis are adequate, and the method permits redesign of stiff tieback wall systems to ensure that strength is adequate for all stages of construction. It also provides information on relative displacement demands (i.e., the effects of system alterations described in terms of changes in computed displacements) and permits redesign of the wall system to meet stringent displacement control performance objectives.

1.7 NLFEM Method

When displacements are important with respect to project performance objectives, a nonlinear finite element soil-structure interaction (SSI) analysis should be performed. In an NLFEM analysis, soil material nonlinearities are considered. Displacements are often of interest when displacement control is required to prevent damage to structures and utilities adjacent to the excavation. To keep displacements within acceptable limits, it may be necessary to increase the level of prestressing beyond that required for basic strength performance. An increase in tieback prestressing is often accompanied by a reduction in tieback spacing. As tieback prestressing is increased, wall lateral movements and ground surface settlements decrease. Associated with an increased level of prestress is an increase in soil pressures. The higher soil pressures increase demands on the structural components of the tieback wall system. General-purpose NLFEM programs for two-dimensional plane strain analyses of SSI problems are available to assess displacement demands on tieback wall systems. These programs can calculate displacements and stresses due to incremental construction and/or load application and are capable of modeling nonlinear stress-strain material behavior. An accurate representation of the nonlinear stress/strain behavior of the soil, as well as proper simulation of the actual (incremental) construction process (excavation, anchor installation, anchor prestress, etc.), in the finite element model is essential if this type of analysis is to provide meaningful results. See Strom and Ebeling (2001) for additional details regarding nonlinear SSI computer programs for displacement prediction.

1.8 Factors Affecting Analysis Methods and Results

1.8.1 Overexcavation

Overexcavation below ground anchor support locations is required to provide space for equipment used to install the ground anchors. It is imperative that the specified construction sequence and excavation methods are adhered to and that overexcavation below the elevation of each anchor is limited to a maximum of 2 ft. Construction inspection requirements in FHWA-SA-99-015 require inspectors to ensure that overexcavation below the elevation of each anchor is limited to 2 ft, or as defined in the specifications. In the Bonneville temporary tieback wall example, an overexcavation of 5.5 ft was considered for the initial design. This should be a "red flag" to the designer that a construction-sequencing evaluation is needed, and that such an evaluation will likely demonstrate that the maximum force demands on the wall and tiebacks will occur during intermediate stages of construction rather than for the final permanent loading condition. For additional information on the effect of overexcavation on tieback wall performance, see Yoo (2001).

1.8.2 Ground anchor preloading

Unless anchored walls are prestressed to specific active stress levels and their movement is consistent with the requirements of the active condition at each construction stage, the lateral earth-pressure distribution will be essentially nonlinear with depth, and largely determined by the interaction of local factors. These may include soil type, degree of fixity or restraint at the top and bottom, wall stiffness, special loads, and construction procedures (Xanthakos 1991). To ensure that ground anchor prestressing is consistent with active state conditions, the designer will generally limit anchor prestress to values that are between 70 and 80 percent of those determined using an equivalent beam on rigid supports analysis based on apparent pressure loadings (FHWA-RD-81-150). However, this may produce wall movements toward the excavation that are larger than tolerable, especially in cases where structures critical to settlement are founded adjacent to the excavation. Larger anchor prestressed loads are generally used when structures critical to settlement are founded adjacent to the excavation. Selection of an arbitrary prestress load can be avoided by using the WINKLER 1 method beam on inelastic foundation analysis described above. This type of analysis permits the designer to relate wall movement to anchor prestress and/or anchor spacing in order to produce tieback wall performance that is consistent with displacement performance objectives.

1.9 Construction Long-Term, Construction Short-Term, and Postconstruction Conditions

For a free-draining granular backfill, the pore water pressure does not usually include excess pore water pressures generated in the soil by changes in the total stress regime due to construction activities (excavation, etc.). This is because the rate of construction is much slower that the ability of a pervious and free-draining granular soil to rapidly dissipate construction-induced excess pore water pressures.

However, for sites containing soils of low permeability (soils that drain slower than the rate of excavation/construction), the total pore water pressures will not have the time to reach a steady-state condition during the construction period. In these types of slow-draining, less permeable soils (often referred to as “cohesive soils”), the shear strength of the soil during wall construction is often characterized in terms of its undrained shear strength. The horizontal earth pressures are often computed using values of the undrained shear strength for these types of soils, especially during the short-term, construction loading condition (sometimes designated as the undrained loading condition—where the term undrained pertains to the state within the soil during this stage of loading).

As time progresses, however, walls retained in these types of soils can undergo two other stages of construction loading: the construction long-term (drained or partially drained) condition and the postconstruction/permanent (drained) condition. Under certain circumstances, earth pressures may be computed in poorly drained soils using the Mohr-Coulomb (effective stress-based) shear strength parameter values for the latter load case(s).

Liao and Neff (1990), along with others, point out that all three stages of loading must be considered when designing tieback wall systems, regardless of soil type. As stated previously, for granular soils, the construction short- and long-term conditions are usually synonymous since drainage in these soils occurs rapidly. Differences in the construction short- and long-term conditions are generally significant only for cohesive soils. Changes in the groundwater level (if present) before and after anchor wall construction, as well as postconstruction/permanent, must be considered in these evaluations. Designers must work closely with geotechnical engineers to develop a soils testing program that will produce soil strength parameters representative of each condition—construction short term, construction long term, and postconstruction. The program should address both laboratory and field testing requirements. Additional information on construction short-term, construction long-term, and postconstruction condition earth-pressure loadings can be found in Strom and Ebeling (2001). Methods used to estimate long-term (drained) shear strength parameters for stiff clay sites are presented in Appendix A of this report.

1.10 Construction-Sequencing Analyses

1.10.1 General

Tieback wall design procedures vary in practice, depending on whether the tieback wall is considered to be flexible or stiff. Flexible wall systems include

a. Vertical sheet-pile systems.

b. Soldier beam and lagging systems.

As stated previously, flexible wall systems are often designed using an equivalent beam on rigid support method of analysis with an apparent earth-pressure envelope loading. The flexible wall system design approach is illustrated herein with respect to the two stiff tieback wall examples in order to be able to compare the results with those obtained using construction-sequencing type analyses. The flexible wall design process is also illustrated in Ebeling, Azene, and Strom (2002).

Stiff tieback wall systems include:

a. Secant cylinder pile systems.

b. Continuous reinforced concrete (tremie wall) systems.

c. Soldier beam–tremie wall systems.

In practice, the stiff tieback wall systems employ some type of construction-sequencing analysis, i.e. staging analysis, in which the anchor loads, wall bending moments, and possibly wall deflections are determined for each construction stage. In general, designers recommend against application of the apparent pressure diagram approach, used for flexible tieback wall systems, for the design of stiff tieback wall systems (Kerr and Tamaro 1990). Equivalent beam on rigid support methods and beam on inelastic foundation methods are those methods most commonly used in the construction-sequencing analysis. Classical earth pressure theories (Rankine, Coulomb, etc.) are generally used in the equivalent beam on rigid support method. Profiles of lateral earth pressures on both sides of the wall are developed by classical theory with active pressures acting on the driving side and passive pressures acting on the resisting side. An at-rest pressure profile may be used to represent driving side earth pressures for stiff wall systems that are required to meet stringent displacement performance objectives. The beam on inelastic foundation method allows displacement performance to be assessed directly (in a relative but ***not*** an absolute sense). It is therefore preferred over the equivalent beam on rigid support method for tieback wall systems where displacement performance is critical. Both the equivalent beam on rigid support method and the beam on inelastic foundation method are demonstrated in a construction-sequencing analysis with respect to the design and evaluation of two stiff tieback wall systems.

1.10.2 Example problems

The design and evaluation of stiff tieback wall systems is illustrated by two example problems. Both wall systems are continuous reinforced concrete (tremie wall) systems.

The first example, identified as the "Soletanche tieback wall example," is a continuous reinforced concrete tremie wall system with a single row of tieback anchors. The example is taken from FHWA-RD-81-150. Results obtained from the equivalent beam on rigid support method and the beam on inelastic foundation method are compared with the results obtained from similar analyses performed by Soletanche and presented in the FHWA report. Results are also compared with those that were obtained by the apparent pressure diagram method. It is important to note that tiebacks for the Soletanche wall were prestressed at low levels to ensure active state conditions in the retained soil. (This would not be the practice for the Corps in a stringent displacement control design situation.). Low prestress levels minimize driving side earth pressures and thereby minimize construction costs.

The second example, identified as the "Bonneville temporary tieback wall example," is a continuous reinforced concrete tremie wall system with four rows of tieback anchors. Results obtained from the equivalent beam on rigid support method and the beam on inelastic foundation method are compared with the results obtained from finite element studies and field measurements. Results are also compared with those that were obtained by the apparent pressure diagram method. The tiebacks for the Bonneville temporary tieback wall were prestressed to minimize wall displacements. At-rest pressures were therefore used on the driving side of the wall for the equivalent beam on rigid support analysis and were used as the basis for constructing apparent pressure diagrams.

Additional details of the two wall systems are provided in the example problems presented in Chapters 2 and 3.

1.11 Research and Development Needs

The design methodologies described herein with respect to flexible and stiff tieback wall systems assume that wall movements are consistent with either the earth pressures assumed for design or are consistent with the earth pressure-displacement response assumed for the analysis. These assumptions are applied to the final excavation stage and to each construction stage analysis. In fact, lateral earth pressures will be essentially nonlinear and dependent on many factors, including soil type, wall fixity and restraint, factors of safety, tieback size and spacing, tieback prestress levels, construction sequencing, overexcavation at anchor locations, and wall performance requirements. Even though the earth pressures used in the simple analysis procedures assume a specified displacement response, it is well known that it is impossible at this time to reasonably predict wall displacements using simplified analysis procedures.

Additional research using nonlinear SSI finite element analyses is needed to validate the use of the design and analysis tools illustrated in this report and the example problems. The research should be directed toward validating the simple design procedures used herein as suitable tools for designing anchors and for estimating wall moment demands. In addition, the research should determine if there are simple analysis procedures that can be used to predict the displacement response for those “Corps of Engineers-type” walls that must meet stringent displacement performance objectives.

2 Example 1 — Soletanche Wall Example

2.1 Introduction

This design example illustrates the various design approaches that can be used with respect to a stiff tieback wall system with a single tieback anchor. It has application with respect to the Corps secant pile and soldier beam-tremie concrete system described in Strom and Ebeling (2001). This particular example was taken from FHWA-RD-81-150. It is identified as the "Soletanche Wall Example" and represents a continuous concrete slurry wall system with wall thickness equal to 1.15 ft and anchors spaced at 8.2 ft on center (OC). Additional details concerning this example can be found in Chapter 6 of FHWA-RD-81-150, Chapter 6 ("Illustration of the design procedure, I-75 Atlanta, Georgia; Wall section A"). The Soletanche wall example is illustrated in Figure 2.1.

Analyses for the Soletanche wall example were based on the following soil strength properties:

$\phi = 35$ deg

$c = 0$

$\gamma = 115$ pcf

The earth pressures were calculated per FHWA-RD-81-150, according to the Caquot-Kerisel (1973) method with the wall friction δ equal to zero for the active earth pressure ($K_a = 0.271$) and equal to -2/3 ϕ for the passive earth pressure ($K_p = 7.346$). A general surcharge of 405 psf was considered in the Soletanche analysis.

The Soletanche design practice as presented in FHWA-RD-81-150 is to initially determine anchor size and wall dimensions using a construction-sequencing analysis based on "classical" methods (identified in the FHWA report as a PEROI 1 analysis). The PEROI 1 analysis is similar to the Rigid 2 method described in Chapter 1. The PEROI 1 analysis is followed by a "Winkler spring"-type analysis. The Winkler spring analysis is used for final evaluation of the wall

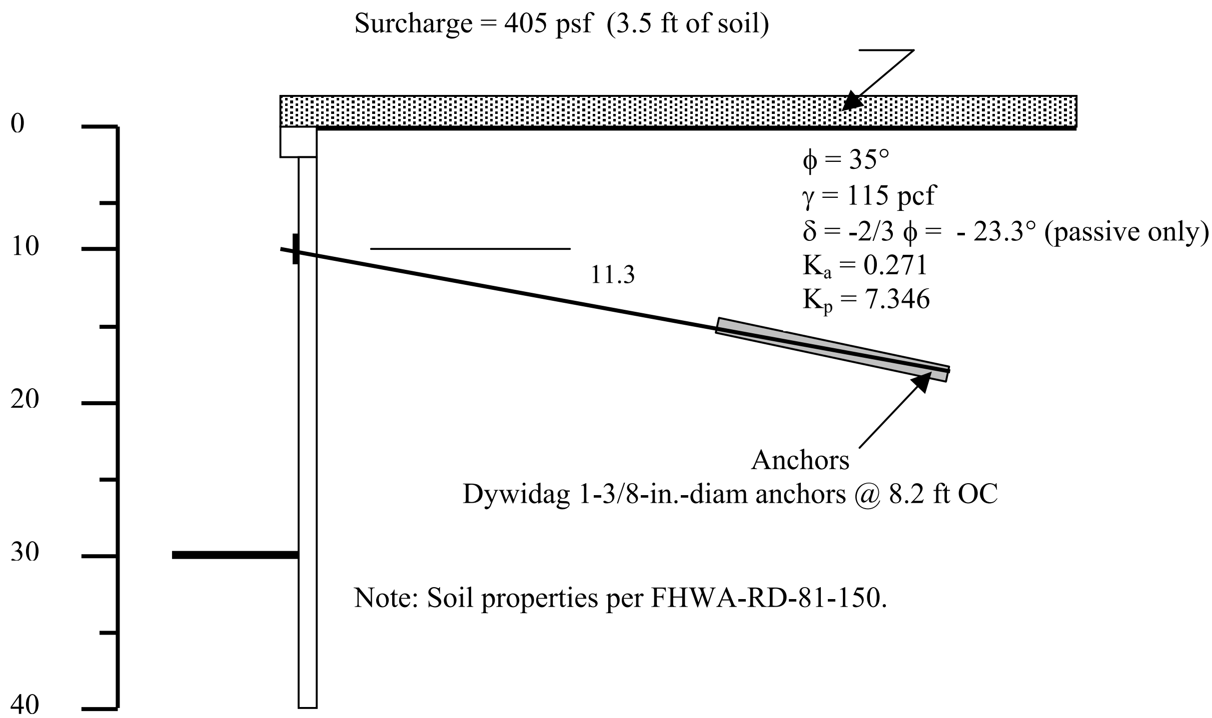


Figure 2.1. Soletanche wall--elevation at final excavation stage

design and for design improvements. Construction-sequencing analysis using Winkler spring methods is identified in FHWA-RD-81-150 as a PEROI 2 analysis. Except for differences in the methods used to develop the elastoplastic soil springs, the PEROI 2 analysis is similar to the Winkler 1 method described in Chapter1. Elastoplastic springs in the PEROI 2 analysis are based on the Pfister method; in the Winkler 1 analysis, these are based on the reference deflection method. (Both methods are described in Chapter 1. The Winkler 1 analyses were performed using a newly developed construction-sequencing computer program entitled CMULTIANC (Dawkins, Strom, and Ebeling, in preparation). This program shifts the soil-structure interaction curves (R-y curves) to account for any plastic movement that may occur in the soil. It is based on the reference deflection method (FHWA-RD-98-066).

The Rigid 2 and Winkler 2 analyses in this report were performed using the Corps' CBEAMC computer software (Dawkins 1994b). The Rigid 2 analyses for a single-anchor tieback wall are statically determinate analyses that can be performed using hand calculations without performing a CBEAMC analysis. However, a CBEAMC analysis was selected because of the additional information it provides with respect to displacements, shears, and moments along the length of the wall. Results for the initial, intermediate, and final construction stages were obtained for each of the three construction-sequencing analyses (Rigid 1, Winkler 1, and Winkler 2).

Results are compared with the Soletanche PEROI 1 and PEROI 2 design results, and differences are discussed. In addition to the three construction-sequencing analyses, an apparent pressure final construction-type analysis (Rigid 1) was also performed. In summary, the following design and evaluation approaches were used:

a. A final excavation-stage analysis using the apparent pressure diagram method (Rigid 1 method).

b. A construction-sequencing analysis, using the "classical" or beam on rigid supports method (Rigid 2 method).

c. A construction-sequencing analysis using Winkler spring analysis methods in combination with soil-structure interaction curve-shifting to account for plastic movement in the soil (Winkler 1–CMULTIANC analyses). Soil-structure interaction curves (soil springs) are used on both the excavated and retained soil sides of the wall.

d. A -sequencing analysis using Winkler spring analysis methods in combination with classical earth pressures (Winkler 2–CBEAMC analyses). Classical earth pressures are applied to the retained soil side of the wall. Soil springs are used on the excavated side of the wall (soil springs located on the front side of the wall and below the excavation).

The soil springs used in the CMULTIANC and CBEAMC analyses were in accordance with the reference deflection method described in FHWA-RD-98-066. In the Winkler 1 analysis, soil springs were used to determine both driving-side and resisting-side earth pressures. In the Winkler 2 analysis, soil springs were used to determine resisting-side earth pressures only, with driving-side pressures applied as loads. This approach is in accordance with SEI/ASCE (2000). The various design analyses used for this example problem are summarized in Table 2.1.

2.2 Tieback Wall Behavior

Excavation begins following placement of the continuous tremie concrete wall by slurry trench methods. The wall is evaluated for three stages of construction:

a. Initial excavation to a depth of 11.5 ft below the ground surface (Stage 1).

b. Placement and prestressing of the tieback anchor at a location of 9.8 ft below the ground surface (Stage 2).

c. Excavation to final grade at a depth of 28.87 ft below the ground surface (Stage 3).

Table 2.1
Analysis Method Summary

Analysis Method	Reference	Description	Solution Method
Soletanche PEROI 1	FHWA-RD-81-150	Classical "Fixed Earth," "Free Earth," and "Equivalent Beam" analyses for first-stage cantilever condition and final-stage anchored wall condition.	* Hand calculations or PEROI 1 software
Soletanche PEROI 2	FHWA-RD-81-150	Soil spring (Winkler) analysis with earth pressure-deflection relationships based on soil shear strength in accordance with FHWA- RD-81-150	PEROI 2 software
Rigid 1 Analysis	Section 2.4	Apparent pressures based on a "total load" approach in accordance with Terzaghi, Peck, and Mesri (1996) and as modified by FHWA-RD-97-130.	Hand calculations
Rigid 2 Analysis	Section 2.5	Classical earth pressure theories used in a construction-sequencing analysis, assuming beam on rigid supports conditions.	Hand calculations or CBEAMC*
Winkler 1 Analysis	Section 2.6	Soil spring (Winkler) analysis with earth pressure-deflection relationships based on the "reference deflection" method per FHWA-RD-98-066.	CMULTIANC**
Winkler 2 Analy sis	Section 2.7	Combination analysis with classical earth pressures on driving side and soil springs on resisting side per SEI/ASCE (2000).	CBEAMC

* For a wall with a single tieback anchor, the solution is statically determinate; can be solved easily by hand calculations.
** Program currently under development.

The computed deflected shapes and wall bending moments for the three stages of construction are illustrated in Figure 2.2. Recall that the deflections computed in a Winkler spring-type analysis are not viewed by the authors of this report to provide an accurate representation of the actual wall displacements that will be encountered in the field.

2.2.1 Stage 1 construction

Comparisons of the maximum wall moments for each method of analysis are provided in Table 2.2. During first-stage excavation, the upper section of the wall moves toward the excavation. Movements are assumed sufficient to develop active pressure conditions behind the wall. All the "classical" methods of analysis provide similar results with respect to the maximum wall bending moment, M_{A1} (see Figure 2.2). The subscript A refers to the maximum moment in the upper region of the wall in the vicinity of the tieback anchor. The subscript 1 refers to the first stage of construction, as defined above. Since the "apparent pressure"

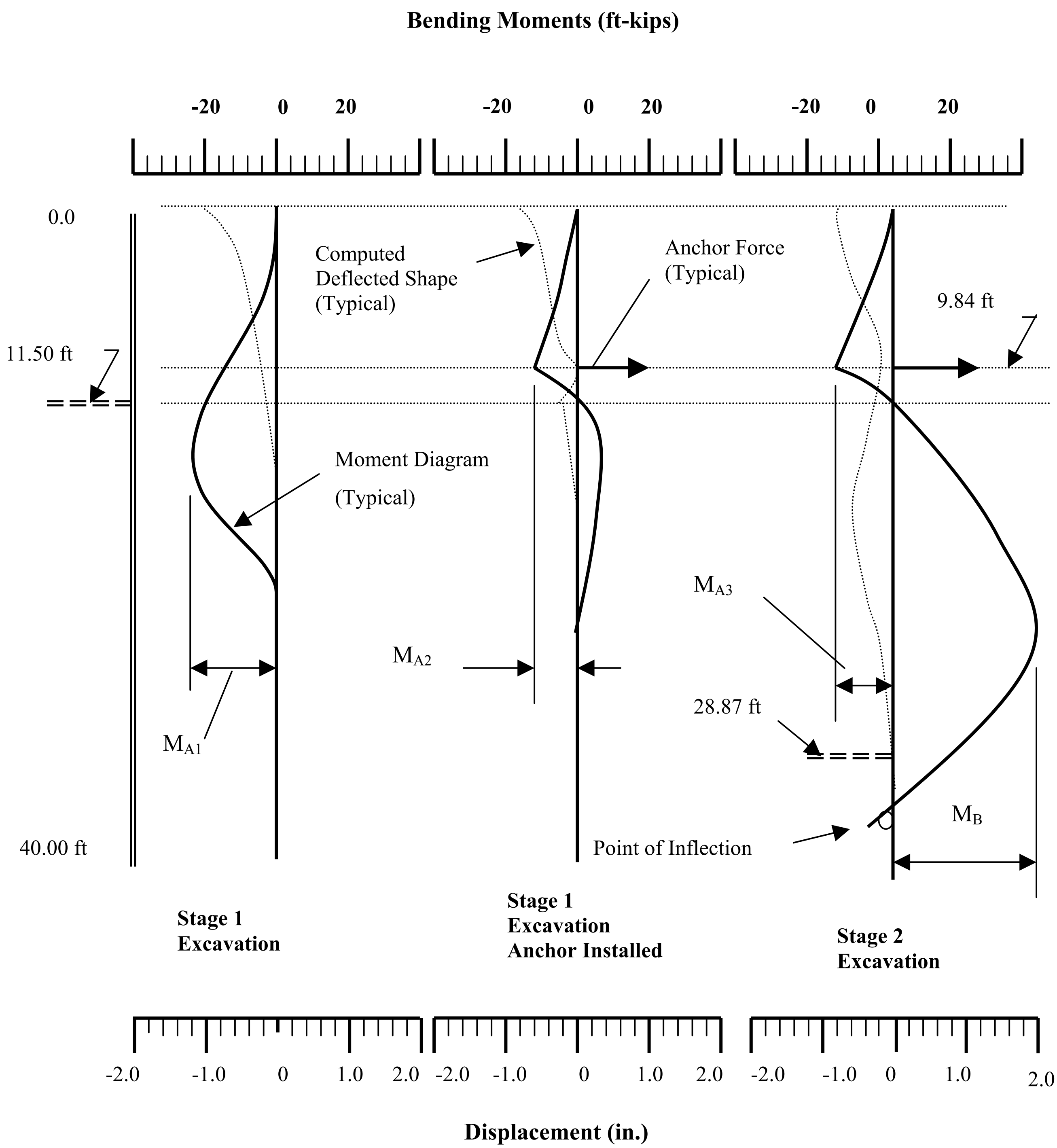


Figure 2.2. Wall construction sequencing behavior

Table 2.2 **First-Stage Construction Analysis Summary**		
Analysis Method	**Maximum Moment Per Foot of Wall M_{A1} (ft-kips)**	**Description**
Soletanche PEROI 1	-23.8	Classical "Fixed Earth" analyses for first-stage cantilever condition.
Soletanche PEROI 2	-29.3	Soil spring (Winkler) analysis with earth pressure-deflection relationships based on soil shear strength in accordance with FHWA-RD-81-150.
Rigid 1	-28.6	Apparent pressures based on a "total load" approach in accordance with Terzaghi, Peck, and Mesri (1996) and as modified by FHWA-RD-97-130.
Rigid 2	-23.5	Classical earth pressure theories using "free" earth assumptions.
Winkler 1	-27.0	Soil spring (Winkler) analysis with earth pressure-deflection relationships based on the "reference deflection" method per FHWA-RD-98-066.
Winkler 2	-33.9	Combination analysis with classical earth pressures on driving side and soil springs on resisting side per SEI/ASCE (2000).

method of analysis is intended to approximate maximum conditions for all stages of construction, the M_{A1} moment does not necessarily represent the first stage of construction.

2.2.2 Stage 2 construction

During Stage 2, the tieback anchor is installed and prestressed. Tieback anchors are usually prestressed to limit wall movement. The level of prestress used by designers is quite variable. The higher the prestress the less the wall movement, especially near the top of the wall. If structures sensitive to settlement are founded adjacent to the excavation, a large prestress loading corresponding to the full apparent pressure diagram (Terzaghi et al. 1996), or about 1.8 times the active pressure distribution, will often be used to limit soil movements (FHWA-RD-81-150). However, in most situations, some wall movement can be tolerated and a smaller prestress load is used. For the Soletanche design, an anchor lock-off load of 8.0 kips per foot of wall was selected. The lock-off prestress is equal to about 85 percent of that required for active pressure conditions as determined by the PERROI 1 analysis. Anchor prestress influences soil pressures behind the wall. In the classical methods of analysis and the SEI/ASCE Winkler spring analysis, active pressures are assumed to exist behind the wall. Pressures higher than active in the anchor region will tend to increase the M_{A3} moments in the upper section of the wall. The actual influence can be best estimated using a Winkler spring analysis in which the soil pressures on both the driving side and resisting side are determined using soil pressure-displacement relationships. Therefore, a Winkler spring analysis is often performed to validate a preliminary design based on either classical, or apparent pressure, methods of analysis. This is the approach used for the Soletanche for the Soletanche example, as described

Table 2.3 Intermediate-Stage Construction Analysis Summary		
Analysis Method	**Maximum Moment Per Foot of Wall M_{A2} (ft-kips)**	**Description**
Soletanche PEROI 2	-26.9	Soil spring (Winkler) analysis with earth pressure-deflection relationships based on soil shear strength in accordance with FHWA-RD-81-150.
Winkler 1	-23.9	Soil spring (Winkler) analysis with earth pressure-deflection relationships based on the “reference deflection” method per FHWA-RD-98-066.

in FHWA-RD-81-150. The M_{A2} moments for the intermediate stage of construction where the anchors are installed and prestressed as determined by the Soletanche PERROI 2 and Winkler 1 analyses are provided in Table 2.3.

2.2.3 Stage 3 construction

Comparisons of the maximum M_{A3} and M_B moments for final construction stage for each method of analysis are provided in Table 2.4. (See Figure 2.2 for a description of the moments M_{A3} and M_B.) During the final stage of excavation, the upper section of the wall moves toward the retained soil and the lower section of wall moves toward the excavation. The amount of the movement will depend on the earth pressure-displacement relationship used in the analysis.

Table 2.4 Final Construction Stage Analysis Summary			
Analysis Method	**Maximum Moment Per Foot of Wall (ft-kips)**		**Description**
	M_{A3}	M_B	
Soletanche PEROI 1	-10.3	33.0	Classical earth pressure theories.
Soletanche PEROI 2	-32.1	27.0	Soil spring (Winkler) analysis with earth pressure-deflection relationships based on soil shear strength in accordance with FHWA-RD-81-150.
Rigid 1	-28.6	23.1	Apparent pressures based on a “total load” approach in accordance with Terzaghi, Peck, and Mesri (1996) and as modified by FHWA-RD-97-130.
Rigid 2	-10.3	33.1	Classical earth pressure theories.
Winkler 1	-31.1	26.7	Soil spring (Winkler) analysis with earth pressure-deflection relationships based on the “reference deflection” method per FHWA-RD-98-066.
Winkler 2	-10.3	52.6	Combination analysis with classical earth pressures on driving side and soil springs on resisting side per SEI/ASCE (2000).

2.3 Elastoplastic Earth Pressure-Deflection Relationships

As can be seen from Table 2.4, the results obtained from the Winkler analyses (i.e. PEROI 2 and Winkler 1 analyses), the moment demands on the walls are somewhat dependent on the elastoplastic earth pressure-deflection relationship used. The Soletanche PEROI 2 analysis used earth pressure-deflection relationships based on soil shear strength in accordance with FHWA-RD-81-150. The Winkler 1 analysis of this report used the reference deflection method of FHWA-RD-98-066 to develop earth pressure-deflection relationships. According to Strom and Ebeling (2001), various elastoplastic earth pressure-displacement relationships are used in Winkler analyses, including

a. The Haliburton (1981) earth pressure-displacement relationship.

b. The FHWA-RD-81-150 earth pressure-displacement relationship.

c. The FHWA-RD-97-130 reference deflection method.

These three earth pressure-displacement relationships are illustrated below with respect to an effective vertical soil pressure of 1,000 psf (soil depth ≈ 8.7 ft). The plateaus representing the active and passive limit states on the elastoplastic earth pressure-displacement relationship for each of the three relationships are identical and equal to

$$p_a = k_a \sigma'_v = 0.271(1.000) = 0.271\, ksf$$

where k_a is the active pressure coefficient assuming a wall friction angle, δ, equal to zero.

$$p_p = k_p \sigma'_v = 7.346(1.000) = 7.346\, ksf$$

and where k_p is the passive pressure coefficient assuming a wall friction angle, δ, equal to –2/3 ϕ.

The at-rest pressure is equal to

$$p_0 = k_0 \sigma'_v = 0.430(1.000) = 0.430\, ksf$$

where k_0 is equal to 1 – sin ϕ (Jaky 1944).

The displacements required to reach the active and passive plateaus of the elastoplastic earth pressure-displacement curve are different, however. The calculations for these are provided below.

2.3.1 Haliburton method

In the Haliburton method, the displacement, Δ_a, required to reach active pressure is equal to

$$\Delta_a = \frac{(p_o - p_a)}{k_h}$$

And to reach passive pressure,

$$\Delta_p = \frac{(p_p - p_o)}{k_h}$$

For continuous walls, the coefficient of horizontal subgrade reaction, k_h (per Terzaghi 1955), is

$$k_h = l_h \frac{z}{D}$$

where

D = effective contact dimension per Haliburton (1971) [see Figure 6.4, Strom and Ebeling 2001, for a pictorial description], which is

= H_a = 9.8 ft [above an anchor depth of 9.8 ft] or

= H + d – H_a [below an anchor depth of 9.8 ft]

l_h = constant of horizontal subgrade reaction = 9 pci = 15.55 ksf (see Table 6.2, Strom and Ebeling 2001, medium dense sand)

z = 8.7 ft (for effective vertical soil pressure of 1,000 psf) (see Figure 6.5, Strom and Ebeling 2001, for a pictorial description of z)

With z = 8.7 ft, which is above the depth of anchor (H_a), or 9.8 ft, the effective contact dimension (D) is 9.8 ft. The horizontal subgrade reaction (k_h) is determined as follows:

$$k_h = l_h \frac{z}{D} = 15.55 \frac{8.7}{9.8} = 13.80 \text{ kcf}$$

$$\Delta_a = \frac{(p_o - p_a)}{k_h} = \frac{(0.430 - 0.217)}{13.80} = 0.0156 \text{ ft} = 0.19 \text{ in.}$$

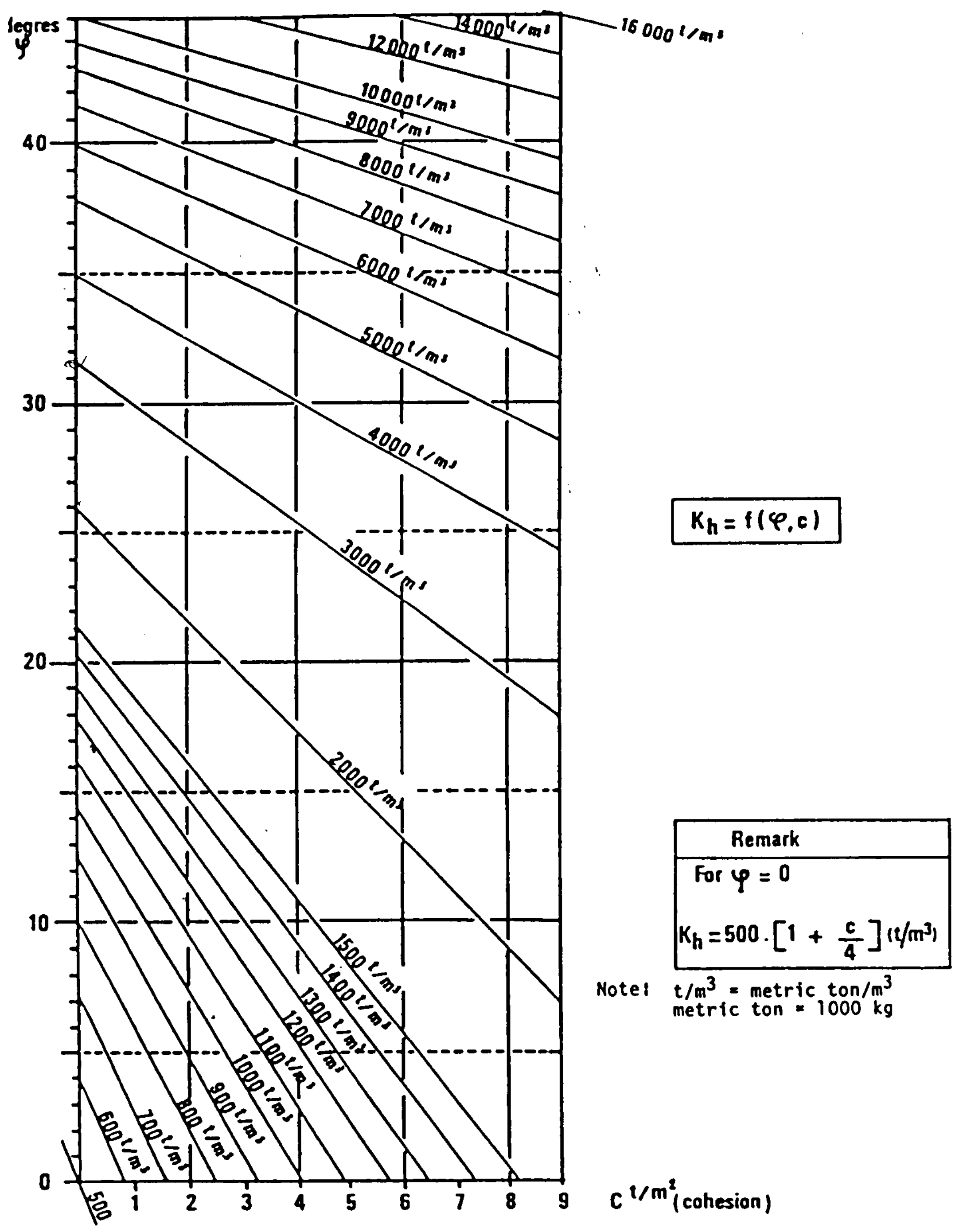


Figure 2.3. Horizontal subgrade moduli, k_h (FHWA-RD-81-150)

$$\Delta_p = \frac{(p_p - p_o)}{k_h} = \frac{(7.346 - 0.430)}{13.80} = 0.50 \text{ ft} = 6.0 \text{ in.}$$

As will be seen, the coefficient of subgrade reaction determined by the Haliburton (1971) method using Terzaghi (1955) constants of subgrade reaction will produce soil springs that are considerably softer than those determined by the FHWA-RD-81-150 (Pfister) method or the FHWA-RD-99-066 (reference deflection) method, described below. This results in somewhat higher wall moment demands and much higher wall deflections.

2.3.2 Pfister method (FHWA-RD-81-150)

In the Soletanche PAROT 2 analysis, the value for the subgrade reaction is a function of the soil strength. Relationships between soil strength and horizontal subgrade reaction for stiff continuous diaphragm walls were developed by Pfister and others and are presented in FHWA-RD-81-150 (shown in Figure 2.3). Soletanche encourages field testing to obtain coefficients of horizontal subgrade reaction. Therefore, the information contained in Figure 2.3 is often modified by Soletanche to be consistent with field test data obtained for similar sites. In this example, the coefficient of subgrade reaction was selected by Soletanche to be equal to 2,000 t/m^3 (125 k/ft^3) at an effective vertical soil pressure of 0 psf. Figure 2.3 indicates a k_h-value of 4,000 t/m^3 (250 k/ft^3); thus, judgment was applied by Soletanche to the parameters used in this analysis.

The coefficient of subgrade reaction was selected by Soletanche to increase at a rate of 200 t/m^3 (12.5 k/ft^3) for each increase in effective vertical soil pressure of 1 t/m^2 (0.207 k/ft^2). Therefore, at an effective vertical soil pressure of 1,000 psf, the coefficient of subgrade reaction, k_h, is 125 + 12.5 (1/0.207) = 185 k/ft^3.

The displacements for the start of the active pressure and passive pressure plateaus are

$$\Delta_a = \frac{(p_o - p_a)}{k_h} = \frac{(0.430 - 0.271)}{185} = 0.0009 \text{ ft} = 0.0103 \text{ in.}$$

$$\Delta_p = \frac{(p_p - p_o)}{k_h} = \frac{(7.346 - 0.430)}{185} = 0.0374 \text{ ft} = 0.4488 \text{ in.}$$

When compared with the reference deflection method, the use of a coefficient of subgrade reaction equal to 125 k/ft^3 at an effective vertical soil pressure of 0 psf will result in higher passive resistances near the ground surface. For the first-, intermediate-, and final-stage construction, this will move the point of contraflexure closer to the ground surface, thereby reducing the M_{A1}, M_{A2}, M_{A3}, and M_B bending moment demands.

2.3.3 Reference deflection method

In the reference deflection method, an active reference deflection of 0.05 in. and a passive reference deflection of 0.5 in. are used to develop elastoplastic earth pressure-deflection relationships for cohesionless soils (FHWA-RD-97-130). These deflections are constant with effective vertical soil pressure. Therefore, at an effective vertical soil pressure of 1,000 psf, the average coefficient of subgrade reaction, k_h, is equal to

$$k_h = \frac{7.346 - 0.217}{(0.50 + 0.05)\frac{1}{12}} = 155 \ \text{k/ft}^3$$

The reference deflections used in establishing the elastoplastic earth pressure-deflection relationships for cohesionless soils were based on measurements obtained from full-scale testing of anchored soldier beam walls (Texas A&M full-scale wall tests, FHWA-RD-97-130). Reference deflections for cohesionless soils are presented in FHWA-RD-97-130 (Section 3.3.2.1). These are considered suitable for use in the soil-structure interaction analysis of anchored soldier beam and continuous diaphragm walls in sand. The reference deflections for anchored soldier beam and continuous diaphragm walls in clay were assumed and verified by comparing predicted behavior with case-history results. The reference deflections for clay are presented in Table 23 of FHWA-RD-97-130.

The following sections provide the calculations and computer analyses used to obtain the force and displacement demands for those analyses other than the Soletanche. The Soletanche analyses can be found in FHWA-RD-81-150.

2.4 Rigid 1 Analysis

The following Rigid 1 analysis (equivalent beam on rigid supports with apparent pressure loading) was performed in accordance with FHWA-RD-97-130 for the Soletanche wall. The Rigid 1 analysis is a final construction-stage analysis that is commonly used for the evaluation of flexible tieback wall systems. The Soletanche wall example is described in Section 2.1. The apparent earth pressure diagram is illustrated in Figure 2.4. The total earth pressure load is equal to an earth pressure factor (EPF) times the square of the wall height. Earth pressure factors for various soils are grouped in a narrow range from 20 to 24 pcf (see Table 8, FHWA-RD-97-130). These EPFs include a factor of safety of about 1.3 on the shear strength of the soil. A value of 23 pcf was selected for the apparent pressure analysis. Calculations for the earth pressure (p_e), the anchor load (T_1), the base reaction (R_B), and the wall maximum bending moments for a wall without surcharge loading are as indicated in Figure 2.5. M_1 is the maximum negative moment occurring at the anchor location. MM_1 is the maximum positive moment occurring at a point of zero shear that is located a distance, x, above the wall toe. Symbols used in these calculations are in accordance with FHWA-RD-97-130.

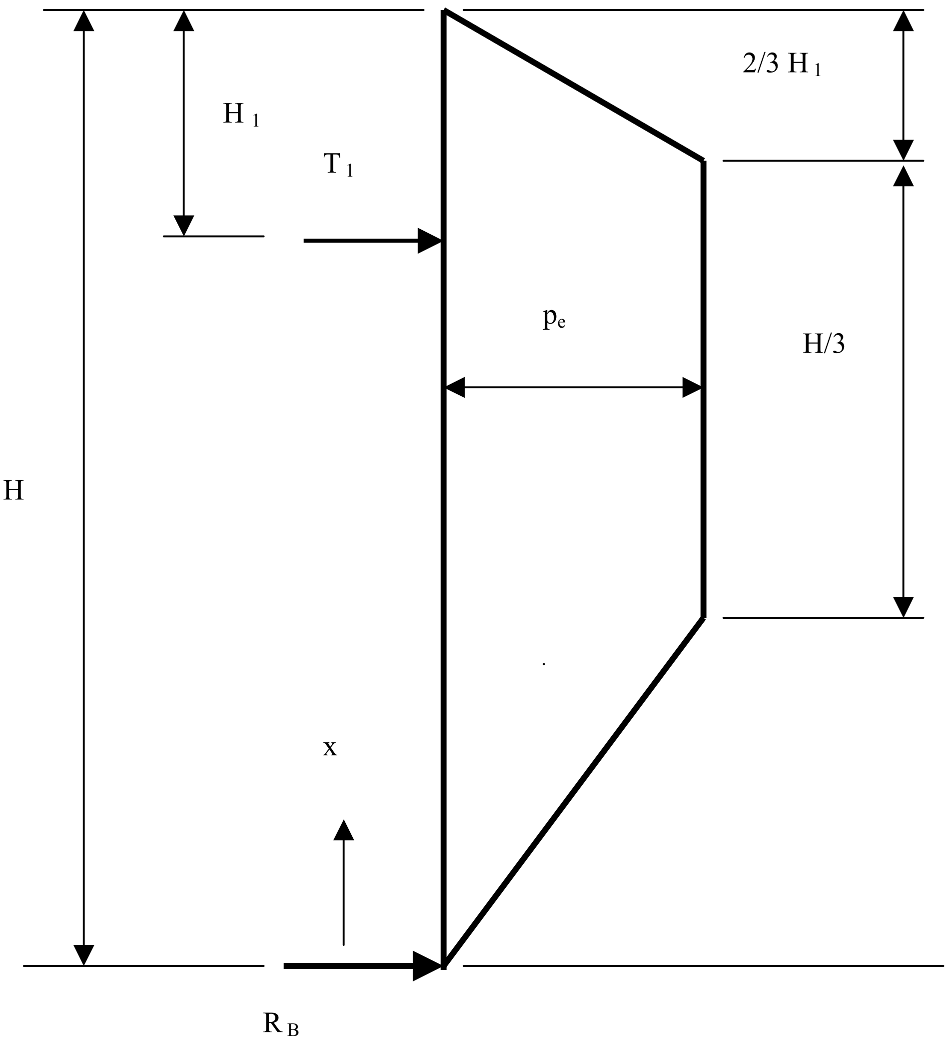


Figure 2.4. Recommended apparent earth pressure diagram for wall supported by one row of anchors–for granular soils (Figure 28, FHWA-RD-97-130)

The calculations for the Rigid 1 analysis (from FHWA-RD-97-130) are provided below. These were performed using Mathcad. The term p_s in the calculations represents the surcharge loading. All pressures, moments, and forces in the following calculations are per foot run of wall. The equations in Figure 2.5 were modified to include surcharge loading effects.

Soldier Beam Moments	Ground Anchor Load
$M_1 = \frac{13}{54} H_1^2 p_e$ **Solve for point of zero shear** $x = \frac{1}{9}\sqrt{26(H)^2 - 52(H)H_1}$ $MM_1 = Rx - \frac{p_e(x)^3}{4(H - H_1)}$	$T_1 = \left(\frac{23H^2 - 10(H)H_1}{54(H - H_1)}\right)p_e$ $R_B = \frac{2}{3}(H)p_e - T_1$

$$p_e = \frac{Total Earth Pressure Load}{\frac{2}{3}H}$$

Earth Pressure "p_e" Determined from total load required to stabilize the cut

Figure 2.5. Recommended apparent earth pressure diagram formulas for wall supported by one row of anchors–for granular soils (Figure 28, FHWA-RD-97-130)

Wall Supported by One Row of Anchors
Apparent Earth Pressure Method
Per Figure 28, Weatherby 1999 and
Sample Problem page 174

File: SOL3

$H_T := 28.9$ Feet

$H_1 := 9.84$ Feet

$\gamma := 0.115$ kcf EPF $:= 0.023$ kips per cubic foot

$P_T := EPF \cdot \left(H_T^{\,2}\right)$ $P_T = 19.21$ Kips

$p_e := \dfrac{P_T}{\left(\frac{2}{3}\right)\cdot H_T}$ $p_e = 0.997$ ksf

$p_s := 0.110$ ksf

$M_1 := \left(\frac{13}{54}\right)\cdot H_1^{\,2}\cdot p_e + \left(\frac{H_1^{\,2}}{2}\right)\cdot p_s$ $M_1 = 28.567$ Ft-Kips

$T_1 := \left[\dfrac{23\cdot H_T^{\,2} - 10\cdot H_T\cdot H_1}{54\cdot\left(H_T - H_1\right)}\right]\cdot p_e + \left[\left(\dfrac{H_T^{\,2}}{2}\right)\cdot\left(\dfrac{1}{H_T - H_1}\right)\right]\cdot p_s$ $T_1 = 18.264$ Kips

$R_B := \left(\frac{2}{3}\right)\cdot H_T\cdot p_e + H_T\cdot p_s - T_1$ $R_B = 4.125$ Kips

Find value of "x" for "y" = 0

Try $x := 8.95$

$$y := R_B - p_s \cdot x - \left[\frac{(p_e)}{\left[\left(\frac{2}{3} \right) \cdot (H_T - H_1) \right]} \right] \cdot \frac{x^2}{2}$$

$y = -2.674 \cdot 10^{-3}$ Approximately equal to zero. Okay

$$MM_1 := R_B \cdot x - p_s \cdot \left(\frac{x^2}{2} \right) - \left[\frac{p_e}{\left(\frac{2}{3} \right) \cdot (H_T - H_1)} \right] \cdot \frac{x^3}{6}$$

$MM_1 = 23.133$ Ft-Kips

A comparison between results obtained by the Rigid 1 analysis and the Winkler 1 analysis is provided in Table 2.5. The Rigid 1 analysis provides a higher anchor force and lower bending moments when compared with the Winkler 1 analysis.

Table 2.5
Rigid 1 – Winkler 1 Comparison

Soletanche Wall Section A	**Rigid 1**	**Winkler 1**
Maximum positive moment	23.1 ft-kips	26.7 ft-kips
Positive moment location	21.0 ft	24.00 ft
Maximum negative moment	-28.57 ft-kips	-31.1 ft-kips
Negative moment location	At anchor location	At anchor location
Anchor force	18.26 kips	12.26 kips

2.5 Rigid 2 Analysis

The Rigid 2 analysis is an equivalent beam on rigid supports construction-sequencing analysis that uses soil-pressure distributions based on classical methods. The use of the classical beam on rigid supports method for evaluating various loading conditions encountered during construction is described in Ratay (1996), Kerr and Tamaro (1990), and FHWA-RD-81-150. In the Rigid 2 analysis method, a vertical strip of the wall is treated as a multispan beam on rigid supports that are located at tieback points. As used in this evaluation, the lowest support is assumed to be below the bottom of the excavation at the point of minimum penetration for the cantilever stage (first-stage excavation) and at the point of zero net earth pressure for subsequent stages of excavation.

2.5.1 Assumptions for construction sequencing analysis

The assumptions used for the Rigid 2 analysis are as described below and as previously described (Section 2.1). The active pressure coefficient (K_a) and the passive pressure coefficient (K_p) are per Caquot-Kerisel (1973) (see Figure 2.6) and as interpreted by Soletanche to be

$\phi = 35$ deg $\quad \delta = -2/3\ \phi = -23.3$ deg $\quad \gamma = 115$ pcf

Surcharge = 405 psf

$K_a = 0.271 \quad (\delta = 0)$

$K_p = 7.346 \quad (\delta = -2/3\ \phi)$

The reader should note that, within the computations contained in this section, a factor of safety of 1 is applied to the shear strength of the soil when computing earth pressure coefficients (i.e., K_a and K_p) used in the subsequent construction sequencing analysis. Additional external stability analyses are required to ensure that the wall penetration depth provides an adequate factor of safety (FS >> 1.0) based on soil shear strength (i.e., FS = $\tan \phi / \tan \phi_{mob}$). External stability calculations for this problem are illustrated in Section 2.9. These calculations assume homogeneous soil conditions.

Earth pressures, shears, and moments for the first stage excavation (cantilever stage) are based on a minimum penetration distance calculated in accordance with Equation 2-4 of Andersen (1956), as presented in Figure 2.7. Shears and moments for subsequent stages of excavation are based on an equivalent beam with the lowermost support located at the point of zero net pressure. This assumes that the point of contraflexure for the equivalent beam occurs at the point of zero net pressure, which is an acceptable assumption provided the penetration below grade is adequate (Andersen 1956).

2.5.2 Stage 1 analysis

The computations for the first excavation stage are provided below. Driving-side earth pressures are indicated in Figure 2.8. Net pressures for the first stage excavation (cantilever stage) are shown on Figure 2.7.

REDUCTION FACTOR (R) OF K_ρ FOR VARIOUS RATIOS OF $-\delta/\phi$

ϕ \ δ/ϕ	−0.7	−0.6	−0.5	−0.4	−0.3	−0.2	−0.1	−0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174

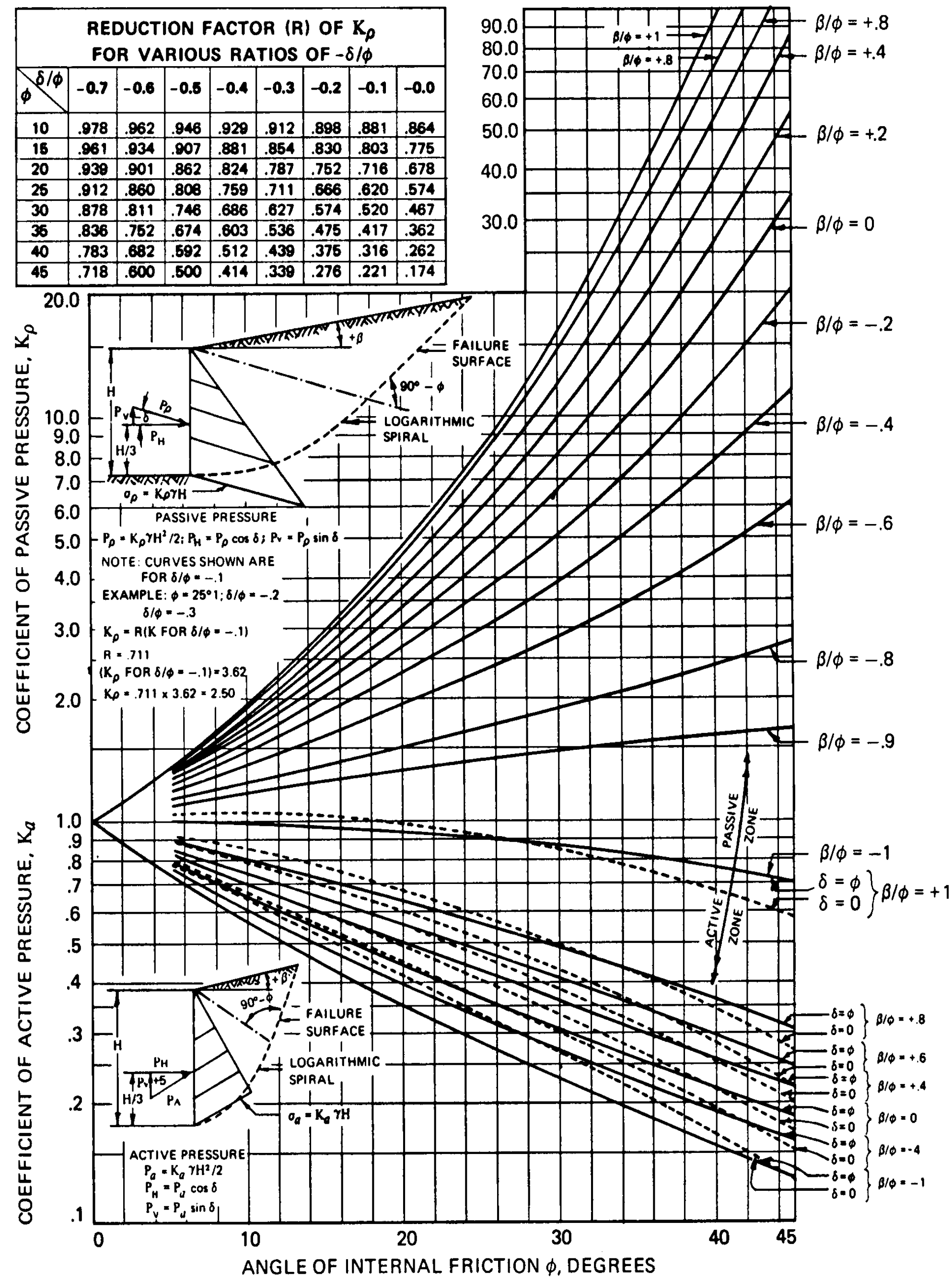
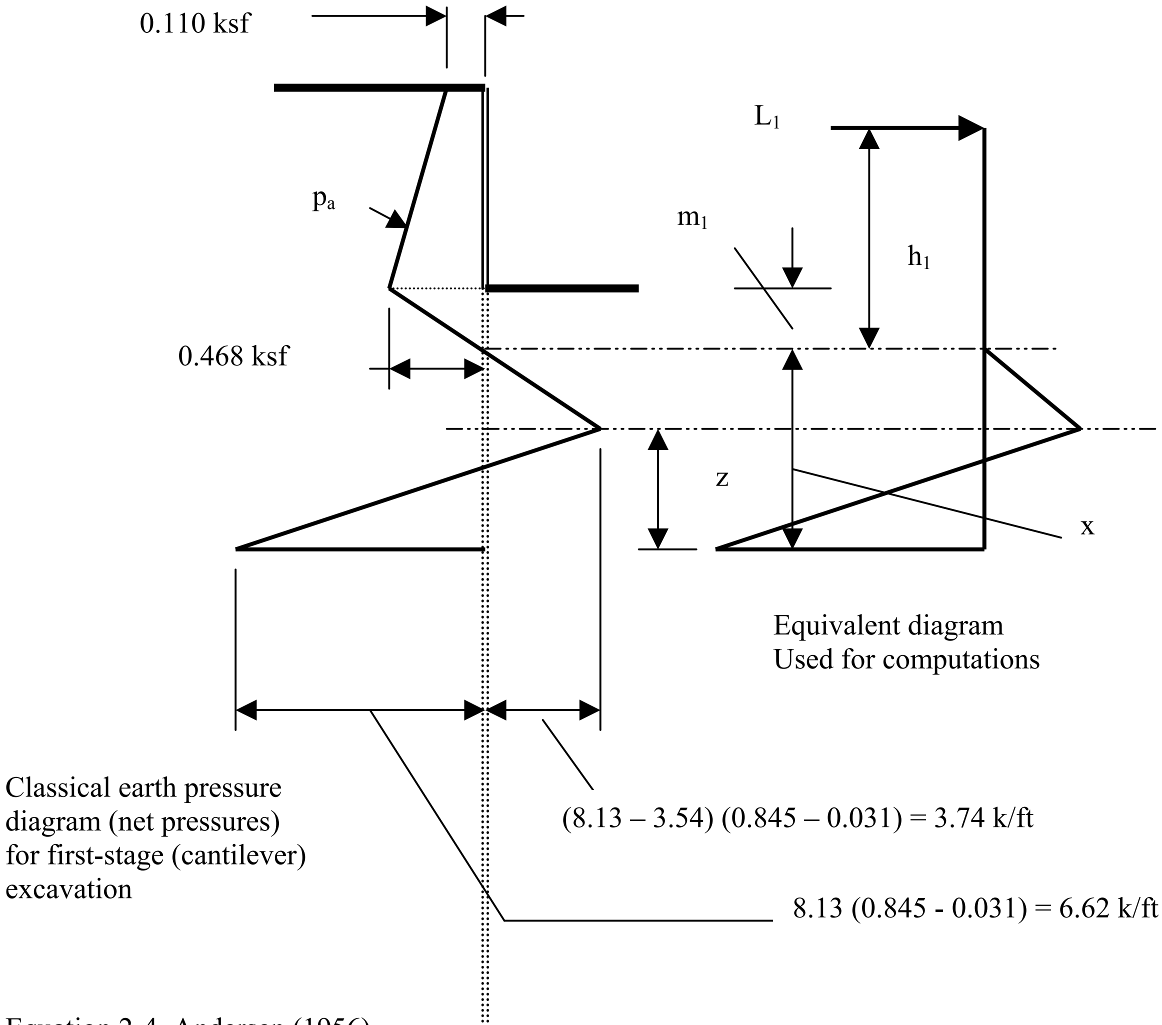


Fig. 5(a) — Active and passive coefficients with wall friction (sloping backfill) (after Caquot and Kerisel[21])

Figure 2.6. Active and passive coefficients (after Caquot and Kerisel 1973)



Equation 2-4, Andersen (1956)

$$x^4 - \left(\frac{8L_1}{p_p - p_a}\right)x^2 - \left[\frac{(12L_1h_1)}{(p_p - p_a)}\right]x - 4\left(\frac{L_1}{p_p - p_a}\right)^2 = 0$$

NOTE: See following pages for calculations of L_1, h_1, m_1, x, and z

Figure 2.7. First-stage excavation-net pressure diagram

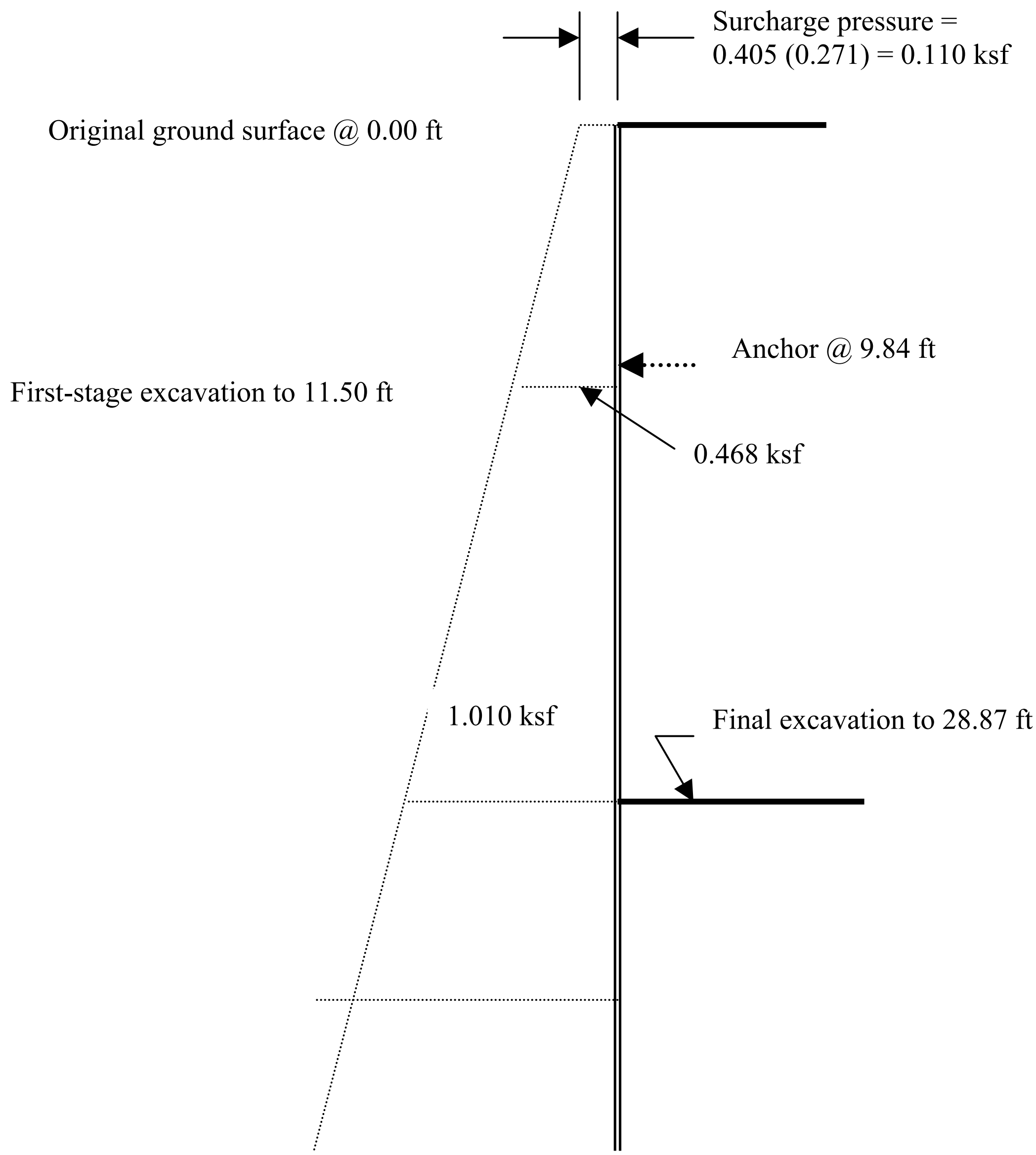


Figure 2.8. Excavation and tieback locations—driving side earth pressures at excavation levels

Computations and CBEAMC analysis for the first-stage excavation are provided below. All pressures, moments, and forces in the following calculations are per foot of run of wall.

Cantilevered Section of Tieback Wall (Uppermost Section)
Determine Minimum Penetration (maximum moment condition)
Per Figure 2-3 Andersen's "Substructure Analysis and Design" 2nd Edition

Soletanche Tieback Wall File: SOL1

$p_a := 0.031$ ksf

$\gamma := 0.115$ kcf

$K_p := 7.346$ After Caquot and Kerisel

$p_p := K_p \cdot \gamma$ $\qquad p_p = 0.845$ ksf

$m_1 := \dfrac{0.468}{(p_p - p_a)}$ $\qquad m_1 = 0.575$ Feet

$L_1 := 0.110 \cdot (11.5) + \left[0.358 \cdot (11.5) \cdot \dfrac{1}{2}\right] + 0.468 \cdot \dfrac{m_1}{2}$ $\qquad L_1 = 3.458$ Kips

$$h_1 := \left[0.11 \cdot (11.5) \cdot \left[(11.5) \cdot \left(\frac{1}{2}\right) + m_1\right] + 0.358 \cdot (11.5) \cdot \left(\frac{1}{2}\right) \cdot \left[11.5 \cdot \left(\frac{1}{3}\right) + m_1\right] + 0.468 \cdot \left(\frac{m_1}{2}\right) \cdot \left(2 \cdot \frac{m_1}{3}\right)\right] \cdot \left(\frac{1}{L_1}\right)$$

$h_1 = 4.953$ Feet

Try various values of "x"
The correct value of "x" is when Y = 0

For *x*, refer to Figure 2.7.

Try $x := 8.13$ Feet

Per Equation 2-4 Andersen (1956) – see Figure 2.7

$$Y := \left[x^4 - \left(\frac{8 \cdot L_1}{p_p - p_a}\right) \cdot x^2 - \left[\frac{(12 \cdot L_1 \cdot h_1)}{(p_p - p_a)}\right] \cdot x - 4 \cdot \left(\frac{L_1}{p_p - p_a}\right)^2\right]$$

$Y = -3.685$ Approximately equal to zero okay

$$z := \left(\frac{x}{2}\right) - \frac{L_1}{(p_p - p_a) \cdot x}$$

$z = 3.542$ Feet

The net earth pressure diagram based on classical methods, established by the above calculations and illustrated in Figure 2.7, is used in a beam-column analysis (CBEAMC analysis) to determine wall bending moments and shears. In the CBEAMC analysis, the wall is provided with a fictitious support. The support is fixed against translation and rotation to provide stability for the beam-column solution. The support is located at a distance equal to 11.50 + m_1 + x, or 20.21 ft, the point of minimum penetration satisfying static equilibrium. (That is, the sum of the forces in the horizontal direction is equal to zero, and the sum of the moments is equal to zero.) If the fictitious support has been selected properly, the results from the CBEAMC analysis should indicate a zero moment and a zero lateral force reaction at the support location. Input and output for the CBEAMC analysis are provided below. Moments obtained from the CBEAMC analysis for the first-stage excavation analysis are plotted in Figure 2.9. Note that the support moment and shear computed by CBEAMC at X = 20.21 ft is approximately equal to zero. The CBEAMC analysis serves as an error check for the m_1-value and x-value computations. A support moment other than zero would indicate an error in these computations. Since the first-stage equivalent beam solution is statically determinant, it is possible by simple hand calculation to determine the maximum moment.

This can be accomplished as specified below:

Let x_o equal the distance below the zero net pressure point where the shear is equal to zero (i.e., point of maximum moment). Summing forces in the lateral direction,

$$L_1 - (p_p - p_a)\frac{x_o^2}{2} = 0$$

$$x_o = \sqrt{\frac{2L_1}{p_p - p_a}} = \sqrt{\frac{2(3.458)}{0.845 - 0.031}} = 2.91\,feet$$

$$M_{\max} = L_1(h_1 + x_o) - (p_p - p_a)\frac{x_o^3}{6} = 23.85\,ft - k$$ Agrees with CBEAMC - OKAY

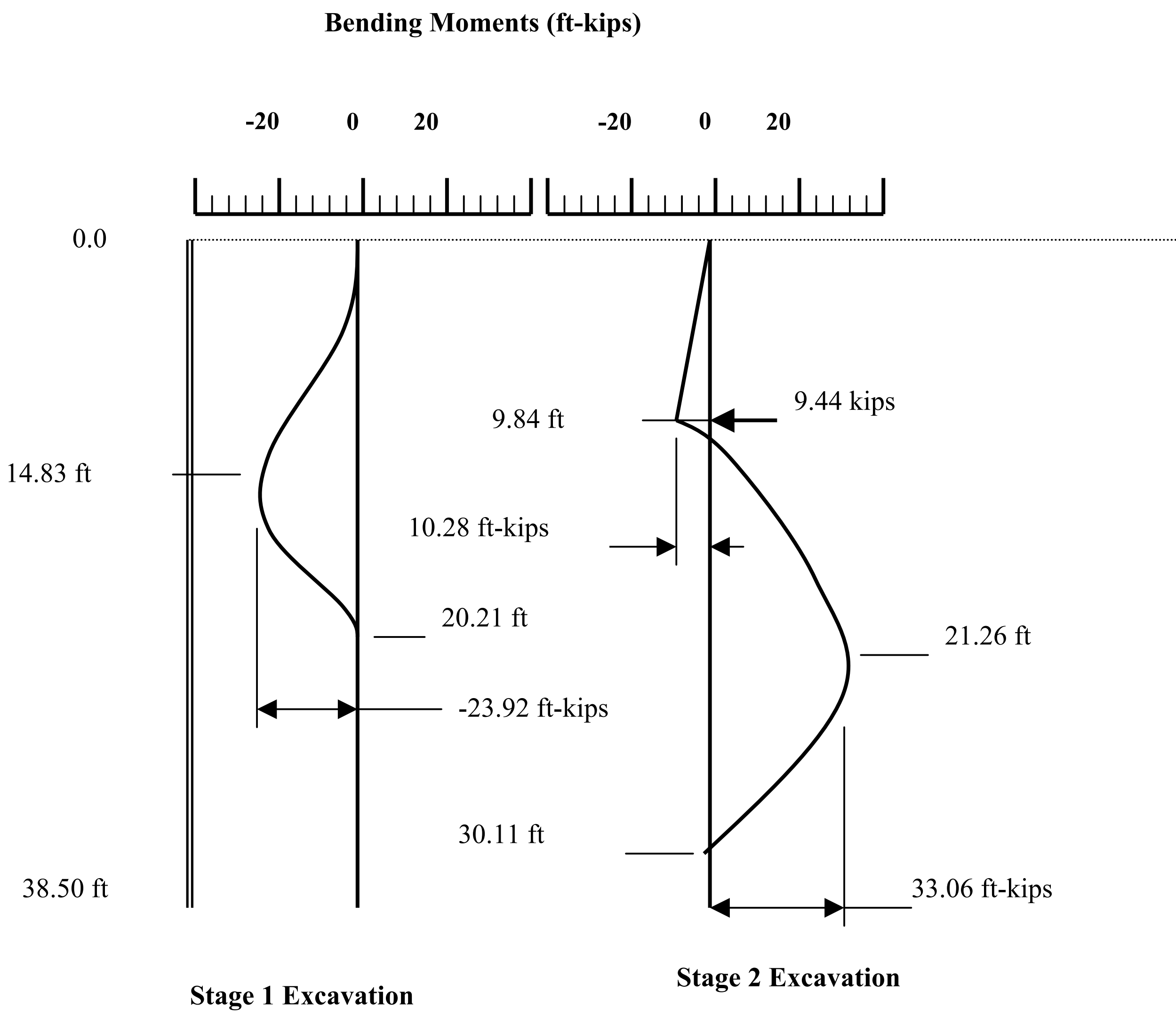


Figure 2.9. Wall bending moments

CBEAMC Input for Rigid 2 analysis
Cantilever Stage
File: SOL1

HEADING	
LN	"Heading Description"
1000	**SOLETANCHE WALL FIRST STAGE EXCAVATION**

BEAM HEADER			
LN	"Beam Title"	"New or Add"	Units "**I**nches or **F**eet" "**P**ounds or **K**ips"
1020	**BEAM**		**F K**

BEAM DATA LINES					
LN	X1	X2	E	A1	SI1
1030	**0.0**	**20.21**	**475000.**	**1.15**	**0.13**

NODE SPACING HEADER			
LN	"NODe"	"New or Add"	Units "**I**nches or **F**eet" "**P**ounds or **K**ips"
1040	**NODES**		**F**

NODE SPACING DATA LINES			
LN	X1 "Coord @ Start"	X2 "Coord @ End"	HMAX "Max dist. betw. nodes"
1050	**0.00**	**20.21**	**1.0**

LOADS HEADER LINE			
LN	"Loads"	"New or Add"	Units "**I**nches or **F**eet" "**P**ounds or **K**ips"
1060	**LOADS**		**F K**

DISTRIBUTED LOADS DATA LINES						
LN	"Distributed"	"Direction"	X1	Q1	X2	Q2
1070	**D**	**Y**	**0.00**	**0.11**	**11.50**	**0.47**
1080	**D**	**Y**	**11.50**	**0.47**	**12.08**	**0.00**
1090	**D**	**Y**	**12.08**	**0.00**	**16.67**	**-3.74**
1100	**D**	**Y**	**16.67**	**-3.74**	**20.21**	**6.62**

FIXED SUPPORTS HEADER			
LN	"FIXed"	"New or Add"	Units "**I**nches or **F**eet" "**P**ounds or **K**ips"
1120	**FIXED**		**F**

FIXED SUPPORTS DATA LINES				
LN	X1 "Coord of support"	XD "Displ. or free"	YD "Displ. or free"	R "Rotation or free"
1130	**20.21**	**0.0**	**0.0**	**0.0**

TERMINATION			
LN	" FINish"	"Rerun"	"Keep"
1150	**FINISH**		

PROGRAM CBEAMC - ANALYSIS OF BEAM-COLUMNS WITH NONLINEAR SUPPORTS

DATE: 14-JANUARY-2001 TIME: 13:19:25

```
                 ***********************
                 *  SUMMARY OF RESULTS  *
                 ***********************

             I.--HEADING
             'SOLETANCHE WALL
             'FIRST STAGE EXCAVATION

            II.--MAXIMA

                                     MAMIMUM  X-COORD     MAXIMUM  X-COORD
                                    POSITIVE     (FT)    NEGATIVE     (FT)
AXIAL DISPLACEMENT (FT)     :      0.000E+00     0.00   0.000E+00     0.00
LATERAL DISPLACEMENT (FT)   :      4.124E-02     0.00   0.000E+00     0.00
ROTATION (RAD)              :      0.000E+00     0.00  -3.089E-03     0.00
AXIAL FORCE (K)             :      0.000E+00     0.00   0.000E+00     0.00
SHEAR FORCE (K)             :      3.471E+00    12.08  -7.276E+00    17.56
BENDING MOMENT (K-FT)       :      2.392E+01    14.83  -2.631E-13     0.00

           III.--REACTIONS AT FIXED SUPPORTS

         X-C00RD    X-REACTION    Y-REACTION    MOMENT-REACTION
          (FT)         (K)           (K)            (K-FT)
          20.21     0.000E+00     1.440E-02        1.039E-01
```

Note: Y-REACTION @ X = 20.21 ≈ zero

```
PROGRAM CBEAMC - ANALYSIS OF BEAM-COLUMNS WITH NONLINEAR
SUPPORTS
DATE: 14-JANUARY-2001


                   *********************
                   *  COMPLETE RESULTS  *
                   *********************


 I.--HEADING
      'SOLETANCHE WALL
      'FIRST STAGE EXCAVATION


II.--DISPLACEMENTS AND INTERNAL FORCES
 <---------DISPLACEMENTS--------->  <----INTERNAL FORCES--->
 X-COORD     LATERAL    ROTATION     SHEAR      MOMENT
   (FT)        (FT)      (RAD)        (K)       (K-FT)
   0.00      4.124E-02 -3.089E-03   1.263E-12 -2.631E-13
   0.96      3.828E-02 -3.089E-03   1.198E-01  5.510E-02
   1.92      3.532E-02 -3.087E-03   2.683E-01  2.388E-01
   2.88      3.237E-02 -3.081E-03   4.456E-01  5.786E-01
   3.83      2.942E-02 -3.068E-03   6.517E-01  1.102E+00
   4.79      2.649E-02 -3.046E-03   8.865E-01  1.837E+00
   5.75      2.359E-02 -3.010E-03   1.150E+00  2.810E+00
   6.71      2.073E-02 -2.957E-03   1.442E+00  4.050E+00
   7.67      1.793E-02 -2.883E-03   1.763E+00  5.584E+00
   8.63      1.521E-02 -2.782E-03   2.113E+00  7.439E+00
   9.58      1.261E-02 -2.650E-03   2.492E+00  9.643E+00
  10.54      1.014E-02 -2.481E-03   2.899E+00  1.222E+01
  11.50      7.865E-03 -2.268E-03   3.335E+00  1.521E+01
  12.08      6.592E-03 -2.116E-03   3.471E+00  1.720E+01
  13.00      4.774E-03 -1.837E-03   3.128E+00  2.028E+01
  13.92      3.232E-03 -1.517E-03   2.098E+00  2.273E+01
  14.83      1.999E-03 -1.168E-03   3.813E-01  2.392E+01
  15.75      1.089E-03 -8.147E-04   2.022E+00  2.322E+01
  16.67      4.942E-04 -4.899E-04   5.112E+00  2.000E+01
  17.56      1.765E-04 -2.416E-04  -7.276E+00  1.435E+01
  18.44      3.983E-05 -8.304E-05  -7.147E+00  7.795E+00
  19.33      3.212E-06 -1.275E-05  -4.727E+00  2.371E+00
  20.21      0.000E+00  0.000E+00   1.440E-02  1.039E-01
```

Note: Moment at X = 20.21 is approximately equal to zero.

2.5.3 Stage 2 analysis

The computations for the final excavation stage (Stage 2) are provided below. Final excavation is at a depth of 28.87 ft. In the analysis, a point of contraflexure is assumed to coincide with the zero net pressure point located at a distance of (28.87 + “m”) ft below the surface. Using this assumption, the upper portion of the anchored tieback wall can be treated as an equivalent beam that is simply supported at the anchor location and at the first point of zero net pressure intensity. The equivalent beam with net pressure loading is shown in Figure 2.10.

As with first-stage excavation analysis, the solution is statically determinate and, therefore, the maximum wall moment can be determined by simple hand calculations. However, this, the final-stage excavation analysis, is performed using the CBEAMC software. The CBEAMC input and output for the final-stage analysis is provided below. Bending moments for the final-stage excavation are plotted in Figure 2.9.

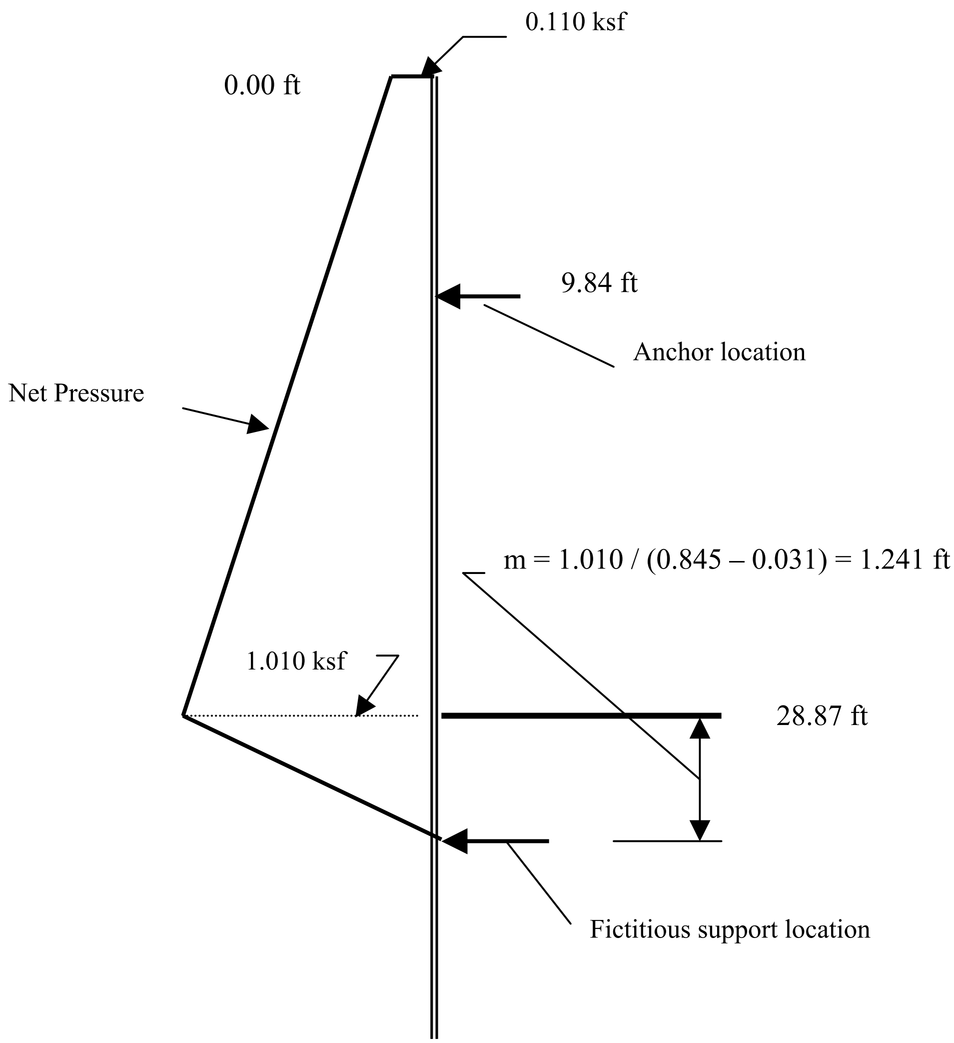


Figure 2.10. Final excavation to 28.87 ft

CBEAMC Input for Rigid 2 analysis
Final excavation stage - Equivalent beam analysis
File: SOL2

HEADING						
LN	"Heading Description"					
1000	**SOLETANCHE WALL TIEBACK WALL SECOND STAGE EXCAVATION**					
BEAM HEADER						
LN	"Beam Title"	"New or Add"	Units "**I**nches or **F**eet" "**P**ounds or **K**ips"			
1020	**BEAM**		**F K**			
BEAM DATA LINES						
LN	X1	X2	E	A1	SI1	
1030	**0.0**	**30.11**	**475000.**	**1.15**	**0.13**	
NODE SPACING HEADER						
	"NODe"	"New or Add"	Units "**I**nches or **F**eet" "**P**ounds or **K**ips"			
	NODES		**F**			
NODE SPACING DATA LINES						
LN	X1 "Coord @ Start"	X2 "Coord @ End"	HMAX "Max dist. betw. nodes"			
1050	**0.00**	**30.11**	**2.0**			
LOADS HEADER LINE						
LN	"Loads"	"New or Add"	Units "**I**nches or **F**eet" "**P**ounds or **K**ips"			
1060	**LOADS**		**F K**			
DISTRIBUTED LOADS DATA LINES						
LN	"Distributed"	"Direction"	X1	Q1	X2	Q2
1070	**D**	**Y**	**0.00**	**0.11**	**28.87**	**1.01**
1080	**D**	**Y**	**28.87**	**1.01**	**30.11**	**0.00**
FIXED SUPPORTS HEADER						
LN	"FIXed"	"New or Add"	Units "**I**nches or **F**eet" "**P**ounds or **K**ips"			
1100	**FIXED**		**F**			
FIXED SUPPORTS DATA LINES						
LN	X1 "Coord of support"	XD "Displ. or free"	YD "Displ. or free"	R "Rotation or free"		
1110	**9.84**	**0.0**	**0.0**	**FREE**		
1120	**30.11**	**0.0**	**0.0**	**FREE**		
TERMINATION						
LN	" FINish"	"Rerun"	"Keep"			
1150	**FINISH**					

```
PROGRAM CBEAMC - ANALYSIS OF BEAM-COLUMNS WITH NONLINEAR
SUPPORTS
DATE: 14-JANUARY-2001                     TIME: 16:37:56

                    ************************
                    *  SUMMARY OF RESULTS  *
                    ************************


  I.--HEADING
      'SOLETANCHE WALL
      'SECOND STAGE EXCAVATION


 II.--MAXIMA

                               MAXIMUM  X-COORD     MAXIMUM X-COORD
                              POSITIVE     (FT)    NEGATIVE    (FT)
AXIAL DISPLACEMENT (FT)    :  0.000E+00     0.00   0.000E+00    0.00
LATERAL DISPLACEMENT (FT)  :  2.157E-02    21.26  -2.450E-02    0.00
ROTATION (RAD)             :  2.984E-03    11.74  -3.646E-03   30.11
AXIAL FORCE (K)            :  0.000E+00     0.00   0.000E+00    0.00
SHEAR FORCE (K)            :  7.350E+00    30.11  -6.852E+00    9.84
BENDING MOMENT (K-FT)      :  1.028E+01     9.84  -3.306E+01   21.26


III.--REACTIONS AT FIXED SUPPORTS

X-C00RD    X-REACTION    Y-REACTION    MOMENT-REACTION
 (FT)         (K)           (K)             (K-FT)

   9.84     0.000E+00    -9.444E+00       0.000E+00
  30.11     0.000E+00    -7.350E+00       0.000E+00


 IV.--FORCES IN LINEAR CONCENTRATED SPRINGS
         NONE


  V.--FORCES IN NONLINEAR CONCENTRATED SPRINGS
         NONE
```

```
PROGRAM CBEAMC - ANALYSIS OF BEAM-COLUMNS WITH NONLINEAR
SUPPORTS
DATE: 14-JANUARY-2001                       TIME: 16:37:56

                    *********************
                    *  COMPLETE RESULTS  *
                    *********************


   I.--HEADING
      'SOLETANCHE WALL
      'SECOND STAGE EXCAVATION


  II.--DISPLACEMENTS AND INTERNAL FORCES
    <-----DISPLACEMENTS------>  <--INTERNAL FORCES--->
 X-COORD  LATERAL    ROTATION     SHEAR      MOMENT
   (FT)     (FT)       (RAD)       (K)        (K-FT)
   0.00  -2.450E-02  2.380E-03   3.929E-13 -2.941E-13
   1.97  -1.982E-02  2.383E-03   2.768E-01  2.526E-01
   3.94  -1.511E-02  2.403E-03   6.744E-01  1.169E+00
   5.90  -1.033E-02  2.467E-03   1.193E+00  2.986E+00
   7.87  -5.356E-03  2.606E-03   1.832E+00  5.943E+00
   9.84   0.000E+00  2.860E-03   2.592E+00  1.028E+01
   9.84   0.000E+00  2.860E-03  -6.852E+00  1.028E+01
  11.74   5.621E-03  2.984E-03  -6.003E+00 -1.974E+00
  13.65   1.113E-02  2.756E-03  -5.040E+00 -1.250E+01
  15.55   1.592E-02  2.233E-03  -3.965E+00 -2.109E+01
  17.45   1.949E-02  1.479E-03  -2.777E+00 -2.752E+01
  19.36   2.145E-02  5.617E-04  -1.476E+00 -3.158E+01
  21.26   2.157E-02 -4.413E-04  -6.166E-02 -3.306E+01
  23.16   1.977E-02 -1.447E-03   1.465E+00 -3.175E+01
  25.06   1.612E-02 -2.367E-03   3.105E+00 -2.741E+01
  26.97   1.087E-02 -3.104E-03   4.858E+00 -1.986E+01
  28.87   4.483E-03 -3.555E-03   6.723E+00 -8.855E+00
  30.11   0.000E+00 -3.646E-03   7.350E+00 -3.026E-14


III.--FORCES IN LINEAR DISTRIBUTED SPRINGS
          NONE


 IV.--FORCES IN NONLINEAR DISTRIBUTED SPRINGS
          NONE
```

If possible, the wall should penetrate to a depth where the angle of rotation at the bottom of the wall is zero. At this depth, the anchor tension and wall bending moments will be at a minimum. This penetration depth is often referred to as the “favorable” or “economic” penetration depth. At this depth the point of contraflexure will approximately coincide with the point of equal active and passive pressure intensity as assumed for the equivalent beam analysis. The economic penetration depth is determined in the following computations. All pressures, moments, and forces in the following calculations are per foot of run of wall.

Lower Section of Tieback Wall
Determine Minimum Penetration for Economical Depth Assumption
Per Figure 2-11 Andersen's "Substructure Analysis and Design" 2nd Editic

$m_1 := 1.24$ Feet $\quad P := 7.35$ kips

$p_p := 0.845$ ksf $\quad p_a := 0.031$ ksf

$$y := 1.1 \cdot \left[m_1 + \left(\frac{6 \cdot P}{p_p - p_a} \right)^{0.5} \right] \qquad y = 9.461 \quad \text{Feet}$$

To reach the economic depth, the wall should extend a distance below the original ground surface equal to 28.87 + 9.63, or 38.50 ft. The actual wall depth used in the final design was equal to 40.0 ft, thereby satisfying economic depth requirements.

Comparisons between results obtained from the Rigid 2 analysis and by Soletanche PEROI 1 analysis are provided in Table 2.6. The analyses should and do produce similar results.

Table 2.6
Rigid 2–PEROI 1 Comparison

Wall Section A	**Rigid 2**	**Soletanche PEROI 1**
Anchor level depth	9.84 ft (given)	9.84 ft
First excavation depth	11.50 ft (given)	11.50 ft
Maximum moment	**23.92 ft-kips**	**23.83 ft-kips**
Final excavation depth	28.87 ft	28.87 ft (given)
Maximum moment	**33.06 ft-kips**	**32.97 ft-kips**
Toe embedment depth	**38.50 ft**	**38.94 ft**
Reaction at tieback	**9.44 kips/foot**	**9.44 kips/foot**

2.6 Winkler 1 Analysis

Soletanche Temporary Tieback Wall
Construction-Sequencing Evaluation
Winkler Spring (Elastoplastic) Method
Illustration of the Design Procedure–I-75, Section A
FHWA-RD-81-150, page 138

The use of the Winkler spring analysis for evaluating various loading conditions encountered during construction is described in Dawkins (1994a), Ratay (1996), Kerr and Tamaro (1990), and FHWA-RD-81-150.

Assumptions for Winkler spring construction-sequencing analysis are in accordance with the reference deflection method described in FHWA-RD-98-066.

The Soletanche tieback wall example is as described in Section 2.1.

The active pressure coefficient (K_a) and the passive pressure coefficient (K_p) are per Caquot-Kerisel (1973), per Soletanche practice. Soil parameters used in the analysis are per the previous Rigid 2 analysis and as indicated below.

ϕ = 35 deg $\delta = - 2/3\ \phi = -23$ deg
γ = 115 pcf   Surcharge = 405 psf

The active pressure coefficient (K_a) and the passive pressure coefficient (K_p) are computed internally within the CMULTIANC program (Dawkins, Strom, and Ebeling, in preparation).

$K_a = 0.225$ $(\delta = -2/3\ \phi)$
$K_p = 8.36$ $(\delta = -2/3\ \phi)$

The above values differ slightly from those used by Soletanche in the Rigid 2 analysis.

The Winkler 1 analysis is performed using CMULTIANC software (Dawkins, Strom, and Ebeling, in preparation). This software is a modification of CBEAMC (Dawkins 1994b). Modifications were necessary to obtain a numerical solution for certain situations where convergence could not be achieved once shifting of the R-y curves occurred. CMULTIANC uses concentrated soil springs in the analysis. A soil spring for each foot of depth is provided to obtain the required accuracy. When convergence cannot be obtained for any particular stage of construction, the CMULTIANC program increments that particular stage of the analysis by either applying anchor loads in small increments or excavating in small increments. This facilitates the solution process such that convergence to the correct solution is obtained. Input to the CMULTIANC program for the Soletanche wall analysis is as shown below.

```
'SOLETANCHE WALL      INPUT FILE: S1    OUTPUT FILE: SO1
'DELTA = 2/3 PHI ON BOTH ACTIVE AND PASSIVE
WALL   0  3.300E+06   2700
WALL  -40
ANCHOR  -10   8000   28900   15189
SOIL RIGHTSIDE  STRENGTHS   1
 0   115   115     0 35   23   23   .05   .5
SOIL LEFTSIDE  STRENGTHS   1
-12.5   115   115     0 35   23   23   .05   .5
VERTICAL UNIFORM   405
EXCAVATION DATA
-30
BOTTOM  FREE
FINISHED
```

2.6.1 Stage 1 excavation

The following describes the CMULTIANC process used to determine the active and passive limiting soil pressures on each side of the wall for the first-stage excavation, i.e., excavation to elevation (el) -11.5.[4] The coordinate system and driving- and resisting-side soil levels for the first-stage excavation are as shown in Figure 2.11.

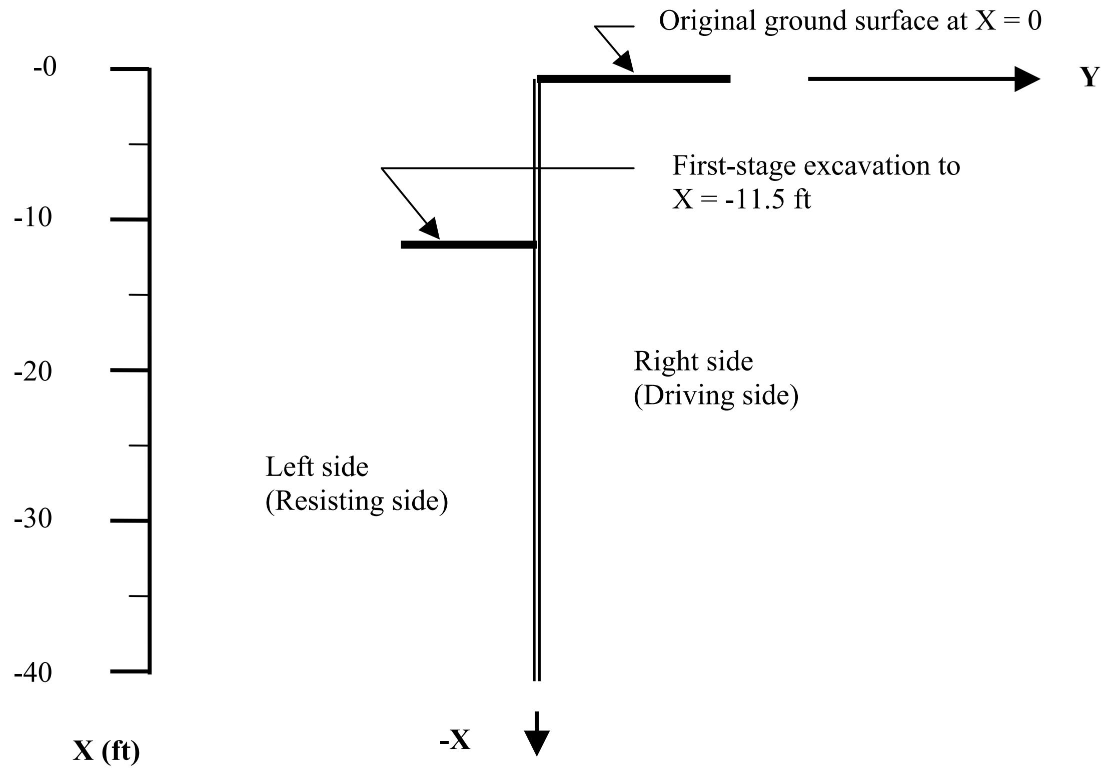


Figure 2.11. First-state excavation, coordinate system per CMULTIANC

[4] All elevations (el) cited herein are in feet referenced to the National Geodetic Vertical Datum.

Calculations of the limiting active and passive soil pressures on each side of the wall are calculated internally within the CMULTIANC program based on input provided by the user. For the left side, the active limit pressure is equal to the soil depth times the unit weight times the active pressure coefficient. For the right side, the active limit pressure is equal to the soil depth plus effective surcharge depth times the unit weight times the active pressure coefficient. Similar calculations are made for left- and right-side passive limit pressures using the passive rather than active pressure coefficient. CMULTIANC input information and the output for the stage-1 excavation left- and right-side limit pressures are provided below.

```
  CMULTIANC: SIMULATION OF CONSTRUCTION SEQUENCE FOR STIFF
        WALL SYSTEMS WITH MULTIPLE LEVELS OF ANCHORS
    DATE: 28-JULY-2002                    TIME: 16:07:31

                      ****************
                      *  INPUT DATA  *
                      ****************

I.--HEADING
    'SOLETANCHE WALL      INPUT FILE: S1    OUTPUT FILE:
SO1
    'DELTA = 2/3 PHI ON BOTH ACTIVE AND PASSIVE

II.--WALL SEGMENT DATA

          ELEVATION
          AT TOP OF       MODULUS OF       MOMENT OF
           SEGMENT        ELASTICITY        INERTIA
             (FT)            (PSI)           (IN^4)
             0.00          3.300E+06        2700.00

          ELEVATION AT BOTTOM OF WALL = -40.00

III.--ANCHOR DATA

 ELEV.     LOCK-OFF     ULTIMATE       ANCHOR
AT WALL      LOAD        TENSION     STIFFNESS
 (FT)       (LBS)         (LBS)      (LBS/IN)       STATUS
-10.0      8.000E+03    2.890E+04    1.519E+04    INACTIVE


IV.--SOIL LAYER DATA


IV.A.1.--RIGHTSIDE PROPERTIES
                             UNDRAINED EFFECTIVE
                             COHESIVE  INTERNAL
LAYER TOP <UNIT WEIGHT(PCF)> STRENGTH  FRICTION WALL  FRICT.
ELEV. (FT) SAT.        MOIST   (PSF)    (DEG)  ACTIVE PASSIVE
   0.0     115.0       115.0     0.0    35.0    23.0    23.0
```

```
IV.A.2.--RIGHTSIDE REFERENCE DISPLACEMENTS
          LAYER TOP      <REFERENCE DISPLACEMENT (IN)>
          ELEV. (FT)         ACTIVE        PASSIVE
              0.0             0.05           0.50

IV.B.1.--LEFTSIDE PROPERTIES

                              UNDRAINED EFFECTIVE
                              COHESIVE  INTERNAL
LAYER TOP <UNIT WEIGHT (PCF)> STRENGTH  FRICTION  WALL FRICT.
ELEV. (FT) SAT.       MOIST    (PSF)      (DEG)  ACTIVE
PASSIVE
 -12.5    115.0         115.0    0.0     35.0     23.0    23.0

NOTE:       SOIL SPRINGS ON LEFTSIDE STARTED AT X = -12.5
            RATHER THAN  X = -11.5 TO ACCOUNT FOR POSSIBLE
            SOIL DISTURBANCE AND POSSIBLE OVEREXCAVATION
            BEYOND THE ESTABLISHED STAGE 1 EXCAVATION DEPTH

V.--INITIAL WATER DATA
          NONE

VI.--VERTICAL SURCHARGE LOADS

VI.A.--VERTICAL LINE LOADS
               NONE

VI.B.--VERTICAL UNIFORM LOADS
                    RIGHTSIDE
                      (PSF)
                     405.00

VI.C.--VERTICAL STRIP LOADS
               NONE

VI.D.--VERTICAL RAMP LOADS
               NONE

VI.E.--VERTICAL TRIANGULAR LOADS
               NONE

VI.F.--VERTICAL VARIABLE LOADS
               NONE

VII.--EXCAVATION DATA
          EXCAVATION       WATER
          ELEVATION      ELEVATION
             (FT)           (FT)
            -30.00         NONE

VII.--WALL BOTTOM CONDITIONS
          FREE
```

```
CMULTIANC: SIMULATION OF CONSTRUCTION SEQUENCE FOR STIFF
      WALL SYSTEMS WITH MULTIPLE LEVELS OF ANCHORS
     DATE: 28-JULY-2002              TIME: 16:07:34

              ******************************
              *      LIMIT PRESSURES       *
              *   FOR INITIAL CONDITIONS   *
              ******************************

I.--HEADING

    'SOLETANCHE WALL   INPUT FILE: S1    OUTPUT FILE: SO1
    'DELTA = 2/3 PHI ON BOTH ACTIVE AND PASSIVE

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

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

ELEV. (FT)	<--LEFTSIDE PRESSURES (PSF)-> WATER	PASSIVE	ACTIVE	<-RIGHTSIDE PRESSURES (PSF)-> WATER	ACTIVE	PASSIVE
0.00	0.00	0.00	0.00	0.00	91.12	3385.8
-1.00	0.00	0.00	0.00	0.00	117.00	4347.20
-2.00	0.00	0.00	0.00	0.00	142.87	5308.60
-3.00	0.00	0.00	0.00	0.00	168.75	6270.00
-4.00	0.00	0.00	0.00	0.00	194.62	7231.40
-5.00	0.00	0.00	0.00	0.00	220.50	8192.80
-6.00	0.00	0.00	0.00	0.00	246.37	9154.20
-7.00	0.00	0.00	0.00	0.00	272.25	10115.60
-8.00	0.00	0.00	0.00	0.00	298.12	11077.00
-9.00	0.00	0.00	0.00	0.00	324.00	12038.40-
-10.00	0.00	0.00	0.00	0.00	349.87	12999.80-
-11.00	0.00	0.00	0.00	0.00	375.75	13961.20
-12.00	0.00	0.00	0.00	0.00	401.62	14922.60
-12.50	0.00	0.00	0.00	0.00	414.56	15403.30
-13.00	0.00	480.70	12.94	0.00	427.50	15884.00
-14.00	0.00	1442.10	38.81	0.00	453.37	16845.40
-15.00	0.00	2403.50	64.69	0.00	479.25	17806.80
-16.00	0.00	3364.90	90.56	0.00	505.12	18768.20
-17.00	0.00	4326.30	116.44	0.00	531.00	19729.60
-18.00	0.00	5287.70	142.31	0.00	556.87	20691.00
-19.00	0.00	6249.10	168.19	0.00	582.75	21652.40
-20.00	0.00	7210.50	194.06	0.00	608.62	22613.80
-21.00	0.00	8171.90	219.94	0.00	634.50	23575.20
-22.00	0.00	9133.30	245.81	0.00	660.37	24536.60
-23.00	0.00	10094.70	271.69	0.00	686.24	25498.00
-24.00	0.00	11056.10	297.56	0.00	712.12	26459.40
-25.00	0.00	12017.50	323.44	0.00	737.99	27420.80
-26.00	0.00	12978.90	349.31	0.00	763.87	28382.20
-27.00	0.00	13940.30	375.18	0.00	789.74	29343.60
-28.00	0.00	14901.70	401.06	0.00	815.62	30305.00
-29.00	0.00	15863.10	426.93	0.00	841.49	31266.40
-30.00	0.00	16824.50	452.81	0.00	867.37	32227.80

-31.00	0.00	17785.90	478.68	0.00	893.24	33189.20
-32.00	0.00	18747.30	504.56	0.00	919.12	34150.60
-33.00	0.00	19708.70	530.43	0.00	944.99	35112.00
-34.00	0.00	20670.10	556.31	0.00	970.87	36073.40
-35.00	0.00	21631.50	582.18	0.00	996.74	37034.80
-36.00	0.00	22592.90	608.06	0.00	1022.62	37996.20
-37.00	0.00	23554.30	633.93	0.00	1048.49	38957.60
-38.00	0.00	24515.70	659.81	0.00	1074.37	39919.00
-39.00	0.00	25477.10	685.68	0.00	1100.24	40880.40
-40.00	0.00	26438.50	711.56	0.00	1126.12	41841.80

Calculations are also made to determine the stage-1 excavation soil springs (soil-structure interaction, SSI, curves). These are determined for each foot of depth using the appropriate reference deflection, and converting the limiting active and passive soil pressures to limiting concentrated spring forces, as described in Figure 2.12. Spring forces are determined at the top and bottom of each 1-ft increment of depth. For the spring at el -2.00 there will be a contribution from the lower half of the soil between el -1.00 and -2.00, (designated as -2.00+), plus a contribution from the upper half of the soil between el -2.00 and -3.00 (designated as -2.00-).

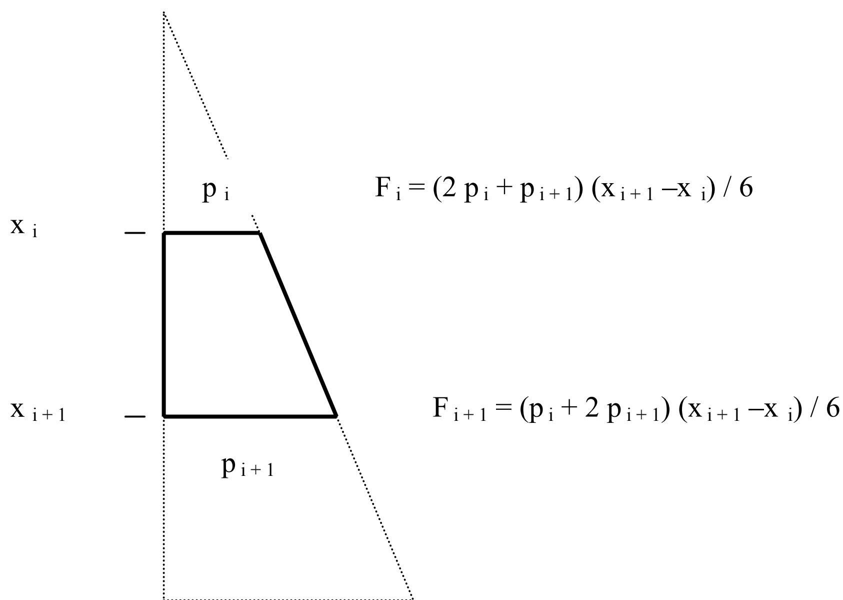


Figure 2.12. Nonlinear spring concentrated limit forces (F) at top and bottom of 1-ft soil increment

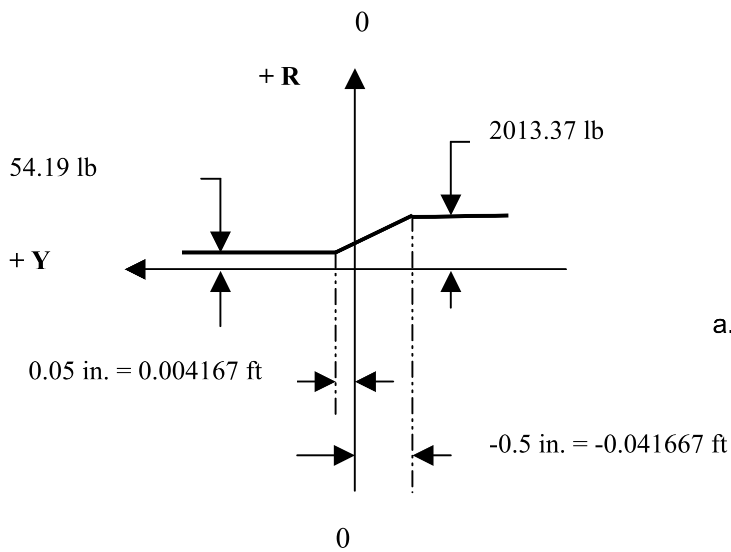
The following computations illustrate the process for the right-side active limit force at the top of the first 1-ft increment (right side 0.00) and at the bottom of the first 1-ft soil increment (right side -1.00+).

$x_i = 0.0$ $p_i = 91.12$ psf (see CMULTIANC right-side soil pressure output)
$x_{i+1} = 1.0$ $p_{i+1} = 117.00$ psf (see CMULTIANC right-side soil pressure output)

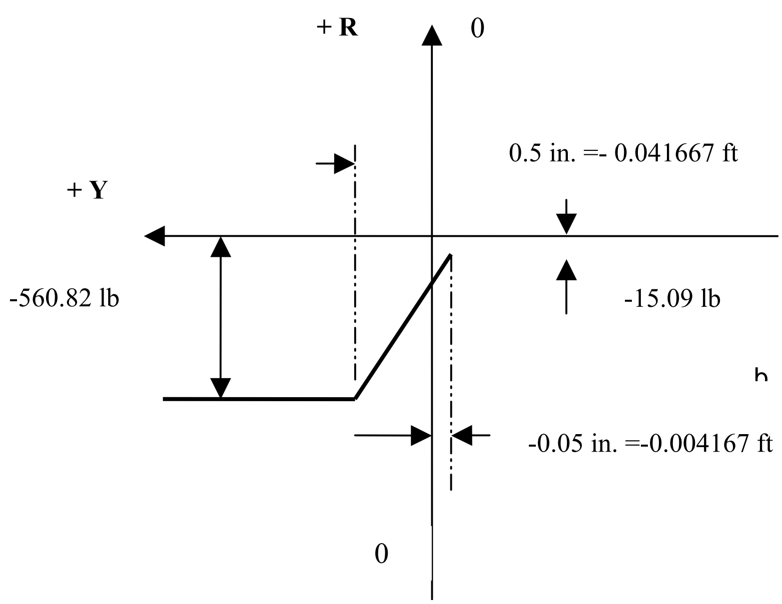
$F_i = (2\,p_i + p_{i+1})\,(x_{i+1} - x_i)/6 = 49.87$ lb

$F_{i+1} = (p_i + 2\,p_{i+1})\,(x_{i+1} - x_i)/6 = 54.19$ lb

A right-side R-y (SSI) curve for el -1.00+ is shown in Figure 2.13a, and a left-side R-y curve for el -14.00+ is shown in Figure 2.13b. Reference deflections for a cohesionless soil are per FHWA-RD-98-066. The active pressure reference deflection is equal to 0.05 in. (0.004167 ft) and the passive pressure reference deflection is equal to 0.50 in. (0.041667 ft). Limiting earth pressures for active and passive state conditions are as determined above. The complete CMULTIANC stage-1 excavation R-y curve output is provided below.



a. Right side at x = -1.00+



b. Left side at x = -14.00+

Figure 2.13. Right-side R-y (SSI) curve for el -1.00+

CMULITANC: SIMULATION OF CONSTRUCTION SEQUENCE FOR STIFF
WALL SYSTEMS WITH MULTIPLE LEVELS OF ANCHORS
DATE: 28-JULY-2002 TIME: 16:07:34

```
***************************************
*         INITIAL SSI CURVES          *
***************************************
```

I.--HEADING

'SOLETANCHE WALL INPUT FILE: S1 OUTPUT FILE: SO1
'DELTA = 2/3 PHI ON BOTH ACTIVE AND PASSIVE

II.--RIGHT SIDE CURVES

ELEV.	<---------ACTIVE--------->		<---------PASSIVE-------->	
(FT)	DISPL. (FT)	FORCE (LB)	DISPL. (FT)	FORCE (LB)
0.00	0.004167	49.87	-0.041667	1853.13
-1.00+	0.004167	54.19	-0.041667	2013.37
-1.00-	0.004167	62.81	-0.041667	2333.83
-2.00+	0.004167	67.12	-0.041667	2494.07
-2.00-	0.004167	75.75	-0.041667	2814.53
-3.00+	0.004167	80.06	-0.041667	2974.77
-3.00-	0.004167	88.69	-0.041667	3295.23
-4.00+	0.004167	93.00	-0.041667	3455.47
-4.00-	0.004167	101.62	-0.041667	3775.93
-5.00+	0.004167	105.94	-0.041667	3936.17
-5.00-	0.004167	114.56	-0.041667	4256.63
-6.00+	0.004167	118.87	-0.041667	4416.87
-6.00-	0.004167	127.50	-0.041667	4737.33
-7.00+	0.004167	131.81	-0.041667	4897.57
-7.00-	0.004167	140.44	-0.041667	5218.03
-8.00+	0.004167	144.75	-0.041667	5378.27
-8.00-	0.004167	153.37	-0.041667	5698.73
-9.00+	0.004167	157.69	-0.041667	5858.97
-9.00-	0.004167	166.31	-0.041667	6179.43
-10.00+	0.004167	170.62	-0.041667	6339.67
-10.00-	0.004167	179.25	-0.041667	6660.13
-11.00+	0.004167	183.56	-0.041667	6820.37
-11.00-	0.004167	192.19	-0.041667	7140.83
-12.00+	0.004167	196.50	-0.041667	7301.07
-12.00-	0.004167	101.48	-0.041667	3770.71
-12.50+	0.004167	102.56	-0.041667	3810.77
-12.50-	0.004167	104.72	-0.041667	3890.88
-13.00+	0.004167	105.80	-0.041667	3930.94
-13.00-	0.004167	218.06	-0.041667	8102.23
-14.00+	0.004167	222.37	-0.041667	8262.47
-14.00-	0.004167	231.00	-0.041667	8582.93
-15.00+	0.004167	235.31	-0.041667	8743.17
-15.00-	0.004167	243.94	-0.041667	9063.63
-16.00+	0.004167	248.25	-0.041667	9223.87
-16.00-	0.004167	256.87	-0.041667	9544.33
-17.00+	0.004167	261.19	-0.041667	9704.57
-17.00-	0.004167	269.81	-0.041667	10025.03
-18.00+	0.004167	274.12	-0.041667	10185.27
-18.00-	0.004167	282.75	-0.041667	10505.73

-19.00+	0.004167	287.06	-0.041667	10665.97
-19.00-	0.004167	295.69	-0.041667	10986.43
-20.00+	0.004167	300.00	-0.041667	11146.67
-20.00-	0.004167	308.62	-0.041667	11467.13
-21.00+	0.004167	312.94	-0.041667	11627.37
-21.00-	0.004167	321.56	-0.041667	11947.83
-22.00+	0.004167	325.87	-0.041667	12108.07
-22.00-	0.004167	334.50	-0.041667	12428.53
-23.00+	0.004167	338.81	-0.041667	12588.77
-23.00-	0.004167	347.43	-0.041667	12909.23
-24.00+	0.004167	351.75	-0.041667	13069.47
-24.00-	0.004167	360.37	-0.041667	13389.93
-25.00+	0.004167	364.68	-0.041667	13550.17
-25.00-	0.004167	373.31	-0.041667	13870.63
-26.00+	0.004167	377.62	-0.041667	14030.87
-26.00-	0.004167	386.25	-0.041667	14351.33
-27.00+	0.004167	390.56	-0.041667	14511.57
-27.00-	0.004167	399.18	-0.041667	14832.03
-28.00+	0.004167	403.50	-0.041667	14992.27
-28.00-	0.004167	412.12	-0.041667	15312.73
-29.00+	0.004167	416.43	-0.041667	15472.97
-29.00-	0.004167	425.06	-0.041667	15793.43
-30.00+	0.004167	429.37	-0.041667	15953.67
-30.00-	0.004167	438.00	-0.041667	16274.13
-31.00+	0.004167	442.31	-0.041667	16434.37
-31.00-	0.004167	450.93	-0.041667	16754.83
-32.00+	0.004167	455.25	-0.041667	16915.07
-32.00-	0.004167	463.87	-0.041667	17235.53
-33.00+	0.004167	468.18	-0.041667	17395.77
-33.00-	0.004167	476.81	-0.041667	17716.23
-34.00+	0.004167	481.12	-0.041667	17876.47
-34.00-	0.004167	489.75	-0.041667	18196.93
-35.00+	0.004167	494.06	-0.041667	18357.17
-35.00-	0.004167	502.68	-0.041667	18677.63
-36.00+	0.004167	507.00	-0.041667	18837.87
-36.00-	0.004167	515.62	-0.041667	19158.33
-37.00+	0.004167	519.93	-0.041667	19318.57
-37.00-	0.004167	528.56	-0.041667	19639.03
-38.00+	0.004167	532.87	-0.041667	19799.27
-38.00-	0.004167	541.50	-0.041667	20119.73
-39.00+	0.004167	545.81	-0.041667	20279.97
-39.00-	0.004167	554.43	-0.041667	20600.43
-40.00	0.004167	558.75	-0.041667	20760.67

III.--LEFT SIDE CURVES

ELEV.	<--------PASSIVE--------->		<---------ACTIVE--------->	
FT)	DISPL. (FT)	FORCE (LB)	DISPL. (FT)	FORCE (LB)
-12.50	0.041667	-40.06	-0.004167	-1.08
-13.00+	0.041667	-80.12	-0.004167	-2.16
-13.00-	0.041667	-400.58	-0.004167	-10.78
-14.00+	0.041667	-560.82	-0.004167	-15.09
-14.00-	0.041667	-881.28	-0.004167	-23.72
-15.00+	0.041667	-1041.52	-0.004167	-28.03
-15.00-	0.041667	-1361.98	-0.004167	-36.66

-16.00+	0.041667	-1522.22	-0.004167	-40.97
-16.00-	0.041667	-1842.68	-0.004167	-49.59
-17.00+	0.041667	-2002.92	-0.004167	-53.91
-17.00-	0.041667	-2323.38	-0.004167	-62.53
-18.00+	0.041667	-2483.62	-0.004167	-66.84
-18.00-	0.041667	-2804.08	-0.004167	-75.47
-19.00+	0.041667	-2964.32	-0.004167	-79.78
-19.00-	0.041667	-3284.78	-0.004167	-88.41
-20.00+	0.041667	-3445.02	-0.004167	-92.72
-20.00-	0.041667	-3765.48	-0.004167	-101.34
-21.00+	0.041667	-3925.72	-0.004167	-105.66
-21.00-	0.041667	-4246.18	-0.004167	-114.28
-22.00+	0.041667	-4406.42	-0.004167	-118.59
-22.00-	0.041667	-4726.88	-0.004167	-127.22
-23.00+	0.041667	-4887.12	-0.004167	-131.53
-23.00-	0.041667	-5207.58	-0.004167	-140.16
-24.00+	0.041667	-5367.82	-0.004167	-144.47
-24.00-	0.041667	-5688.28	-0.004167	-153.09
-25.00+	0.041667	-5848.52	-0.004167	-157.41
-25.00-	0.041667	-6168.98	-0.004167	-166.03
-26.00+	0.041667	-6329.22	-0.004167	-170.34
-26.00-	0.041667	-6649.68	-0.004167	-178.97
-27.00+	0.041667	-6809.92	-0.004167	-183.28
-27.00-	0.041667	-7130.38	-0.004167	-191.90
-28.00+	0.041667	-7290.62	-0.004167	-196.22
-28.00-	0.041667	-7611.08	-0.004167	-204.84
-29.00+	0.041667	-7771.32	-0.004167	-209.15
-29.00-	0.041667	-8091.78	-0.004167	-217.78
-30.00+	0.041667	-8252.02	-0.004167	-222.09
-30.00-	0.041667	-8572.48	-0.004167	-230.72
-31.00+	0.041667	-8732.72	-0.004167	-235.03
-31.00-	0.041667	-9053.18	-0.004167	-243.65
-32.00+	0.041667	-9213.42	-0.004167	-247.97
-32.00-	0.041667	-9533.88	-0.004167	-256.59
-33.00+	0.041667	-9694.12	-0.004167	-260.90
-33.00-	0.041667	-10014.58	-0.004167	-269.53
-34.00+	0.041667	-10174.82	-0.004167	-273.84
-34.00-	0.041667	-10495.28	-0.004167	-282.47
-35.00+	0.041667	-10655.52	-0.004167	-286.78
-35.00-	0.041667	-10975.98	-0.004167	-295.40
-36.00+	0.041667	-11136.22	-0.004167	-299.72
-36.00-	0.041667	-11456.68	-0.004167	-308.34
-37.00+	0.041667	-11616.92	-0.004167	-312.65
-37.00-	0.041667	-11937.38	-0.004167	-321.28
-38.00+	0.041667	-12097.62	-0.004167	-325.59
-38.00-	0.041667	-12418.08	-0.004167	-334.22
-39.00+	0.041667	-12578.32	-0.004167	-338.53
-39.00-	0.041667	-12898.78	-0.004167	-347.15
-40.00	0.041667	-13059.02	-0.004167	-351.47

The CMULTIANC results for the first-stage excavation analysis are provided below.

```
  CMULITANC: SIMULATION OF CONSTRUCTION SEQUENCE FOR STIFF
        WALL SYSTEMS WITH MULTIPLE LEVELS OF ANCHORS

    DATE: 28-JULY-2002                    TIME: 16:07:38

     *************************************************
     *        RESULTS FOR INITIAL SSI CURVES         *
     *************************************************

I.--HEADING

   'SOLETANCHE WALL      INPUT FILE: S1    OUTPUT FILE: SO1
   'DELTA = 2/3 PHI ON BOTH ACTIVE AND PASSIVE

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

II.--MAXIMA

                                     MAXIMUM       MINIMUM
    DEFLECTION (FT)             : 9.084E-02      1.258E-03
         AT ELEVATION (FT)      :      0.00         -40.00

    BENDING MOMENT (LB-FT)      : 2.700E+04     -8.464E+02
         AT ELEVATION (FT)      :    -18.00         -32.00

    SHEAR (LB)                  :   3290.32       -3660.16
         AT ELEVATION (FT)      :    -13.00         -23.00

    RIGHTSIDE SOIL PRESSURE (PSF):   3710.37
         AT ELEVATION (FT)      :    -40.00

    LEFTSIDE SOIL PRESSURE (PSF) :   3756.27
         AT ELEVATION (FT)      :    -40.00

III.--ANCHOR FORCES


      ELEVATION      ANCHOR       ANCHOR            ANCHOR
      AT ANCHOR      STATUS     DEFORMATION          FORCE
        (FT)                       (FT)              (LB)
         -10.00     INACTIVE
```

IV.--COMPLETE RESULTS

ELEV. (FT)	DEFLECTION (FT)	SHEAR FORCE (LB)	BENDING MOMENT (LB-FT)	<-SOIL PRESS.(PSF)-> LEFT	RIGHT
0.00	9.084E-02	0.00	0.00	0.00	91.12
-1.00	8.558E-02	104.06	49.87	0.00	117.00
-2.00	8.032E-02	234.00	216.75	0.00	142.87
-3.00	7.506E-02	389.81	526.50	0.00	168.75
-4.00	6.981E-02	571.50	1004.99	0.00	194.62
-5.00	6.458E-02	779.06	1678.11	0.00	220.50
-6.00	5.937E-02	1012.49	2571.73	0.00	246.37
-7.00	5.421E-02	1271.80	3711.72	0.00	272.25
-8.00	4.911E-02	1556.99	5123.96	0.00	298.12
-9.00	4.409E-02	1868.05	6834.33	0.00	324.00
-10.00	**3.918E-02**	2204.98	8868.69	0.00	349.87
-11.00	3.442E-02	2567.79	11252.92	0.00	375.75
-12.00	2.984E-02	2956.48	14012.90	0.00	401.62
-12.50	2.763E-02	3160.52	15541.88	0.00	414.56
-13.00	2.549E-02	3290.32	17160.44	315.59	427.50
-14.00	2.141E-02	3148.09	20405.82	822.00	453.37
-15.00	1.767E-02	2601.25	23282.59	1178.83	479.25
-16.00	1.429E-02	1787.82	25459.77	1409.46	505.12
-17.00	1.133E-02	821.05	26732.61	1540.02	531.00
-18.00	8.799E-03	-212.55	26996.42	1597.82	556.87
-19.00	6.698E-03	-1253.66	26219.28	1609.71	582.75
-20.00	5.019E-03	-2268.69	24415.19	1600.31	608.62
-21.00	3.732E-03	-3139.59	21619.40	1590.36	851.98
-22.00	2.792E-03	-3624.66	18085.22	1595.24	1376.27
-23.00	2.144E-03	-3660.16	14332.09	1624.26	1781.03
-24.00	1.728E-03	-3384.32	10735.72	1681.30	2081.88
-25.00	1.487E-03	-2919.59	7539.93	1765.90	2298.08
-26.00	1.369E-03	-2366.70	4876.31	1874.62	2449.84
-27.00	1.331E-03	-1802.32	2787.90	2002.30	2556.36
-28.00	1.340E-03	-1279.59	1253.55	2143.17	2634.47
-29.00	1.370E-03	-830.61	210.50	2291.66	2697.89
-30.00	1.405E-03	-470.25	-426.31	2443.00	2757.01
-31.00	1.434E-03	-200.31	-749.12	2593.47	2818.95
-32.00	1.451E-03	-13.70	-846.45	2740.55	2888.06
-33.00	1.455E-03	101.88	-796.27	2882.78	2966.37
-34.00	1.446E-03	160.90	-662.49	3019.62	3054.19
-35.00	1.427E-03	177.80	-494.15	3151.23	3150.71
-36.00	1.400E-03	165.47	-326.32	3278.23	3254.46
-37.00	1.367E-03	134.51	-182.27	3401.44	3363.74
-38.00	1.332E-03	93.00	-75.92	3521.75	3476.92
-39.00	1.295E-03	46.77	-14.39	3639.87	3592.73
-40.00	1.258E-03	0.00	0.00	3756.27	3710.37

The computed displacement results for the first-stage CMULTIANC excavation analysis are illustrated in Figure 2.14, and the moment results are illustrated in Figure 2.15. The computed displacements in Figure 2.13 show the wall moving toward the excavation, as expected. Recall that displacements computed using a Winkler spring analysis are not intended to be a predictor of actual wall movement.

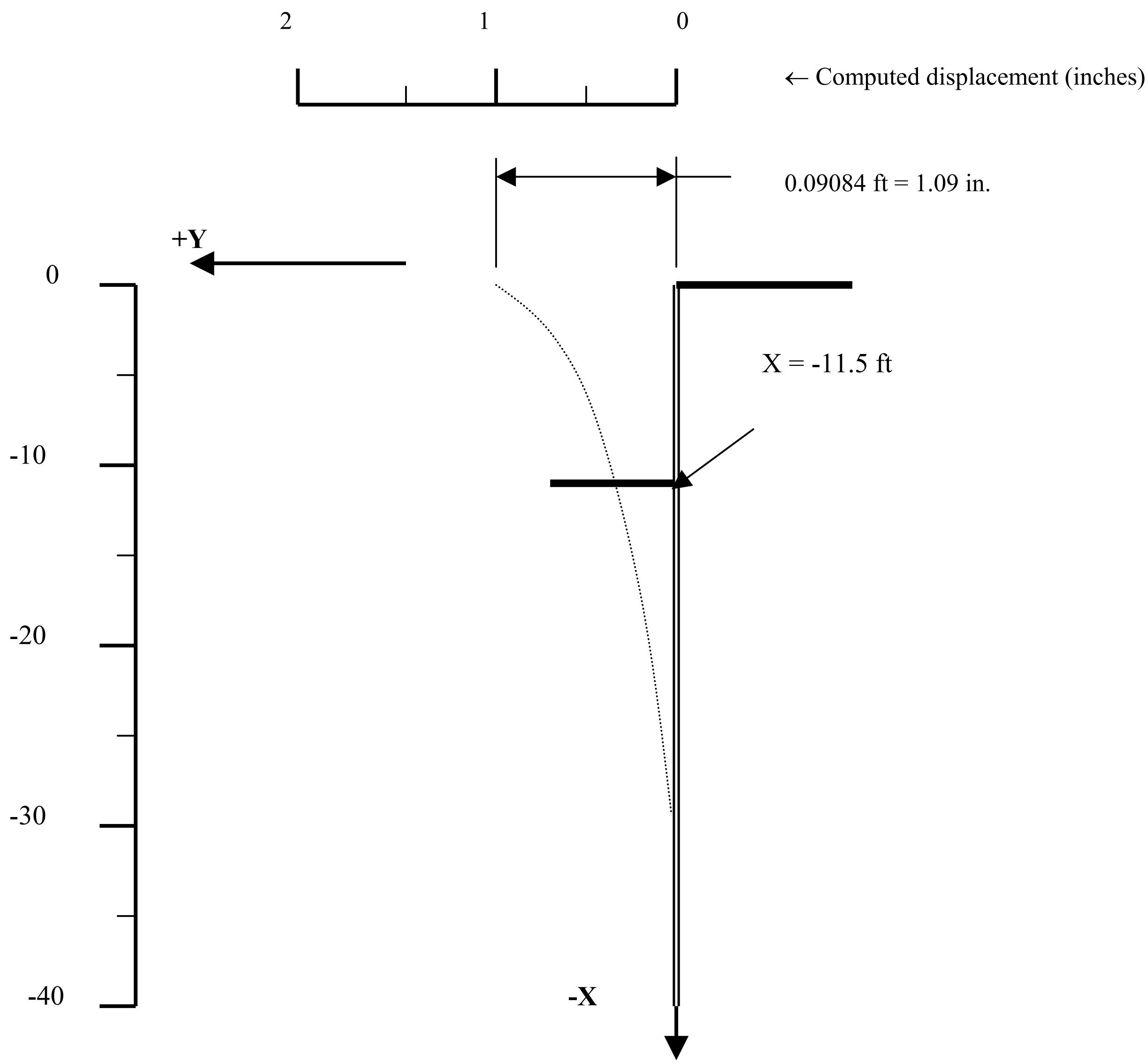


Figure 2.14. First-stage excavation computed displacements–coordinate system per CMULTIANC

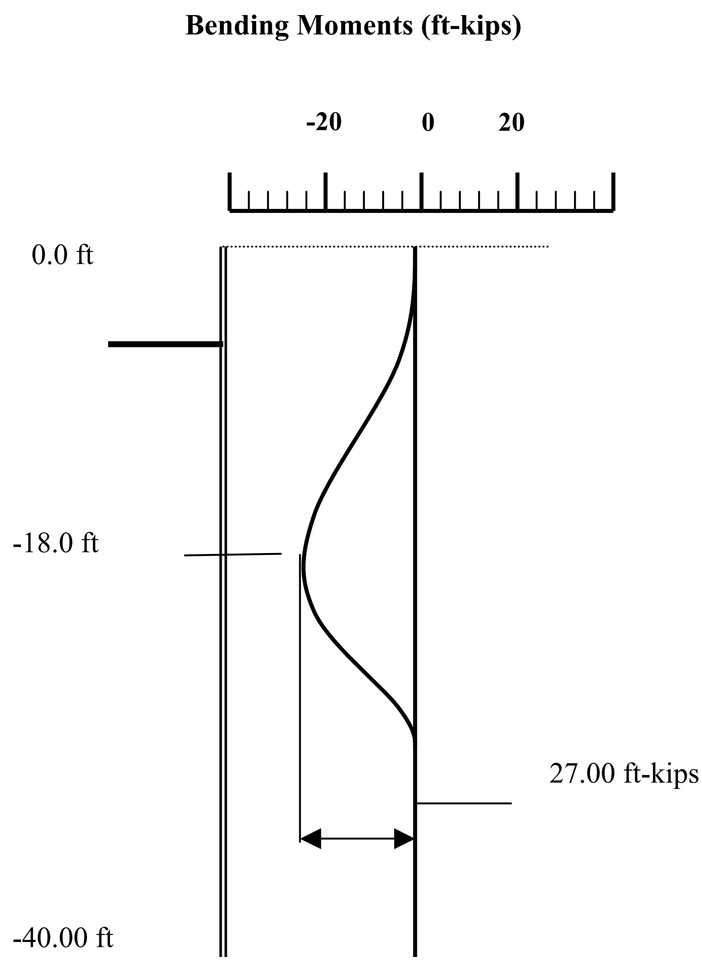


Figure 2.15. Stage-1 excavation moments

The results obtained from the Winkler 1 analysis are compared with those from the "Soletanche" PEROI 2 Elasto-Plastic Computer Program for the first-stage excavation analysis (see Table 2.7 below). Results are in good agreement.

Table 2.7
Winkler 1 – PEROI 2 Comparison

Wall Section A	**Winkler 1**	**PEROI 2**
First excavation depth	11.50 ft	11.50 ft (given)
Maximum moment	**27.00 ft-kips**	**29.33 ft-kips**
Maximum moment location	**18.0 ft**	**15.0 ft**
Maximum shear	**3.29 kips**	**3.32 kips**
Max. computed deflection (top)	**1.09 in.**	**1.16 in.**
Toe embedment (SE)	**40.0 ft**	**38.94 ft**

The R-Y curves in subsequent analyses must be shifted to capture the plastic movement that takes place in the soil as wall displaces toward the excavation for those conditions where actual wall computed displacements exceed active computed displacements of 0.05 in. Shifting of the soil and anchor load-displacement elastoplastic curves may be necessary for each stage of the analysis occurring after the first-stage excavation. (The first-stage excavation starts at an at-rest pressure condition.) Shifting is necessary in order to start each construction sequence considering the plastic (nonrecoverable) displacements accumulated from previous construction stages. The method for doing this is described in FHWA-RD-98-066. In the CMULTIANC analyses, this is accomplished by shifting the soil load displacement curves for the first-stage cantilever excavation as shown in Figure 2.16 for el -1.00+. This accounts for the active state plastic yielding that occurred during the first-stage excavation. When plastic deformations occur, i.e., computed displacements greater than those represented by the active or passive pressure limits, the soil-displacement curves are shifted so as to place the start point for the subsequent analysis at the approximate elastic/elastoplastic intersection point. This is necessary because plastic deformations cannot be recovered. As an example, in those cases where the previous construction-stage computed displacements toward the excavation exceeded active pressure limit state conditions, passive resistance in the retained soil will immediately mobilize as the anchor is tensioned and as the wall moves back toward the retained soil. With respect to the right-side soil spring (R-y curve) at el -1.00+ (see Figure 2.13a), the wall deflection is 0.08558 ft (1.03 in.), which exceeds the active displacement of 0.004167 ft (0.05 in.). The R-y curve, therefore, must be shifted as illustrated in Figure 2.16. The shifted R-y curve information is provided as output for the CMULTIANC analysis. This information for the Soletanche wall example is provided below.

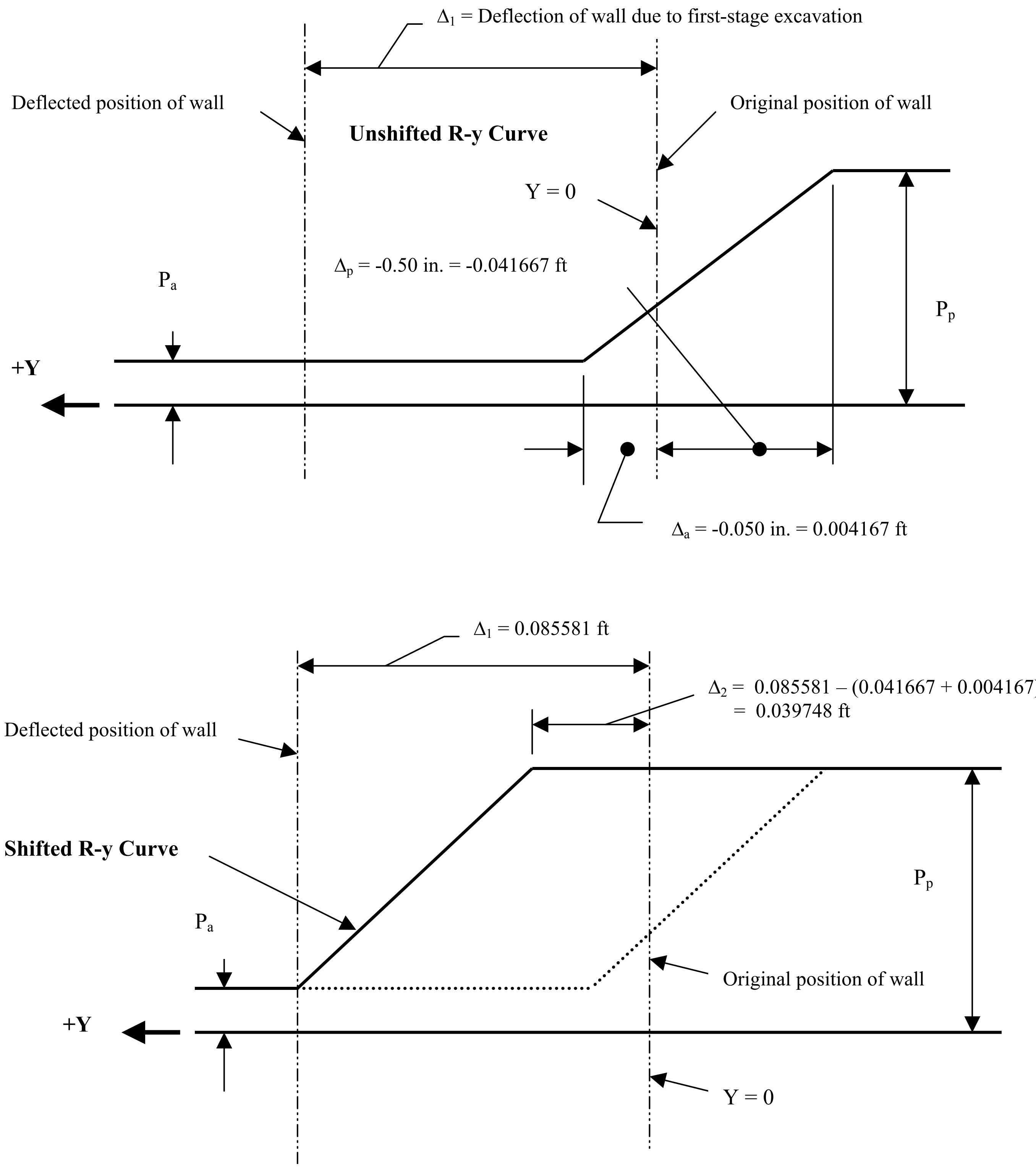


Figure 2.16. Shifted R-y curve for right side at el -1.00+

CUMULITANC: SIMULATION OF CONSTRUCTION SEQUENCE FOR STIFF
WALL SYSTEMS WITH MULTIPLE LEVELS OF ANCHORS

DATE: 28-JULY-200 TIME: 16:07:41

****
* SHIFTED SSI CURVES *
****

I.--HEADING

'SOLETANCHE WALL INPUT FILE: S1 OUTPUT FILE: SO1

'DELTA = 2/3 PHI ON BOTH ACTIVE AND PASSIVE

II.--RIGHT SIDE CURVES

ELEV.	<---------ACTIVE--------->		<---------PASSIVE-------->	
(FT)	DISPL. (FT)	FORCE (LB)	DISPL. (FT)	FORCE (LB)
0.00*	0.090845	49.87	0.045011	1853.13
-1.00+*	0.085581	54.19	0.039748	2013.37
-1.00-*	0.085581	62.81	0.039748	2333.83
-2.00+*	0.080319	67.12	0.034486	2494.07
-2.00-*	0.080319	75.75	0.034486	2814.53
-3.00+*	0.075061	80.06	0.029227	2974.77
-3.00-*	0.075061	88.69	0.029227	3295.23
-4.00+*	0.069811	93.00	0.023978	3455.47
-4.00-*	0.069811	101.62	0.023978	3775.93
-5.00+*	0.064579	105.94	0.018745	3936.17
-5.00-*	0.064579	114.56	0.018745	4256.63
-6.00+*	0.059374	118.87	0.013540	4416.87
-6.00-*	0.059374	127.50	0.013540	4737.33
-7.00+*	0.054211	131.81	0.008378	4897.57
-7.00-*	0.054211	140.44	0.008378	5218.03
-8.00+*	0.049109	144.75	0.003276	5378.27
-8.00-*	0.049109	153.37	0.003276	5698.73
-9.00+*	0.044091	157.69	-0.001743	5858.97
-9.00-*	0.044091	166.31	-0.001743	6179.43
-10.00+*	0.039184	170.62	-0.006650	6339.67
-10.00-*	0.039184	179.25	-0.006650	6660.13
-11.00+*	0.034421	183.56	-0.011412	6820.37
-11.00-*	0.034421	192.19	-0.011412	7140.83
-12.00+*	0.029841	196.50	-0.015992	7301.07
-12.00-*	0.029841	101.48	-0.015992	3770.71
-12.50+*	0.027633	102.56	-0.018200	3810.77
-12.50-*	0.027633	104.72	-0.018200	3890.88
-13.00+*	0.025488	105.80	-0.020345	3930.94
-13.00-*	0.025488	218.06	-0.020345	8102.23
-14.00+*	0.021413	222.37	-0.024420	8262.47
-14.00-*	0.021413	231.00	-0.024420	8582.93
-15.00+*	0.017667	235.31	-0.028166	8743.17
-15.00-*	0.017667	243.94	-0.028166	9063.63
-16.00+*	0.014295	248.25	-0.031538	9223.87
-16.00-*	0.014295	256.87	-0.031538	9544.33
-17.00+*	0.011332	261.19	-0.034501	9704.57
-17.00-*	0.011332	269.81	-0.034501	10025.03
-18.00+*	0.008799	274.12	-0.037035	10185.27

-18.00-*	0.008799	282.75	-0.037035	10505.73
-19.00+*	0.006698	287.06	-0.039135	10665.97
-19.00-*	0.006698	295.69	-0.039135	10986.43
-20.00+*	0.005019	300.00	-0.040814	11146.67
-20.00-*	0.005019	308.62	-0.040814	11467.13
-21.00+	0.004167	312.94	-0.041667	11627.37
-21.00-	0.004167	321.56	-0.041667	11947.83
-22.00+	0.004167	325.87	-0.041667	12108.07
-22.00-	0.004167	334.50	-0.041667	12428.53
-23.00+	0.004167	338.81	-0.041667	12588.77
-23.00-	0.004167	347.43	-0.041667	12909.23
-24.00+	0.004167	351.75	-0.041667	13069.47
-24.00-	0.004167	360.37	-0.041667	13389.93
-25.00+	0.004167	364.68	-0.041667	13550.17
-25.00-	0.004167	373.31	-0.041667	13870.63
-26.00+	0.004167	377.62	-0.041667	14030.87
-26.00-	0.004167	386.25	-0.041667	14351.33
-27.00+	0.004167	390.56	-0.041667	14511.57
-27.00-	0.004167	399.18	-0.041667	14832.03
-28.00+	0.004167	403.50	-0.041667	14992.27
-28.00-	0.004167	412.12	-0.041667	15312.73
-29.00+	0.004167	416.43	-0.041667	15472.97
-29.00-	0.004167	425.06	-0.041667	15793.43
-30.00+	0.004167	429.37	-0.041667	15953.67
-30.00-	0.004167	438.00	-0.041667	16274.13
-31.00+	0.004167	442.31	-0.041667	16434.37
-31.00-	0.004167	450.93	-0.041667	16754.83
-32.00+	0.004167	455.25	-0.041667	16915.07
-32.00-	0.004167	463.87	-0.041667	17235.53
-33.00+	0.004167	468.18	-0.041667	17395.77
-33.00-	0.004167	476.81	-0.041667	17716.23
-34.00+	0.004167	481.12	-0.041667	17876.47
-34.00-	0.004167	489.75	-0.041667	18196.93
-35.00+	0.004167	494.06	-0.041667	18357.17
-35.00-	0.004167	502.68	-0.041667	18677.63
-36.00+	0.004167	507.00	-0.041667	18837.87
-36.00-	0.004167	515.62	-0.041667	19158.33
-37.00+	0.004167	519.93	-0.041667	19318.57
-37.00-	0.004167	528.56	-0.041667	19639.03
-38.00+	0.004167	532.87	-0.041667	19799.27
-38.00-	0.004167	541.50	-0.041667	20119.73
-39.00+	0.004167	545.81	-0.041667	20279.97
-39.00-	0.004167	554.43	-0.041667	20600.43
-40.00	0.004167	558.75	-0.041667	20760.67

III.--LEFT SIDE CURVES

ELEV.	<--------PASSIVE--------->		<---------ACTIVE--------->	
(FT)	DISPL. (FT)	FORCE (LB)	DISPL. (FT)	FORCE (LB)
-12.50	0.041667	-40.06	-0.004167	-1.08
-13.00+	0.041667	-80.12	-0.004167	-2.16
-13.00-	0.041667	-400.58	-0.004167	-10.78
-14.00+	0.041667	-560.82	-0.004167	-15.09
-14.00-	0.041667	-881.28	-0.004167	-23.72
-15.00+	0.041667	-1041.52	-0.004167	-28.03
-15.00-	0.041667	-1361.98	-0.004167	-36.66

-16.00+	0.041667	-1522.22	-0.004167	-40.97
-16.00-	0.041667	-1842.68	-0.004167	-49.59
-17.00+	0.041667	-2002.92	-0.004167	-53.91
-17.00-	0.041667	-2323.38	-0.004167	-62.53
-18.00+	0.041667	-2483.62	-0.004167	-66.84
-18.00-	0.041667	-2804.08	-0.004167	-75.47
-19.00+	0.041667	-2964.32	-0.004167	-79.78
-19.00-	0.041667	-3284.78	-0.004167	-88.41
-20.00+	0.041667	-3445.02	-0.004167	-92.72
-20.00-	0.041667	-3765.48	-0.004167	-101.34
-21.00+	0.041667	-3925.72	-0.004167	-105.66
-21.00-	0.041667	-4246.18	-0.004167	-114.28
-22.00+	0.041667	-4406.42	-0.004167	-118.59
-22.00-	0.041667	-4726.88	-0.004167	-127.22
-23.00+	0.041667	-4887.12	-0.004167	-131.53
-23.00-	0.041667	-5207.58	-0.004167	-140.16
-24.00+	0.041667	-5367.82	-0.004167	-144.47
-24.00-	0.041667	-5688.28	-0.004167	-153.09
-25.00+	0.041667	-5848.52	-0.004167	-157.41
-25.00-	0.041667	-6168.98	-0.004167	-166.03
-26.00+	0.041667	-6329.22	-0.004167	-170.34
-26.00-	0.041667	-6649.68	-0.004167	-178.97
-27.00+	0.041667	-6809.92	-0.004167	-183.28
-27.00-	0.041667	-7130.38	-0.004167	-191.90
-28.00+	0.041667	-7290.62	-0.004167	-196.22
-28.00-	0.041667	-7611.08	-0.004167	-204.84
-29.00+	0.041667	-7771.32	-0.004167	-209.15
-29.00-	0.041667	-8091.78	-0.004167	-217.78
-30.00+	0.041667	-8252.02	-0.004167	-222.09
-30.00-	0.041667	-8572.48	-0.004167	-230.72
-31.00+	0.041667	-8732.72	-0.004167	-235.03
-31.00-	0.041667	-9053.18	-0.004167	-243.65
-32.00+	0.041667	-9213.42	-0.004167	-247.97
-32.00-	0.041667	-9533.88	-0.004167	-256.59
-33.00+	0.041667	-9694.12	-0.004167	-260.90
-33.00-	0.041667	-10014.58	-0.004167	-269.53
-34.00+	0.041667	-10174.82	-0.004167	-273.84
-34.00-	0.041667	-10495.28	-0.004167	-282.47
-35.00+	0.041667	-10655.52	-0.004167	-286.78
-35.00-	0.041667	-10975.98	-0.004167	-295.40
-36.00+	0.041667	-11136.22	-0.004167	-299.72
-36.00-	0.041667	-11456.68	-0.004167	-308.34
-37.00+	0.041667	-11616.92	-0.004167	-312.65
-37.00-	0.041667	-11937.38	-0.004167	-321.28
-38.00+	0.041667	-12097.62	-0.004167	-325.59
-38.00-	0.041667	-12418.08	-0.004167	-334.22
-39.00+	0.041667	-12578.32	-0.004167	-338.53
-39.00-	0.041667	-12898.78	-0.004167	-347.15
-40.00	0.041667	-13059.02	-0.004167	-351.47

(Note: * Indicates shifted curve.)

In CMULTIANC, the first-stage excavation analysis is rerun with the shifted R-y curves to verify the results are unchanged from those obtained with the unshifted curves.

2.6.2 Stage 2 construction analysis

The purpose of this analysis stage is to determine the wall deflection with the total anchor lock-off load applied. The wall computed-displacement occurring at the anchor location with the lock-off load, applied as a force, will be used in the development of an anchor spring (CMULTIANC concentrated spring) for subsequent excavation analyses. Representation of the anchor by a concentrated spring allows the anchor load deformation characteristics to be considered in the excavation analysis and permits a determination of the final anchor load based on these load-deformation characteristics.

The anchor properties used are in accordance with those selected for the Soletanche example, specifically:

a. Dywidag 1-3/8-in.-diam anchors at 8.2 ft on center

b. Anchor load at lock-off = 66 kips

c. Ultimate anchor load = 237 kips

d. Effective unbonded length = 30 ft

e. Bar area = 1.58 in.^{2}

f. Anchor inclination = 11.3 deg

g. Spacing = 8.2 ft

The horizontal component of lock-off load (66 kips) is applied as a horizontal force to first-stage Winkler spring analytical model.

$F_H = 66/8.2 = 8.0$ kips/ft

The anchor load is applied as shown in Figure 2.17.

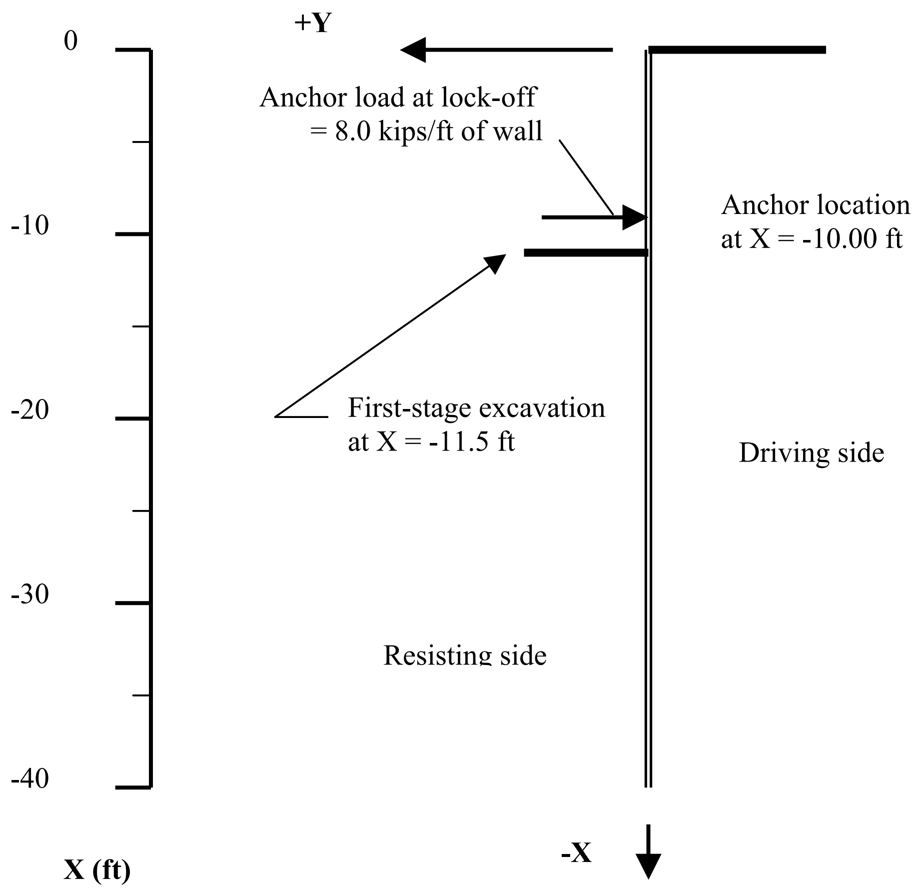


Figure 2.17. Intermediate construction stage–(first-stage excavation with anchor lock-off load), coordinate system per CMULTIANC

The CMULTIANC results for the intermediate construction stage with the anchor lock-off load are provided below.

```
  CMULITANC: SIMULATION OF CONSTRUCTION SEQUENCE FOR STIFF
        WALL SYSTEMS WITH MULTIPLE LEVELS OF ANCHORS
     DATE: 28-JULY-2002                 TIME: 16:07:48


     *************************************************
     *  RESULTS AFTER ANCHOR LOCK OFF LOAD AT EL -10 *
     *************************************************

I.--HEADING
   'SOLETANCHE WALL      INPUT FILE: S1    OUTPUT FILE: SO1
   'DELTA = 2/3 PHI ON BOTH ACTIVE AND PASSIVE
```

II.--MAXIMA

		MAXIMUM	MINIMUM
DEFLECTION (FT)	:	9.057E-02	1.254E-03
AT ELEVATION (FT)	:	0.00	-40.00
BENDING MOMENT (LB-FT)	:	2.388E+04	-7.166E+02
AT ELEVATION (FT)	:	-18.00	-32.00
SHEAR (LB)	:	5767.03	-3258.12
AT ELEVATION (FT)	:	-10.00	-23.00
RIGHTSIDE SOIL PRESSURE (PSF)	:	3713.21	
AT ELEVATION (FT)	:	-40.00	
LEFTSIDE SOIL PRESSURE (PSF)	:	3754.48	
AT ELEVATION (FT)	:	-40.00	

III.--ANCHOR FORCES

ELEVATION AT ANCHOR (FT)	ANCHOR STATUS	ANCHOR DEFORMATION (FT)	ANCHOR FORCE (LB)
-10.00	INACTIVE		

IV.--COMPLETE RESULTS

ELEV. (FT)	DEFLECTION (FT)	SHEAR FORCE (LB)	BENDING MOMENT (LB-FT)	<-SOIL PRESS. (PSF)-> LEFT	RIGHT
0.00	9.057E-02	0.00	0.00	0.00	110.94
-1.00	8.498E-02	140.76	60.72	0.00	172.81
-2.00	7.939E-02	350.09	294.25	0.00	248.09
-3.00	7.380E-02	641.31	775.87	0.00	336.58
-4.00	6.823E-02	1027.36	1594.07	0.00	437.73
-5.00	6.268E-02	1520.32	2850.00	0.00	550.30
-6.00	5.718E-02	2130.35	4656.22	0.00	671.77
-7.00	5.176E-02	2864.22	7134.22	0.00	797.71
-8.00	4.646E-02	3722.81	10409.93	0.00	920.84
-9.00	4.133E-02	4697.80	14606.48	0.00	1029.92
-10.00	**3.644E-02**	5767.03	19832.94	0.00	1108.46
-10.00	3.644E-02	-2232.97	19832.94	0.00	1108.46
-11.00	3.184E-02	-1108.30	18167.87	0.00	1139.71
-12.00	2.755E-02	26.48	17642.50	0.00	1127.91
-12.50	2.551E-02	585.93	17798.24	0.00	1109.31
-13.00	2.354E-02	1058.85	18218.15	295.72	1084.17
-14.00	1.983E-02	1564.77	19583.59	773.52	1019.69
-15.00	1.644E-02	1591.89	21195.21	1115.99	944.75
-16.00	1.338E-02	1258.96	22635.58	1344.35	868.32
-17.00	1.070E-02	670.67	23599.93	1481.59	797.46
-18.00	8.388E-03	-85.73	23880.14	1551.73	737.22
-19.00	6.464E-03	-942.91	23345.83	1578.57	690.64
-20.00	4.914E-03	-1854.33	21923.59	1584.25	658.99
-21.00	3.717E-03	-2686.26	19576.10	1587.72	859.60
-22.00	2.834E-03	-3180.63	16500.49	1603.27	1354.68
-23.00	2.217E-03	-3258.12	13176.28	1639.79	1741.81

-24.00	1.813E-03	-3043.27	9954.09	1701.17	2034.34
-25.00	1.571E-03	-2647.40	7065.08	1787.33	2249.18
-26.00	1.444E-03	-2162.87	4637.91	1895.42	2404.36
-27.00	1.394E-03	-1660.64	2719.69	2020.93	2517.15
-28.00	1.389E-03	-1190.50	1297.67	2158.72	2602.85
-29.00	1.406E-03	-783.17	319.79	2303.76	2674.04
-30.00	1.429E-03	-453.57	-287.81	2451.71	2740.31
-31.00	1.449E-03	-204.53	-606.81	2599.15	2808.37
-32.00	1.459E-03	-30.54	-716.59	2743.70	2882.33
-33.00	1.458E-03	78.89	-687.74	2883.98	2964.23
-34.00	1.446E-03	136.48	-578.64	3019.43	3054.51
-35.00	1.425E-03	155.08	-434.46	3150.15	3152.57
-36.00	1.397E-03	146.36	-287.86	3276.63	3257.14
-37.00	1.364E-03	119.96	-160.75	3399.62	3366.76
-38.00	1.328E-03	83.36	-66.50	3519.86	3480.00
-39.00	1.291E-03	42.04	-12.10	3638.01	3595.72
-40.00	1.254E-03	0.00	0.00	3754.48	3713.21

Note: Very little inward deflection relative to the first-stage excavation occurred in the vicinity of the applied anchor load. The first-stage excavation computed displacement at X = 10.00 ft is approximately 0.03918 ft (0.470 in.), and the computed displacement at the X = 10.00 ft with the applied anchor load is approximately 0.03644 ft (0.437 in.). For the final construction stage analysis, the anchor load at lock-off will be assumed to occur at a wall computed-displacement of about 0.44 in.

The computed displacement results for the intermediate-stage CMULTIANC construction analysis are illustrated in Figure 2.18, and the moment results are illustrated in Figure 2.19.

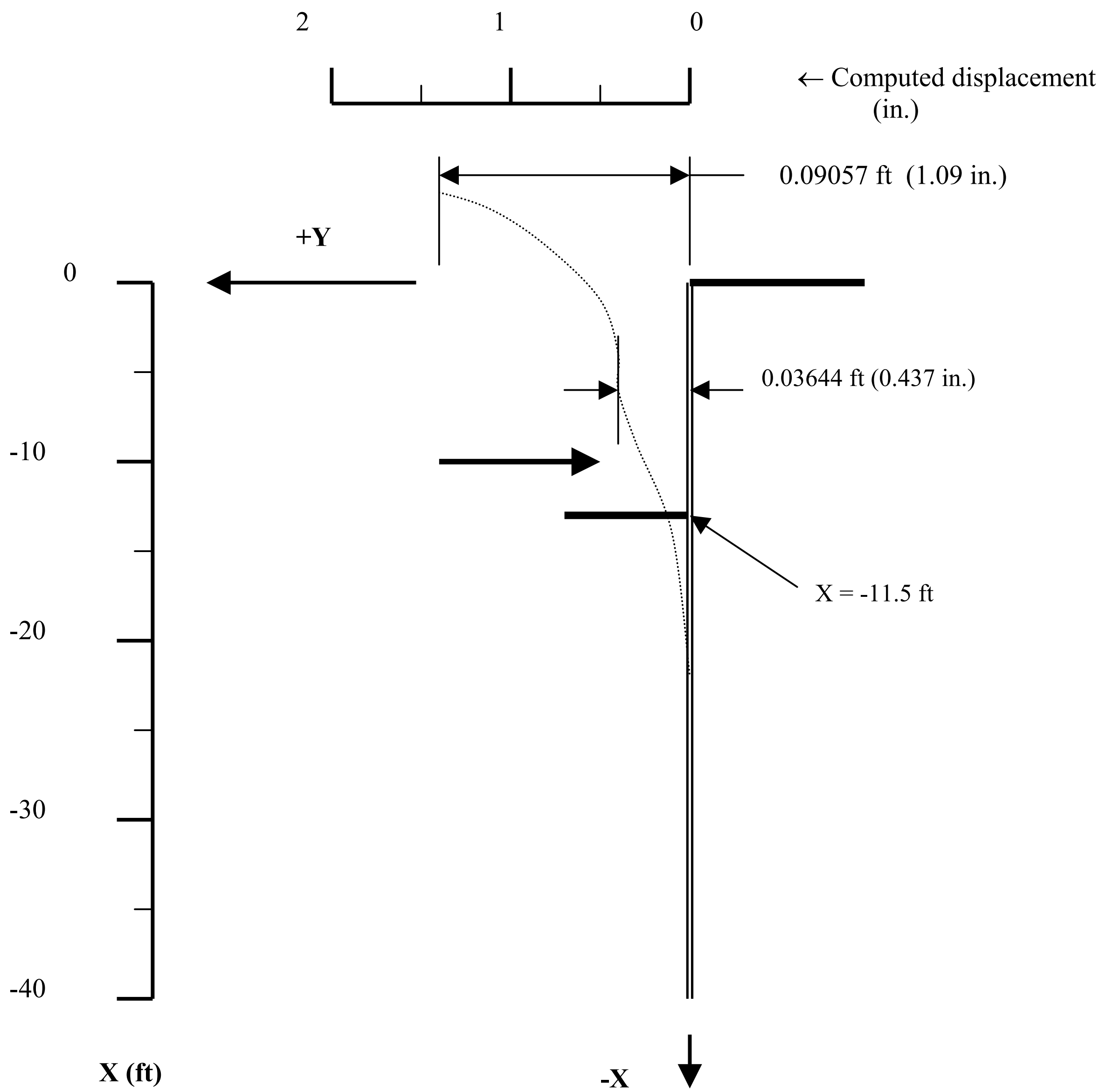


Figure 2.18. Wall computed-displacements–intermediate construction stage

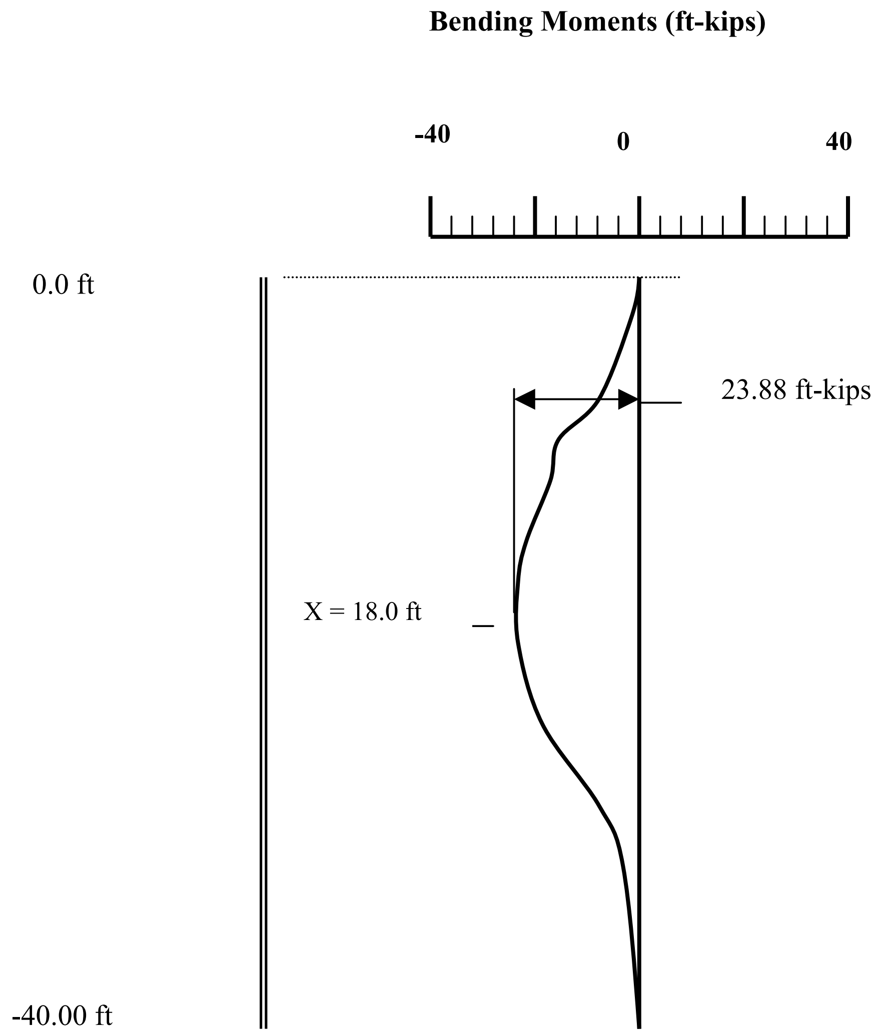


Figure 2.19. Wall moments–intermediate construction stage

The results obtained from the Winkler 1 analysis are compared with those from the “Soletanche” PEROI 2 Elasto-Plastic Computer Program for the second-stage construction (first-stage excavation with anchor force as load). The results (presented in Table 2.8) are in reasonable agreement.

Table 2.8
Winkler 1 – PEROI 2 Comparison

Wall Section A	Winkler 1	PAROI 2
First excavation depth	11.50 ft	11.50 ft (given)
Maximum moment	23.9 ft-kips	26.9 ft-kips
Maximum moment location	18.0 feet	15.0 ft
Maximum shear	5.77 kips	6.14 kips
Max. computed deflection (top)	1.09 in.	1.17 in.

2.6.3 Stage 3 construction analysis

The purpose of the Stage 3 construction analysis (final-stage excavation analysis) is to determine the forces on the wall and to determine the additional force that occurs in the anchor after excavation has been completed to final grade.

The anchor lock-off load was set equal to 8.0 kips, corresponding to a wall deflection of 0.437 in.—the deflection that takes place at the anchor location during the intermediate stage of construction (see Figure 2.18).

The anchor properties used are in accordance with those selected for the Soletanche example and are as follows:

a. Dywidag 1-3/8-in.-diam anchors at 8.2 ft on center

b. Anchor load at lock-off = 66 kips

c. Ultimate anchor load = 237 kips

d. Effective unbonded length = 30 ft

e. Bar area = 1.58 in.^{2}

f. Anchor inclination = 11.3 deg

g. Spacing = 8.2 ft

Computations for the Winkler 1, Stage 3, construction analysis are provided below. Stage 3, which represents the final excavation to grade, is depicted in Figure 2.20.

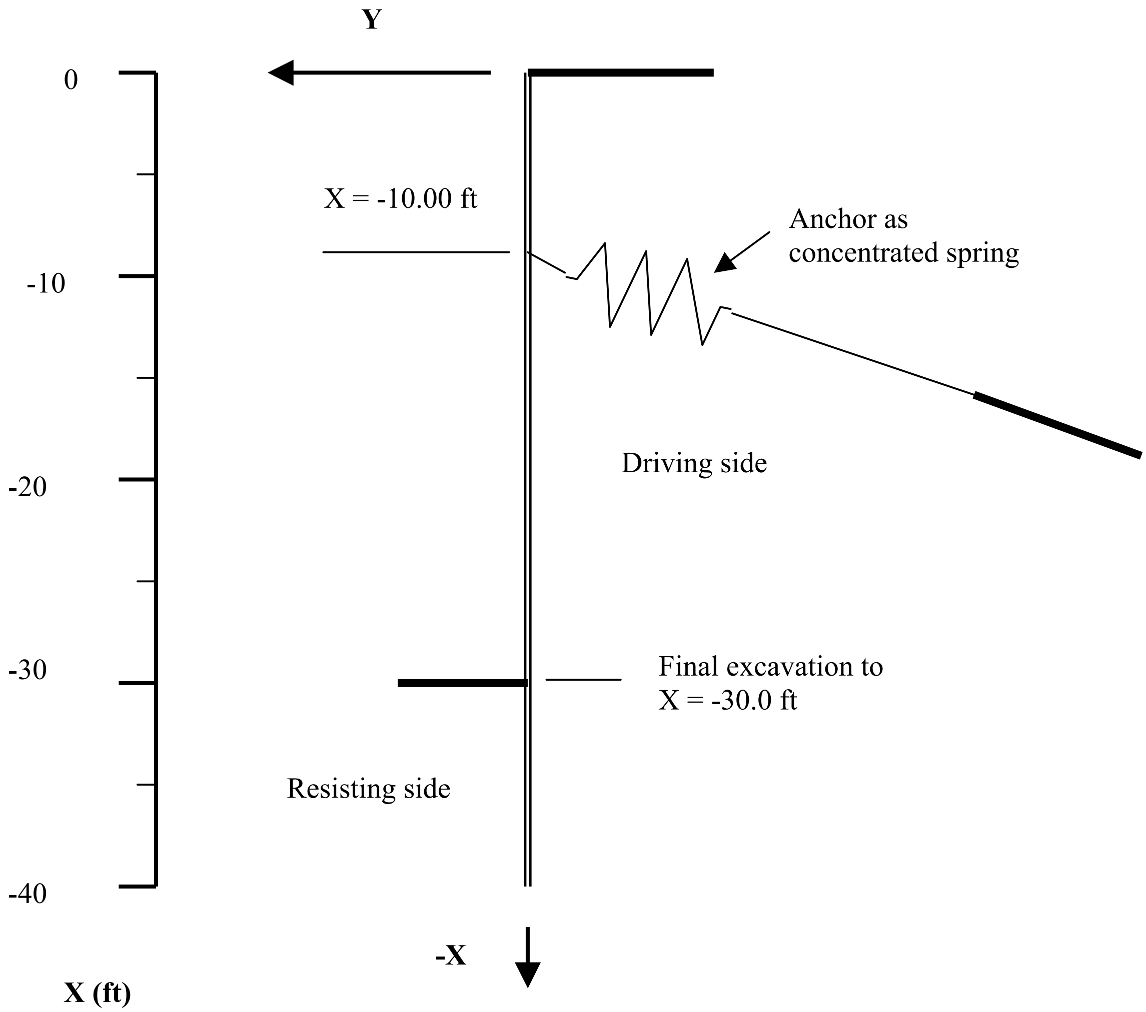
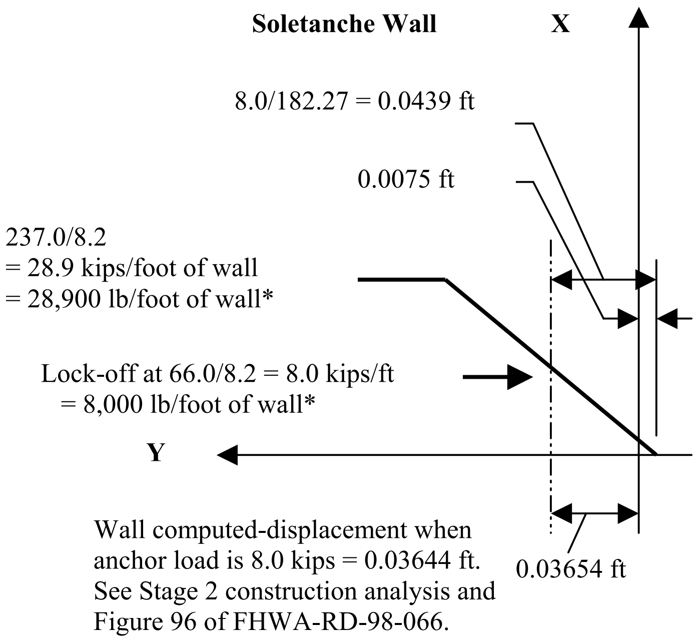


Figure 2.20. Stage 3 construction analysis (final excavation), coordinate system per CMULTIANC

The development of the concentrated spring used to represent the anchor for the final excavation stage is illustrated in Figure 2.21. This is accomplished internally within the CMULTIANC program. The final wall computed-displacement after excavating to el -30 will be used to calculate the final anchor force.



Compute anchor stiffness
for CMULTIANC analysis
(stiffness in y-direction)

P = anchor load
δ_L = anchor elongation, longitud. direction
A = anchor cross-sectional area
L_U = effective anchor length
E_s = anchor modulus of elasticity
S = anchor spacing (in plan)

Displacement at anchor location for final excavation is 0.05981 ft. Therefore, the load in the anchor after final excavation is
182.27 (0.5981+0.0075) = 12.26 kips
which agrees with CMULTIANC results.

$$\delta_L = \frac{PL_U}{AE_s} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots P = \left(\frac{AE_s}{L_U}\right)\delta_Y \cos\alpha \qquad P_Y = \frac{AE_s}{L_U}\delta_Y \cos^2\alpha$$

Therefore, on a per-foot-of-wall basis, $k_Y = \dfrac{AE_s}{L_U S}\cos^2\alpha \qquad k_Y = \dfrac{1.58(29{,}000)}{29.48(8.2)}\cos^2(11.3^o)$

Dywidag 1-3/8-in.-diam anchors
Lock-off anchor load = 66.0 kips
Ultimate anchor load = 237 kips
Effective unbonded length = 29.48 ft
Bar area = 1.58 in.^{2}
Anchor inclination = 11.3 deg
Spacing (S) = 8.2 ft

$k_Y = 182.27$ kips/foot/foot of wall

$k_Y = 15189$ pounds/inch/foot of wall*

* Denotes CMULTIANC input.

Note: The tieback lock-off load and ultimate tension load used as input to this version of CMULTIANC are expressed as y-direction forces per foot of wall. Anchor forces provided as output are also expressed as y-direction forces per foot of wall. In the new version of CMULTIANC (under development), the tieback lock-off load and ultimate tension load used as input are to be expressed as “total” anchor forces acting along the axis of the tieback. The anchor forces provided as output are also expressed as “total” anchor forces along the axis of the tieback. The above changes will be more consistent with design practice and eliminate the need for

Figure 2.21. Anchor spring load-displacement curve (anchor lock-off load = 8.0 kips per foot of wall in y-direction)

Input and output for the CMULTIANC final-excavation stage analysis is provided below.

```
  CMULTIANC: SIMULATION OF CONSTRUCTION SEQUENCE FOR STIFF
        WALL SYSTEMS WITH MULTIPLE LEVELS OF ANCHORS
     DATE: 28-JULY-2002                 TIME: 16:07:52

                 ******************************
                 *      LIMIT PRESSURES       *
                 * AFTER EXCAVATE TO EL -30   *
                 ******************************

I.--HEADING
   'SOLETANCHE WALL      INPUT FILE: S1    OUTPUT FILE: SO1
   'DELTA = 2/3 PHI ON BOTH ACTIVE AND PASSIVE

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

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

ELEV.	<--LEFTSIDE PRESSURES (PSF)->			<-RIGHTSIDE PRESSURES (PSF)->		
(FT)	WATER	PASSIVE	ACTIVE	WATER	ACTIVE	PASSIVE
0.00	0.00	0.00	0.00	0.00	91.12	3385.80
-1.00	0.00	0.00	0.00	0.00	117.00	4347.20
-2.00	0.00	0.00	0.00	0.00	142.87	5308.60
-3.00	0.00	0.00	0.00	0.00	168.75	6270.00
-4.00	0.00	0.00	0.00	0.00	194.62	7231.40
-5.00	0.00	0.00	0.00	0.00	220.50	8192.80
-6.00	0.00	0.00	0.00	0.00	246.37	9154.20
-7.00	0.00	0.00	0.00	0.00	272.25	10115.60
-8.00	0.00	0.00	0.00	0.00	298.12	11077.00
-9.00	0.00	0.00	0.00	0.00	324.00	12038.40
-10.00	0.00	0.00	0.00	0.00	349.87	12999.80
-11.00	0.00	0.00	0.00	0.00	375.75	13961.20
-12.00	0.00	0.00	0.00	0.00	401.62	14922.60
-12.50	0.00	0.00	0.00	0.00	414.56	15403.30
-13.00	0.00	0.00	0.00	0.00	427.50	15884.00
-14.00	0.00	0.00	0.00	0.00	453.37	16845.40
-15.00	0.00	0.00	0.00	0.00	479.25	17806.80
-16.00	0.00	0.00	0.00	0.00	505.12	18768.20
-17.00	0.00	0.00	0.00	0.00	531.00	19729.60
-18.00	0.00	0.00	0.00	0.00	556.87	20691.00
-19.00	0.00	0.00	0.00	0.00	582.75	21652.40
-20.00	0.00	0.00	0.00	0.00	608.62	22613.80
-21.00	0.00	0.00	0.00	0.00	634.50	23575.20
-22.00	0.00	0.00	0.00	0.00	660.37	24536.60
-23.00	0.00	0.00	0.00	0.00	686.24	25498.00
-24.00	0.00	0.00	0.00	0.00	712.12	26459.40
-25.00	0.00	0.00	0.00	0.00	737.99	27420.80
-26.00	0.00	0.00	0.00	0.00	763.87	28382.20
-27.00	0.00	0.00	0.00	0.00	789.74	29343.60
-28.00	0.00	0.00	0.00	0.00	815.62	30305.00
-29.00	0.00	0.00	0.00	0.00	841.49	31266.40

-30.00	0.00	0.00	0.00	0.00	867.37	32227.80
-31.00	0.00	961.40	25.87	0.00	893.24	33189.20
-32.00	0.00	1922.80	51.75	0.00	919.12	34150.60
-33.00	0.00	2884.20	77.62	0.00	944.99	35112.00
-34.00	0.00	3845.60	103.50	0.00	970.87	36073.40
-35.00	0.00	4807.00	129.37	0.00	996.74	37034.80
-36.00	0.00	5768.40	155.25	0.00	1022.62	37996.20
-37.00	0.00	6729.80	181.12	0.00	1048.49	38957.60
-38.00	0.00	7691.20	207.00	0.00	1074.37	39919.00
-39.00	0.00	8652.60	232.87	0.00	1100.24	40880.40
-40.00	0.00	9614.00	258.75	0.00	1126.12	41841.80

CMULITANC: SIMULATION OF CONSTRUCTION SEQUENCE FOR STIFF
ALL SYSTEMS WITH MULTIPLE LEVELS OF ANCHORS

DATE: 28-JULY-2002 TIME: 16:07:52

```
**************************************
* SSI CURVES AFTER EXCAVATE TO EL -30 *
**************************************
```

I.--HEADING
'SOLETANCHE WALL INPUT FILE: S1 OUTPUT FILE: SO1
'DELTA = 2/3 PHI ON BOTH ACTIVE AND PASSIVE

II.--RIGHT SIDE CURVES

ELEV.	<---------ACTIVE--------->		<---------PASSIVE-------->	
(FT)	DISPL. (FT)	FORCE (LB)	DISPL. (FT)	FORCE (LB)
0.00*	0.090845	49.87	0.045011	1853.13
-1.00+*	0.085581	54.19	0.039748	2013.37
-1.00-*	0.085581	62.81	0.039748	2333.83
-2.00+*	0.080319	67.12	0.034486	2494.07
-2.00-*	0.080319	75.75	0.034486	2814.53
-3.00+*	0.075061	80.06	0.029227	2974.77
-3.00-*	0.075061	88.69	0.029227	3295.23
-4.00+*	0.069811	93.00	0.023978	3455.47
-4.00-*	0.069811	101.62	0.023978	3775.93
-5.00+*	0.064579	105.94	0.018745	3936.17
-5.00-*	0.064579	114.56	0.018745	4256.63
-6.00+*	0.059374	118.87	0.013540	4416.87
-6.00-*	0.059374	127.50	0.013540	4737.33
-7.00+*	0.054211	131.81	0.008378	4897.57
-7.00-*	0.054211	140.44	0.008378	5218.03
-8.00+*	0.049109	144.75	0.003276	5378.27
-8.00-*	0.049109	153.37	0.003276	5698.73
-9.00+*	0.044091	157.69	-0.001743	5858.97
-9.00-*	0.044091	166.31	-0.001743	6179.43
-10.00+*	0.039184	170.62	-0.006650	6339.67
-10.00-*	0.039184	179.25	-0.006650	6660.13
-11.00+*	0.034421	183.56	-0.011412	6820.37
-11.00-*	0.034421	192.19	-0.011412	7140.83
-12.00+*	0.029841	196.50	-0.015992	7301.07
-12.00-*	0.029841	101.48	-0.015992	3770.71
-12.50+*	0.027633	102.56	-0.018200	3810.77
-12.50-*	0.027633	104.72	-0.018200	3890.88
-13.00+*	0.025488	105.80	-0.020345	3930.94

-13.00-*	0.025488	218.06	-0.020345	8102.23
-14.00+*	0.021413	222.37	-0.024420	8262.47
-14.00-*	0.021413	231.00	-0.024420	8582.93
-15.00+*	0.017667	235.31	-0.028166	8743.17
-15.00-*	0.017667	243.94	-0.028166	9063.63
-16.00+*	0.014295	248.25	-0.031538	9223.87
-16.00-*	0.014295	256.87	-0.031538	9544.33
-17.00+*	0.011332	261.19	-0.034501	9704.57
-17.00-*	0.011332	269.81	-0.034501	10025.03
-18.00+*	0.008799	274.12	-0.037035	10185.27
-18.00-*	0.008799	282.75	-0.037035	10505.73
-19.00+*	0.006698	287.06	-0.039135	10665.97
-19.00-*	0.006698	295.69	-0.039135	10986.43
-20.00+*	0.005019	300.00	-0.040814	11146.67
-20.00-*	0.005019	308.62	-0.040814	11467.13
-21.00+	0.004167	312.94	-0.041667	11627.37
-21.00-	0.004167	321.56	-0.041667	11947.83
-22.00+	0.004167	325.87	-0.041667	12108.07
-22.00-	0.004167	334.50	-0.041667	12428.53
-23.00+	0.004167	338.81	-0.041667	12588.77
-23.00-	0.004167	347.43	-0.041667	12909.23
-24.00+	0.004167	351.75	-0.041667	13069.47
-24.00-	0.004167	360.37	-0.041667	13389.93
-25.00+	0.004167	364.68	-0.041667	13550.17
-25.00-	0.004167	373.31	-0.041667	13870.63
-26.00+	0.004167	377.62	-0.041667	14030.87
-26.00-	0.004167	386.25	-0.041667	14351.33
-27.00+	0.004167	390.56	-0.041667	14511.57
-27.00-	0.004167	399.18	-0.041667	14832.03
-28.00+	0.004167	403.50	-0.041667	14992.27
-28.00-	0.004167	412.12	-0.041667	15312.73
-29.00+	0.004167	416.43	-0.041667	15472.97
-29.00-	0.004167	425.06	-0.041667	15793.43
-30.00+	0.004167	429.37	-0.041667	15953.67
-30.00-	0.004167	438.00	-0.041667	16274.13
-31.00+	0.004167	442.31	-0.041667	16434.37
-31.00-	0.004167	450.93	-0.041667	16754.83
-32.00+	0.004167	455.25	-0.041667	16915.07
-32.00-	0.004167	463.87	-0.041667	17235.53
-33.00+	0.004167	468.18	-0.041667	17395.77
-33.00-	0.004167	476.81	-0.041667	17716.23
-34.00+	0.004167	481.12	-0.041667	17876.47
-34.00-	0.004167	489.75	-0.041667	18196.93
-35.00+	0.004167	494.06	-0.041667	18357.17
-35.00-	0.004167	502.68	-0.041667	18677.63
-36.00+	0.004167	507.00	-0.041667	18837.87
-36.00-	0.004167	515.62	-0.041667	19158.33
-37.00+	0.004167	519.93	-0.041667	19318.57
-37.00-	0.004167	528.56	-0.041667	19639.03
-38.00+	0.004167	532.87	-0.041667	19799.27
-38.00-	0.004167	541.50	-0.041667	20119.73
-39.00+	0.004167	545.81	-0.041667	20279.97
-39.00-	0.004167	554.43	-0.041667	20600.43
-40.00	0.004167	558.75	-0.041667	20760.67

III.--LEFT SIDE CURVES

ELEV. (FT)	PASSIVE DISPL. (FT)	PASSIVE FORCE (LB)	ACTIVE DISPL. (FT)	ACTIVE FORCE (LB)
-30.00	0.041667	-160.23	-0.004167	-4.31
-31.00+	0.041667	-320.47	-0.004167	-8.62
-31.00-	0.041667	-640.93	-0.004167	-17.25
-32.00+	0.041667	-801.17	-0.004167	-21.56
-32.00-	0.041667	-1121.63	-0.004167	-30.19
-33.00+	0.041667	-1281.87	-0.004167	-34.50
-33.00-	0.041667	-1602.33	-0.004167	-43.12
-34.00+	0.041667	-1762.57	-0.004167	-47.44
-34.00-	0.041667	-2083.03	-0.004167	-56.06
-35.00+	0.041667	-2243.27	-0.004167	-60.37
-35.00-	0.041667	-2563.73	-0.004167	-69.00
-36.00+	0.041667	-2723.97	-0.004167	-73.31
-36.00-	0.041667	-3044.43	-0.004167	-81.94
-37.00+	0.041667	-3204.67	-0.004167	-86.25
-37.00-	0.041667	-3525.13	-0.004167	-94.87
-38.00+	0.041667	-3685.37	-0.004167	-99.19
-38.00-	0.041667	-4005.83	-0.004167	-107.81
-39.00+	0.041667	-4166.07	-0.004167	-112.12
-39.00-	0.041667	-4486.53	-0.004167	-120.75
-40.00	0.041667	-4646.77	-0.004167	-125.06

(Note: * Indicates shifted curve.)

```
   CMULITANC: SIMULATION OF CONSTRUCTION SEQUENCE FOR STIFF
         WALL SYSTEMS WITH MULTIPLE LEVELS OF ANCHORS
     DATE: 28-JULY-2002                    TIME: 16:07:52

      *************************************************
      *        RESULTS AFTER EXCAVATE TO EL -30       *
      *************************************************

I.--HEADING
   'SOLETANCHE WALL      INPUT FILE: S1    OUTPUT FILE: SO1
   'DELTA = 2/3 PHI ON BOTH ACTIVE AND PASSIVE

II.--MAXIMA
                                  MAXIMUM       MINIMUM
   REFLECTION (FT)             :  7.873E-02     -8.061E-04
        AT ELEVATION (FT)      :       0.00         -40.00

   BENDING MOMENT (LB-FT)      :  3.109E+04     -2.666E+04
        AT ELEVATION (FT)      :     -10.00         -24.00

   SHEAR (LB)                  :    5198.81       -7419.27
        AT ELEVATION (FT)      :     -31.00         -10.00

   RIGHTSIDE SOIL PRESSURE (PSF):    5543.65
        AT ELEVATION (FT)      :     -40.00

   LEFTSIDE SOIL PRESSURE (PSF) :    2458.29
        AT ELEVATION (FT)      :     -35.00
```

III.--ANCHOR FORCES

ELEVATION AT ANCHOR (FT)	ANCHOR STATUS	ANCHOR DEFORMATION (FT)	ANCHOR FORCE (LB)
-10.00	ACTIVE	5.981E-02	12260.01

IV.--COMPLETE RESULTS

ELEV. (FT)	DEFLECTION (FT)	SHEAR FORCE (LB)	BENDING MOMENT (LB-FT)	<-SOIL PRESS. (PSF)- LEFT	RIGHT
0.00	7.873E-02	0.00	0.00	0.00	961.98
-1.00	7.629E-02	977.66	526.52	0.00	974.09
-2.00	7.387E-02	1909.15	2027.12	0.00	869.59
-3.00	7.148E-02	2676.25	4397.32	0.00	645.07
-4.00	6.917E-02	3155.57	7412.59	0.00	293.60
-5.00	6.697E-02	3414.81	10721.47	0.00	220.50
-6.00	6.495E-02	3648.25	14250.84	0.00	246.37
-7.00	6.316E-02	3907.56	18026.58	0.00	272.25
-8.00	6.166E-02	4192.74	22074.58	0.00	298.12
-9.00	6.052E-02	4503.80	26420.69	0.00	324.00
-10.00	5.981E-02	4840.74	31090.81	0.00	349.87
-10.00	5.981E-02	-7419.27	31090.81	0.00	349.87
-11.00	5.956E-02	-7056.46	23850.78	0.00	375.75
-12.00	5.971E-02	-6667.78	16986.51	0.00	401.62
-12.50	5.989E-02	-6463.73	13703.36	0.00	414.56
-13.00	6.013E-02	-6253.22	10523.86	0.00	427.50
-14.00	6.072E-02	-5812.78	4488.70	0.00	453.37
-15.00	6.138E-02	-5346.47	-1093.08	0.00	479.25
-16.00	6.203E-02	-4854.29	-6195.62	0.00	505.12
-17.00	6.258E-02	-4336.23	-10793.03	0.00	531.00
-18.00	6.296E-02	-3792.30	-14859.45	0.00	556.87
-19.00	6.309E-02	-3222.49	-18369.00	0.00	582.75
-20.00	6.293E-02	-2626.80	-21295.80	0.00	608.62
-21.00	6.243E-02	-2005.25	-23613.99	0.00	634.50
-22.00	6.155E-02	-1357.81	-25297.67	0.00	660.37
-23.00	6.027E-02	-684.51	-26320.99	0.00	686.24
-24.00	5.856E-02	14.68	-26658.06	0.00	712.12
-25.00	5.642E-02	739.73	-26283.01	0.00	737.99
-26.00	5.385E-02	1490.67	-25169.97	0.00	763.87
-27.00	5.088E-02	2267.47	-23293.06	0.00	789.74
-28.00	4.754E-02	3070.15	-20626.40	0.00	815.62
-29.00	4.387E-02	3898.71	-17144.13	0.00	841.49
-30.00	3.992E-02	4753.14	-12820.36	0.00	867.37
-31.00	3.577E-02	5198.81	-7783.52	841.03	893.24
-32.00	3.149E-02	4916.23	-2694.46	1507.38	919.12
-33.00	2.717E-02	4081.73	1806.33	1996.30	944.99
-34.00	2.287E-02	2871.46	5255.82	2310.87	970.87
-35.00	1.865E-02	1456.34	7365.31	2458.29	996.74
-36.00	1.455E-02	-0.93	8013.24	2447.70	1022.62
-37.00	1.058E-02	-1346.57	7236.09	2287.65	1048.49
-38.00	6.715E-03	-2434.03	5219.79	1983.86	1074.37
-39.00	2.935E-03	-2589.93	2294.00	1537.47	2169.26
-40.00	-8.061E-04	0.00	0.00	944.69	5543.65

The computed displacement results for the final-stage CMULTIANC construction analysis are illustrated in Figure 2.22, the moment and shear results are illustrated in Figure 2.23, and final net earth pressures are shown in Figure 2.24.

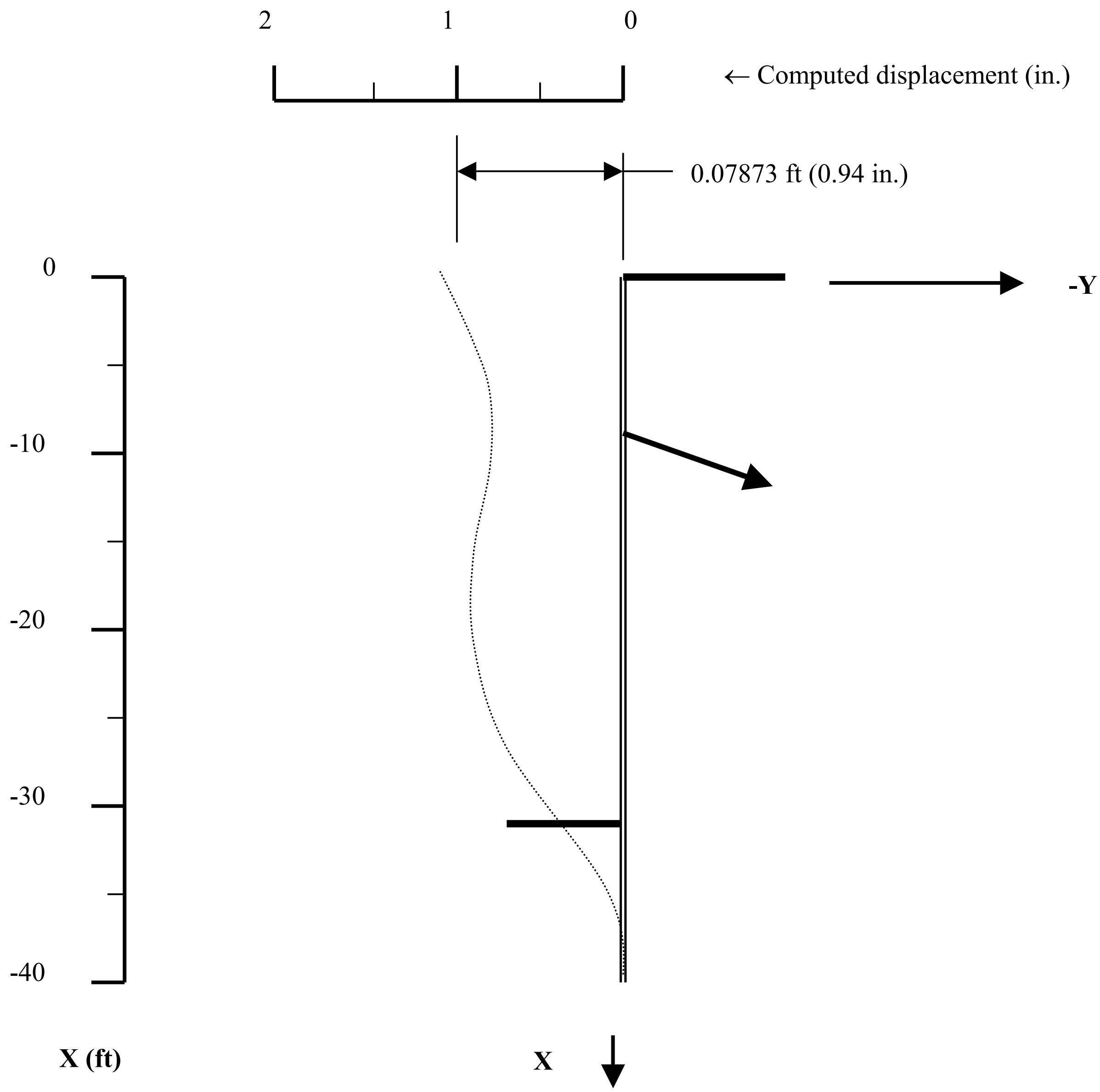


Figure 2.22. Wall computed displacements–Stage 3 construction

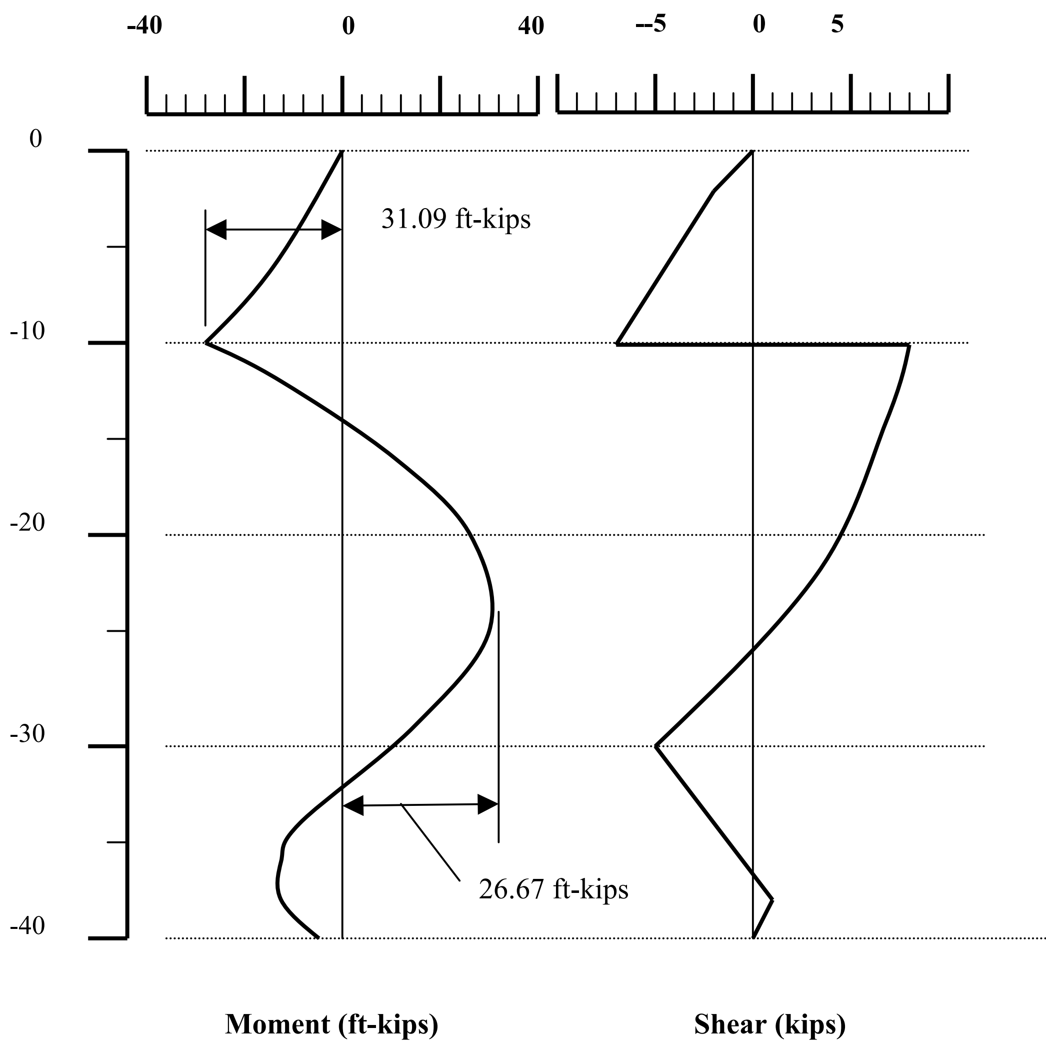


Figure 2.23. Wall moments and shears–Stage 3 construction

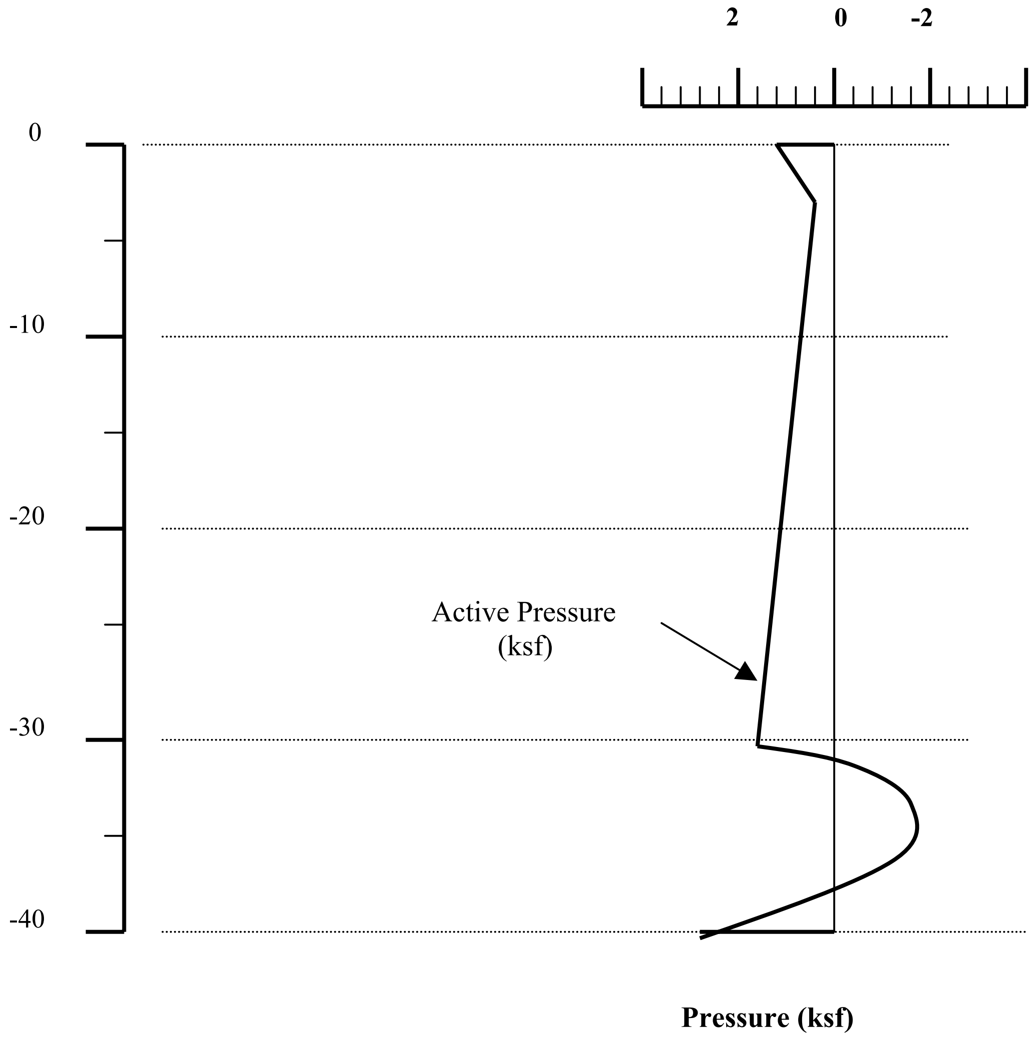


Figure 2.24. Soil pressures–Stage 3 construction

The results obtained from the Winkler 1 analysis are compared with those from the Soletanche PEROI 2 Elasto-Plastic Computer Program for Stage 3 construction (final-stage excavation with anchor as spring). The results, as shown in Table 2.9, are in reasonable agreement.

Table 2.9
Results Comparison–Winkler 1 Versus PEROI 2

Wall Section A	Winkler 1	PEROI 2
Maximum positive moment	**26.67 ft-kips**	**27.00 ft-kips**
Positive moment location	**24.00 ft**	**21.92 ft**
Maximum negative moment	**31.09 ft-kips**	**32.08 ft-kips**
Negative moment location	**At anchor location**	**At anchor location**
Maximum shear (at anchor)	**7.42 kips**	**8.11 kips**
Max. computed deflection (top)	**0.94 in. at top**	**1.05 in. at top**
Anchor force	**12.26 kips**	**13.15 kips**

2.7 Winkler 2 Analysis

Construction-sequencing analysis by SEI/ASCE method
(Reference: SEI/ASCE 2000)
Soil springs on resisting side only
Active earth pressures on driving side
Anchor springs per FHWA-RD-98-066 reference deflection method

2.7.1 First-stage excavation analysis

In this analysis, only the soil on the passive (excavation) side is modeled as elastoplastic springs. Active soil pressures (i.e., full mobilization of soil shear strength, FS = 1.0) were applied as loads on the retained soil side. This process is similar to that used in SEI/ACSE (2000). As per the Winkler 1 analysis, the elastoplastic springs used for the Winkler 2 construction-sequencing analysis are in accordance with the reference deflection method described in FHWA-RD-98-066.

As before, the active pressure coefficient (K_a) and the passive pressure coefficient (K_p) are per Caquot-Kerisel (1973), according to Soletanche practice. The following soil properties, consistent with previous analyses, were used in the analysis:

ϕ = 35 deg

δ = - 2/3 ϕ = -23.3 deg (resisting side only)

γ = 115 pcf

Surcharge = 405 psf

K_a = 0.271 (δ = 0)

K_p = 7.346 (δ = -2/3 ϕ)

The first-stage excavation analysis is illustrated in Figure 2.25. Soil springs for the Winkler 2 analysis for the first-stage excavation are shown in Figure 2.26. This analysis was performed using CBEAMC (Dawkins 1994b) and, as such, the conventions used are per the CBEAMC software rather than CMULTIANC.

Input and output for the first-stage excavation analysis are presented on the following pages.

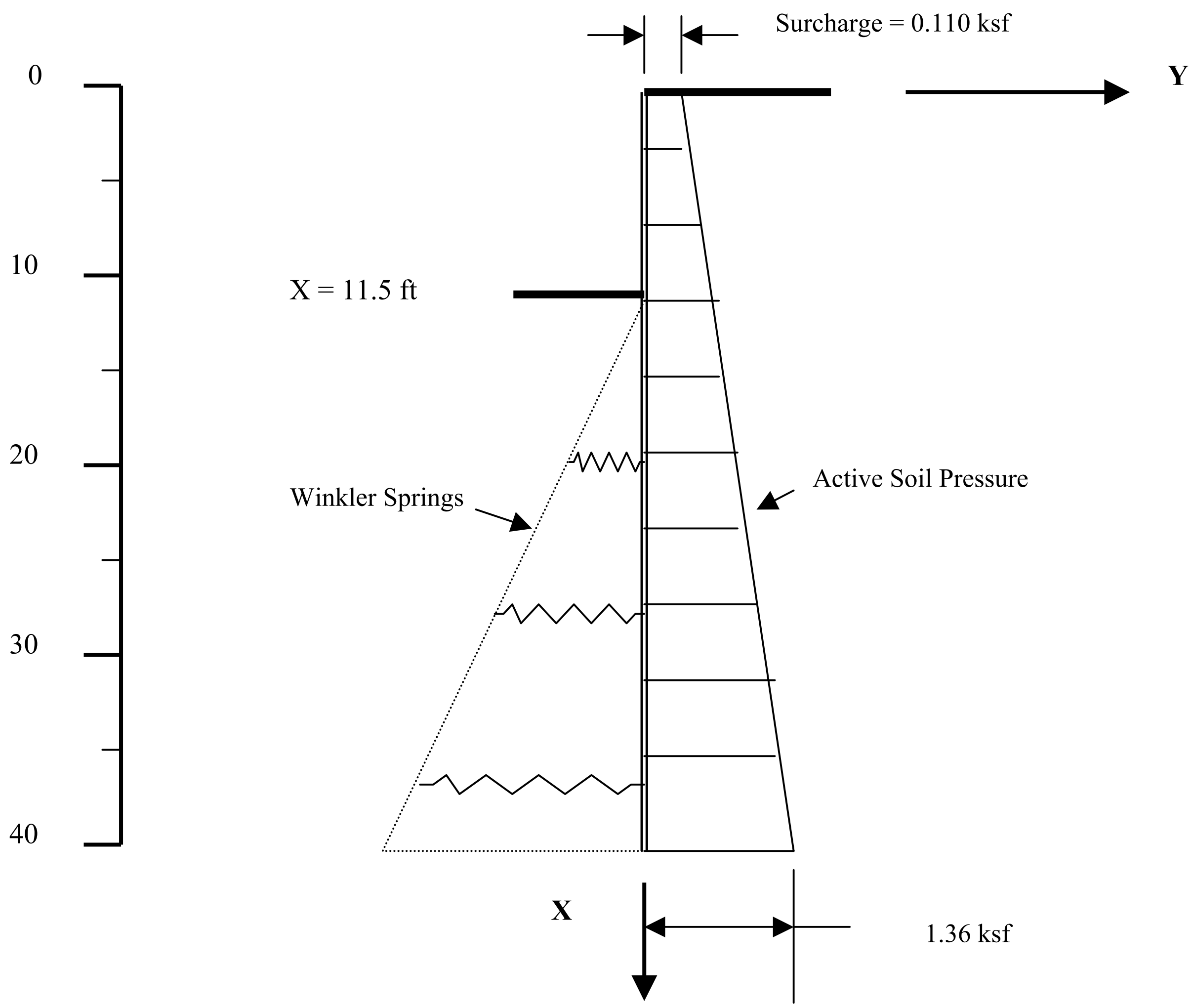
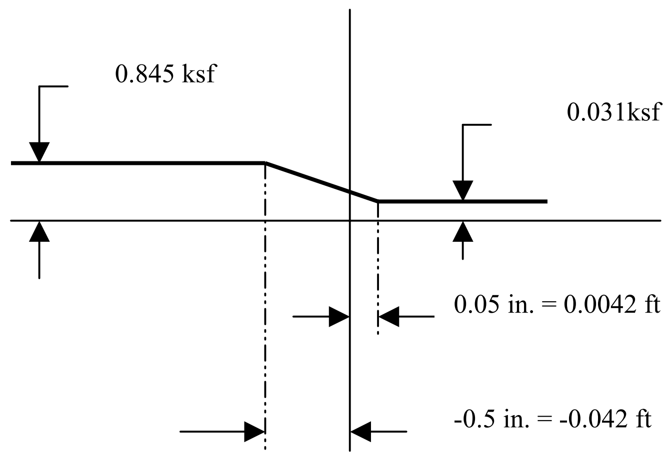
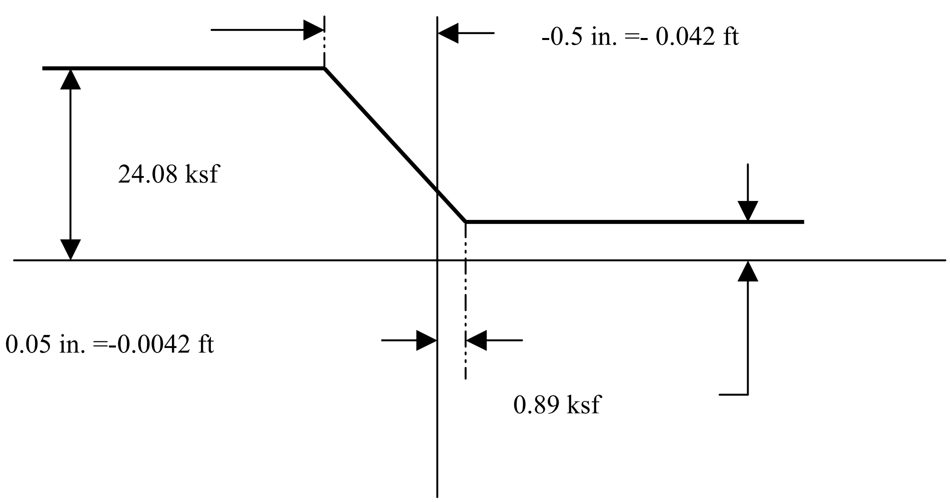


Figure 2.25. First-stage excavation analysis, SEI/ASCE method (Winkler 2), coordinate system per CBEAMC



Left side at X = 12.5



Left side at X = 40.0

Figure 2.26. Soil springs for Winkler 2 analysis–first-stage excavation analysis

INPUT FILE: SOL15

```
'SOLETANCHE WALL
'SEI - ASCE METHOD
'FIRST STAGE EXCAVATION
BEAM  FT  KSF  FT
   0    40   4.750E+05   1.15    0.13    1.15    0.13
NODES  FT  FT
   0    40    2
LOADS DISTRIBUTED  FT  K/FT
   0    0   -0.11    40    0   -1.36
FIXED  FT  FT
   40   0.000  FREE  FREE
NONLINEAR DISTRIBUTED  FT  FT  K/F
  Y   12.5    2    1    1
          -0.042    0.845
           0.0042    0.031
      40    2    1    1
          -0.042    24.08
           0.0042    0.89
FINISHED
```

OUTPUT FILE: SOL15.OUT

```
  PROGRAM CBEAMC - ANALYSIS OF BEAM-COLUMNS WITH NONLINEAR
                          SUPPORTS

  DATE: 25-JANUARY-2001                     TIME: 14:51:34

                  ************************
                  *  SUMMARY OF RESULTS  *
                  ************************


I.--HEADING
      'SOLETANCHE WALL
      'SEI - ASCE METHOD
      'FIRST STAGE EXCAVATION


II.--MAXIMA

                              MAXIMUM -COORD       MAXIMUM     X-COORD
                             POSITIVE   (FT)       NEGATIVE      (FT)
AXIAL DISPLACEMENT (IN)  :  0.000E+00   0.00     0.000E+00       0.00
LATERAL DISPLACEMENT (IN):  4.822E-02  28.00    -1.490E+00       0.00
ROTATION (RAD)           :  7.200E-03   0.00    -1.199E-04      32.00
AXIAL FORCE (K)          :  0.000E+00   0.00     0.000E+00       0.00
SHEAR FORCE (K)          :  3.500E+00  24.00    -4.603E+00      14.00
BENDING MOMENT (K-FT)    :  5.148E-01  36.00    -3.387E+01      18.00


III.--REACTIONS AT FIXED SUPPORTS
           X-C00RD  X-REACTION  Y-REACTION  MOMENT-REACTION
              (FT)         (K)         (K)            (K-FT)
             40.00   0.000E+00   0.000E+00         0.000E+00
```

PROGRAM CBEAMC - ANALYSIS OF BEAM-COLUMNS WITH NONLINEAR SUPPORTS

DATE: 25-JANUARY-2001 TIME: 14:51:34

```
**********************
*  COMPLETE RESULTS  *
**********************
```

I.--HEADING

'SOLETANCHE WALL
'SEI - ASCE METHOD
'FIRST STAGE EXCAVATION

II.--DISPLACEMENTS AND INTERNAL FORCES

<------DISPLACEMENTS------>			<------INTERNAL FORCES------>	
X-COORD (FT)	LATERAL (IN)	ROTATION (RAD)	SHEAR (K)	MOMENT (K-FT)
0.00	-1.490E+00	7.200E-03	-4.836E-13	-2.209E-13
2.00	-1.317E+00	7.198E-03	-2.825E-01	-2.617E-01
4.00	-1.145E+00	7.176E-03	-6.900E-01	-1.213E+00
6.00	-9.732E-01	7.109E-03	-1.223E+00	-3.105E+00
8.00	-8.041E-01	6.962E-03	-1.880E+00	-6.187E+00
10.00	-6.400E-01	6.693E-03	-2.662E+00	-1.071E+01
12.00	-4.843E-01	6.250E-03	-3.570E+00	-1.692E+01
14.00	-3.419E-01	5.576E-03	-4.603E+00	-2.507E+01
16.00	-2.188E-01	4.638E-03	-2.223E+00	-3.201E+01
18.00	-1.204E-01	3.558E-03	3.066E-01	-3.387E+01
20.00	-4.793E-02	2.494E-03	2.247E+00	-3.119E+01
22.00	4.659E-04	1.570E-03	3.300E+00	-2.549E+01
24.00	2.913E-02	8.558E-04	3.500E+00	-1.856E+01
26.00	4.334E-02	3.648E-04	3.076E+00	-1.190E+01
28.00	4.822E-02	7.120E-05	2.317E+00	-6.478E+00
30.00	4.796E-02	-7.264E-05	1.482E+00	-2.684E+00
32.00	4.550E-02	-1.199E-04	7.478E-01	-4.814E-01
34.00	4.259E-02	-1.177E-04	2.106E-01	4.400E-01
36.00	3.997E-02	-1.006E-04	-9.664E-02	5.148E-01
38.00	3.772E-02	-8.844E-05	-1.688E-01	2.101E-01
40.00	3.564E-02	-8.595E-05	0.000E+00	3.752E-14

III.--FORCES IN LINEAR DISTRIBUTED SPRINGS

NONE

IV.--FORCES IN NONLINEAR DISTRIBUTED SPRINGS

X-COORD (FT)	AXIAL (K/FT)	LATERAL (K/FT)
0.00	0.000E+00	0.000E+00
2.00	0.000E+00	0.000E+00
4.00	0.000E+00	0.000E+00
6.00	0.000E+00	0.000E+00
8.00	0.000E+00	0.000E+00
10.00	0.000E+00	0.000E+00
12.00	0.000E+00	0.000E+00
14.00	0.000E+00	0.000E+00

14.00	0.000E+00	1.517E+00
16.00	0.000E+00	1.919E+00
18.00	0.000E+00	1.832E+00
20.00	0.000E+00	1.492E+00
22.00	0.000E+00	1.097E+00
24.00	0.000E+00	7.805E-01
26.00	0.000E+00	6.029E-01
28.00	0.000E+00	5.680E-01
30.00	0.000E+00	6.440E-01
32.00	0.000E+00	7.874E-01
34.00	0.000E+00	9.604E-01
36.00	0.000E+00	1.140E+00
38.00	0.000E+00	1.321E+00
40.00	0.000E+00	1.507E+00

Wall bending moments for the first-stage excavation are shown in Figure 2.27.

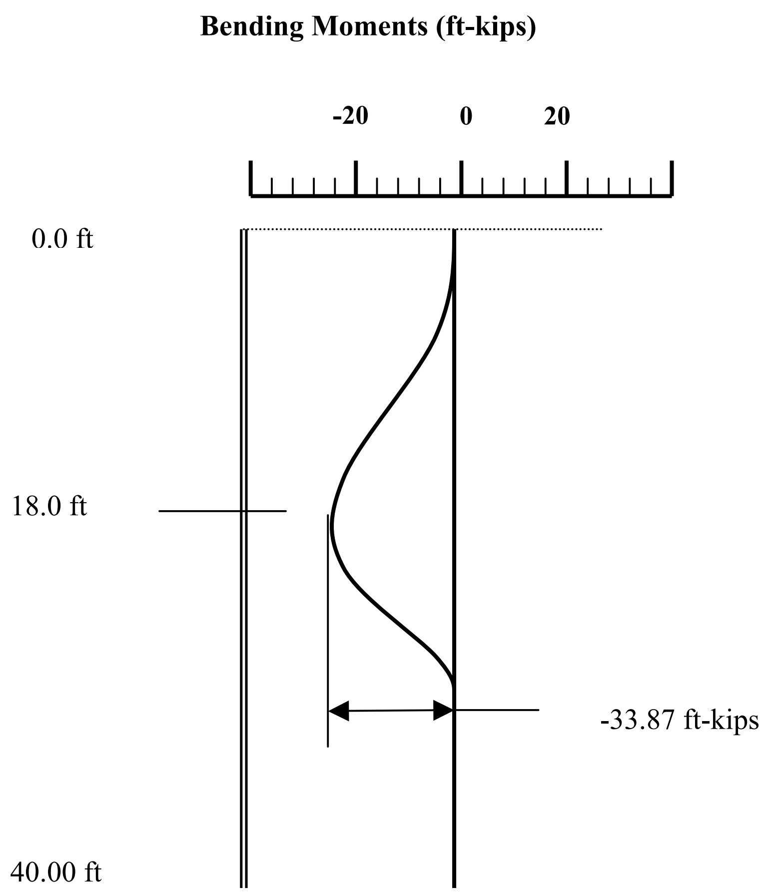


Figure 2.27. Wall moments, first-stage excavation analysis, SEI/ASCE method (Winkler 2)

The results obtained by the Winkler 1 method (SEI/ASCE method) are compared with those from the Winkler 1 method for the first-stage excavation analysis. The results are presented in Table 2.10. As expected, for the Stage 1 excavation, the results are in good agreement.

Table 2.10
Winkler 2–Winkler 1 Comparison for Stage 1 Excavation

Wall Section A	Winkler 2	Winkler 1
First excavation depth	12.50 ft (given)	12.50 ft
Maximum moment	**33.9 ft-kips**	**27.00 ft-kips**
Maximum moment location	**18.0 ft**	**18.0 ft**
Maximum shear	**4.60 kips**	**3.66 kips**
Max. computed deflection (top)	**1.49 in.**	**1.09 in.**
Toe embedment (SE)	**40.00 ft**	**40.00 ft**

2.7.2 Final-stage excavation analysis

The final-stage excavation analysis is illustrated in Figure 2.28. Note that the anchor is actually installed immediately following the first-stage excavation but is modeled only in the final-stage excavation of a Winkler 2 analysis. The analytical model is similar to that used for the first-stage excavation analysis. Soil springs for the final-stage excavation analysis are shown in Figure 2.29. The anchor spring (CBEAMC concentrated spring) used for the final-stage excavation analysis is shown in Figure 2.30. A disadvantage of the Winkler 2 analysis is that the computed wall displacement at lock-off load cannot be determined by the methods used in the Winkler 1 analysis. Instead, it is assumed in the development of the anchor spring that the computed wall displacement at lock-off is equal to zero.

CBEAMC calculations for the final stage excavation follow.

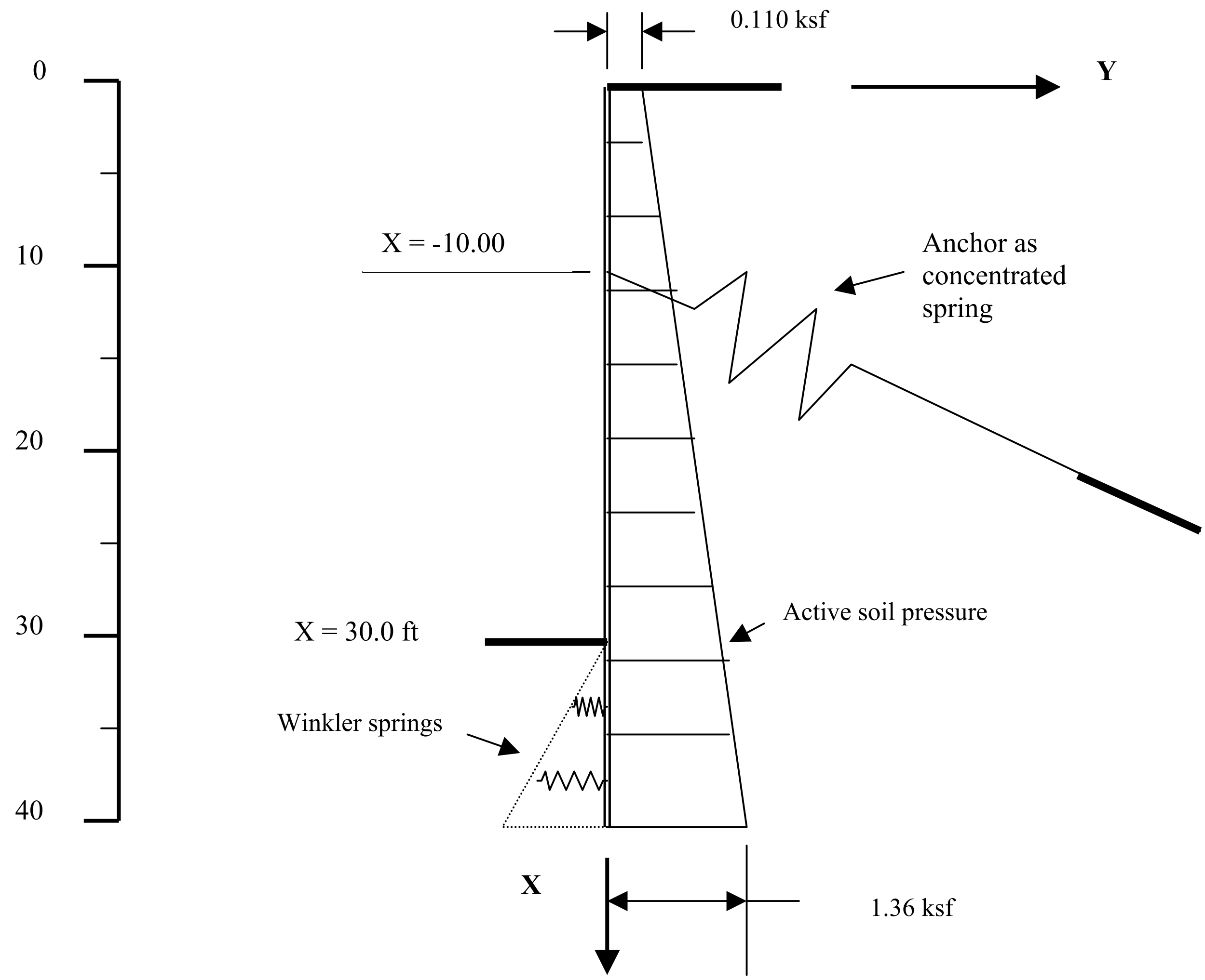
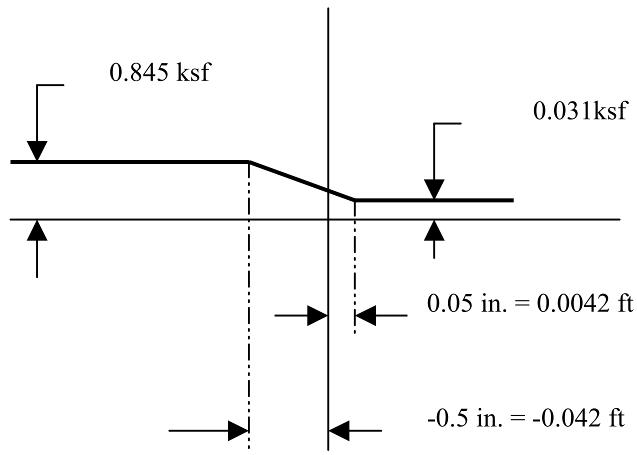
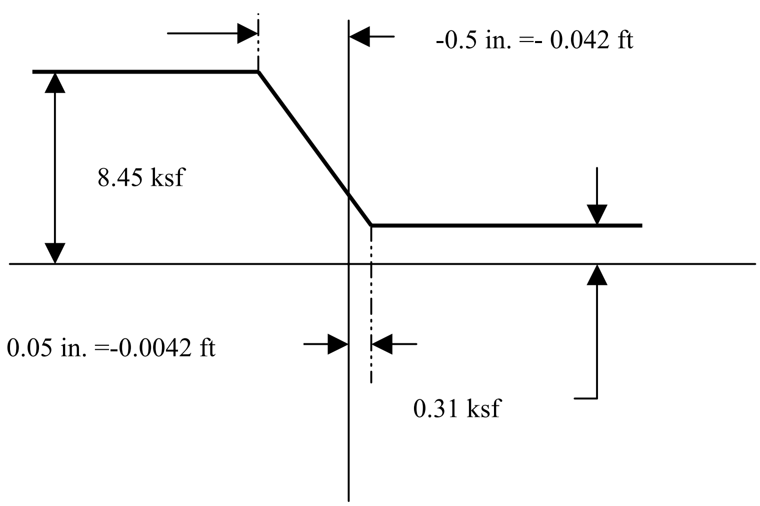


Figure 2.28. Final stage excavation analysis SEI/ASCE Method (Winkler 2), coordinate system per CBEAMC



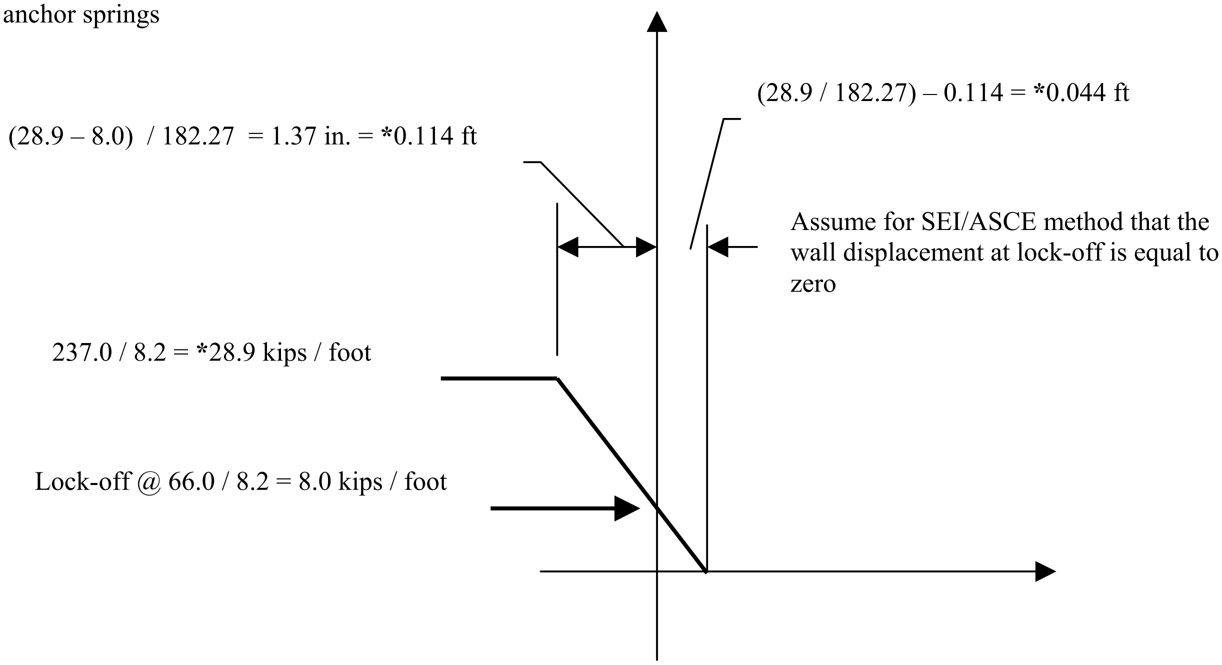
Left side at X = 30.0



Left side at X = 40.0

Figure 2.29. Soil springs for Winkler 2 analysis, final-stage excavation

* Asterisked values indicate information used in CBEAMC to develop nonlinear concentrated anchor springs



Anchor spring stiffness, on a per foot-of-wall-basis per Figure 2.22

$$k_y = \frac{AE_s}{L_U S}\cos\alpha = \frac{1.58(29{,}000)}{29.48(8.2)}\cos^2 11.3^o = 182.27 \text{ kips / foot / foot of wall}$$

Dywidag 1-3/8-in.-diam anchors @ 8.2 ft on center
Lock-off anchor load = 66.0 kips
Ultimate anchor load = 237 kips
Effective unbonded length = 29.48 ft
Bar area = 1.58 in.^{2}
Anchor inclination = 11.3 deg
Spacing = 8.2 ft

Figure 2.30. Anchor spring for Winkler 2 analysis, final-stage excavation

CBEAMC input and output for the final stage excavation is provided below.

INPUT FILE: SOL16

```
'SOLETANCHE WALL
'SEI - ASCE METHOD
'FINAL STAGE EXCAVATION
BEAM  FT  KSF  FT
   0    40   4.750E+05   1.15    0.13    1.15    0.13
NODES  FT  FT
   0    40    2
LOADS DISTRIBUTED  FT  K/FT
   0    0   -0.11    40    0   -1.36
FIXED  FT  FT
   40   0.000  FREE  FREE
NONLINEAR CONSENTRATED  FT  FT  K
   9.84  90  2  1  1
         -0.114  28.9
          0.044   0
NONLINEAR DISTRIBUTED  FT  FT  K/F
  Y   30.0    2    1    1
          -0.042    0.845
           0.0042    0.031
      40    2    1    1
          -0.042     8.45
           0.0042    0.31
FINISHED
```

OUTPUT FILE: SOL16.OUT

```
  PROGRAM CBEAMC - ANALYSIS OF BEAM-COLUMNS WITH NONLINEAR
                           SUPPORTS

  DATE: 26-JANUARY-2001                     TIME: 10:29:46

                  ************************
                  *  SUMMARY OF RESULTS  *
                  ************************

I.--HEADING
      'SOLETANCHE WALL
      'SEI - ASCE METHOD
      'FINAL STAGE EXCAVATION

II.--MAXIMA
                              MAXIMUM   -COORD      MAXIMUM    X-COORD
                             POSITIVE    (FT)      NEGATIVE      (FT)
AXIAL DISPLACEMENT (IN)  :  0.000E+00     0.00    0.000E+00       0.00
LATERAL DISPLACEMENT (IN):  5.314E-01     0.00   -8.954E-01      23.91
ROTATION (RAD)           :  6.339E-03    35.98   -6.599E-03      11.85
AXIAL FORCE (K)          :  0.000E+00     0.00    0.000E+00       0.00
SHEAR FORCE (K)          :  8.440E+00     9.84   -8.437E+00      31.96
BENDING MOMENT (K-FT)    :  5.263E+01    23.91   -1.029E+01       9.84
```

III.--REACTIONS AT FIXED SUPPORTS

X-C00RD (FT)	X-REACTION (K)	Y-REACTION (K)	MOMENT-REACTION (K-FT)
40.00	0.000E+00	0.000E+00	0.000E+00

IV.--FORCES IN LINEAR CONCENTRATED SPRINGS

NONE

V.--FORCES IN NONLINEAR CONCENTRATED SPRINGS

X-COORD (FT)	ANGLE (DEG)	DEFORMATION (FT)	FORCE (K)
9.84	90.00	-1.633E-02	1.104E+01

PROGRAM CBEAMC - ANALYSIS OF BEAM-COLUMNS WITH NONLINEAR SUPPORTS

DATE: 26-JANUARY-2001 TIME: 10:29:46

****
* COMPLETE RESULTS *
****

I.--HEADING

'SOLETANCHE WALL
'SEI - ASCE METHOD
'FINAL STAGE EXCAVATION

II.--DISPLACEMENTS AND INTERNAL FORCES

	<---------DISPLACEMENTS-------->		<---INTERNAL FORCES--->	
X-COORD (FT)	LATERAL (IN)	ROTATION (RAD)	SHEAR (K)	MOMENT (K-FT)
0.00	5.314E-01	-6.050E-03	-1.673E-13	-2.723E-13
1.97	3.885E-01	-6.052E-03	-2.770E-01	-2.527E-01
3.94	2.454E-01	-6.073E-03	-6.750E-01	-1.170E+00
5.90	1.013E-01	-6.137E-03	-1.194E+00	-2.989E+00
7.87	-4.506E-02	-6.276E-03	-1.834E+00	-5.949E+00
9.84	-1.960E-01	-6.530E-03	-2.595E+00	-1.029E+01
9.84	-1.960E-01	-6.530E-03	8.440E+00	-1.029E+01
11.85	-3.554E-01	-6.599E-03	7.538E+00	5.797E+00
13.86	-5.105E-01	-6.174E-03	6.509E+00	1.994E+01
15.87	-6.500E-01	-5.324E-03	5.354E+00	3.189E+01
17.88	-7.646E-01	-4.124E-03	4.072E+00	4.138E+01
19.89	-8.468E-01	-2.658E-03	2.664E+00	4.818E+01
21.90	-8.914E-01	-1.019E-03	1.130E+00	5.201E+01
23.91	-8.954E-01	6.939E-04	-5.310E-01	5.263E+01
25.93	-8.582E-01	2.371E-03	-2.318E+00	4.979E+01
27.94	-7.822E-01	3.896E-03	-4.231E+00	4.323E+01
29.95	-6.725E-01	5.143E-03	-6.271E+00	3.269E+01
31.96	-5.373E-01	5.979E-03	-8.437E+00	1.793E+01
33.97	-3.880E-01	6.321E-03	-4.942E+00	4.242E+00
35.98	-2.349E-01	6.339E-03	-1.237E+00	-1.877E+00
37.99	-8.284E-02	6.268E-03	1.102E+00	-1.650E+00
40.00	6.805E-02	6.248E-03	0.000E+00	-2.861E-14

```
III.--FORCES IN LINEAR DISTRIBUTED SPRINGS
          NONE



IV.--FORCES IN NONLINEAR DISTRIBUTED SPRINGS
                     X-COORD      AXIAL        LATERAL
                       (FT)      (K/FT)        (K/FT)
                       0.00     0.000E+00     0.000E+00
                       1.97     0.000E+00     0.000E+00
                       3.94     0.000E+00     0.000E+00
                       5.90     0.000E+00     0.000E+00
                       7.87     0.000E+00     0.000E+00
                       9.84     0.000E+00     0.000E+00
                      11.85     0.000E+00     0.000E+00
                      13.86     0.000E+00     0.000E+00
                      15.87     0.000E+00     0.000E+00
                      17.88     0.000E+00     0.000E+00
                      19.89     0.000E+00     0.000E+00
                      21.90     0.000E+00     0.000E+00
                      23.91     0.000E+00     0.000E+00
                      25.93     0.000E+00     0.000E+00
                      27.94     0.000E+00     0.000E+00
                      29.95     0.000E+00     0.000E+00
                      31.96     0.000E+00     0.000E+00
                      31.96     0.000E+00     2.334E+00
                      33.97     0.000E+00     3.084E+00
                      35.98     0.000E+00     2.871E+00
                      37.99     0.000E+00     1.856E+00
                      40.00     0.000E+00     3.100E-01
```

Wall moments and shears for the final stage excavation analysis are plotted in Figure 2.31. Net soil pressures are shown in Figure 2.32.

The results obtained by the Winkler 1 method (SEI/ASCE method) were compared with those from the Winkler 1 method for the final-stage excavation analysis, and are presented in Table 2.11. The Winkler 2 analysis produces a significantly higher positive moment and a significantly lower negative moment. This difference is in part attributed to the R-y curve-shifting that is used in the Winkler 1 analysis to account for plastic deformations that occur in the soil during first-stage excavation.

Tieback wall systems design must include an evaluation of internal and external stability. Simple limiting equilibrium procedures are available for evaluating internal and external stability for tieback wall systems constructed at sites with reasonably homogeneous soil profiles. The simple procedures are illustrated for Example 1 in the succeeding sections of Chapter 2. (Section 2.8 covers the internal stability evaluation, and Section 2.9 covers the external stability evaluation.) Additional information on internal and external stability can be found in Strom and Ebeling (2002).

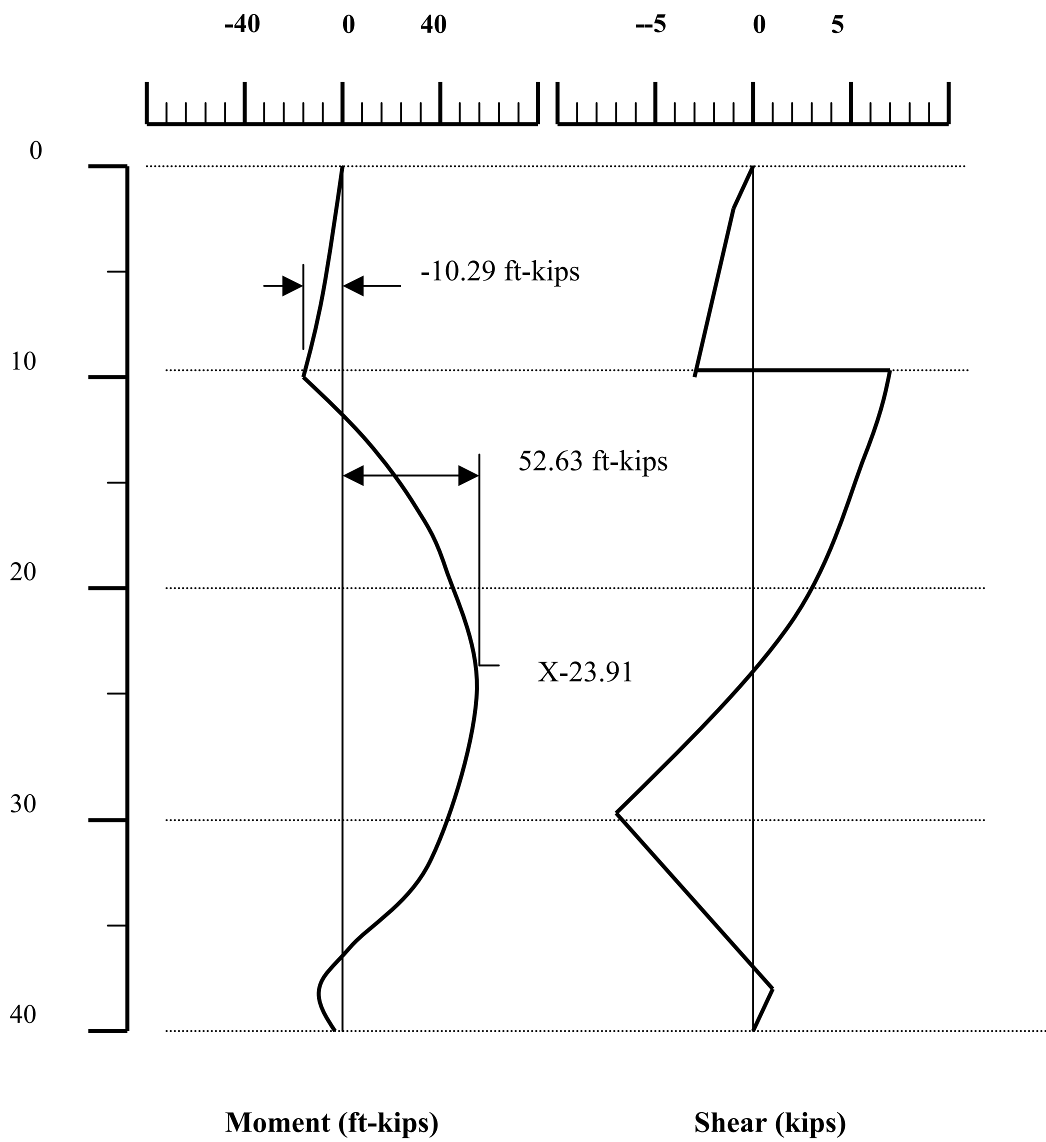


Figure 2.31. Wall moments and shears for Winkler 2 analysis–final construction stage

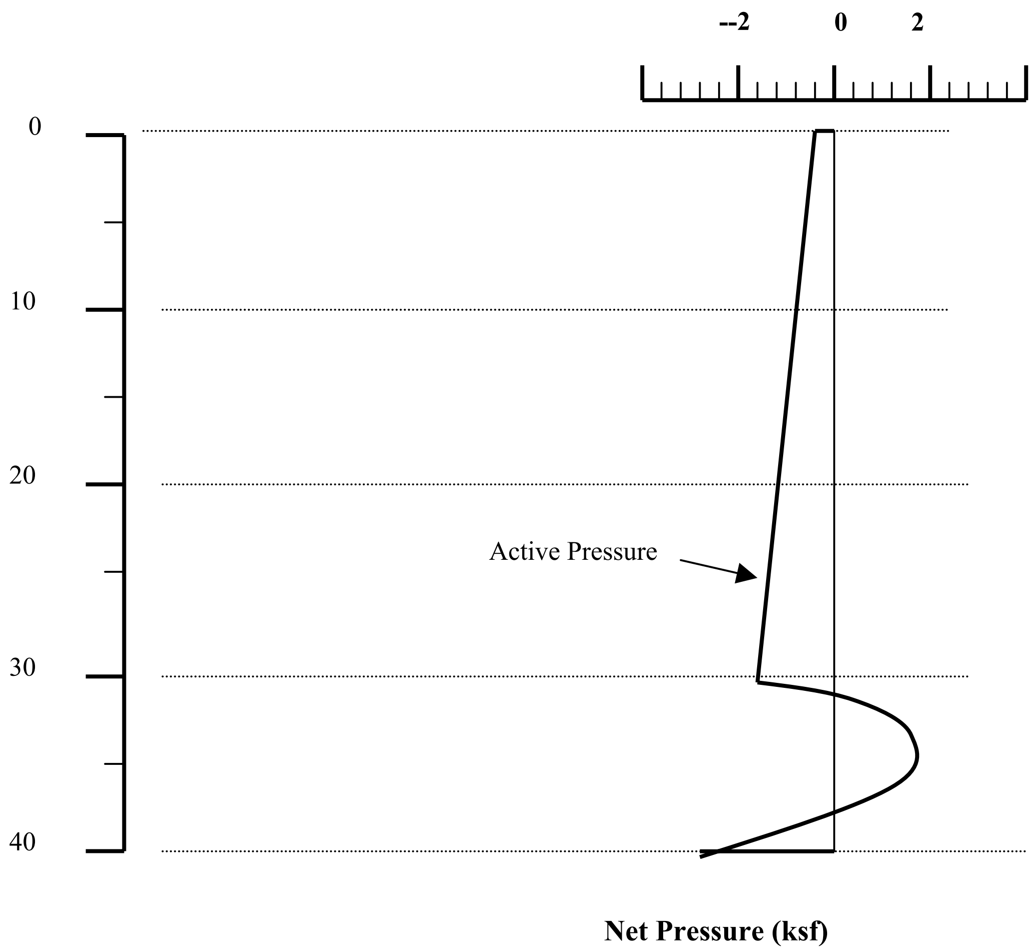


Figure 2.32. Soil pressures for Winkler 2 analysis–final construction stage

Table 2.11
Winkler 2–Winkler 1 Comparison for Final-Stage Excavation

Wall Section A	SEI/ASCE (Winkler 2)	Winkler 1
Maximum positive moment	**52.6 ft-kips**	**26.7 ft-kips**
Positive moment location	**23.91 ft**	**24.00 ft**
Maximum negative moment	**10.3 ft-kips**	**31.1 ft-kips**
Negative moment location	**At anchor location**	**At anchor location**
Maximum shear (at anchor)	**8.44 kips**	**7.42 kips**
Anchor force	**10.04 kips**	**12.26 kips**

2.8 Internal Stability

2.8.1 General

A simplified limiting equilibrium approach is used to check internal stability for the Soletanche wall of Example 1. This approach is described in FHWA-RD-98-065 and is limited to walls with reasonably homogeneous soil profiles. For complicated stratification, irregular ground surface, or irregular surcharge loading, the lateral force required to stabilize the excavation must be determined by a trial wedge stability analysis.

2.8.2 Earth pressure coefficient method of analysis

Force equilibrium methods are used to determine the total force required to stabilize the excavation. In the following analysis, P_{reqd} represents the external force required to provide stability to the vertical cut. This force represents the combined resistance provided by the horizontal component of the anchor force, T cos (I), and the lateral resistance provided by the embedded portion of the wall. The assumption that P_{reqd} is horizontal implies that the vertical resistance provided by the wall, SP_V, is equal and opposite in sign to the vertical component of the ground anchor loads, T sin (I). The unbonded length of the anchor must extend beyond the failure plane to permit the full anchor load to contribute toward internal stability. The potential failure plane passes through the toe at depth, d, and mobilizes a passive resistance from the soil, P_p, and a horizontal resistance from the tremie concrete/slurry trench wall (SP_H and SP_V, respectively). The internal stability failure plane for the Soletanche wall is illustrated in Figure 2.33. The free-body diagram is illustrated in Figure 2.34, and the force vector diagram shown is in Figure 2.35.

Equation 3.19 of FHWA-RD-98-065 (given below) is solved for various values of ξ and α to determine the maximum value for P_{reqd}, where

ξ = ratio d to H

α = angle of the failure plane with respect to horizontal

$$P_{reqd} = \frac{1}{2}\gamma H^2 \left[\frac{(1+\xi)^2}{\tan(\alpha) - \tan(\beta)} - K_{Pmob}\xi^2 \left(\sin(\delta_{mob}) + \frac{\cos(\delta_{mob})}{\tan(\alpha - \phi_{mob})} \right) \right] \tan(\alpha - \phi_{mob})$$

procedure for determining P_{reqd} is illustrated below. In the procedure, the friction angle ϕ is replaced by the mobilized friction angle, ϕ_{mob}, determined using a safety factor of 1.3 applied to the shear strength of the soil. Current design practice according to FHWA-RD-98-065 (see paragraph 3.3.1, page 35) is to use

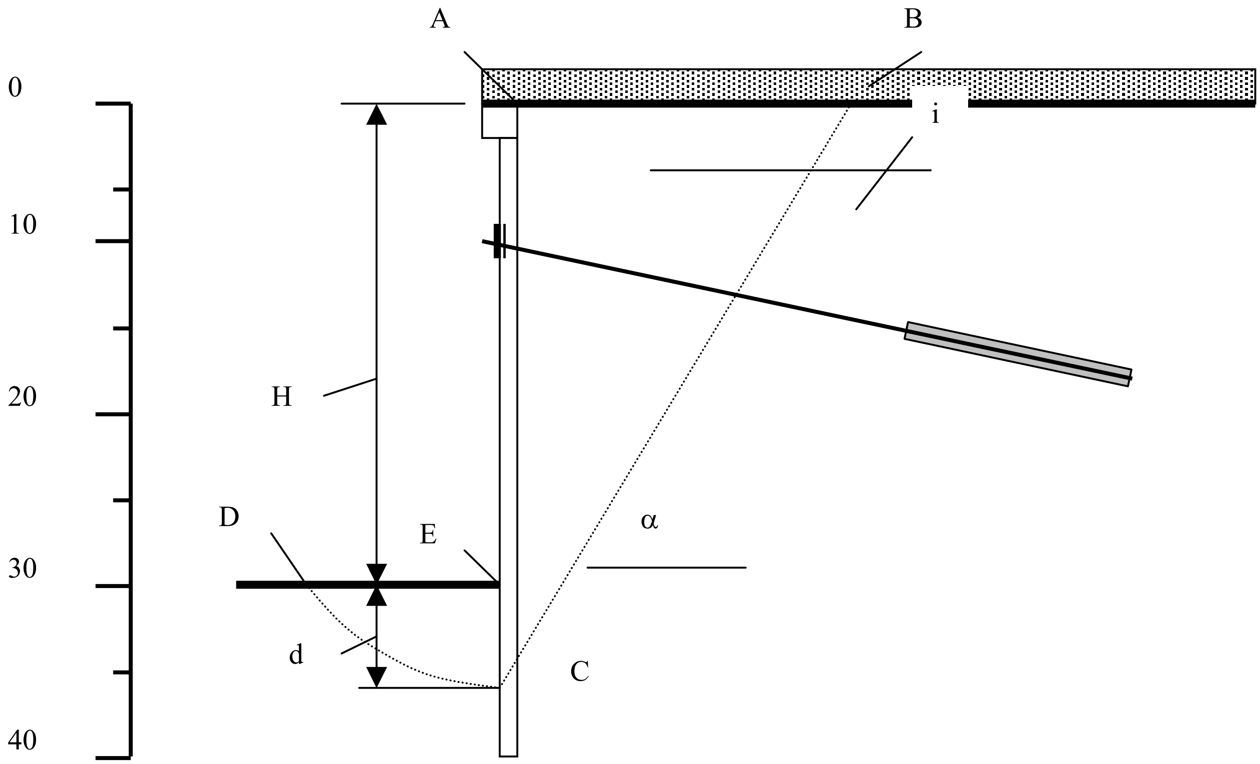


Figure 2.33. Soletanche wall internal stability, elevation at final excavation stage

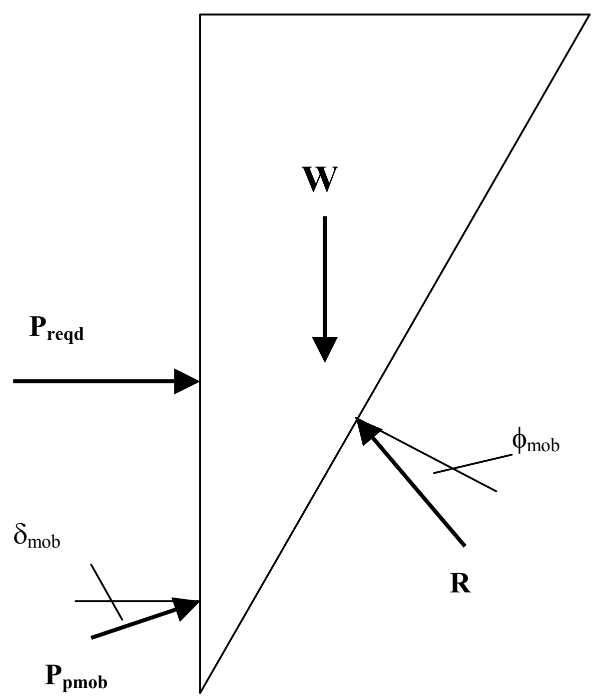


Figure 2.34. Free-body diagram

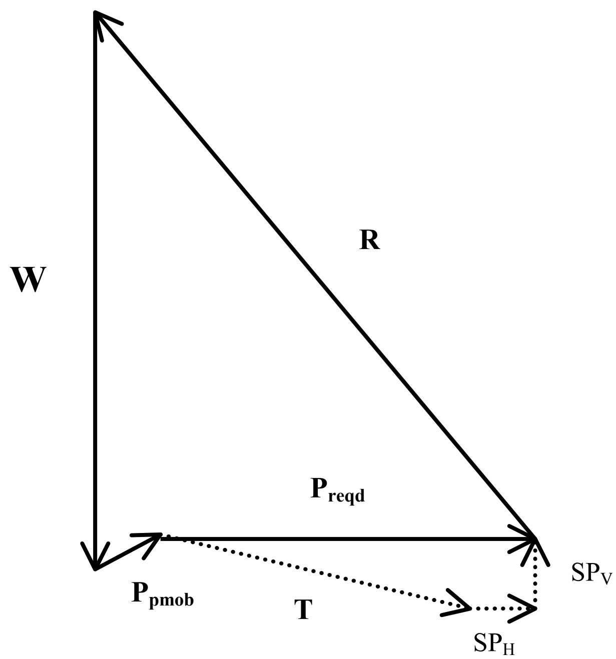


Figure 2.35. Force vector diagram

a safety factor between 1.2 and 1.5. A safety factor of 1.3 produces design loads similar to those for walls designed to support apparent earth pressures. Walls designed to minimize lateral movements in the retained soil are often designed with a safety factor equal to 1.5 (i.e. stringent displacement control design). The mobilized passive earth pressure coefficient, K_{pmob}, is determined, assuming the angle of internal friction is equal to ϕ_{mob} and that the mobilized interface friction angle between the embedded portion of the wall and the mobilized passive zone of soil is also equal to ϕ_{mob}. Note that in these calculations, which follow the engineering calculation procedures of outlined in FHWA-RD-98-065, paragraph 3.5.2.1, page 61), the soil-to-structure interface friction, δ, is assumed equal to the angle of internal friction, ϕ, of the soil and δ_{mob} is assumed equal to ϕ_{mob}. It can be understood how this assumption may be valid with respect to the soil-to-concrete interface for walls constructed by slurry trench methods. However, the authors of this report are concerned about the reasonableness of this assumption for soil-to-steel interfaces such as occur with soldier beam and lagging systems and with sheet-pile systems.

Soletanche Wall Internal Stability
Determine (P_{reqd}) the force required to provide stability
to the vertical cut using limiting equilibrium methods

October 9, 2001

Surcharge effects not included

Equation 3.19, FHWA-RD-98-065 **File: STABLE 1A**

$\gamma := 0.115$ kcf $\quad H := 28.9$ feet $\quad \phi := 35 \cdot deg$

$SF := 1.3$ Safety Factor = 1.3 applied to shear strength of soil

$\dfrac{\tan(\phi)}{SF} = 0.539 \qquad \phi_{mob} := \operatorname{atan}(0.539) \cdot \dfrac{180}{\pi} \qquad \phi_{mob} = 28.325$

$\phi_{mob} := 28.325 deg \qquad K_{pmob} := 5.5 \qquad \delta_{mob} := 28.325 deg \qquad \beta := 0 \cdot deg$

Mobilized δ the interface friction angle between the embedded portion of the wall and the passive zone of soil is set equal to the mobilized ϕ

From Figure 27, FHWA-RD-98-065, for mobilized $-\delta / \phi$ = -1, K_{pmob} = 5.5

Try different values of α and ξ to find the maximum value for P_{reqd}
Use Table 4 of FHWA-RD-98-065 to find the proper range

$\alpha := 54 \cdot deg, 55 \cdot deg .. 60 \cdot deg$

Try $\quad \xi := 0.060$

$$P_{reqd}(\alpha) := \frac{1}{2} \cdot \gamma \cdot H^2 \left[\frac{(1+\xi)^2}{\tan(\alpha) - \tan(\beta)} - K_{pmob} \cdot \xi^2 \left(\sin(\delta_{mob}) + \frac{\cos(\delta_{mob})}{\tan(\alpha - \phi_{mob})} \right) \right] \cdot \tan(\alpha - \phi_{mob})$$

$P_{reqd}(\alpha) =$ Kips / Foot

17.793
17.919
18.015
18.081
18.119
18.127
18.107

Try $\xi := 0.055$

$$P_{reqd}(\alpha) := \frac{1}{2}\cdot\gamma\cdot H^2\left[\frac{(1+\xi)^2}{\tan(\alpha)-\tan(\beta)} - K_{pmob}\cdot\xi^2\left(\sin\left(\delta_{mob}\right)+\frac{\cos\left(\delta_{mob}\right)}{\tan\left(\alpha-\phi_{mob}\right)}\right)\right]\cdot\tan\left(\alpha-\phi_{mob}\right)$$

$P_{reqd}(\alpha) =$ Kips / Foot

17.784
17.91
18.007
18.074
18.113
18.123
18.104

Try $\xi := 0.065$

$$P_{reqd}(\alpha) := \frac{1}{2}\cdot\gamma\cdot H^2\left[\frac{(1+\xi)^2}{\tan(\alpha)-\tan(\beta)} - K_{pmob}\cdot\xi^2\left(\sin\left(\delta_{mob}\right)+\frac{\cos\left(\delta_{mob}\right)}{\tan\left(\alpha-\phi_{mob}\right)}\right)\right]\cdot\tan\left(\alpha-\phi_{mob}\right)$$

$P_{reqd}(\alpha) =$ Kips / Foot

17.788
17.914
18.009
18.075
18.111
18.117
18.095

Use P_{reqd} = 18.1 kips per foot
α = 59 deg, ξ = 0.060

CHECK

Using a K = 0.375 for base failure obtained from Figure 30 of FHWA-RD-98-065

$$P_{reqd} := \left(0.375 \cdot \gamma \cdot H^2\right) \cdot \frac{1}{2}$$

$P_{reqd} = 18.009$ Kips / Foot - Checks

$EPF := \dfrac{P_{reqd}}{H^2}$ $EPF = 0.022$ kcf

EPF of 0.023 kcf selected for Rigid 1 analysis - okay

Internal stability is okay provided the anchor force used for the design is greater than the anchor force determined from the Rigid 1 analysis and the anchorage zone falls outside the falure plane determined above. α = 59 deg, ξ = 0.060

2.9 External Stability

A simplified force equilibrium approach is used to check external stability for the Soletanche wall of Example 1. This approach is described in FHWA-RD-98-065 and is limited to walls with reasonably homogeneous soil profiles. For complicated stratification, irregular ground surface, or irregular surcharge loading, the lateral force required to stabilize the excavation must be determined by a trial wedge stability analysis.

The external stability of an anchored wall system is determined by assuming the potential plane of sliding passes behind the anchor and below the bottom of the wall. Since anchors are spaced at a horizontal distance, "s", the potential failure surface may assume a three-dimensional shape rather than the two-dimensional shape used as an idealized basis for the following analysis. When a two-dimensional surface is used to approximate a three-dimensional failure surface it is commonly assumed that the idealized two-dimensional failure plane intersects the ground anchor at a distance s /3 from the back of the anchor as shown in Figure 2.36. The stability for the soil mass is determined by requiring horizontal and vertical force equilibrium. The soil mass under consideration is the soil prism ABCDEG, as shown in Figure 2.36.

2.9.1 Simplified force limit equilibrium approach for homogeneous soil sites

Forces on the soil mass are shown in Figure 2.37, and the force vectors on area ABCDEG are shown in Figure 2.38. The soil mass acts downward with a magnitude equal to its weight. On the left face, the mobilized passive soil resistance, K_{mob}, acts at a mobilized angle of interface friction, δ_{mob}. Active soil pressure is assumed to act on the right vertical face. On the bottom, soil resistance acts at an angle ϕ_{mob} from the perpendicular to the failure plane. The forces will sum to zero in the horizontal and vertical directions for a safety factor equal to one and a friction angle ϕ_{mob}. Additional details pertaining to the force equilibrium analysis can be found in FHWA-RD-98-065. Equation 3.22 of FHWA-RD-98-065 is used to determine the friction angle ϕ_{mob} needed to produce force equilibrium for the soil mass ABDEG. In Equation 3.22 the friction angle ϕ is replaced by the mobilized friction angle, ϕ_{mob}. The resulting factor of safety based on strength, $FS_{STRENGTH}$, is equal to tan (ϕ)/tan (ϕ_{mob}). A value for $FS_{STRENGTH}$ equal to 1.3 is often used in practice according to FHWA-RD-98-065 (paragraph 3.3.1, page 35), and such a factor of safety would be appropriate for "safety with economy" type designs.

$$\left(1+\xi+\lambda\right)X - K_{Pmob}\xi^2 \sin(\delta_{mob}) + \frac{K_{Pmob}\xi^2 \cos(\delta_{mob}) - K_{Amob}\lambda^2}{\tan(\phi_{mob} - \alpha)} = 0$$

where

$X = x/H$

$\lambda = y/H$

$\xi = d/H$

See comments in Section 3.1 of this report regarding $\delta = \phi$ and $\delta_{mob} = \phi_{mob}$.

The dimensions x, y, d, and H are shown in Figure 2.36.

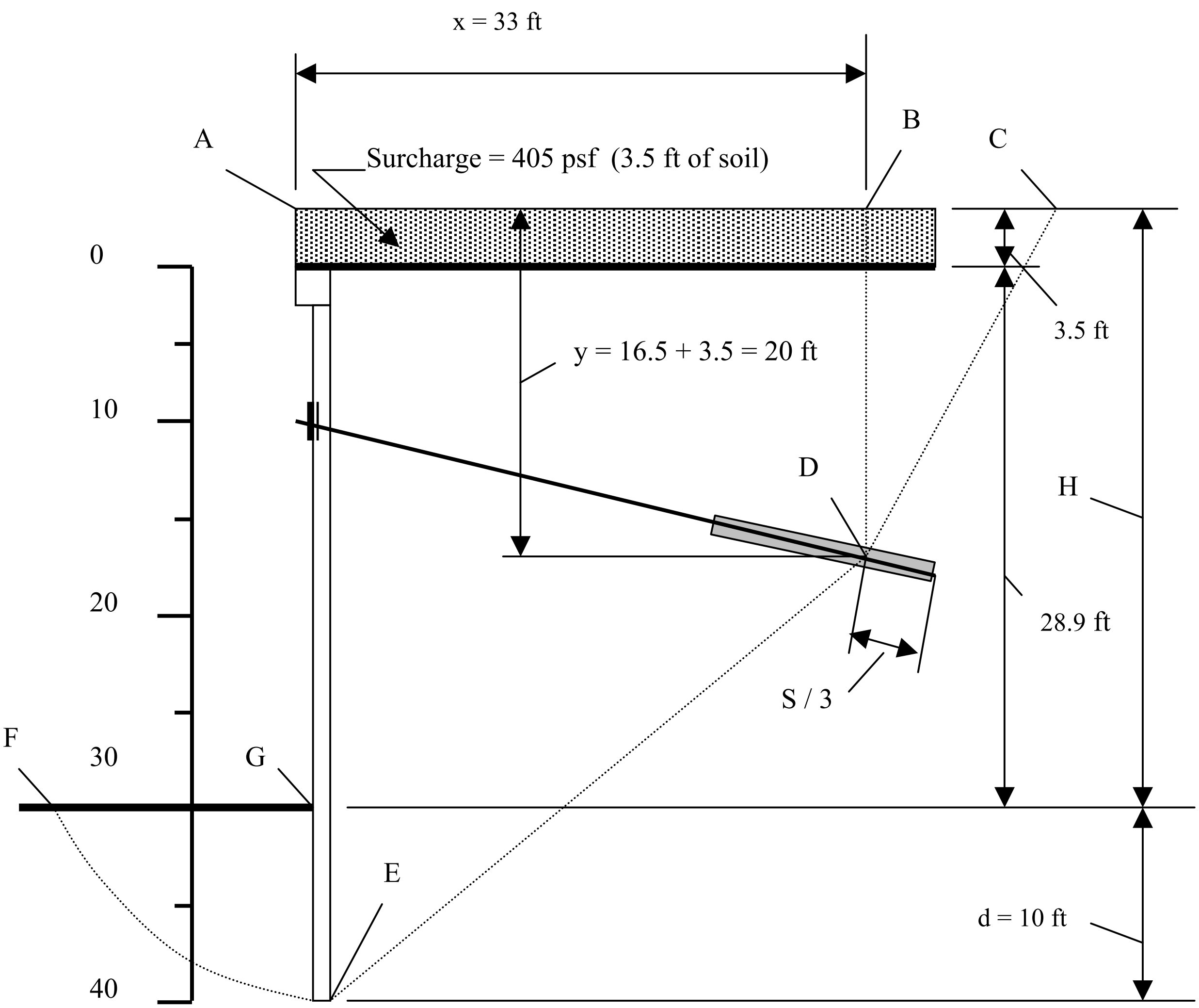


Figure 2.36. Soletanche wall, elevation at final excavation stage

Equation 3.22 is solved to find the mobilized friction angle, ϕ_{mob}. These calculations are provided in the following pages. The correct solution for ϕ_{mob} is provided when the term *A* in the following caclulations becomes equal to zero.

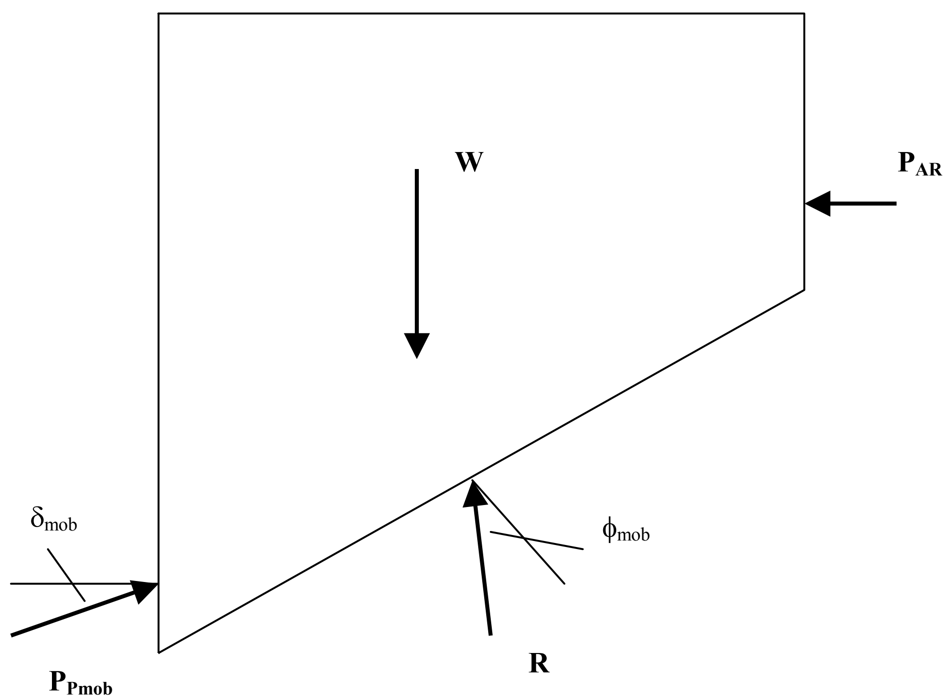


Figure 2.37. Force-body diagram

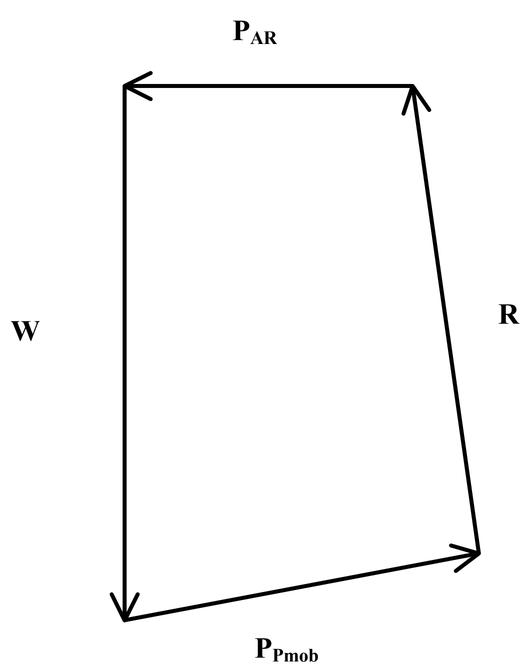


Figure 2.38. Force vectors acting on area ABCDEG

Soletanche Wall External Stability

October 9, 2001

Consider surcharge

Equation 3.22, FHWA-RD-98-065

File: STABLE 2A

$H := 28.9 + 3.5$ $H = 32.4$ feet

$\gamma := 0.115$ kcf $\phi := 35 \cdot deg$

Mobilized δ, the interface friction angle betwwen the embedded portion of the wall and the passive zone of soil is set equal to the mobilized ϕ.

$x := 33.0$ feet $X := \frac{x}{H}$ $X = 1.019$

$y := 20.0$ feet $\lambda := \frac{y}{H}$ $\lambda = 0.617$

$d := 10$ feet $\xi := \frac{d}{H}$ $\xi = 0.309$

$\alpha := \operatorname{atan}\left[\frac{(H + d - y)}{x}\right] \cdot \frac{180}{\pi}$ $\alpha = 34.168$ degrees

$\alpha := 34.168 deg$

Try $\phi_{mob} := 28.325 deg$ $\delta_{mob} := 28.325 deg$

From Figure 27, FHWA-RD-98-065, for mobilized $-\delta / \phi$ = -1, K_{pmob} = 5.5, K_{amob} = 0.34

$K_{pmob} := 5.5$ $K_{amob} := 0.34$

$$A := (1 + \xi + \lambda) \cdot X - K_{pmob} \cdot \xi^2 \cdot \sin(\delta_{mob}) + \frac{\left(K_{pmob} \cdot \xi^2 \cdot \cos(\delta_{mob}) - K_{amob} \cdot \lambda^2\right)}{\tan(\phi_{mob} - \alpha)}$$

$A = -1.528$ A must equal zero to satisfy Equation 3.22 requirements

Try $\phi_{mob} := 24.0 \cdot deg$ $\delta_{mob} := 24.0 \cdot deg$

$K_{pmob} := 4.0$ $K_{amob} := 0.38$

$$A := (1 + \xi + \lambda) \cdot X - K_{pmob} \cdot \xi^2 \cdot \sin(\delta_{mob}) + \frac{\left(K_{pmob} \cdot \xi^2 \cdot \cos(\delta_{mob}) - K_{amob} \cdot \lambda^2\right)}{\tan(\phi_{mob} - \alpha)}$$

$A = 0.673$

Try $\phi_{mob} := 26.0 \cdot deg$ $\delta_{mob} := 26.0 \cdot deg$

$K_{pmob} := 4.5$ $K_{amob} := 0.35$

$$A := (1 + \xi + \lambda) \cdot X - K_{pmob} \cdot \xi^2 \cdot \sin(\delta_{mob}) + \frac{\left(K_{pmob} \cdot \xi^2 \cdot \cos(\delta_{mob}) - K_{amob} \cdot \lambda^2\right)}{\tan(\phi_{mob} - \alpha)}$$

$A = 0.019$ Approximately = 0

ϕmob required for external stability is less than ϕmob with a Factor of Safety = 1.3
Meets external stability requirements.

$$FS_{STRENGTH} := \frac{\tan(\phi)}{\tan(\phi_{mob})}$$

$FS_{STRENGTH} = 1.436$ Okay

2.9.2 Simplified force limit equilibrium approach for nonhomogeneous soil sites using CSLIDE

The Corps program CSLIDE can be used to assess the stability of a tieback wall system. It is based on the equations of horizontal and vertical equilibrium applied to the soil wedges. It does not include the equation of moment equilibrium between wedges. CSLIDE can accommodate water loads, surcharge loads, and layered soil systems. Since there is no interaction of vertical shear force effects between wedges, the passive resistance must act horizontally (i.e., $\delta_{mob} = 0$) rather than at an angle $\delta_{mob} > 0$. This will result in a conservative factor of safety. Also, the CSLIDE program satisfies force equilibrium only. Moment equilibrium is not considered. This is also true for the FHWA-RD-98-065 simplified external stability analysis.

The use of the CSLIDE program is demonstrated with respect to the Soletanche wall example. The analysis is also performed using the simplified external stability analysis procedure of FHWA-RD-98-065, assuming that δ_{mob}

equals zero. The results in terms of the factor of safety are similar.

The CSLIDE and FHWA-RD-98-065 analyses for this example (shown in Figure 2.39) are presented on the following pages.

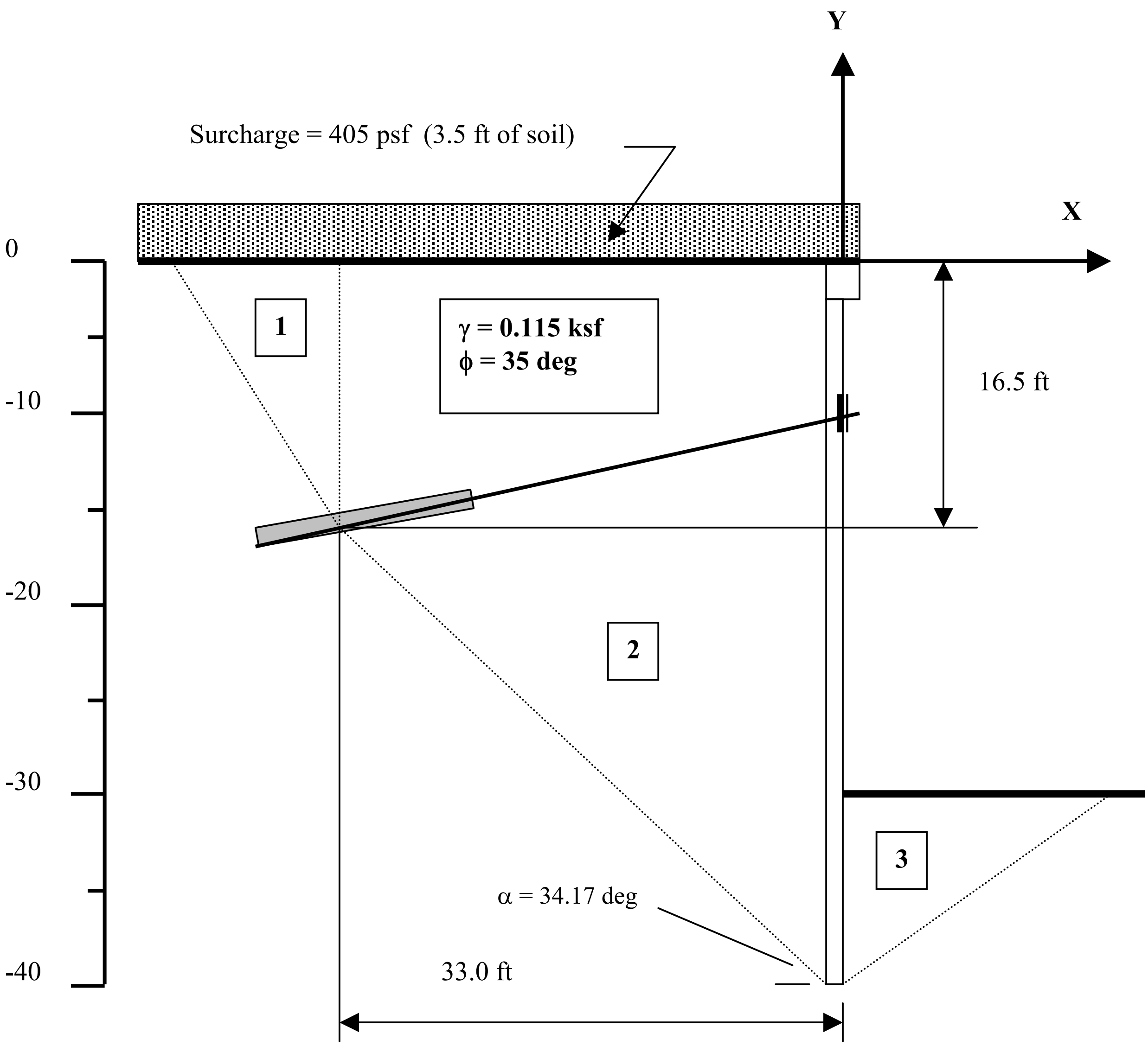


Figure 2.39. Soletanche wall, elevation at final excavation stage

CSLIDE Input and Output

```
10010 TITL SOLETANCHE WALL FILE: SOL1.IN
10020 STRU  4      .150
10030        -1.00       -38.90
10040        -1.00         0.00
10050         0.00         0.00
10060         0.00       -38.90
10090 SOLT  1  1       35.00      0.00      0.115
0.00
10095      -100.00       0.00
10100 SOLT  2  1       35.00      0.00      0.115       -
16.50
10105      -100.00       -16.50
10110 SORT  1  1       35.00      0.00      0.115       -
28.90
10120       150.00        -28.90
10130 SOST        0.00      0.00
10135 WATR       -38.90   -38.90     0.0625
10140 METH  2
10150 WEDG  2  -34.17
10155 VULO  L  0.405
10160 FACT        1.0        2.0      1.00
10170 END
```

```
 ------------------------------
   PROGRAM CSLIDE - ECHOPRINT
 ------------------------------

 DATE: 06-27-01                       TIME: 10:23:09

 SOLETANCHE WALL FILE: SOL1.IN

 MULTI  FAILURE PLANE ANALYSIS

 HYDROSTATIC WATER FORCE COMPUTED FOR WEDGES


 NO OF CORNERS IN STRUCTURE ----------       4
 DENSITY OF CONCRETE -----------------   .1500(KCF)
 DENSITY OF WATER --------------------   .0625(KCF)
 WATER LEVEL LEFT SIDE ---------------  -38.90(FT)
 WATER LEVEL RIGHT SIDE --------------  -38.90(FT)
 NO. OF SOIL LAYERS LEFT SIDE --------       2
 NO. OF SOIL LAYERS RIGHT SIDE -------       1
```

STRUCTURE INFORMATION

POINT	X-COORD	Y-COORD
1	-1.00	-38.90
2	-1.00	.00
3	.00	.00
4	.00	-38.90

LEFTSIDE SOIL DATA

LAYER NO.	FRICTION ANGLE (DEG)	COHESION (KSF)	UNIT WEIGHT (KCF)	ELEV AT STRUCTURE (FT)
1	35.00	.0000	.115	.00
2	35.00	.0000	.115	-16.50

LAYER NO	POINT NO. 1 X-COORD	POINT NO. 1 Y-COORD
1	-100.00	.00
2	-100.00	-16.50

SOIL DATA BELOW STRUCTURE

FRICTION ANGLE --------- .00
COHESION --------------- .0000

RIGHTSIDE SOIL DATA

LAYER NO.	FRICTION ANGLE (DEG)	COHESION (KSF)	UNIT WEIGHT (KCF)	ELEV AT STRUCTURE (FT)
1	35.00	.0000	.115	-28.90

LAYER NO	POINT NO. 1 X-COORD	POINT NO. 1 Y-COORD
1	150.00	-28.90

WEDGE NO.	ANGLE
2	-34.17

SAFETY FACTOR DESCRIPTION

LOWER LIMIT OF F.S. ----- 1.00
UPPER LIMIT OF F.S. ----- 2.00

VERTICAL UNIFORM LOADS

SIDE	MAGNITUDE
L	.405

```
------------------------------------
   PROGRAM CSLIDE - FINAL RESULTS
------------------------------------


 DATE: 06-27-01                          TIME: 10:23:22


 SOLETANCHE WALL FILE: SOL1.IN



 MULTIPLE FAILURE PLANE ANALYSIS

 HYDROSTATIC WATER FORCE COMPUTED FOR WEDGES



                  HORIZONTAL LOADS
                  ----------------          VERTICAL
     WEDGE      LEFT  SIDE    RIGHT SIDE      LOAD
     NUMBER       (KIPS)        (KIPS)       (KIPS)
 ---------------------------------------------------


       1           .000          .000         3.887
       2           .000          .000        13.364
       3           .000          .000          .000
       4           .000          .000          .000



        WATER PRESSURES ON WEDGES
        -------------------------

             LEFTSIDE WEDGES
             ---------------

 WEDGE NO.  TOP PRESSURE  BOTTOM PRESSURE
               (KSF)           (KSF)


      1          .000          .000
      2          .000          .000

             STRUCTURAL WEDGE
             ----------------

           X-COORD.   PRESSURE
            (FT)        (KSF)


           -1.00        .000
             .00        .000
```

RIGHTSIDE WEDGES

WEDGE NO.	TOP PRESSURE (KSF)	BOTTOM PRESSURE (KSF)
4	.000	.000

WEDGE NUMBER	FAILURE ANGLE (DEG)	TOTAL LENGTH (FT)	WEIGHT OF WEDGE (KIPS)	SUBMERGED LENGTH (FT)	UPLIFT FORCE (KIPS)
1	-59.8	19.089	9.107	.000	.000
2	-34.2	39.883	105.114	.000	.000
3	.000	1.000	5.835	.000	.000
4	30.2	19.887	9.884	.000	.000

WEDGE NUMBER	NET FORCE ON WEDGE (KIPS)
1	-7.560
2	-9.428
3	.000
4	16.988

SUM OF FORCES ON SYSTEM ---- .000

FACTOR OF SAFETY ----------- **1.232**

Soletanche Wall External Stability

Consider surcharge

October 9, 2001

Equation 3.22, FHWA-RD-98-065

File: STABLE 3A

$H := 28.9 + 3.5$ $\quad H = 32.4$ feet

$\gamma := 0.115$ kcf $\quad \phi := 35 \cdot deg$

Mobilized δ, the interface friction angle betwwen the embedded portion of the wall and the passive zone of soil is set equal to zero

$x := 33.0$ feet $\quad X := \frac{x}{H}$ $\quad X = 1.019$

$y := 20.0$ feet $\quad \lambda := \frac{y}{H}$ $\quad \lambda = 0.617$

$d := 10$ feet $\quad \xi := \frac{d}{H}$ $\quad \xi = 0.309$

$\alpha := \operatorname{atan}\left[\frac{(H + d - y)}{x}\right] \cdot \frac{180}{\pi}$ $\quad \alpha = 34.168$ degrees

$\alpha := 34.168 deg$

Try $\quad \phi_{mob} := 28.325 deg$ $\quad \delta_{mob} := 0 \cdot deg$

From Figure 27, FHWA-RD-98-065, for mobiolized δ / ϕ = 0,
K_{pmob} = 5.5 (0.5) = 2.75, K_{amob} = 0.34

$K_{pmob} := 2.75$ $\quad K_{amob} := 0.34$

$$A := (1 + \xi + \lambda) \cdot X - K_{pmob} \cdot \xi^2 \cdot \sin\left(\delta_{mob}\right) + \frac{\left(K_{pmob} \cdot \xi^2 \cdot \cos\left(\delta_{mob}\right) - K_{amob} \cdot \lambda^2\right)}{\tan\left(\phi_{mob} - \alpha\right)}$$

$A = 0.668$ $\quad$ A must equal zero to satisfy Equation 3.22 requirements

Try $\phi_{mob} := 29.0 \cdot deg$ $\delta_{mob} := 0 \cdot deg$

$K_{pmob} := 6.0 \cdot (0.488)$

$K_{pmob} = 2.928$ $K_{amob} := 0.347$

$$A := (1 + \xi + \lambda) \cdot X - K_{pmob} \cdot \xi^2 \cdot \sin(\delta_{mob}) + \frac{\left(K_{pmob} \cdot \xi^2 \cdot \cos(\delta_{mob}) - K_{amob} \cdot \lambda^2\right)}{\tan(\phi_{mob} - \alpha)}$$

$A = 0.34$

Try $\phi_{mob} := 30.0 \cdot deg$ $\delta_{mob} := 0 \cdot deg$

$K_{pmob} := 6.5 \cdot (0.467)$

$K_{pmob} = 3.036$ $K_{amob} := 0.333$

$$A := (1 + \xi + \lambda) \cdot X - K_{pmob} \cdot \xi^2 \cdot \sin(\delta_{mob}) + \frac{\left(K_{pmob} \cdot \xi^2 \cdot \cos(\delta_{mob}) - K_{amob} \cdot \lambda^2\right)}{\tan(\phi_{mob} - \alpha)}$$

$A = -0.265$

Use ϕ mob = 29.5 deg

$\phi_{mob} := 29.5 \cdot deg$

$$FS_{STRENGTH} := \frac{\tan(\phi)}{\tan(\phi_{mob})}$$

$FS_{STRENGTH} = 1.238$

2.10 Discussion of Results–Example 1

Various methods were demonstrated for potential application with respect to the design and evaluation of a ***stiff*** tieback wall with a single anchor. Except for the apparent pressure method (i.e., Rigid 1 analysis), all the methods used involve a construction-sequencing type analysis. Soil arching tends to develop both horizontally and vertically with ***flexible*** wall systems, resulting in earth-pressure concentration at tieback locations. This reduces the moment and shear demands on the wall system. Since the Rigid 1 approach is based on the measured response of systems, it produces a reliable design for flexible wall systems. With stiff wall systems, soil arching is less pronounced, and soil pressures at the facing tend to be more uniform with little tendency to concentrate at tieback locations. Therefore, the design of stiff wall systems should more closely follow classical earth pressure theory and should consider construction-sequencing effects. (Note: Construction-sequencing effects are inherently included in the apparent pressure method.) The Rigid 2, Winkler 1, and Winkler 2 analysis methods, which were demonstrated with respect to Example 1, are methods that are available to evaluate the performance of stiff tieback wall systems.

In the introduction to this report it was pointed out that many designers feel that the apparent pressure diagram approach used for flexible tieback wall systems is ill advised for use in the design of stiff tieback wall systems (Kerr and Tamaro 1990). These investigators also indicated that the apparent pressure approach for stiff wall systems will underpredict loads in the lower tiebacks and underpredict negative moments at the tieback anchor locations. It can be seen for this single tieback anchor example that the apparent pressure method (i.e., Rigid 1 analysis) does produce lower bending moments when compared with the Rigid 2, Winkler 1, and Winkler 2 analysis methods. However, the anchor force determined by the Rigid 1 analysis is slightly higher that the values determined by the aforementioned stiff wall analysis methods. The apparent pressure diagram approach (i.e., Rigid 1 analysis) may be needed in many instances to ensure that the upper anchor design loads, as determined by construction-sequencing analyses, are adequate to meet “safety with economy” and “stringent displacement control” performance objectives.

Until additional research is conducted to evaluate the validity of these construction-sequencing methods, the designer must determine which method is most suitable and applicable to his or her particular wall system and site conditions.

3 Example 2 — Bonneville Temporary Tieback Wall

3.1 Introduction

3.1.1 Tieback wall description

During construction of the second navigation lock at Bonneville Lock and Dam, a 50-ft-high temporary tieback reinforced concrete diaphragm wall was built to retain the foundation of the Union Pacific Railroad. Wall movement was of great concern, so tiebacks were prestressed to levels consistent with at-rest earth pressure conditions to minimize wall deflections. This is referred to by the authors as a "stringent displacement control" design approach. A vertical section through the wall is illustrated in Figure 3.1. Additional information concerning the wall design, construction, and instrumentation can be found in Munger,

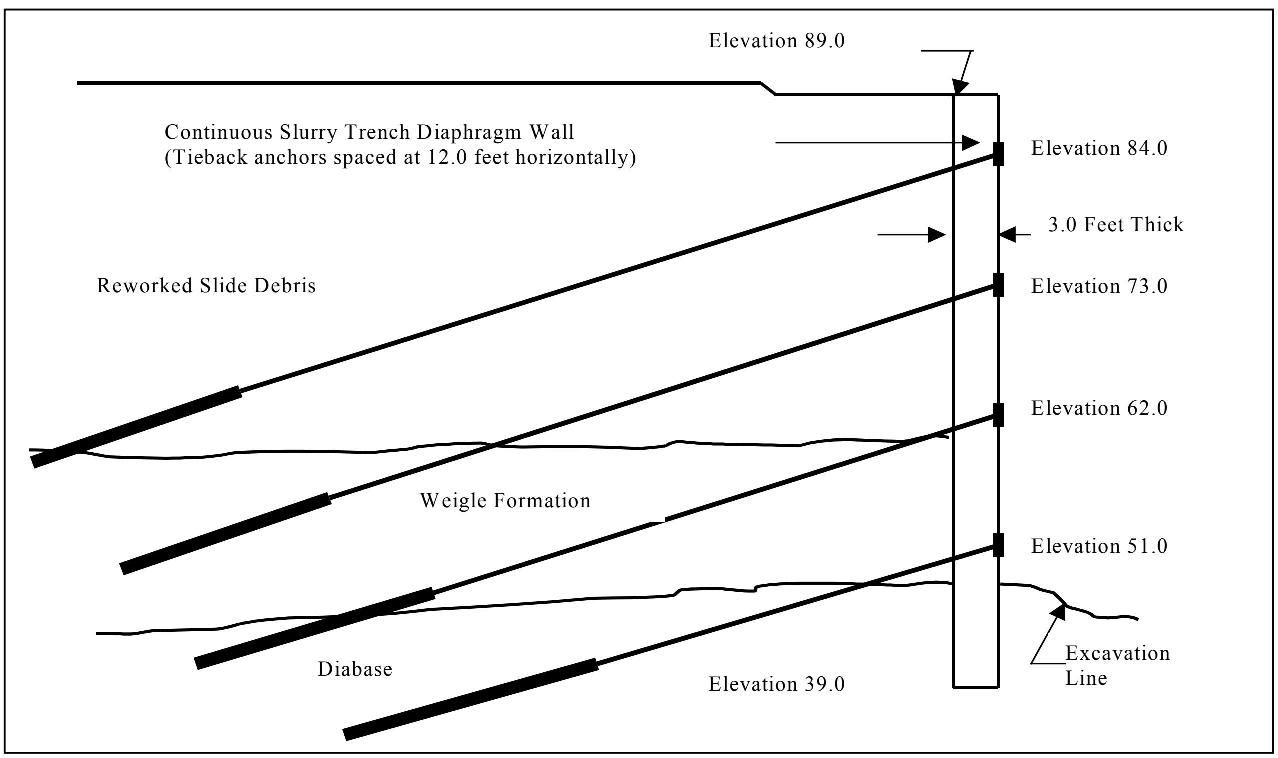


Figure 3.1. Bonneville Navigation Lock temporary tieback wall–vertical section

Jones, and Johnson (1991). In addition, a finite element (FEM) soil-structure interaction (SSI) analysis was performed to evaluate each phase of wall construction (Mosher and Knowles 1990). The results of this FEM study, as well as instrumentation measurements, are used to evaluate the various tieback wall design and analysis procedures performed as part of Example 2.

3.1.2 Analysis procedures

Various tieback wall design and analysis procedures were described in Strom and Ebeling (2001). Three of these procedures (identified as Rigid 1, Rigid 2, and Winkler 1) were used to evaluate the temporary tieback wall described above. Descriptions of the various design and analysis procedures are provided below. The temporary tieback wall is a stiff wall system in which the excavation that takes place prior to tieback installation occurs to a depth of 5.5 ft below the tieback location. This suggests that the largest force demands (moments and shears) on the wall will occur at intermediate construction stages rather than at the final excavation stage and, as such, only those analysis procedures considering construction sequencing will provide reasonable results. The stages of construction for the wall are described in Table 3.1.

Table 3.1
Construction Sequencing

Stage	Description
1	Construct surcharge to pre-excavation grade (four increments)
2	Excavate for railroad relocation
3	Construct slurry trench temporary tieback wall
4	Excavate in front of wall to el 78.5 **(Stage 1 excavation)**
5	Install upper tieback anchor at el 84 and prestress to 150 percent of the design load
6	Excavate in front of wall to el 67.5 and lock off upper anchor at design load **(Stage 2 excavation)**
7	Install second tieback anchor at el 73 and prestress to 150 percent of the design load
8	Excavate in front of wall to el 56.5 and lock off second anchor at design load **(Stage 3 excavation)**
9	Install third tieback anchor at el 62 and prestress to 150 percent of the design load
10	Excavate in front of wall to el 45 and lock off third anchor at design load **(Stage 4 excavation)**
11	Install fourth tieback anchor at el 51 and prestress to 150 percent of design load
12	Excavate to bottom of wall at el 39 and lock off fourth anchor at design load **(Stage 5 excavation)**

3.1.3 Rigid 1 analysis description

The procedure labeled as Rigid 1 is an equivalent beam on rigid supports analysis in which the tieback wall modeled as a continuous beam on rigid supports is loaded with an apparent pressure diagram. Apparent pressures are intended to represent a load envelope, and not the actual loads that might exist on the wall at any time. The analysis, therefore, is a final-excavation analysis that indirectly considers the effects of construction sequencing. This approach provides good results for ***flexible*** walls constructed in competent soils where excavation below the point of tieback prestress application is minimal (±1.5 ft). However, many designers of stiff wall systems believe that the use of apparent pressure diagrams for the design of stiff wall systems is ill advised (Kerr and Tamaro 1990). Therefore, the Rigid 1 approach is not expected to provide valid results for the Bonneville tieback wall, which is characterized as a stiff wall system. The purpose of this example is to illustrate the problems that can occur in using the apparent pressure approach to design stiff tieback wall systems. The apparent pressure diagram is based on a total load approach in accordance with procedures presented in FHWS-RD-98-066. At-rest earth pressure coefficients are the basis for determining the total load, since tiebacks will be sized and prestressed to minimize wall movement. The Rigid 1 analysis is illustrated in Section 3.2.

3.1.4 Rigid 2 analysis description

The use of the beam on rigid supports (Rigid) method for evaluating various loading conditions encountered during construction is described in Ratay (1996), Kerr and Tamaro (1990), and FHWA-RD-81-150. In the Rigid method, a vertical strip of the wall is treated as a multispan beam on rigid supports that are located at tieback points. The analysis is a construction-sequencing analysis in which the earth loads are applied according to classical earth pressure theory (see Ebeling and Morrison 1992, Chapter 3). An equivalent cantilever beam method is used to evaluate wall-bending moments for the initial excavation (cantilever) stage of construction. For subsequent stages of excavation, where the wall is anchored, it is assumed that the depth of penetration below grade is sufficient to cause the point of contraflexure to coincide with the point of zero net pressure intensities. This allows the use of an equivalent beam supported at anchor locations and at the point of zero net earth pressure where the wall moment can be assumed to be zero. In this Rigid 2 analysis, the driving side earth pressures are assumed to be equal to at-rest pressure, since tiebacks will be sized and prestressed to minimize wall movement. The Rigid 2 analysis is illustrated in Section 3.3.

3.1.5 Winkler 1 analysis description

The Winkler 1 analysis is a beam on elastic foundation analysis where the soil springs are based on the referenced deflection method in accordance with FHWA-RD-98-066. Wall deflections greater than the reference deflections are considered to be plastic (nonrecoverable) movements. The earth pressure-deflection curves (R-y curves) are shifted following each excavation stage to account for those nonrecoverable displacements that are larger than the active state yield displacement. For cohesionless soils, active state yielding is considered to occur whenever the wall displacement exceeds 0.05 in. The shifted R-y curve approach is used to capture the buildup of earth pressure in the upper sections of the wall. Tiebacks are represented by anchor springs in the Winkler 1 analysis. Anchor loads are initially applied to determine the wall displacement at lock off. The wall displacement at the anchor lock-off load is used to establish the anchor load with respect to zero wall displacement. With this information, the anchor spring can then be properly introduced into the Winkler analysis. The computer program CMULTIANC (Dawkins, Strom, and Ebeling, in preparation) is used for the analysis. The Winkler 1 analysis is illustrated in Section 3.4.

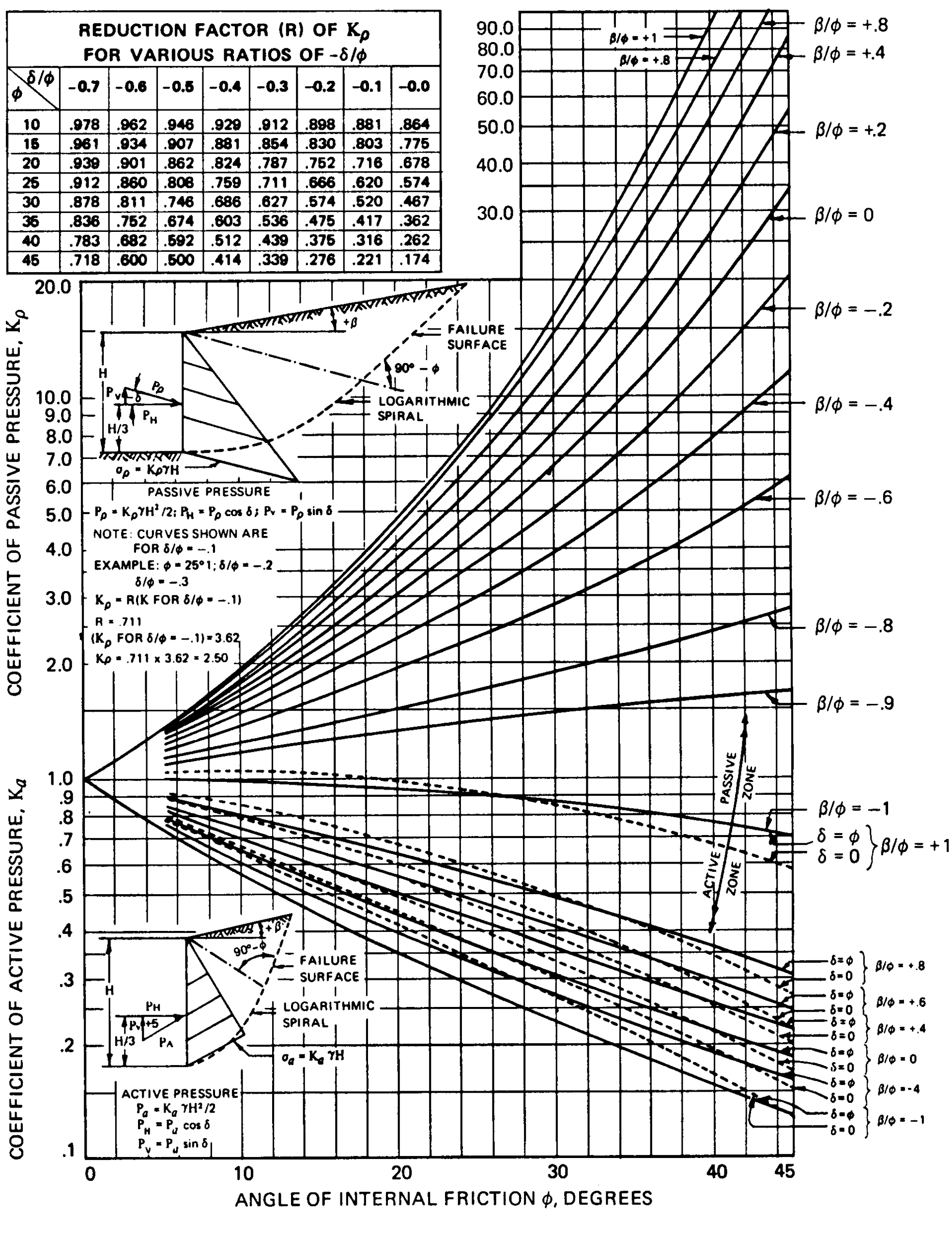
3.1.6 Assumptions

For the Rigid 1 and Rigid 2 analyses, at-rest pressure coefficients are used in driving-side earth pressure computations. Coefficients are in accordance with Munger, Jones, and Johnson (1991). The soil is assumed to have a moist unit weight (γ_{moist}) of 125 pcf and an angle of internal friction (ϕ) of 30 deg. Horizontal earth pressures were increased by 0.44 ksf because of surcharge loads from trains and equipment. The at-rest pressure coefficient (K_o) is equal to 0.50, calculated per Jaky (1944). The wall friction angle for passive resistance is equal to $-\phi/2$, or -15 deg.

Active pressure coefficients are based on Coulomb. Passive pressure coefficients (K_p) are per Caquot and Kerisel (1973), per 1982 Soletanche practice. (See Figure 3.2 and the calculations on the subsequent page.)

Since the base of the wall is keyed into rock, it was assumed for the various analyses that the wall is fixed against translation at its base (pinned condition at wall base).

This example follows the construction sequencing for the FEM study analysis of the Bonneville tieback, wall section 6 (Mosher and Knowles 1990). The actual excavation depths and overexcavation (excavations below anchor locations to facilitate anchor installation) for each excavation stage are somewhat different than those used in Mosher and Knowles (1990). For instance, in the actual construction, the first anchor was located 4 ft below the top of the wall. (The first anchor was located 5 ft below the top of the wall for the nonlinear FEM study.) Overexcavation was limited to 2 ft (5.5 ft of overexcavation was used for the FEM study). Final excavation was to a depth of 42 ft (whereas final excavation was to a depth of 50 ft in the nonlinear FEM study). All the measures used in the actual construction will substantially improve wall performance. A staged-excavation analysis modeling the actual construction stages can be found in FHWA-RD-98-066.



ϕ \ δ/ϕ	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	-0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174

Fig. 5(a) — Active and passive coefficients with wall friction (sloping backfill) (after Caquot and Kerisel[21])

Figure 3.2. Active and passive coefficients (after Caquot and Kerisel 1973)

Static Soil Pressure - Coulomb Active and Passive
The Seismic Design of Waterfront Retaining Structures
Equations 16 and 29, Bonneville Temporary Tieback Wall

File: BAA2\Coulomb k_a and k_p

$\phi := 30 \cdot \deg \qquad \theta := 0 \cdot \deg$

$\beta := 0 \cdot \deg \qquad \delta := 0 \cdot \deg$

$$K_a := \frac{\cos(\phi - \theta)^2}{\cos(\theta)^2 \cdot \cos(\theta + \delta) \cdot \left[1 + \sqrt{\frac{(\sin(\phi + \delta) \cdot \sin(\phi - \beta))}{\cos(\delta + \theta) \cdot \cos(\beta - \theta)}}\right]^2}$$

$K_a = 0.333$ **Use K_a = 0.333**

$\phi := 30 \cdot \deg \qquad \theta := 0 \cdot \deg$

$\beta := 0 \cdot \deg \qquad \delta := 15 \cdot \deg$

$$K_p := \frac{\cos(\phi + \theta)^2}{\cos(\theta)^2 \cdot \cos(\delta - \theta) \cdot \left[1 - \sqrt{\frac{(\sin(\phi + \delta) \cdot \sin(\phi + \beta))}{\cos(\delta - \theta) \cdot \cos(\beta - \theta)}}\right]^2}$$

$K_p = 4.977$

By Log Spiral (Caqout and Kerisel)

$K_{pls} := 6.5 \qquad R_{pls} := 0.746$

$K_p := R_{pls} \cdot K_{pls} \qquad K_p = 4.849$ **Use K_p = 4.85**

Driving-side at-rest earth pressures at elevations significant to the Rigid 1 and Rigid 2 analyses are shown in Figure 3.3.

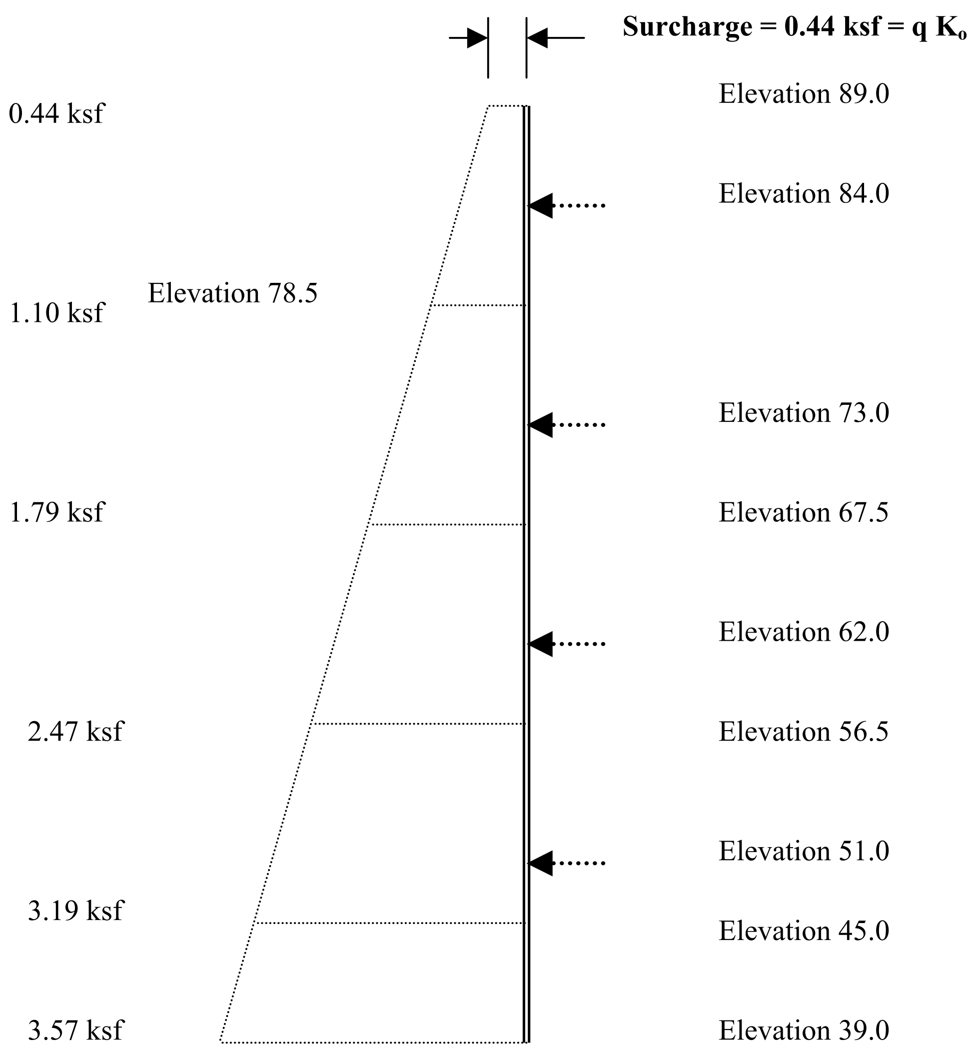


Figure 3.3. Excavation and tieback locations—driving-side at-rest earth pressure distribution at excavation levels, for Rigid 2 analyses

3.1.7 Observations

Using the analysis results contained in the following sections, wall computed maximum displacements, wall maximum positive and negative bending moments, and maximum anchor forces for each excavation stage for the Rigid 1, Rigid 2, and Winkler 1 analyses are compared with the results from the Mosher and Knowles (1990) FEM study. This information is provided in Table 3.2. A positive deflection indicates movement toward the retained soil, and a negative displacemment indicates a displacement toward the excavation. This is opposite the sign convention for displacement used in CMULTIANC. Positive moments are those that place the excavation side of the wall in tension, and negative moments are those that place the retained earth side of the wall in tension. It should be noted that these moment signs are opposite of those obtained from CBEAMC for the Rigid 2 analysis and from CMULTIANC for the Winkler 1 analysis.

The Rigid 1 analysis significantly underestimated the moment demands on the wall for both the final-excavation condition and intermediate-excavation conditions. This occurred even though a total load approach based on at-rest pressures was used for the development of the apparent pressure diagram. However, the method may serve as a means for initially determining ground anchor requirements. It should be noted that the original design used an earth pressure diagram that was a composite of the at-rest pressure diagram and the apparent pressure diagram based on an at-rest pressure coefficient (see Figure 3-8 of Strom and Ebeling 2001). Note that, at each elevation, the largest pressure obtained from the two pressure diagrams (i.e., apparent pressure diagram or at-rest pressure diagram) was used. Since the Rigid 1 analysis provides no usable information on wall displacements, it cannot be used as a final design method for walls that must meet displacement performance objectives.

The Rigid 2 analysis provided reasonable estimates of moment demands for both the final-excavation condition and intermediate-excavation conditions. However, as with the Rigid 1 analysis, it provides no usable information on wall displacements. Wall behavior, according to the Rigid 1 analysis results, was not consistant with the results obtained from the FEM study.

The Winkler 1 analysis provided reasonable estimates of moment demands for both the final-excavation condition and intermediate-excavation conditions. It also provided wall displacement behavior close to that predicted by the FEM study and close to measured behavior. Additional comparisons between the Winkler 1 analysis and FEM study can be found in Section 3.4. The purpose of a Winkler 1 analysis, however, should be primarily to evaluate the influence of anchor prestress and anchor spacing on wall displacements. A nonlinear finite element SSI analysis is often required to reasonably predict wall displacements, and such an analysis is generally a requirement in those cases where it is necessary to ensure that a particular tieback wall meets displacement performance objectives. However, such an analysis should not be undertaken until the anchor prestress and spacing has been optimized using a Winkler 1 analysis.

Table 3.2
Excavation Stages 1-5 – Analysis Results

	RIGID 1	RIGID 2	Winkler 1	FEM Study
Stage 1				
Maximum anchor load (kips)	NA	NA	NA	NA
Max. + moment (ft-kips)	NA	0	0	---
Max. - moment (ft-kips)	NA	89.9	83.7	50
Max. computed displacement (in.)	NA	NA	-0.50	-0.5
Stage 2				
Maximum anchor load (kips)	NA	14.7	28.2	29.8
Max. + moment (ft-kips)	NA	62.7	63.6	110
Max. - moment (ft-kips)	NA	6.8	27.8	10
Max. computed displacement (in.)	NA	NA	-0.26	0.2
Stage 3				
Maximum anchor load (kips)	NA	36.0	28.6	30.2
Max. + moment (ft-kips)	NA	77.9	121.7	120
Max. - moment (ft-kips)	NA	76.6	40.9	20
Max. computed displacement (in.)	NA	NA	-0.29	0.6
Stage 4				
Maximum anchor load (kips)	NA	57.6	28.3	29.7
Max. + moment (ft-kips)	NA	117.9	104.3	120
Max. - moment (ft-kips)	NA	127.7	61.0	20
Max. computed displacement (in.)	NA	NA	-0.22	0.6
Stage 5				
Maximum anchor load (kips)	25.4	38.7	28.1	36.1
Max. + moment (ft-kips)	27.2	35.7	55.7	75
Max. - moment (ft-kips)	23.6	43.7	67.5	20
Max. computed displacement (in.)	NA	NA	-0.25	0.6

3.2 RIGID 1 analysis (Equivalent Beam on Rigid Supports Analysis Using Apparent Pressures)

An equivalent beam on rigid supports analysis loaded with an apparent pressure diagram is used for the Rigid 1 analysis. For this analysis, the total load used to construct the apparent pressure diagram is based on an at-rest earth pressure coefficient (i.e., approximate factor of safety of 1.5 on the shear strength of the soil) (see discussion in Strom and Ebeling 2001). The resulting apparent pressure diagram is shown in Figure 3.4. Anchor forces and wall bending moment calculations follow. These calculations are similar to those for a multi-anchor wall system (Chapter 10 of FHWA-RD-97-130).

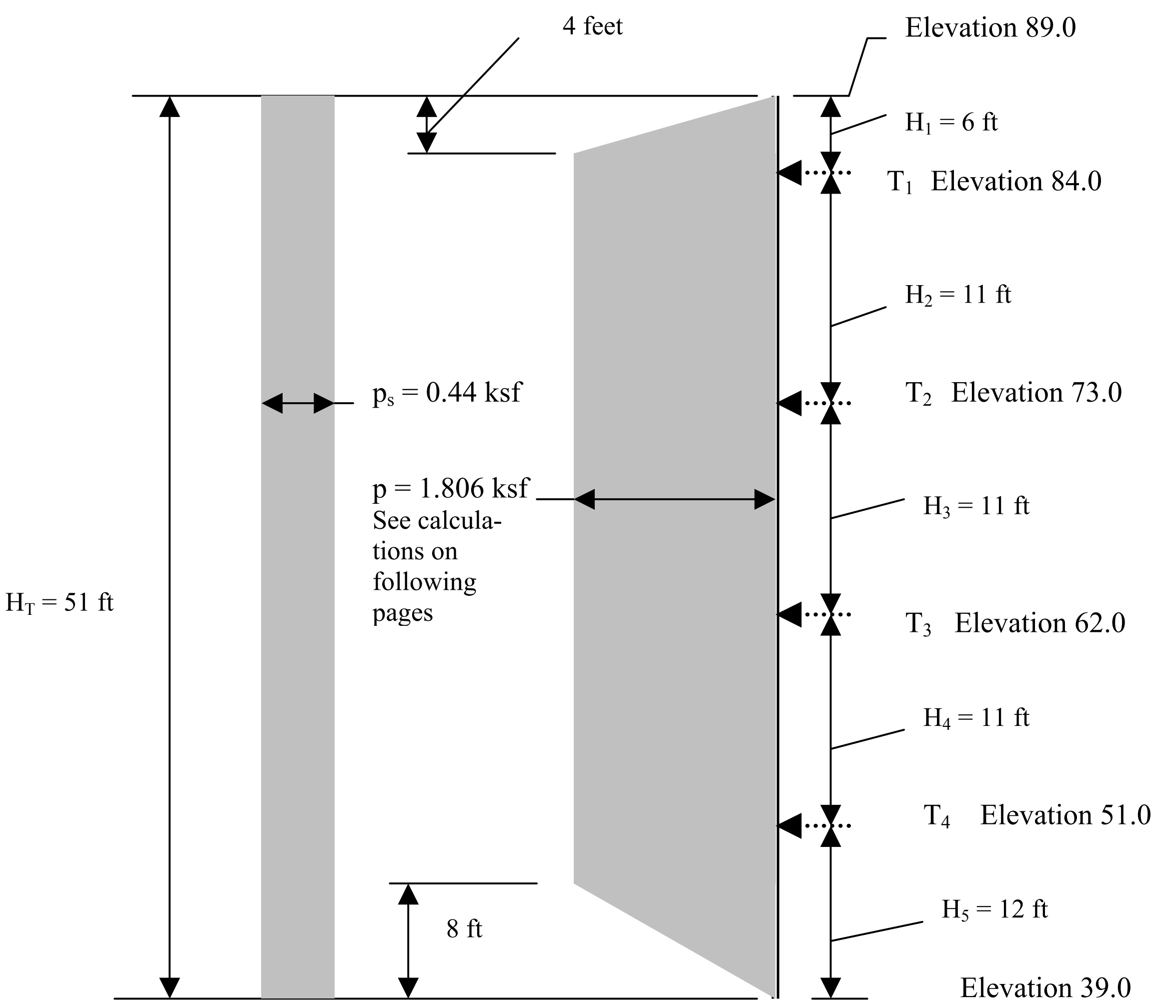


Figure 3.4. Apparent pressure and surcharge loadings

File: Bonneville Apparent Pressure

Multiple Anchor Locations
Surcharge

Soil Properties

Friction Angle = 30 Degees
Total weight (γ) = 125 pcf
ka = 0.333
k_p = 4.85

Determine Total Earth Pressure Load
Based on At-Rest Pressure

$k_0 := 0.50$ $\qquad \gamma := 0.125$ ksf $\qquad H_T := 51$

$$T_L := \frac{k_0 \cdot \gamma \cdot (H_T)^2}{2} \qquad T_L = 81.281$$ kips/foot

Earth pressure to stabilize cut (p)

$H_1 := 6 \qquad H_2 := 11 \qquad H_3 := 11 \qquad H_4 := 11 \qquad H_5 := 12$

$$p := \frac{T_L}{(H_T - 0.333H_1 - 0.333H_5)}$$

$p = 1.806$ ksf

Surcharge Pressure (p_s)

$p_s := 0.44$ ksf

Surcharge load from trains and construction equipment (see Figure 3-4)

Calculate Bending Moment at Upper Ground Anchor (M_1)

$$M_1 := \left(\frac{13}{54}\right)\cdot H_1^2\cdot p + \left(p_s\cdot H_1\cdot\frac{H_1}{2}\right)$$

$M_1 = 23.572$ ft-kips per foot of wall

Calculate the Ground Anchor Loads by the Tributary Area Method

$$T_1 := \left[\left(\frac{2}{3}\right)\cdot H_1 + \left(\frac{1}{2}\right)\cdot H_2\right]\cdot p + \left(H_1 + \frac{H_2}{2}\right)\cdot p_s$$

$T_1 = 22.217$ kips per foot of wall

$$T_2 := \left[\left(\frac{1}{2}\right)\cdot H_2 + \left(\frac{1}{2}\right)\cdot H_3\right]\cdot p + \left(\frac{H_2}{2} + \frac{H_3}{2}\right)\cdot p_s$$

$T_2 = 24.706$ kips per foot of wall

$$T_3 := \left[\left(\frac{1}{2}\right)\cdot H_3 + \left(\frac{1}{2}\right)\cdot H_4\right]\cdot p + \left(\frac{H_3}{2} + \frac{H_4}{2}\right)\cdot p_s$$

$T_2 = 24.706$ kips per foot of wall

$$T_4 := \left[\left(\frac{1}{2}\right)\cdot H_4 + \left(\frac{23}{48}\right)\cdot H_5\right]\cdot p + \left(\frac{H_4}{2} + \frac{H_5}{2}\right)\cdot p_s$$

$T_4 = 25.378$ kips per foot of wall

Calculate the Subgrade Reaction at Base of Wall (R_B)

$$R_B := \left[\left(\frac{3}{16}\right)\cdot H_5\right]\cdot(p) + \left(\frac{H_5}{2}\right)\cdot p_s$$

$R_B = 6.704$ kips per foot of wall

Calculate Maximum Bending Moment Below Upper Anchor

$$MM_1 := \left(\frac{1}{10}\right) \cdot H_2^{\,2} \cdot \left(p + p_s\right)$$

$MM_1 = 27.177$ ft-kips per foot

3.3 RIGID 2 Analysis

Construction-sequencing analysis
Equivalent beam on rigid supports (Rigid)
Classical methods

The RIGID 2 analysis is an equivalent beam on rigid supports excavation sequencing analysis. Earth pressures, however, are in accordance with classical earth pressure theory, assuming a wall retaining nonyielding soil backfill (at-rest earth pressure distribution).

3.3.1 First-stage excavation analysis

The first-stage net pressure conditions are illustrated in Figure 3.5. Calculations for quantities of interest follow, along with a CBEAMC analysis used to determine wall bending moments and shears.

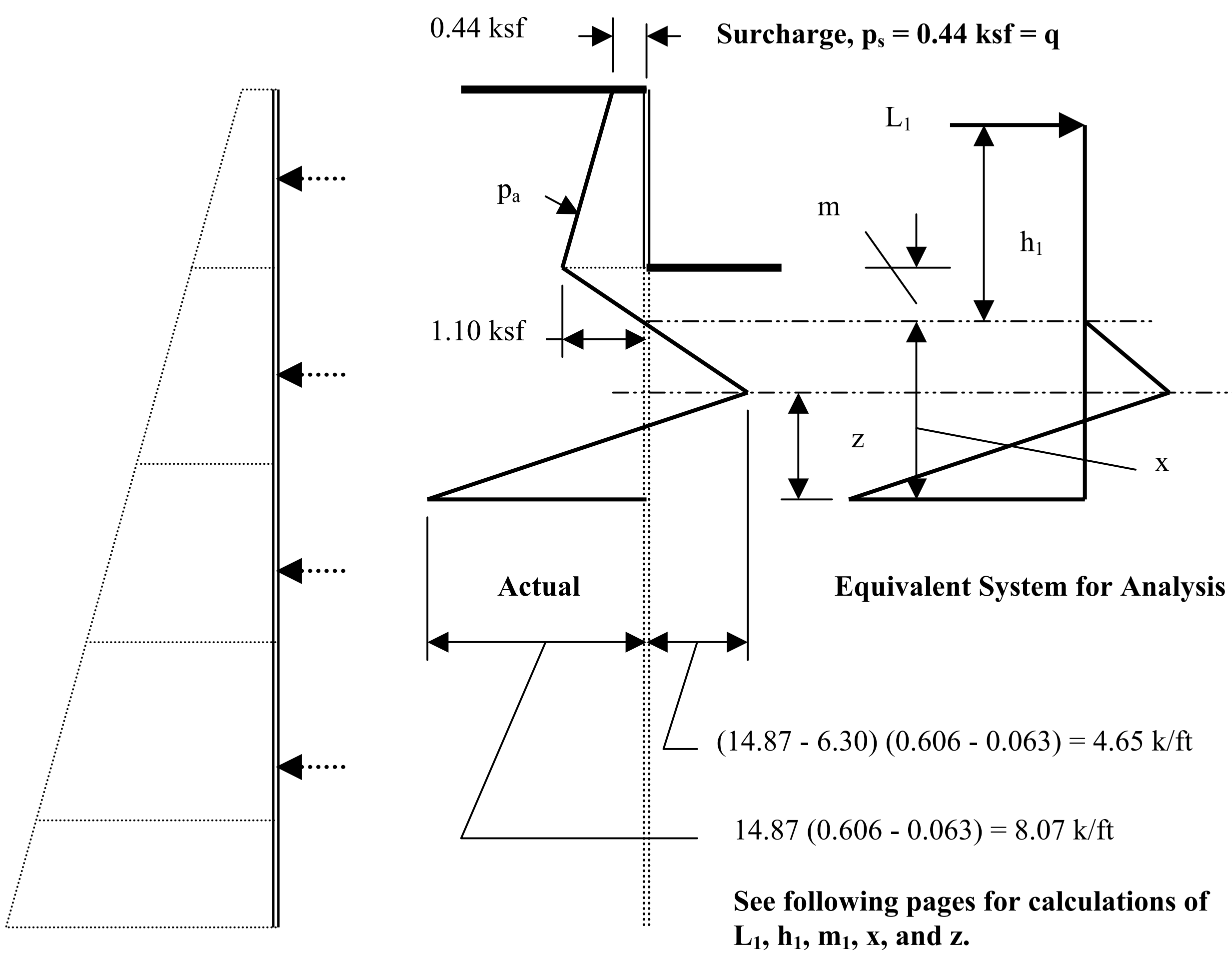


Figure 3.5. Step 4–excavate to el 78.5, first-stage excavation

Cantilevered Section of Tieback Wall (Uppermost Section)
Determine Minimum Penetration (maximum moment condition)
Per Figure 2-3 Andersen's "Substructure Analysis and Design" 2nd Edition

Bonneville Tieback Wall File: BTW1

$p_a := \frac{(3.57 - 0.44)}{50}$ $\quad p_a = 0.063$ kips / ft.

$\gamma := 0.125$ kips / ft.

$K_p := 4.85$ After Caquot and Kerisel

$p_p := K_p \cdot \gamma$ $\quad p_p = 0.606$ kips / ft.

$m_1 := \frac{1.10}{(p_p - p_a)}$ $\quad m_1 = 2.023$ Feet

$L_1 := 0.44(10.5) + \left[0.66(10.5) \cdot \frac{1}{2}\right] + 1.10 \cdot \frac{m_1}{2}$ $\quad L_1 = 9.198$ Kips

$$h_1 := \left[0.44(10.5) \cdot \left[(10.5) \cdot \left(\frac{1}{2}\right) + m_1\right] + 0.66(10.5) \cdot \left(\frac{1}{2}\right) \cdot \left[10.5 \cdot \left(\frac{1}{3}\right) + m_1\right] + 1.10 \cdot \left(\frac{m_1}{2}\right) \cdot \left(2 \cdot \frac{m_1}{3}\right)\right] \cdot \left(\frac{1}{L_1}\right)$$

$h_1 = 5.897$ Feet

Try various values of "x"
The correct value of "x" is when Y = 0

Try $x := 14.868$ Feet

$$Y := \left[x^4 - \left(\frac{8 \cdot L_1}{p_p - p_a}\right) \cdot x^2 - \left[\frac{(12 \cdot L_1 \cdot h_1)}{(p_p - p_a)}\right] \cdot x - 4 \cdot \left(\frac{L_1}{p_p - p_a}\right)^2\right]$$

$Y = -6.247 \cdot 10^{-3}$ Approximately equal to zero okay

$$z := \left(\frac{x}{2}\right) - \frac{L_1}{(p_p - p_a) \cdot x}$$ $z = 6.296$ Feet

Use x = 14.686 ft. For x, refer to Figure 3.5.

The net earth pressure diagram based on classical methods, established by the above calculations and illustrated in Figure 3.5, is used in a beam-column analysis (CBEAMC analysis) to determine wall bending moments and shears. In the CBEAMC analysis, the wall is provided with a fictitious support. This support is fixed against translation and rotation to provide stability for the beam-column solution. The support is located at a distance equal to (89.0 - 78.5) + m_1 + x, or 27.39 ft below the top of the wall. This is the point that first provides static equilibrium and thus produces the minimum penetration depth required for system stability. At this depth the moment should be zero, and provided the fictitious support depth has been properly determined, the CBEAMC analysis should confirm this. Input and output for the CBEAMC analysis are provided on the following pages. Wall bending moments obtained from the CBEAMC analysis for the first-stage excavation analysis (and for Stages 2-5) are plotted in Figure 3.6.

CBEAMC Input for Classical Tieback Wall Analysis
First Stage Excavation Analysis (Cantilever Stage)
File: BW1

HEADING	
LN	"Heading Description"
1000	**'BONNEVILLE TIEBACK WALL FIRST STAGE EXCAVATION**

BEAM HEADER			
LN	"Beam Title"	"New or Add"	Units "**I**nches or **F**eet" "**P**ounds or **K**ips"
1020	**BEAM**		**F K**

BEAM DATA LINES					
LN	X1	X2	E	A1	SI1
1030	**0.0**	**27.39**	**475000.**	**3.00**	**2.25**

NODE SPACING HEADER			
LN	"NODe"	"New or Add"	Units "**I**nches or **F**eet" "**P**ounds or **K**ips"
1040	**NODES**		**F**

NODE SPACING DATA LINES			
LN	X1 "Coord @ Start"	X2 "Coord @ End"	HMAX "Max dist. betw. nodes"
1050	**0.00**	**27.39**	**1.0**

LOADS HEADER LINE			
LN	"Loads"	"New or Add"	Units "**I**nches or **F**eet" "**P**ounds or **K**ips"
1060	**LOADS**		**F K**

DISTRIBUTED LOADS DATA LINES						
LN	"Distributed"	"Direction"	X1	Q1	X2	Q2
1070	**D**	**Y**	**0.00**	**0.44**	**10.50**	**1.10**
1080	**D**	**Y**	**10.50**	**1.10**	**12.52**	**0.00**
1090	**D**	**Y**	**12.52**	**0.00**	**21.09**	**-4.65**
1100	**D**	**Y**	**21.09**	**-4.65**	**27.39**	**8.07**

FIXED SUPPORTS HEADER			
LN	"FIXed"	"New or Add"	Units "**I**nches or **F**eet" "**P**ounds or **K**ips"
1120	**FIXED**		**F**

FIXED SUPPORTS DATA LINES				
LN	X1 "Coord of support"	XD "Displ. or free"	YD "Displ. or free"	R "Rotation or free"
1130	**27.39**	**0.0**	**0.0**	**0.0**

TERMINATION			
LN	" FINish"	"Rerun"	"Keep"
1150	**FINISH**		

CBEAMC RESULTS   File: BW1

```
'BONNEVILLE TIEBACK WALL
'FIRST STAGE EXCAVATION
BEAM  FT  KSF  FT
   0    27.39   4.750E+05   3    2.25    3    2.25
NODES  FT  FT
   0    27.39    1
LOADS DISTRIBUTED  FT  K/FT
   0    0    0.44    10.5    0    1.1
   10.5    0    1.1    12.52    0    0
   12.52    0    0    21.09    0   -4.65
   21.09    0   -4.65    27.39    0    8.07
FIXED  FT  FT
   27.39   0.000  0.000  0.000
FINISHED


  PROGRAM CBEAMC - ANALYSIS OF BEAM-COLUMNS WITH NONLINEAR
                          SUPPORTS

   DATE: 11-FEBRUARY-2001                TIME: 11:23:49

                  ************************
                  *  SUMMARY OF RESULTS  *
                  ************************

I.--HEADING
      'BONNEVILLE TIEBACK WALL
      'FIRST STAGE EXCAVATION

II.--MAXIMA

                          MAXIMUM  X-COORD  MAXIMUM X-COORD
                         POSITIVE   (FT)   NEGATIVE   (FT)
AXIAL DISPLACEMENT (FT)  : 0.000E+00  0.00  0.000E+00  0.00
LATERAL DISPLACEMENT (FT): 1.711E-02  0.00  0.000E+00  0.00
ROTATION (RAD)           : 0.000E+00  0.00 -1.023E-03  0.00
AXIAL FORCE (K)          : 0.000E+00  0.00  0.000E+00  0.00
SHEAR FORCE (K)          : 9.196E+00 12.52 -1.592E+01 23.79
BENDING MOMENT (K-FT)    : 8.989E+01 18.23 -2.754E-13  0.00

III.--REACTIONS AT FIXED SUPPORTS
             X-C00RD X-REACTION Y-REACTION MOMENT-REACTION
              (FT)       (K)        (K)        (K-FT)
               27.39  0.000E+00 -4.375E-02       3.695E-01


IV.--FORCES IN LINEAR CONCENTRATED SPRINGS
          NONE


V.--FORCES IN NONLINEAR CONCENTRATED SPRINGS
          NONE
```

PROGRAM CBEAMC - ANALYSIS OF BEAM-COLUMNS WITH NONLINEAR SUPPORTS

DATE: 11-FEBRUARY-2001 TIME: 11:23:49

```
*********************
*  COMPLETE RESULTS  *
*********************
```

I.--HEADING
'BONNEVILLE TIEBACK WALL
'FIRST STAGE EXCAVATION

II.--DISPLACEMENTS AND INTERNAL FORCES

	<------DISPLACEMENTS------>		<----INTERNAL FORCES---->	
X-COORD (FT)	LATERAL (FT)	ROTATION (RAD)	SHEAR (K)	MOMENT (K-FT)
0.00	1.711E-02	-1.023E-03	-2.162E-11	-2.754E-13
0.95	1.614E-02	-1.023E-03	4.486E-01	2.096E-01
1.91	1.516E-02	-1.023E-03	9.545E-01	8.747E-01
2.86	1.418E-02	-1.021E-03	1.518E+00	2.050E+00
3.82	1.321E-02	-1.019E-03	2.138E+00	3.790E+00
4.77	1.224E-02	-1.014E-03	2.816E+00	6.150E+00
5.73	1.127E-02	-1.008E-03	3.551E+00	9.184E+00
6.68	1.032E-02	-9.977E-04	4.343E+00	1.295E+01
7.64	9.371E-03	-9.842E-04	5.193E+00	1.749E+01
8.59	8.440E-03	-9.662E-04	6.100E+00	2.288E+01
9.55	7.528E-03	-9.431E-04	7.064E+00	2.916E+01
10.50	6.642E-03	-9.139E-04	8.085E+00	3.638E+01
11.51	5.737E-03	-8.755E-04	8.918E+00	4.502E+01
12.52	4.876E-03	-8.286E-04	9.196E+00	5.421E+01
13.47	4.111E-03	-7.764E-04	8.950E+00	6.289E+01
14.42	3.400E-03	-7.167E-04	8.212E+00	7.110E+01
15.38	2.749E-03	-6.500E-04	6.982E+00	7.837E+01
16.33	2.164E-03	-5.775E-04	5.260E+00	8.424E+01
17.28	1.650E-03	-5.005E-04	3.046E+00	8.823E+01
18.23	1.211E-03	-4.209E-04	3.403E-01	8.989E+01
19.19	8.487E-04	-3.411E-04	-2.858E+00	8.873E+01
20.14	5.610E-04	-2.638E-04	-6.547E+00	8.429E+01
21.09	3.445E-04	-1.920E-04	-1.073E+01	7.610E+01
21.99	1.992E-04	-1.325E-04	-1.410E+01	6.481E+01
22.89	1.028E-04	-8.354E-05	-1.583E+01	5.122E+01
23.79	4.525E-05	-4.647E-05	-1.592E+01	3.681E+01
24.69	1.559E-05	-2.137E-05	-1.439E+01	2.304E+01
25.59	3.526E-06	-7.069E-06	-1.121E+01	1.140E+01
26.49	3.322E-07	-1.160E-06	-6.402E+00	3.353E+00
27.39	0.000E+00	0.000E+00	4.375E-02	3.695E-01

Shear and moment approximately equal to zero. Okay.

II.--FORCES IN LINEAR DISTRIBUTED SPRINGS
NONE

IV.--FORCES IN NONLINEAR DISTRIBUTED SPRINGS
NONE

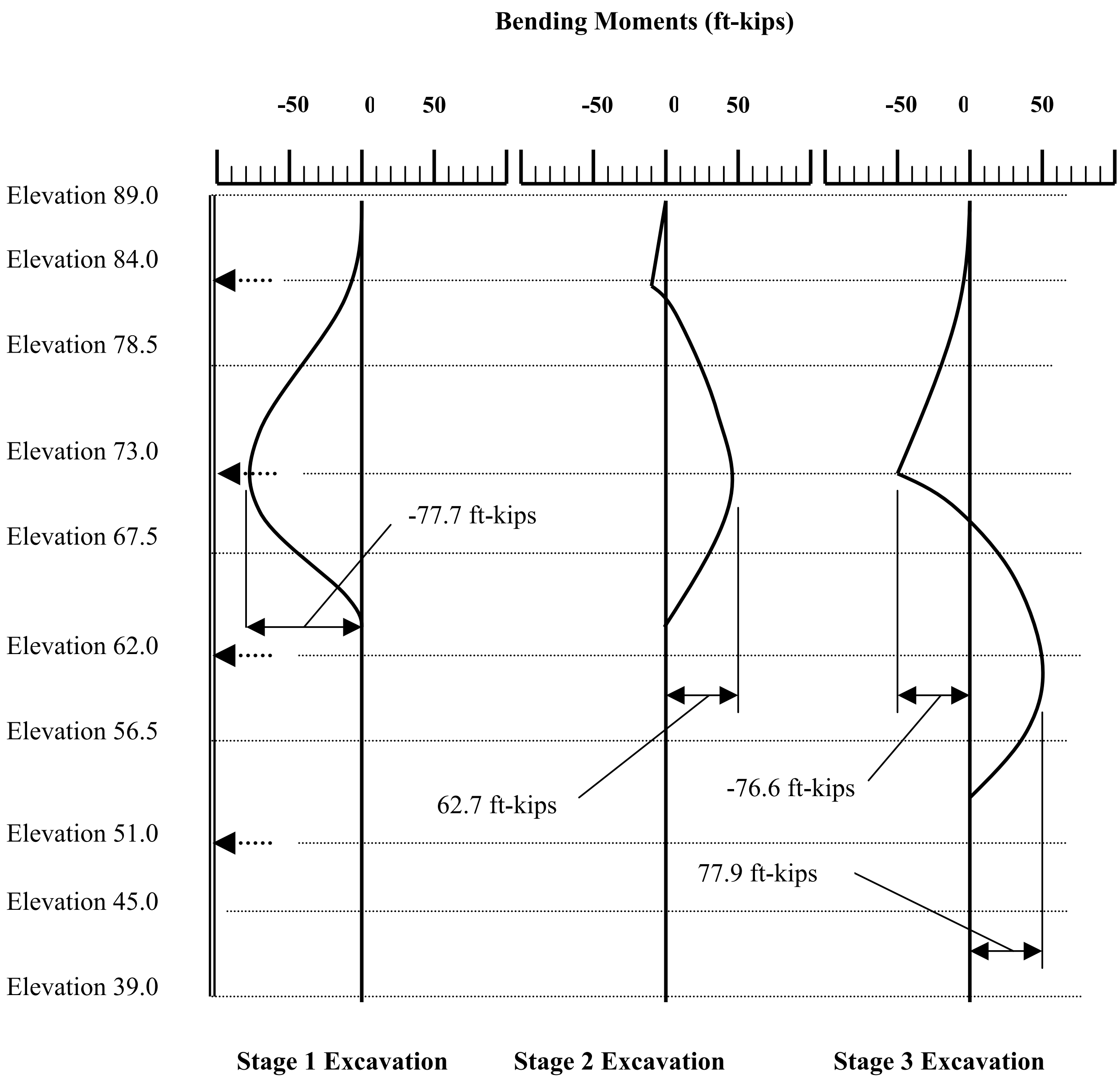


Figure 3.6. Wall bending moments (Continued)

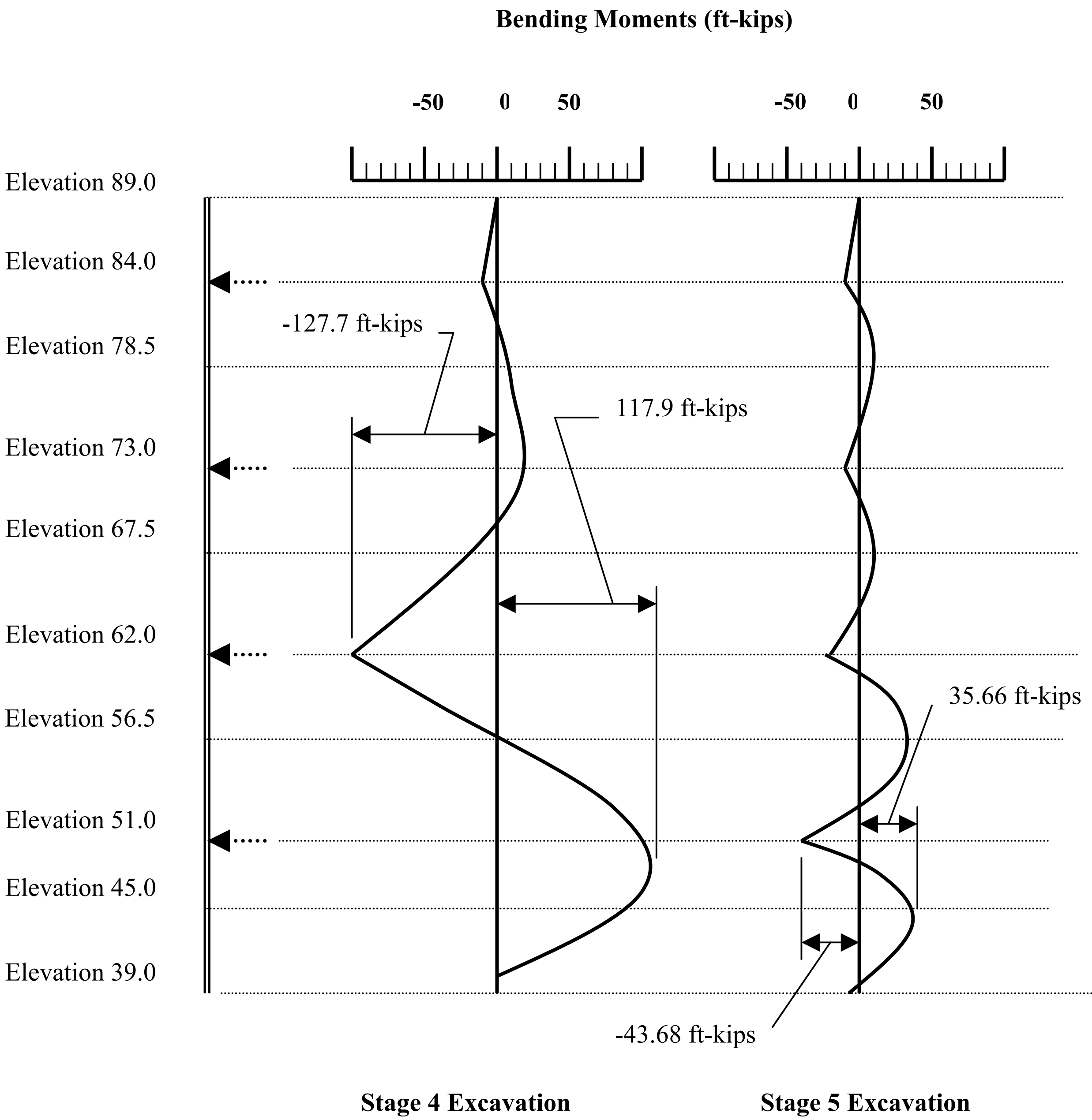


Figure 3.6. (Concluded)

3.3.2 Second-stage excavation analysis

The computations for the second excavation stage (Stage 2) are provided below. Second-stage excavation is at a depth of 89.0 - 67.5 = 21.5 ft. In the analysis, a point of contraflexure is assumed to coincide with zero net pressure point located at a distance of "21.5 + *m* ft" below the surface. Using this assumption, the upper portion of the anchored tieback wall can be treated as an equivalent beam that is simply supported at the anchor location and at the first point of zero net pressure intensity. The equivalent beam with net pressure loading is shown in Figure 3.7. As with first-stage excavation, the second-stage excavation analysis is performed using the CBEAMC software. The CBEAMC input and output for the final stage analysis is provided below. Bending moments for the second-stage excavation are plotted in Figure 3.6a.

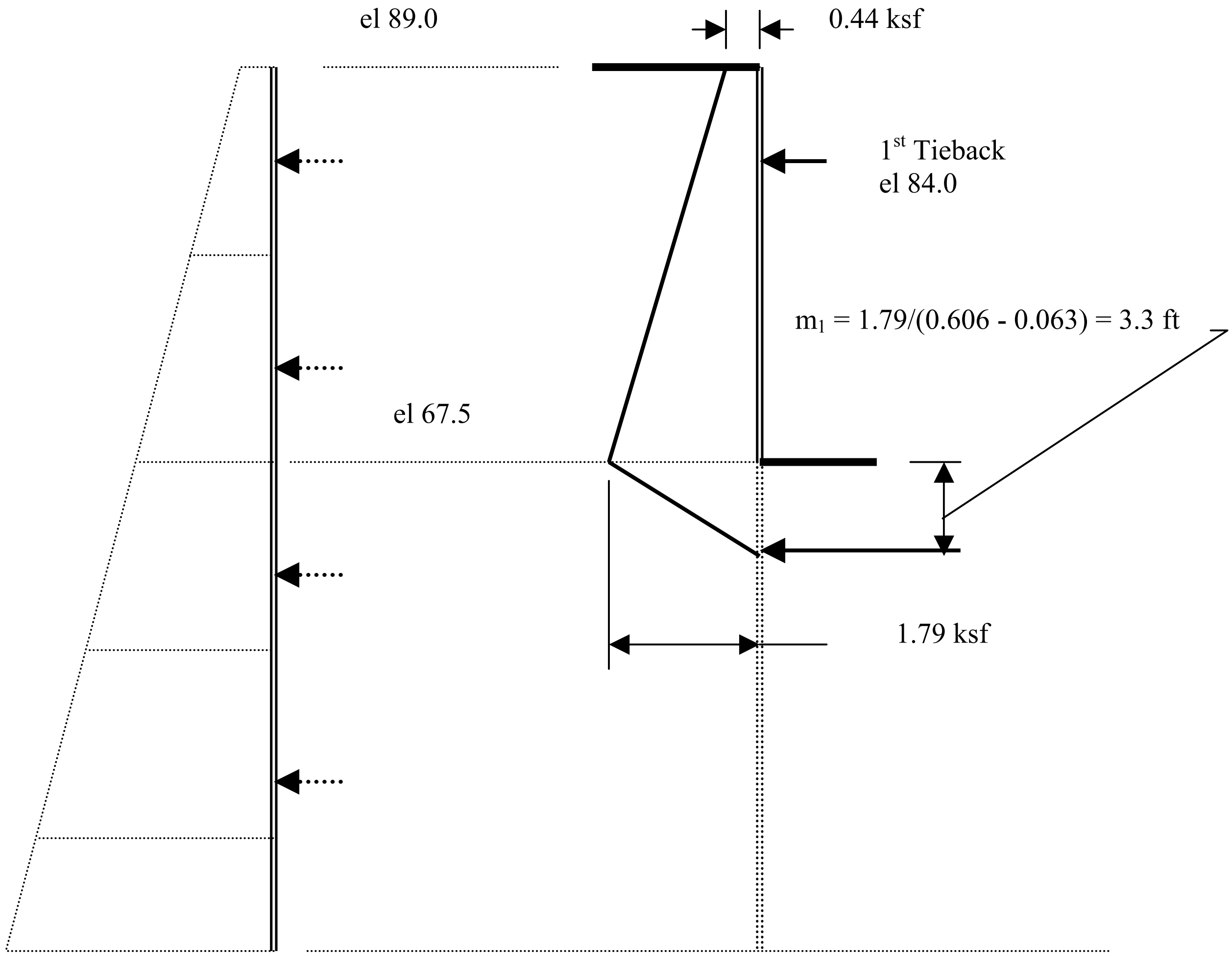


Figure 3.7. Step 6, excavate to el 67.5, second-stage excavation

CBEAMC Input for Classical Tieback Wall Analysis
Second Stage Excavation Analysis (Equivalent Beam Analysis)
File: BW2

HEADING	
LN	"Heading Description"
1000	**BONNEVILLE TIEBACK WALL SECOND STAGE EXCAVATION**

BEAM HEADER			
LN	"Beam Title"	"New or Add"	Units "**I**nches or **F**eet" "**P**ounds or **K**ips"
1020	**BEAM**		**F K**

BEAM DATA LINES					
LN	X1	X2	E	A1	SI1
1030	**0.0**	**24.80**	**475000.**	**3.00**	**2.25**

NODE SPACING HEADER			
LN	"NODe"	"New or Add"	Units "**I**nches or **F**eet" "**P**ounds or **K**ips"
1040	**NODES**		**F**

NODE SPACING DATA LINES			
LN	X1 "Coord @ Start"	X2 "Coord @ End"	HMAX "Max dist. betw. nodes"
1050	**0.00**	**24.80**	**2.0**

LOADS HEADER LINE			
LN	"Loads"	"New or Add"	Units "**I**nches or **F**eet" "**P**ounds or **K**ips"
1060	**LOADS**		**F K**

DISTRIBUTED LOADS DATA LINES						
LN	"Distributed"	"Direction"	X1	Q1	X2	Q2
1070	**D**	**Y**	**0.00**	**0.44**	**21.50**	**1.79**
1080	**D**	**Y**	**21.50**	**1.79**	**24.80**	**0.00**

FIXED SUPPORTS HEADER			
LN	"FIXed"	"New or Add"	Units "**I**nches or **F**eet" "**P**ounds or **K**ips"
1100	**FIXED**		**F**

FIXED SUPPORTS DATA LINES				
LN	X1 "Coord of support"	XD "Displ. or free"	YD "Displ. or free"	R "Rotation or free"
1110	**5.00**	**0.0**	**0.0**	**FREE**
1120	**24.80**	**0.0**	**0.0**	**FREE**

TERMINATION		
LN	" FINish"	"Rerun"	"Keep"
1150	**FINISH**		

CBEAMC RESULTS **File: BW2**

```
'BONNEVILLE TIEBACK WALL
'SECOND STAGE EXCAVATION
BEAM  FT  KSF  FT
   0    24.8   4.750E+05   3    2.25    3    2.25
NODES  FT  FT
   0    24.8    2
LOADS DISTRIBUTED  FT  K/FT
   0    0    0.44    21.5    0    1.79
   21.5    0    1.79    24.8    0    0
FIXED  FT  FT
   5   0.000  0.000  FREE
   24.8   0.000  0.000  FREE
FINISHED
```

```
  PROGRAM CBEAMC - ANALYSIS OF BEAM-COLUMNS WITH NONLINEAR
                          SUPPORTS

   DATE: 12-FEBRUARY-2001                  TIME: 9:51:21

                ************************
                *  SUMMARY OF RESULTS  *
                ************************

I.--HEADING
      'BONNEVILLE TIEBACK WALL
      'SECOND STAGE EXCAVATION


II.--MAXIMA
```

	MAXIMUM POSITIVE	X-COORD (FT)	MAXIMUM NEGATIVE	X-COORD (FT)
AXIAL DISPLACEMENT (FT) :	0.000E+00	0.00	0.000E+00	0.00
LATERAL DISPLACEMENT (FT):	2.317E-03	16.00	-1.715E-03	0.00
ROTATION (RAD) :	3.506E-04	5.00	-3.881E-04	24.80
AXIAL FORCE (K) :	0.000E+00	0.00	0.000E+00	0.00
SHEAR FORCE (K) :	1.221E+01	24.80	-1.173E+01	5.00
BENDING MOMENT (K-FT) :	6.808E+00	5.00	-6.265E+01	16.00

III.--REACTIONS AT FIXED SUPPORTS

X-COORD (FT)	X-REACTION (K)	Y-REACTION (K)	MOMENT-REACTION (K-FT)
5.00	0.000E+00	-1.471E+01	0.000E+00
24.80	0.000E+00	-1.221E+01	0.000E+00

```
IV.--FORCES IN LINEAR CONCENTRATED SPRINGS
          NONE


V.--FORCES IN NONLINEAR CONCENTRATED SPRINGS
          NONE
```

```
PROGRAM CBEAMC - ANALYSIS OF BEAM-COLUMNS WITH NONLINEAR
                       SUPPORTS

 DATE: 12-FEBRUARY-2001                  TIME: 9:51:21

                 **********************
                 *  COMPLETE RESULTS  *
                 **********************

I.--HEADING
      'BONNEVILLE TIEBACK WALL
      'SECOND STAGE EXCAVATION


II.--DISPLACEMENTS AND INTERNAL FORCES
  <---DISPLACEMENTS--------> <----INTERNAL FORCES------->
 X-COORD     LATERAL    ROTATION     SHEAR      MOMENT
   (FT)        (FT)       (RAD)        (K)       (K-FT)
   0.00    -1.715E-03  3.405E-04    -1.648E-13 -4.657E-14
   1.67    -1.147E-03  3.408E-04     8.205E-01  6.596E-01
   3.33    -5.774E-04  3.433E-04     1.816E+00  2.832E+00
   5.00     0.000E+00  3.506E-04     2.985E+00  6.808E+00
   5.00     0.000E+00  3.506E-04    -1.173E+01  6.808E+00
   6.83     6.426E-04  3.446E-04    -1.024E+01 -1.336E+01
   8.67     1.244E-03  3.064E-04    -8.541E+00 -3.061E+01
  10.50     1.750E-03  2.415E-04    -6.631E+00 -4.455E+01
  12.33     2.117E-03  1.557E-04    -4.510E+00 -5.479E+01
  14.17     2.312E-03  5.582E-05    -2.178E+00 -6.096E+01
  16.00     2.317E-03 -5.087E-05     3.647E-01 -6.265E+01
  17.83     2.126E-03 -1.564E-04     3.119E+00 -5.949E+01
  19.67     1.750E-03 -2.520E-04     6.084E+00 -5.109E+01
  21.50     1.214E-03 -3.284E-04     9.260E+00 -3.706E+01
  23.15     6.319E-04 -3.727E-04     1.148E+01 -1.975E+01
  24.80     0.000E+00 -3.881E-04     1.221E+01  6.970E-14


II.--FORCES IN LINEAR DISTRIBUTED SPRINGS
          NONE


IV.--FORCES IN NONLINEAR DISTRIBUTED SPRINGS
          NONE
```

3.3.3 Third-stage excavation analysis

The third-stage excavation is to el 56.5. The equivalent beam with net pressure loading is shown in Figure 3.8. Calculations for the third-stage excavation are performed in the same manner as for the second-stage excavation. Bending moments are plotted in Figure 3.6a.

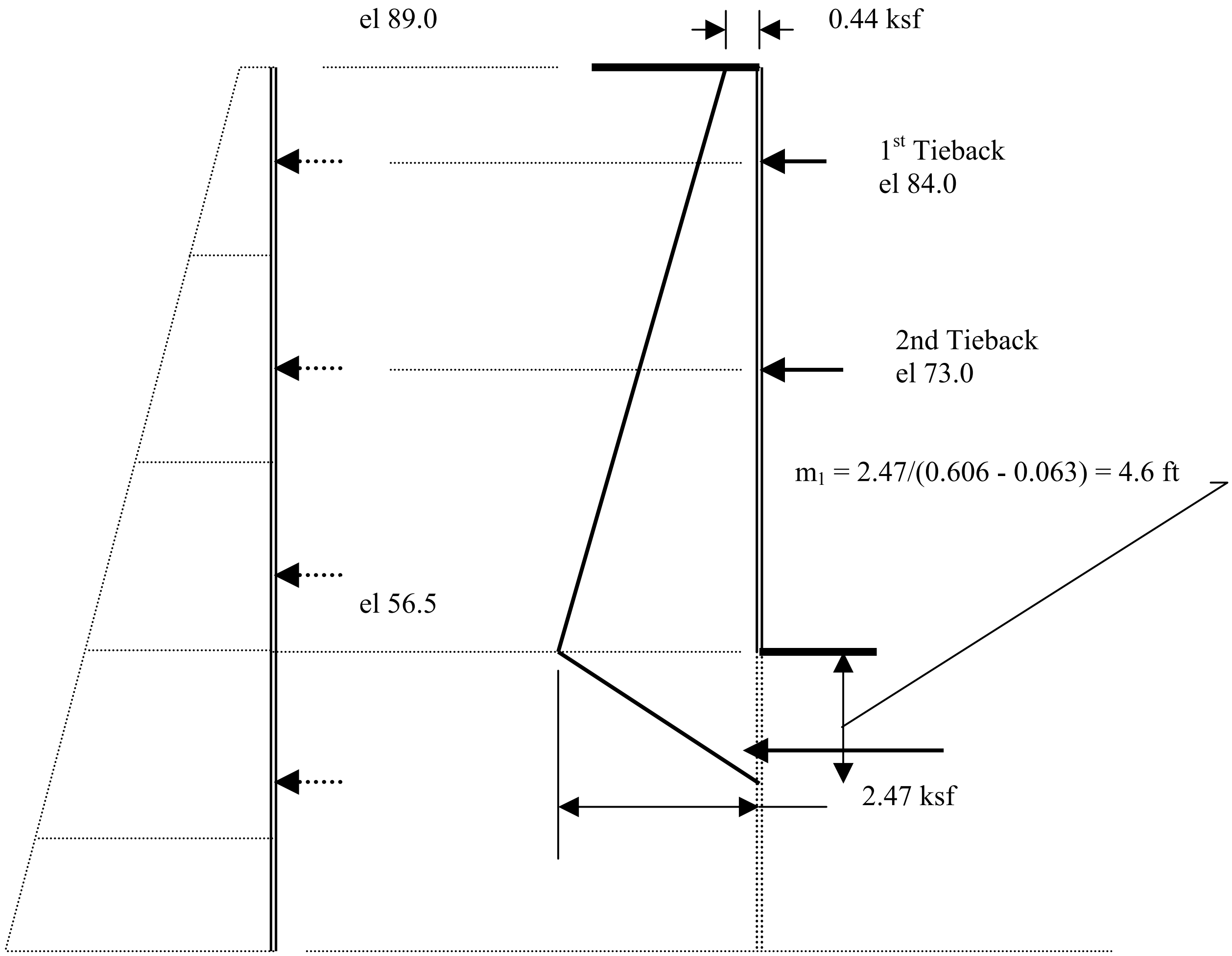


Figure 3.8. Step 8, excavate to el 56.5, third-stage excavation

CBEAMC Input for Classical Tieback Wall Analysis
Third Stage Excavation Analysis (Equivalent Beam Analysis)
File: BW3

HEADING

LN	"Heading Description"
1000	**BONNEVILLE TIEBACK WALL THIRD STAGE EXCAVATION**

BEAM HEADER

LN	"Beam Title"	"New or Add"	Units "**I**nches or **F**eet" "**P**ounds or **K**ips"
1020	**BEAM**		**F K**

BEAM DATA LINES

LN	X1	X2	E	A1	SI1
1030	**0.0**	**37.1**	**475000.**	**3.00**	**2.25**

NODE SPACING HEADER

LN	"NODe"	"New or Add"	Units "**I**nches or **F**eet" "**P**ounds or **K**ips"
1040	**NODES**		**F**

NODE SPACING DATA LINES

LN	X1 "Coord @ Start"	X2 "Coord @ End"	HMAX "Max dist. betw. nodes"
1050	**0.00**	**37.1**	**2.0**

LOADS HEADER LINE

LN	"Loads"	"New or Add"	Units "**I**nches or **F**eet" "**P**ounds or **K**ips"
1060	**LOADS**		**F K**

DISTRIBUTED LOADS DATA LINES

LN	"Distributed"	"Direction"	X1	Q1	X2	Q2
1070	**D**	**Y**	**0.00**	**0.44**	**32.50**	**2.47**
1080	**D**	**Y**	**32.50**	**2.47**	**37.1**	**0.00**

FIXED SUPPORTS HEADER

LN	"FIXed"	"New or Add"	Units "**I**nches or **F**eet" "**P**ounds or **K**ips"
1100	**FIXED**		**F**

FIXED SUPPORTS DATA LINES

LN	X1 "Coord of support"	XD "Displ. or free"	YD "Displ. or free"	R "Rotation or free"
1110	**5.00**	**0.0**	**0.0**	**FREE**
1120	**16.00**	**0.0**	**0.0**	**FREE**
1130	**37.1**	**0.0**	**0.0**	**FREE**

TERMINATION

LN	" FINish"	"Rerun"	"Keep"
1150	**FINISH**		

CBEAMC RESULTS   File: BW3

```
'BONNEVILLE TIEBACK WALL
'THIRD STAGE EXCAVATION
BEAM  FT  KSF  FT
   0    37.1   4.750E+05   3    2.25    3    2.25
NODES  FT  FT
   0    37.1    2
LOADS DISTRIBUTED  FT  K/FT
   0    0    0.44    32.5    0    2.47
   32.5    0    2.47    37.1    0    0
FIXED  FT  FT
   5   0.000  0.000  FREE
   16   0.000  0.000  FREE
   37.1   0.000  0.000  FREE
FINISHED
```

```
  PROGRAM CBEAMC - ANALYSIS OF BEAM-COLUMNS WITH NONLINEAR
                        SUPPORTS

     DATE: 12-FEBRUARY-2001                TIME: 9:52:52

                  ************************
                  *  SUMMARY OF RESULTS  *
                  ************************

I.--HEADING
      'BONNEVILLE TIEBACK WALL
      'THIRD STAGE EXCAVATION

II.--MAXIMA
                              MAXIMUM  X-COORD    MAXIMUM   X-COORD
                             POSITIVE   (FT)      NEGATIVE   (FT)
AXIAL DISPLACEMENT (FT)  :  0.000E+00     0.00   0.000E+00     0.00
LATERAL DISPLACEMENT (FT):  2.848E-03    27.00  -4.021E-04    12.33
ROTATION (RAD)           :  3.463E-04    19.67  -4.784E-04    37.10
AXIAL FORCE (K)          :  0.000E+00     0.00   0.000E+00     0.00
SHEAR FORCE (K)          :  1.494E+01    37.10  -2.299E+01    16.00
BENDING MOMENT (K-FT)    :  7.664E+01    16.00  -7.789E+01    28.83


III.--REACTIONS AT FIXED SUPPORTS
          X-C00RD  X-REACTION  Y-REACTION  MOMENT-REACTION
           (FT)       (K)         (K)          (K-FT)
            5.00   0.000E+00  -2.030E+00        0.000E+00
           16.00   0.000E+00  -3.600E+01        0.000E+00
           37.10   0.000E+00  -1.494E+01        0.000E+00


IV.--FORCES IN LINEAR CONCENTRATED SPRINGS
          NONE


V.--FORCES IN NONLINEAR CONCENTRATED SPRINGS
          NONE
```

PROGRAM CBEAMC - ANALYSIS OF BEAM-COLUMNS WITH NONLINEAR SUPPORTS

DATE: 12-FEBRUARY-2001 TIME: 9:52:52

```
*********************
*  COMPLETE RESULTS  *
*********************
```

I.--HEADING
'BONNEVILLE TIEBACK WALL
'THIRD STAGE EXCAVATION

II.--DISPLACEMENTS AND INTERNAL FORCES

<-------DISPLACEMENTS------>			<----INTERNAL FORCES---->	
X-COORD (FT)	LATERAL (FT)	ROTATION (RAD)	SHEAR (K)	MOMENT (K-FT)
0.00	5.338E-04	-1.092E-04	1.115E-13	-1.188E-14
1.67	3.520E-04	-1.089E-04	8.201E-01	6.593E-01
3.33	1.721E-04	-1.064E-04	1.814E+00	2.830E+00
5.00	0.000E+00	-9.912E-05	2.981E+00	6.801E+00
5.00	0.000E+00	-9.912E-05	9.512E-01	6.801E+00
6.83	-1.698E-04	-8.520E-05	2.435E+00	9.874E+00
8.67	-3.077E-04	-6.358E-05	4.130E+00	1.586E+01
10.50	-3.949E-04	-2.891E-05	6.034E+00	2.514E+01
12.33	-4.021E-04	2.479E-05	8.148E+00	3.811E+01
14.17	-2.883E-04	1.042E-04	1.047E+01	5.515E+01
16.00	0.000E+00	2.165E-04	1.301E+01	7.664E+01
16.00	0.000E+00	2.165E-04	-2.299E+01	7.664E+01
17.83	4.960E-04	3.133E-04	-2.025E+01	3.697E+01
19.67	1.110E-03	3.463E-04	-1.729E+01	2.522E+00
21.50	1.733E-03	3.251E-04	-1.413E+01	-2.631E+01
23.33	2.275E-03	2.595E-04	-1.076E+01	-4.916E+01
25.17	2.664E-03	1.601E-04	-7.173E+00	-6.563E+01
27.00	2.848E-03	3.821E-05	-3.379E+00	-7.533E+01
28.83	2.797E-03	-9.426E-05	6.242E-01	-7.789E+01
30.67	2.504E-03	-2.247E-04	4.838E+00	-7.291E+01
32.50	1.983E-03	-3.399E-04	9.261E+00	-6.002E+01
34.03	1.401E-03	-4.145E-04	1.242E+01	-4.324E+01
35.57	7.252E-04	-4.621E-04	1.431E+01	-2.259E+01
37.10	0.000E+00	-4.784E-04	1.494E+01	5.736E-14

II.--FORCES IN LINEAR DISTRIBUTED SPRINGS
NONE

IV.--FORCES IN NONLINEAR DISTRIBUTED SPRINGS
NONE

3.3.4 Fourth-stage excavation analysis

Fourth-stage excavation is to el 45.0. The equivalent beam with net pressure loading is shown in Figure 3.9. Calculations for the third-stage excavation are performed in the same manner as for the second and third stages. Bending moments are plotted in Figure 3.6b.

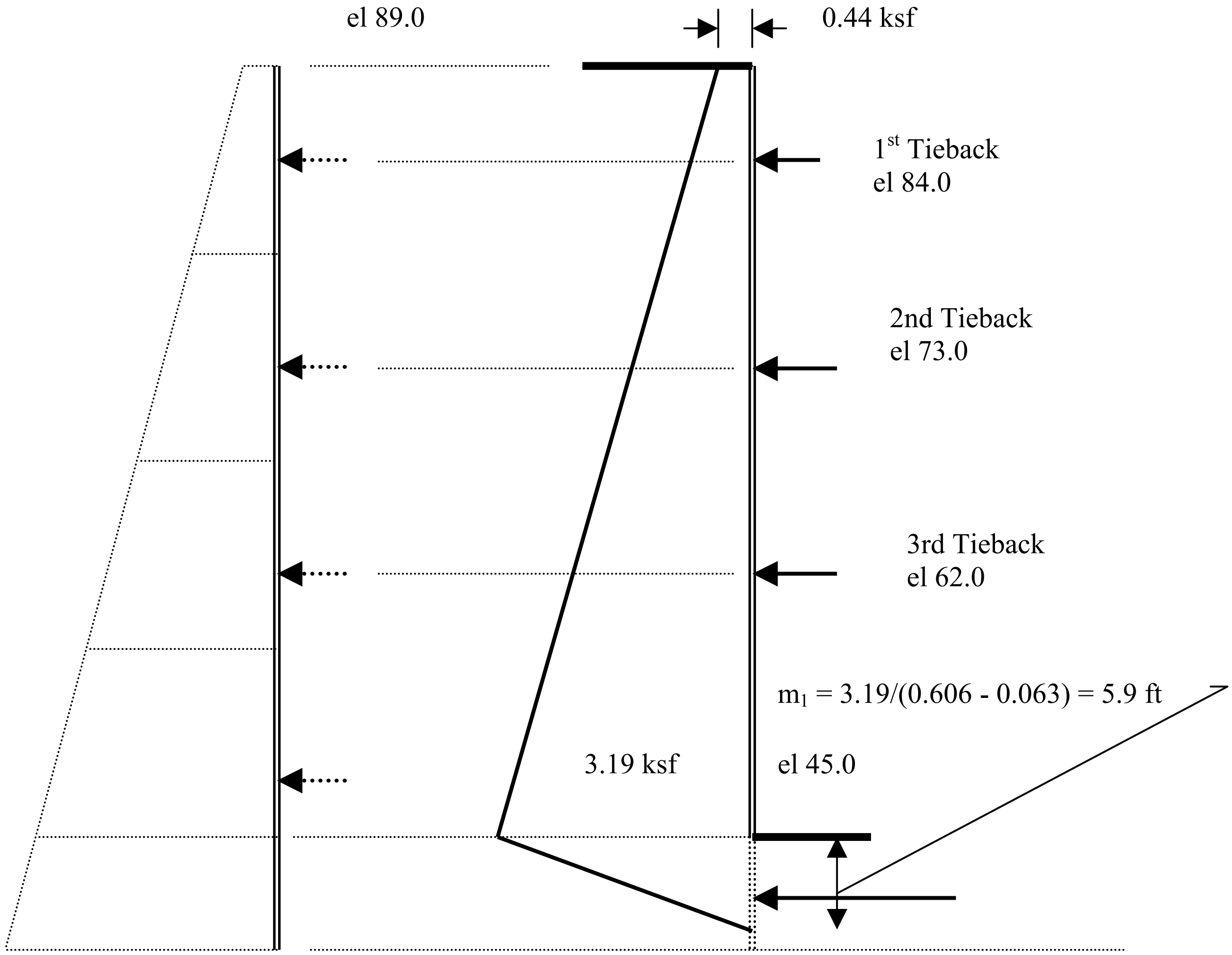


Figure 3.9. Step 10, excavate to el 45.0, fourth-stage excavation

CBEAMC Input for Classical Tieback Wall Analysis
Fourth Stage Excavation Analysis (Equivalent Beam Analysis)
File: BW4

HEADING						
LN	"Heading Description"					
1000	**BONNEVILLE TIEBACK WALL FOURTH STAGE EXCAVATION**					
BEAM HEADER						
LN	"Beam Title"	"New or Add"	Units "**I**nches or **F**eet" "**P**ounds or **K**ips"			
1020	**BEAM**		**F K**			
BEAM DATA LINES						
LN	X1	X2	E	A1	SI1	
1030	**0.0**	**49.9**	**475000.**	**3.00**	**2.25**	
NODE SPACING HEADER						
LN	"NODe"	"New or Add"	Units "**I**nches or **F**eet" "**P**ounds or **K**ips"			
1040	**NODES**		**F**			
NODE SPACING DATA LINES						
LN	X1 "Coord @ Start"	X2 "Coord @ End"	HMAX "Max dist. betw. nodes"			
1050	**0.00**	**49.9**	**3.0**			
LOADS HEADER LINE						
LN	"Loads"	"New or Add"	Units "**I**nches or **F**eet" "**P**ounds or **K**ips"			
1060	**LOADS**		**F K**			
DISTRIBUTED LOADS DATA LINES						
LN	"Distributed"	"Direction"	X1	Q1	X2	Q2
1070	**D**	**Y**	**0.00**	**0.44**	**44.00**	**3.19**
1080	**D**	**Y**	**44.00**	**3.19**	**49.9**	**0.00**
FIXED SUPPORTS HEADER						
LN	"FIXed"	"New or Add"	Units "**I**nches or **F**eet" "**P**ounds or **K**ips"			
1100	**FIXED**		**F**			
FIXED SUPPORTS DATA LINES						
LN	X1 "Coord of support"	XD "Displ. or free"	YD "Displ. or free"	R "Rotation or free"		
1110	**5.00**	**0.0**	**0.0**	**FREE**		
1120	**16.00**	**0.0**	**0.0**	**FREE**		
1130	**27.00**	**0.0**	**0.0**	**FREE**		
1140	**49.9**	**0.0**	**0.0**	**FREE**		
TERMINATION						
LN	" FINish"	"Rerun"	"Keep"			
1150	**FINISH**					

CBEAMC RESULTS File: BW4

```
'BONNEVILLE TIEBACK WALL
'FOURTH STAGE EXCAVATION
BEAM  FT  KSF  FT
   0    49.9   4.750E+05   3    2.25    3    2.25
NODES  FT  FT
   0    49.9    3
LOADS DISTRIBUTED  FT  K/FT
   0    0    0.44    44    0    3.19
   44    0    3.19    49.9    0    0
FIXED  FT  FT
   5   0.000  0.000  FREE
   16   0.000  0.000  FREE
   27   0.000  0.000  FREE
   49.9   0.000  0.000  FREE
FINISHED

  PROGRAM CBEAMC - ANALYSIS OF BEAM-COLUMNS WITH NONLINEAR
                          SUPPORTS

    DATE: 12-FEBRUARY-2001                TIME: 9:54:34

                  ***********************
                  *  SUMMARY OF RESULTS  *
                  ***********************

I.--HEADING
      'BONNEVILLE TIEBACK WALL
      'FOURTH STAGE EXCAVATION


II.--MAXIMA
```

	MAXIMUM POSITIVE	X-COORD (FT)	MAXIMUM NEGATIVE	X-COORD (FT)
AXIAL DISPLACEMENT (FT) :	0.000E+00	0.00	0.000E+00	0.00
LATERAL DISPLACEMENT (FT):	4.978E-03	38.33	-5.099E-04	24.25
ROTATION (RAD) :	5.461E-04	32.67	-7.768E-04	49.90
AXIAL FORCE (K) :	0.000E+00	0.00	0.000E+00	0.00
SHEAR FORCE (K) :	2.313E+01	27.00	-3.450E+01	27.00
BENDING MOMENT (K-FT) :	1.277E+02	27.00	-1.179E+02	41.17

III.--REACTIONS AT FIXED SUPPORTS

X-COORD (FT)	X-REACTION (K)	Y-REACTION (K)	MOMENT-REACTION (K-FT)
5.00	0.000E+00	-1.008E+01	0.000E+00
16.00	0.000E+00	-1.453E+00	0.000E+00
27.00	0.000E+00	-5.763E+01	0.000E+00
49.90	0.000E+00	-2.011E+01	0.000E+00

IV.--FORCES IN LINEAR CONCENTRATED SPRINGS
NONE

V.--FORCES IN NONLINEAR CONCENTRATED SPRINGS

NONE

PROGRAM CBEAMC - ANALYSIS OF BEAM-COLUMNS WITH NONLINEAR SUPPORTS

DATE: 12-FEBRUARY-2001 TIME: 9:54:34

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*********************
*  COMPLETE RESULTS  *
*********************
```

I.--HEADING

'BONNEVILLE TIEBACK WALL
'FOURTH STAGE EXCAVATION

II.--DISPLACEMENTS AND INTERNAL FORCES

	<------DISPLACEMENTS------>		<----INTERNAL FORCES---->	
X-COORD (FT)	LATERAL (FT)	ROTATION (RAD)	SHEAR (K)	MOMENT (K-FT)
0.00	-2.252E-04	4.260E-05	0.000E+00	-2.554E-14
2.50	-1.180E-04	4.376E-05	1.295E+00	1.538E+00
5.00	0.000E+00	5.270E-05	2.981E+00	6.802E+00
5.00	0.000E+00	5.270E-05	-7.095E+00	6.802E+00
7.75	1.477E-04	4.768E-05	-4.790E+00	-9.648E+00
10.50	2.313E-04	9.042E-06	-2.011E+00	-1.911E+01
13.25	1.846E-04	-4.354E-05	1.240E+00	-2.028E+01
16.00	0.000E+00	-8.708E-05	4.963E+00	-1.186E+01
16.00	0.000E+00	-8.708E-05	3.510E+00	-1.186E+01
18.75	-2.667E-04	-1.004E-04	7.706E+00	3.459E+00
21.50	-5.018E-04	-5.882E-05	1.238E+01	3.096E+01
24.25	-5.099E-04	7.055E-05	1.752E+01	7.196E+01
27.00	0.000E+00	3.242E-04	2.313E+01	1.277E+02
27.00	0.000E+00	3.242E-04	-3.450E+01	1.277E+02
29.83	1.281E-03	5.409E-04	-2.822E+01	3.877E+01
32.67	2.865E-03	5.461E-04	-2.144E+01	-3.170E+01
35.50	4.224E-03	3.905E-04	-1.416E+01	-8.225E+01
38.33	4.978E-03	1.288E-04	-6.373E+00	-1.115E+02
41.17	4.909E-03	-1.804E-04	1.912E+00	-1.179E+02
44.00	3.969E-03	-4.749E-04	1.070E+01	-1.001E+02
46.95	2.212E-03	-6.966E-04	1.776E+01	-5.701E+01
49.90	0.000E+00	-7.768E-04	2.011E+01	-9.557E-14

II.--FORCES IN LINEAR DISTRIBUTED SPRINGS

NONE

IV.--FORCES IN NONLINEAR DISTRIBUTED SPRINGS

NONE

3.3.5 Final (fifth-stage) excavation analysis

Final excavation is to el 39.0. The equivalent beam with net pressure loading is shown in Figure 3.10. Calculations for final-stage excavation are similar to those performed in Stages 2 through 4. Bending moments are plotted in Figure 3.6b.

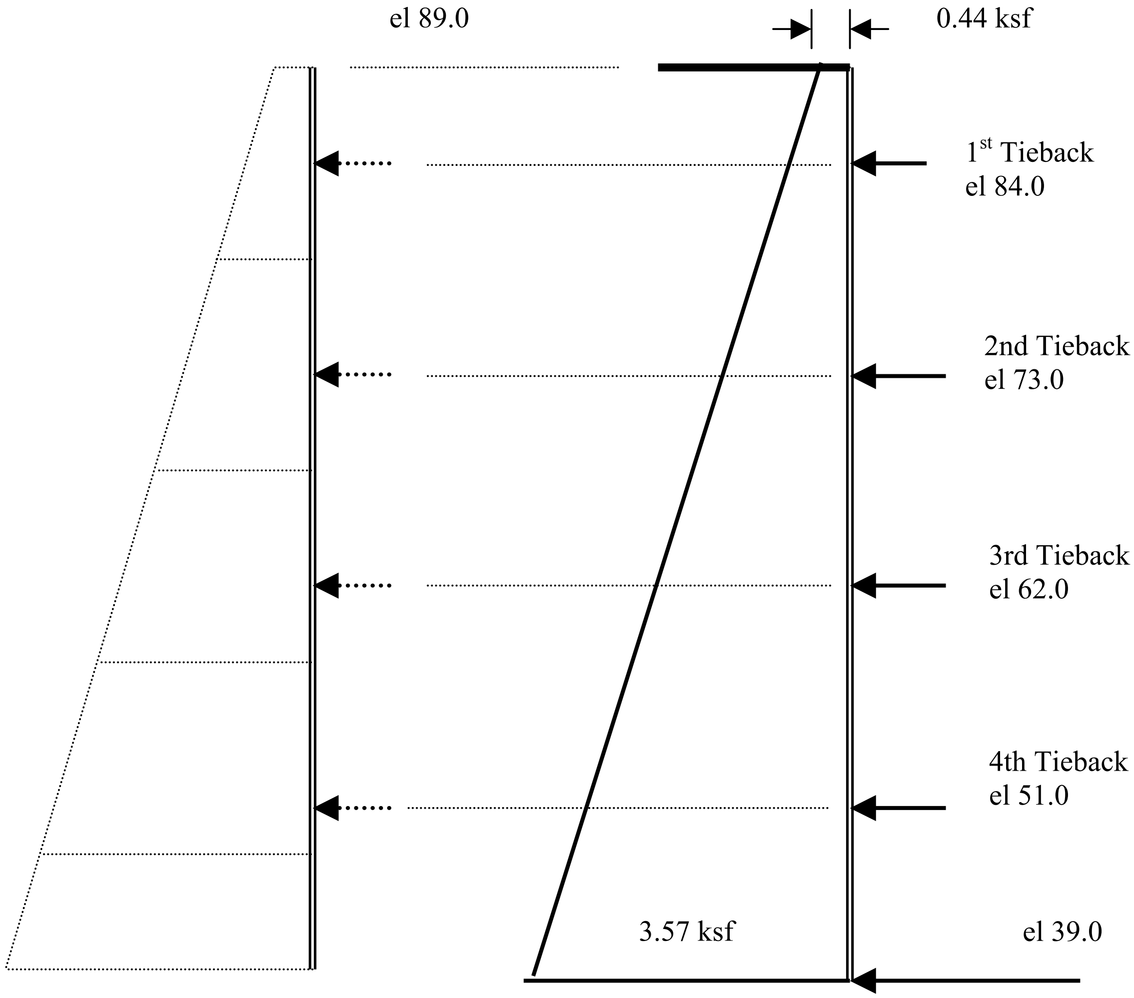

Figure 3.10. Step 12, excavate to el 39.0, final stage

CBEAMC Input for Classical Tieback Wall Analysis
Final Stage Excavation Analysis (Equivalent Beam Analysis)
File: BW5

HEADING	
LN	"Heading Description"
1000	**BONNEVILLE TIEBACK WALL FINAL STAGE EXCAVATION**

BEAM HEADER			
LN	"Beam Title"	"New or Add"	Units "**I**nches or **F**eet" "**P**ounds or **K**ips"
1020	**BEAM**		**F K**

BEAM DATA LINES					
LN	X1	X2	E	A1	SI1
1030	**0.0**	**50.00**	**475000.**	**3.00**	**2.25**

NODE SPACING HEADER			
LN	"NODe"	"New or Add"	Units "**I**nches or **F**eet" "**P**ounds or **K**ips"
1040	**NODES**		**F**

NODE SPACING DATA LINES			
LN	X1 "Coord @ Start"	X2 "Coord @ End"	HMAX "Max dist. betw. nodes"
1050	0.00	50.00	3.0

LOADS HEADER LINE			
LN	"Loads"	"New or Add"	Units "**I**nches or **F**eet" "**P**ounds or **K**ips"
1060	**LOADS**		**F K**

DISTRIBUTED LOADS DATA LINES						
LN	"Distributed"	"Direction"	X1	Q1	X2	Q2
1070	**D**	**Y**	0.00	0.44	50.00	3.57

FIXED SUPPORTS HEADER			
LN	"FIXed"	"New or Add"	Units "**I**nches or **F**eet" "**P**ounds or **K**ips"
1100	**FIXED**		**F**

FIXED SUPPORTS DATA LINES				
LN	X1 "Coord of support"	XD "Displ. or free"	YD "Displ. or free"	R "Rotation or free"
1110	5.00	0.0	0.0	**FREE**
1120	16.00	0.0	0.0	**FREE**
1130	27.00	0.0	0.0	**FREE**
1140	38.00	0.0	0.0	**FREE**
1140	50.00	0.0	0.0	**FREE**

TERMINATION			
LN	" FINish"	"Rerun"	"Keep"
1160	**FINISH**		

CBEAMC RESULTS File: BW5

```
'BONNEVILLE TIEBACK WALL
'FINAL STAGE EXCAVATION
BEAM  FT  KSF  FT
   0    50   4.750E+05   3    2.25    3    2.25
NODES  FT  FT
   0    50    3
LOADS DISTRIBUTED  FT  K/FT
   0    0    0.44    50    0    3.57
FIXED  FT  FT
   5   0.000  0.000  FREE
   16   0.000  0.000  FREE
   27   0.000  0.000  FREE
   38   0.000  0.000  FREE
   50   0.000  0.000  FREE
FINISHED
```

OUTPUT

```
I.--HEADING
      'BONNEVILLE TIEBACK WALL
      'FINAL STAGE EXCAVATION

II.--MAXIMA
                         MAXIMUM   X-COORD   MAXIMUM   X-COORD
                         POSITIVE    (FT)    NEGATIVE    (FT)
AXIAL DISPLACEMENT (FT)  :  0.000E+00    0.00   0.000E+00    0.00
LATERAL DISPLACEMENT (FT):  4.392E-04   44.00  -3.080E-05   35.25
ROTATION (RAD)           :  9.185E-05   41.00  -1.351E-04   50.00
AXIAL FORCE (K)          :  0.000E+00    0.00   0.000E+00    0.00
SHEAR FORCE (K)          :  1.663E+01   38.00  -2.206E+01   38.00
BENDING MOMENT (K-FT)    :  4.368E+01   38.00  -3.566E+01   44.00

III.--REACTIONS AT FIXED SUPPORTS
           X-COORD X-REACTION  Y-REACTION  MOMENT-REACTION
             (FT)     (K)         (K)          (K-FT)
             5.00   0.000E+00  -7.572E+00     0.000E+00
            16.00   0.000E+00  -1.653E+01     0.000E+00
            27.00   0.000E+00  -2.118E+01     0.000E+00
            38.00   0.000E+00  -3.869E+01     0.000E+00
            50.00   0.000E+00  -1.628E+01     0.000E+00
```

```
I.--HEADING
      'BONNEVILLE TIEBACK WALL
      'FINAL STAGE EXCAVATION

      II.--DISPLACEMENTS AND INTERNAL FORCES
    <------DISPLACEMENTS------>   <----INTERNAL FORCES---->
 X-COORD      LATERAL    ROTATION      SHEAR      MOMENT
   (FT)         (FT)       (RAD)         (K)        (K-FT)
    0.00      1.144E-05 -4.738E-06   0.000E+00  0.000E+00
    2.50      3.163E-07 -3.571E-06   1.296E+00  1.538E+00
    5.00      0.000E+00  5.364E-06   2.983E+00  6.804E+00
    5.00      0.000E+00  5.364E-06  -4.589E+00  6.804E+00
    7.75      2.570E-05  9.216E-06  -2.282E+00 -2.753E+00
   10.50      3.604E-05 -2.800E-06   4.989E-01 -5.313E+00
   13.25      1.369E-05 -1.101E-05   3.753E+00  4.252E-01
   16.00      0.000E+00  7.622E-06   7.481E+00  1.576E+01
   16.00      0.000E+00  7.622E-06  -9.046E+00  1.576E+01
   18.75      5.069E-05  2.099E-05  -4.845E+00 -3.446E+00
   21.50      8.419E-05  3.581E-07  -1.706E-01 -1.045E+01
   24.25      5.170E-05 -2.120E-05   4.977E+00 -3.950E+00
   27.00      0.000E+00 -7.268E-06   1.060E+01  1.736E+01
   27.00      0.000E+00 -7.268E-06  -1.059E+01  1.736E+01
   29.75      1.192E-05  6.993E-06  -4.491E+00 -3.480E+00
   32.50      9.483E-06 -1.025E-05   2.077E+00 -6.908E+00
   35.25     -3.080E-05 -1.251E-05   9.119E+00  8.378E+00
   38.00      0.000E+00  5.004E-05   1.663E+01  4.368E+01
   38.00      0.000E+00  5.004E-05  -2.206E+01  4.368E+01
   41.00      2.502E-04  9.185E-05  -1.332E+01 -9.520E+00
   44.00      4.392E-04  2.191E-05  -4.016E+00 -3.566E+01
   47.00      3.481E-04 -8.145E-05   5.849E+00 -3.305E+01
   50.00      0.000E+00 -1.351E-04   1.628E+01 -4.669E-14
```

3.4 Winkler 1 Construction-Sequencing Analysis for Bonneville Navigation Lock Temporary Tieback Wall

A construction-sequencing analysis using beam on elastic foundation techniques (Winkler spring analysis) was performed for the Bonneville Navigation Lock temporary tieback wall. As with Example 1, the analysis was accomplished using the computer program CMULTIANC (Dawkins, Strom, and Ebeling, in preparation). Following each excavation stage of the analysis, the soil load-displacement curves (R-y curves) were shifted to account for active state plastic yielding that occurs in the soil as the wall moves toward the excavation. This R-y curve shifting is necessary to ensure that, as the wall is pulled back into the soil by the upper ground anchor prestress force, soil pressures behind the wall immediately increase above active earth pressure. The soil springs are based on the reference deflection method (FHWA-RD-98-066). According to this document (FHWA-RD-98-066), these reference deflection values do not change with effective overburden pressure. For cohesionless soils, the reference displacement indicating active state first yield is 0.05 in., and the reference deflection indicating passive state first yield is 0.50 in. Ground anchors are represented as springs that are preloaded to produce a lock-off load at a wall deflection consistent with that obtained when the lock-off load is applied as a force. Several analyses are required to perform the construction-sequencing analysis for all five excavation stages. These occur internally within the CMULTIANC program and are summarized in Table 3.3.

Table 3.3
CMULTIANC Steps Used in Construction-Sequencing Analysis

Internal Analysis Step	Description
1	Develop soil springs and wall stiffness from input data and run Winker analysis to determine wall displacements and forces for the first-stage excavation (excavation to el 78.5).
2	Shift R-y curves in locations where displacements exceed active state yielding, i.e., 0.05 in. for cohesionless soils. Rerun first-stage excavation analysis with the shifted R-y curves to verify force results are consistent with Step 1.
3	Apply the Anchor 1 lock-off load (lock-off load provided as input) as a concentrated force and run the Winker analysis to determine the wall displacement at the Anchor 1 location.
4	Replace the anchor lock-off load with a concentrated anchor spring and repeat Steps 1 through 4 for the remaining excavation stages. The concentrated anchor spring is fitted to the lock-off load/wall displacement point obtained from Step 3 using the anchor spring stiffness and yield plateau information supplied as input.

R-y curve development, curve shifting, and anchor spring development for each excavation stage are as demonstrated in Example 1. First-stage excavation to el 78.5 is illustrated in Figure 3.11. The coordinate system for the CMULTIANC construction sequencing analysis is as shown in Figure 3.11.

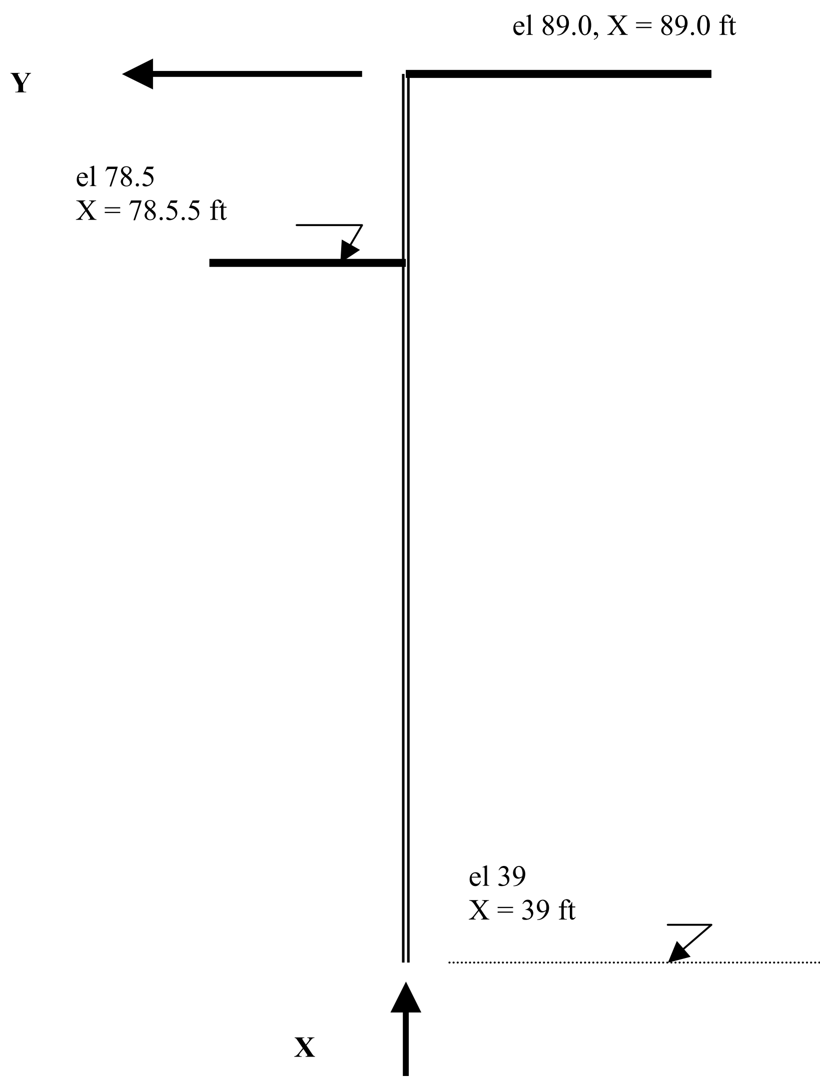

Figure 3.11. Step 4, excavate to el 78.5, first-stage excavation

The distributed soil springs representing the R-y curves are shifted at those locations where displacements from the Step 1 analysis indicate active state yielding has occurred. The analysis is rerun (Step 2) using the shifted R-y curves to ensure that the results are consistent with the Step 1 analysis. In Step 3, the anchor load at lock-off (28.1 kips/ft) is applied as a concentrated force. The wall configuration for Step 3 is as shown in Figure 3.12.

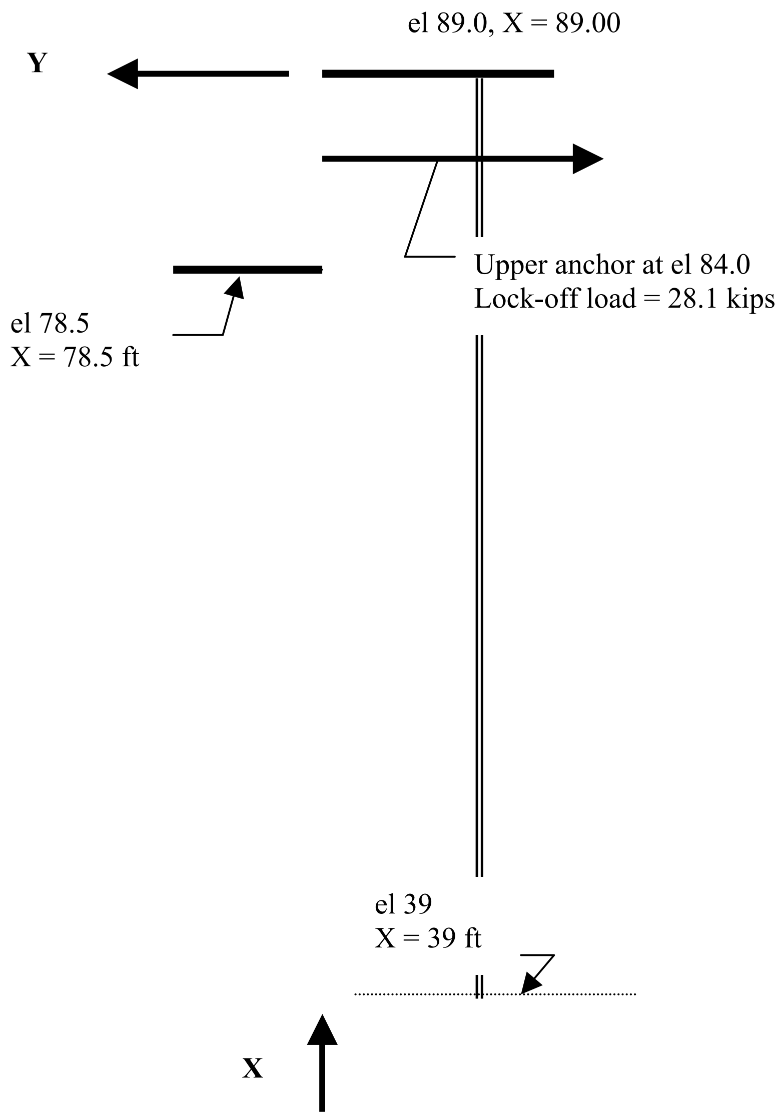

Figure 3.12. Step 3, upper anchor as concentrated force

The displacement from the Step 3 analysis is used to establish the anchor force at a zero wall displacement. With this information, an anchor spring can be developed and inserted into the Step 4 analysis as a replacement for the Step 3 anchor force. This is illustrated in Figure 3.13.

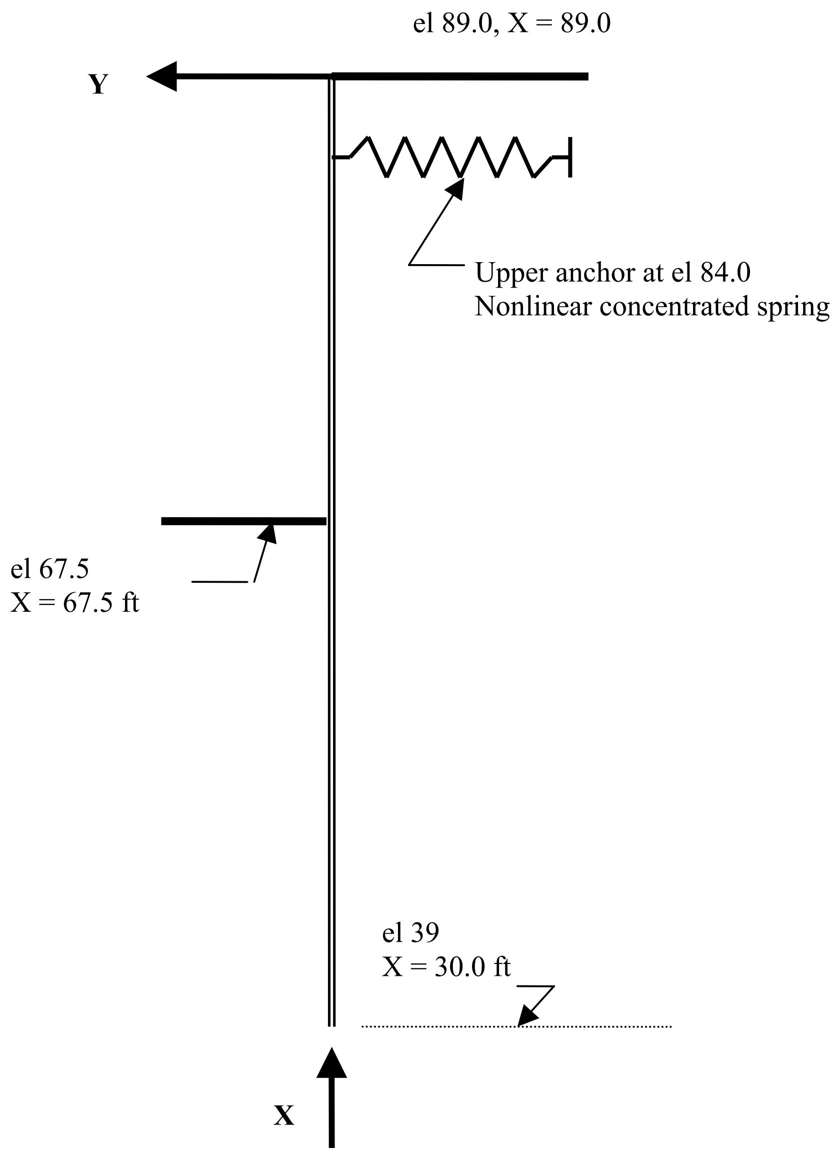

Figure 3.13. Step 4, excavate to el 67.5, anchor as nonlinear concentrated spring

The CMULTIANC program provides results for each of the steps described in Table 3.3. The input for the analysis and the results for each excavation stage are shown on the following pages. These are compared to the results obtained from a nonlinear finite element soil-structure interaction analysis (see Section 3.5).

INPUT

```
'BONNEVILLE TIEBACK WALL
'INPUT FILE: B1  OUTPUT FILE BO1
WALL   89  3.300E+06   46656
WALL   39
ANCHOR   84   28100   34200   4912
ANCHOR   73   28100   34200   4912
ANCHOR   62   28100   34200   4912
ANCHOR   51   28100   34200   4912
SOIL RIGHTSIDE  STRENGTHS   1
 89   125   125     0 30   0   15   .05   .5
SOIL LEFTSIDE  STRENGTHS   1
 78   125   125     0 30   0   15   .05   .5
VERTICAL UNIFORM   875
EXCAVATION DATA
 67
 56
 45
 40
BOTTOM  PINNED
FINISHED
```

OUTPUT

```
  CMULITANC: SIMULATION OF CONSTRUCTION SEQUENCE FOR STIFF
        WALL SYSTEMS WITH MULTIPLE LEVELS OF ANCHORS

     DATE: 28-JULY-2002                  TIME: 19:08:21

      *************************************************
      *        RESULTS FOR INITIAL SSI CURVES         *
      *************************************************

I.--HEADING
       'BONNEVILLE TIEBACK WALL
       'INPUT FILE: B1  OUTPUT FILE BO1

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

II.--MAXIMA
                                 MAXIMUM        MINIMUM
DEFLECTION (FT)            : 4.136E-02      0.000E+00
     AT ELEVATION (FT)     :     89.00          39.00

BENDING MOMENT (LB-FT)     : 8.366E+04     -1.063E+04
     AT ELEVATION (FT)     :     66.00          43.00

SHEAR (LB)                 :   6647.34       -6545.25
     AT ELEVATION (FT)     :     75.00          56.00

RIGHTSIDE SOIL PRESSURE (PSF):   5272.67
     AT ELEVATION (FT)     :     39.00

LEFTSIDE SOIL PRESSURE (PSF) :   3607.62
     AT ELEVATION (FT)     :     39.00
```

III.--ANCHOR FORCES

ELEVATION AT ANCHOR (FT)	ANCHOR STATUS	ANCHOR DEFORMATION (FT)	ANCHOR FORCE (LB)
84.00	INACTIVE		
73.00	INACTIVE		
62.00	INACTIVE		
51.00	INACTIVE		

IV.--COMPLETE RESULTS

ELEV. (FT)	DEFLECTION (FT)	SHEAR FORCE (LB)	BENDING MOMENT (LB-FT)	<-SOIL PRESS. (PSF)-> LEFT	RIGHT
89.00	4.136E-02	0.00	0.00	0.00	291.67
88.00	3.969E-02	312.50	152.78	0.00	333.33
87.00	3.802E-02	666.67	638.89	0.00	375.00
86.00	3.635E-02	1062.50	1500.00	0.00	416.67
85.00	3.468E-02	1500.00	2777.78	0.00	458.33
84.00	3.301E-02	1979.17	4513.89	0.00	500.00
83.00	3.135E-02	2500.00	6750.00	0.00	541.67
82.00	2.969E-02	3062.50	9527.78	0.00	583.33
81.00	2.805E-02	3666.67	12888.89	0.00	625.00
80.00	2.641E-02	4312.50	16875.00	0.00	666.67
79.00	2.479E-02	5000.00	21527.78	0.00	708.33
78.00	2.319E-02	5729.17	26888.89	0.00	750.00
77.00	2.162E-02	6318.65	32937.42	356.30	791.67
76.00	2.008E-02	6612.39	39421.32	674.96	833.33
75.00	1.857E-02	6647.34	46063.58	957.35	875.00
74.00	1.711E-02	6458.98	52623.51	1205.10	916.67
73.00	1.570E-02	6080.97	58895.00	1420.17	958.33
72.00	1.434E-02	5544.90	64704.66	1604.78	1000.00
71.00	1.304E-02	4880.00	69909.54	1761.42	1041.67
70.00	1.181E-02	4112.90	74394.67	1892.76	1083.33
69.00	1.065E-02	3267.50	78070.37	2001.65	1125.00
68.00	9.557E-03	2364.76	80869.42	2091.06	1166.67
67.00	8.543E-03	1422.66	82744.08	2164.01	1208.33
66.00	7.605E-03	456.17	83663.06	2223.50	1250.00
65.00	6.746E-03	-522.75	83608.53	2272.51	1291.67
64.00	5.965E-03	-1505.04	82573.17	2313.88	1333.33
63.00	5.261E-03	-2484.39	80557.25	2350.30	1375.00
62.00	4.632E-03	-3457.09	77566.04	2384.21	1416.67
61.00	4.075E-03	-4402.38	73607.29	2417.79	1497.40
60.00	3.587E-03	-5213.77	68728.14	2452.88	1754.45
59.00	3.164E-03	-5812.97	63150.56	2490.94	1994.44
58.00	2.799E-03	-6220.52	57076.47	2533.06	2217.41
57.00	2.488E-03	-6457.63	50686.74	2579.94	2423.91
56.00	2.224E-03	-6545.25	44140.99	2631.95	2614.88
55.00	2.001E-03	-6503.41	37578.17	2689.09	2791.65
54.00	1.814E-03	-6350.53	31117.90	2751.10	2955.82
53.00	1.655E-03	-6102.88	24862.35	2817.42	3109.29
52.00	1.520E-03	-5774.07	18898.67	2887.27	3254.12
51.00	1.403E-03	-5374.71	13301.84	2959.66	3392.50
50.00	1.298E-03	-4912.08	8137.85	3033.46	3526.73

49.00	1.201E-03	-4390.00	3467.13	3107.43	3659.11
48.00	1.107E-03	-3808.70	-651.91	3180.24	3791.91
47.00	1.013E-03	-3164.88	-4159.28	3250.56	3927.30
46.00	9.144E-04	-2451.81	-6989.91	3317.11	4067.31
45.00	8.099E-04	-1659.62	-9070.34	3378.68	4213.71
44.00	6.970E-04	-775.71	-10315.75	3434.23	4367.95
43.00	5.746E-04	214.72	-10627.43	3482.97	4531.10
42.00	4.424E-04	1327.87	-9890.98	3524.43	4703.68
41.00	3.012E-04	2580.41	-7975.29	3558.57	4885.56
40.00	1.527E-04	3988.27	-4732.62	3585.90	5075.85
39.00	0.000E+00	5565.14	0.00	3607.62	5272.67

```
CMULITANC: SIMULATION OF CONSTRUCTION SEQUENCE FOR STIFF
      WALL SYSTEMS WITH MULTIPLE LEVELS OF ANCHORS

 DATE: 28-JULY-2002                    TIME: 19:08:36

     *************************************************
     *      RESULTS AFTER EXCAVATE TO EL  67       *
     *************************************************


I.--HEADING
       'BONNEVILLE TIEBACK WALL
       'INPUT FILE: B1  OUTPUT FILE BO1


II.--MAXIMA
                                    MAXIMUM        MINIMUM
DEFLECTION (FT)             :   2.167E-02      0.000E+00
     AT ELEVATION (FT)      :       78.00          39.00

BENDING MOMENT (LB-FT)      :   2.781E+04     -6.362E+04
     AT ELEVATION (FT)      :       51.00          71.00

SHEAR (LB)                  :    11080.25      -17140.68
     AT ELEVATION (FT)      :       84.00          84.00

RIGHTSIDE SOIL PRESSURE (PSF):     5272.67
     AT ELEVATION (FT)      :       39.00

LEFTSIDE SOIL PRESSURE (PSF) :     2901.10
     AT ELEVATION (FT)      :       47.00

III.--ANCHOR FORCES

     ELEVATION      ANCHOR      ANCHOR           ANCHOR
     AT ANCHOR      STATUS    DEFORMATION         FORCE
       (FT)                      (FT)              (LB)
        84.00       ACTIVE     2.122E-02        28220.94
        73.00     INACTIVE
        62.00     INACTIVE
        51.00     INACTIVE
```

IV.--COMPLETE RESULTS

ELEV. (FT)	DEFLECTION (FT)	SHEAR FORCE (LB)	BENDING MOMENT (LB-FT)	<-SOIL PRESS. (PSF)-> LEFT	RIGHT
89.00	2.101E-02	0.00	0.00	0.00	2029.36
88.00	2.104E-02	2094.67	1063.00	0.00	2153.05
87.00	2.108E-02	4292.18	4279.04	0.00	2235.05
86.00	2.111E-02	6550.69	9730.14	0.00	2275.03
85.00	2.116E-02	8827.83	17456.26	0.00	2272.26
84.00	2.122E-02	11080.25	27454.65	0.00	2225.57
84.00	2.122E-02	-17140.68	27454.65	0.00	2225.57
83.00	2.131E-02	-14957.39	11457.66	0.00	2133.92
82.00	2.141E-02	-12887.45	-2405.41	0.00	1998.84
81.00	2.150E-02	-10972.97	-14269.64	0.00	1823.02
80.00	2.158E-02	-9253.15	-24310.85	0.00	1609.65
79.00	2.164E-02	-7763.73	-32742.41	0.00	1362.36
78.00	2.167E-02	-6536.60	-39811.61	0.00	1085.28
77.00	2.166E-02	-5595.03	-45795.52	0.00	791.67
76.00	2.161E-02	-4782.53	-50987.77	0.00	833.33
75.00	2.151E-02	-3928.36	-55346.68	0.00	875.00
74.00	2.136E-02	-3032.53	-58830.60	0.00	916.67
73.00	2.115E-02	-2095.03	-61397.85	0.00	958.33
72.00	2.089E-02	-1115.86	-63006.76	0.00	1000.00
71.00	2.057E-02	-95.03	-63615.68	0.00	1041.67
70.00	2.019E-02	967.47	-63182.93	0.00	1083.33
69.00	1.975E-02	2071.64	-61666.84	0.00	1125.00
68.00	1.925E-02	3217.47	-59025.76	0.00	1166.67
67.00	1.870E-02	4404.97	-55218.01	0.00	1208.33
66.00	1.809E-02	5476.29	-50255.36	313.25	1250.00
65.00	1.744E-02	6283.87	-44355.95	610.61	1291.67
64.00	1.675E-02	6844.36	-37775.50	890.59	1333.33
63.00	1.602E-02	7175.78	-30752.30	1151.95	1375.00
62.00	1.527E-02	7297.20	-23506.04	1393.81	1416.67
61.00	1.449E-02	7228.41	-16236.93	1615.61	1458.33
60.00	1.369E-02	6989.59	-9125.10	1817.13	1500.00
59.00	1.289E-02	6601.01	-2330.39	1998.44	1541.67
58.00	1.209E-02	6082.68	4007.54	2159.95	1583.33
57.00	1.129E-02	5454.10	9768.85	2302.28	1625.00
56.00	1.050E-02	4734.05	14852.89	2426.28	1666.67
55.00	9.719E-03	3940.33	19177.32	2532.99	1708.33
54.00	8.959E-03	3089.68	22677.08	2623.55	1750.00
53.00	8.220E-03	2197.65	25303.29	2699.17	1791.67
52.00	7.505E-03	1278.59	27022.01	2761.06	1833.33
51.00	6.815E-03	345.62	27813.00	2810.40	1875.00
50.00	6.151E-03	-589.23	27668.59	2848.27	1916.67
49.00	5.512E-03	-1514.95	26592.57	2875.58	1958.33
48.00	4.898E-03	-2421.34	24599.32	2893.04	2000.00
47.00	4.307E-03	-3298.78	21713.02	2901.10	2041.67
46.00	3.736E-03	-4007.61	17967.30	2899.91	2345.76
45.00	3.182E-03	-4362.86	13667.42	2889.28	2737.43
44.00	2.641E-03	-4308.07	9215.70	2868.71	3134.54
43.00	2.108E-03	-3826.42	5029.81	2837.46	3539.25
42.00	1.581E-03	-2898.13	1545.70	2794.69	3953.79
41.00	1.054E-03	-1500.44	-779.30	2739.55	4380.12
40.00	5.277E-04	391.80	-1463.74	2671.42	4819.61
39.00	0.000E+00	2805.04	0.00	2590.08	5272.67

CMULITANC: SIMULATION OF CONSTRUCTION SEQUENCE FOR STIFF
WALL SYSTEMS WITH MULTIPLE LEVELS OF ANCHORS

DATE: 28-JULY-2002 TIME: 19:08:45

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**********************************************
*      RESULTS AFTER EXCAVATE TO EL  56      *
**********************************************
```

I.--HEADING

'BONNEVILLE TIEBACK WALL
'INPUT FILE: B1 OUTPUT FILE BO1

II.--MAXIMA

		MAXIMUM	MINIMUM
DEFLECTION (FT)	:	2.401E-02	0.000E+00
AT ELEVATION (FT)	:	62.00	39.00
BENDING MOMENT (LB-FT)	:	4.090E+04	-1.217E+05
AT ELEVATION (FT)	:	73.00	58.00
SHEAR (LB)	:	14715.58	-19588.97
AT ELEVATION (FT)	:	84.00	73.00
RIGHTSIDE SOIL PRESSURE (PSF)	:	5272.67	
AT ELEVATION (FT)	:	39.00	
LEFTSIDE SOIL PRESSURE (PSF)	:	2312.93	
AT ELEVATION (FT)	:	45.00	

III.--ANCHOR FORCES

ELEVATION AT ANCHOR (FT)	ANCHOR STATUS	ANCHOR DEFORMATION (FT)	ANCHOR FORCE (LB)
84.00	ACTIVE	1.546E-02	27881.12
73.00	ACTIVE	1.974E-02	28578.37
62.00	INACTIVE		
51.00	INACTIVE		

IV.--COMPLETE RESULTS

ELEV. (FT)	DEFLECTION (FT)	SHEAR FORCE (LB)	BENDING MOMENT (LB-FT)	<-SOIL PRESS. (PSF)-> LEFT	RIGHT
89.00	1.414E-02	0.00	0.00	0.00	2615.90
88.00	1.439E-02	2712.94	1370.23	0.00	2802.17
87.00	1.464E-02	5588.64	5542.64	0.00	2941.42
86.00	1.490E-02	8579.85	12656.46	0.00	3033.16
85.00	1.517E-02	11638.58	22803.44	0.00	3076.43
84.00	1.546E-02	14715.58	36026.84	0.00	3069.61
84.00	1.546E-02	-13165.55	36026.84	0.00	3069.61

83.00	1.578E-02	-10121.21	24438.73	0.00	3011.00
82.00	1.612E-02	-7161.08	15861.62	0.00	2901.13
81.00	1.648E-02	-4335.77	10185.65	0.00	2741.32
80.00	1.685E-02	-1694.54	7251.00	0.00	2533.01
79.00	1.723E-02	714.83	6849.35	0.00	2277.61
78.00	1.761E-02	2845.92	8725.32	0.00	1976.51
77.00	1.800E-02	4653.65	12577.80	0.00	1630.96
76.00	1.840E-02	6094.12	18061.23	0.00	1242.07
75.00	1.882E-02	7156.07	24786.74	0.00	875.00
74.00	1.927E-02	8051.90	32387.25	0.00	916.67
73.00	1.974E-02	8989.40	40904.43	0.00	958.33
73.00	1.974E-02	-19588.97	40904.43	0.00	958.33
72.00	2.025E-02	-18609.80	21801.57	0.00	1000.00
71.00	2.078E-02	-17588.97	3698.71	0.00	1041.67
70.00	2.131E-02	-16526.47	-13362.48	0.00	1083.33
69.00	2.183E-02	-15422.30	-29340.33	0.00	1125.00
68.00	2.232E-02	-14276.47	-44193.19	0.00	1166.67
67.00	2.277E-02	-13088.97	-57879.38	0.00	1208.33
66.00	2.317E-02	-11859.80	-70357.24	0.00	1250.00
65.00	2.350E-02	-10588.97	-81585.10	0.00	1291.67
64.00	2.376E-02	-9276.47	-91521.29	0.00	1333.33
63.00	2.393E-02	-7922.30	-100124.14	0.00	1375.00
62.00	2.401E-02	-6526.47	-107352.00	0.00	1416.67
61.00	2.398E-02	-5088.97	-113163.19	0.00	1458.33
60.00	2.386E-02	-3609.80	-117516.05	0.00	1500.00
59.00	2.362E-02	-2088.97	-120368.91	0.00	1541.67
58.00	2.327E-02	-526.47	-121680.10	0.00	1583.33
57.00	2.280E-02	1077.70	-121407.96	0.00	1625.00
56.00	2.222E-02	2723.53	-119510.81	0.00	1666.67
55.00	2.153E-02	4232.01	-116007.61	355.24	1708.33
54.00	2.074E-02	5436.42	-111151.32	691.02	1750.00
53.00	1.984E-02	6358.14	-105236.04	1003.54	1791.67
52.00	1.884E-02	7022.21	-98532.65	1289.27	1833.33
51.00	1.774E-02	7457.02	-91285.18	1544.99	1875.00
50.00	1.657E-02	7694.05	-83707.72	1767.83	1916.67
49.00	1.531E-02	7767.45	-75981.41	1955.26	1958.33
48.00	1.399E-02	7713.72	-68252.04	2105.13	2000.00
47.00	1.260E-02	7571.35	-60627.79	2215.62	2041.67
46.00	1.115E-02	7380.47	-53177.50	2285.27	2083.33
45.00	9.652E-03	7182.49	-45929.15	2312.93	2125.00
44.00	8.112E-03	7019.85	-38868.71	2297.77	2166.67
43.00	6.536E-03	6935.62	-31939.39	2239.28	2208.33
42.00	4.931E-03	6973.26	-25041.01	2137.23	2250.00
41.00	3.301E-03	7464.93	-18029.86	1991.70	2872.42
40.00	1.655E-03	9021.66	-10138.00	1803.16	4049.29
39.00	0.000E+00	11988.06	0.00	1572.55	5272.67

```
CMULITANC: SIMULATION OF CONSTRUCTION SEQUENCE FOR STIFF
     WALL SYSTEMS WITH MULTIPLE LEVELS OF ANCHORS

  DATE: 28-JULY-2002                  TIME: 19:08:53

    *********************************************
    *      RESULTS AFTER EXCAVATE TO EL  45       *
    *********************************************


I.--HEADING
       'BONNEVILLE TIEBACK WALL
       'INPUT FILE: B1  OUTPUT FILE BO1


II.--MAXIMA

                                   MAXIMUM        MINIMUM
DEFLECTION (FT)              :   1.817E-02      0.000E+00
     AT ELEVATION (FT)       :       89.00          39.00

BENDING MOMENT (LB-FT)       :   6.104E+04     -1.043E+05
     AT ELEVATION (FT)       :       73.00          49.00

SHEAR (LB)                   :    24516.57      -22098.12
     AT ELEVATION (FT)       :       39.00          62.00

RIGHTSIDE SOIL PRESSURE (PSF):     5272.67
     AT ELEVATION (FT)       :       39.00

LEFTSIDE SOIL PRESSURE (PSF) :      555.02
     AT ELEVATION (FT)       :       39.00


III.--ANCHOR FORCES

      ELEVATION      ANCHOR       ANCHOR            ANCHOR
      AT ANCHOR      STATUS     DEFORMATION          FORCE
        (FT)                       (FT)               (LB)
          84.00      ACTIVE      1.556E-02        27887.28
          73.00      ACTIVE      1.092E-02        28058.46
          62.00      ACTIVE      9.936E-03        28318.09
          51.00     INACTIVE


IV.--COMPLETE RESULTS

                                SHEAR       BENDING
ELEV.     DEFLECTION        FORCE         MOMENT   <-SOIL PRESS. (PSF)->
(FT)         (FT)           (LB)         (LB-FT)       LEFT          RIGHT
89.00      1.817E-02          0.00           0.00          0.00      2272.06
88.00      1.764E-02       2381.15        1190.13          0.00      2485.61
87.00      1.710E-02       4961.89        4865.86          0.00      2671.25
86.00      1.658E-02       7714.14       11212.85          0.00      2828.58
85.00      1.606E-02      10609.14       20388.42          0.00      2956.75
84.00      1.556E-02      13617.05       32520.74          0.00      3054.32
83.00      1.509E-02     -11180.74       19820.09          0.00      3119.82
```

82.00	1.464E-02	-8041.34	10239.25	0.00	3154.09
81.00	1.420E-02	-4882.46	3812.51	0.00	3158.77
80.00	1.376E-02	-1732.83	544.54	0.00	3135.62
79.00	1.333E-02	1380.55	412.18	0.00	3086.34
78.00	1.289E-02	4432.32	3366.16	0.00	3012.46
77.00	1.246E-02	7398.47	9332.60	0.00	2915.22
76.00	1.204E-02	10256.07	18214.25	0.00	2795.41
75.00	1.163E-02	12982.64	29891.32	0.00	2653.26
74.00	1.126E-02	15555.58	44221.65	0.00	2488.21
73.00	1.092E-02	17951.25	61040.18	0.00	2298.75
72.00	1.064E-02	-7913.88	52099.01	0.00	2083.54
71.00	1.041E-02	-5947.17	45241.37	0.00	1845.54
70.00	1.022E-02	-4227.90	40229.28	0.00	1588.75
69.00	1.006E-02	-2773.01	36805.93	0.00	1316.93
68.00	9.947E-03	-1530.02	34699.52	0.00	1166.67
67.00	9.862E-03	-342.52	33759.77	0.00	1208.33
66.00	9.809E-03	886.64	34028.36	0.00	1250.00
65.00	9.787E-03	2157.48	35546.95	0.00	1291.67
64.00	9.799E-03	3469.98	38357.21	0.00	1333.33
63.00	9.848E-03	4824.14	42500.79	0.00	1375.00
62.00	9.936E-03	6219.98	48019.38	0.00	1416.67
62.00	9.936E-03	-22098.12	48019.38	0.00	1416.67
61.00	1.006E-02	-20660.62	26636.54	0.00	1458.33
60.00	1.022E-02	-19181.45	6712.04	0.00	1500.00
59.00	1.038E-02	-17660.62	-11712.47	0.00	1541.67
58.00	1.053E-02	-16098.12	-28595.31	0.00	1583.33
57.00	1.065E-02	-14493.95	-43894.81	0.00	1625.00
56.00	1.074E-02	-12848.12	-57569.32	0.00	1666.67
55.00	1.076E-02	-11160.62	-69577.16	0.00	1708.33
54.00	1.073E-02	-9431.45	-79876.67	0.00	1750.00
53.00	1.062E-02	-7660.62	-88426.17	0.00	1791.67
52.00	1.043E-02	-5848.12	-95184.01	0.00	1833.33
51.00	1.014E-02	-3993.95	-100108.52	0.00	1875.00
50.00	9.770E-03	-2098.12	-103158.02	0.00	1916.67
49.00	9.299E-03	-160.62	-104290.86	0.00	1958.33
48.00	8.731E-03	1818.55	-103465.37	0.00	2000.00
47.00	8.066E-03	3839.38	-100639.87	0.00	2041.67
46.00	7.308E-03	5901.88	-95772.71	0.00	2083.33
45.00	6.460E-03	8006.05	-88822.22	0.00	2125.00
44.00	5.530E-03	10070.01	-79775.28	159.97	2166.67
43.00	4.525E-03	12027.77	-68721.64	295.42	2208.33
42.00	3.456E-03	14137.61	-55755.09	404.02	2718.03
41.00	2.336E-03	16808.17	-40474.53	484.01	3520.24
40.00	1.178E-03	20242.06	-22157.74	534.38	4375.33
39.00	0.000E+00	24516.57	0.00	555.02	5272.67

```
**********************************************
*      RESULTS AFTER EXCAVATE TO EL  40      *
**********************************************


I.--HEADING
       'BONNEVILLE TIEBACK WALL
       'INPUT FILE: B1  OUTPUT FILE BO1
```

II.--MAXIMA

		MAXIMUM	MINIMUM
DEFLECTION (FT)	:	2.045E-02	0.000E+00
AT ELEVATION (FT)	:	89.00	39.00
BENDING MOMENT (LB-FT)	:	6.747E+04	-5.565E+04
AT ELEVATION (FT)	:	62.00	44.00
SHEAR (LB)		22760.16	-20296.37
AT ELEVATION (FT)	:	39.00	51.00
RIGHTSIDE SOIL PRESSURE (PSF)	:	5272.67	
AT ELEVATION (FT)	:	39.00	
LEFTSIDE SOIL PRESSURE (PSF)	:	92.50	
AT ELEVATION (FT)	:	39.00	

II.--ANCHOR FORCES

ELEVATION AT ANCHOR (FT)	ANCHOR STATUS	ANCHOR DEFORMATION (FT)	ANCHOR FORCE (LB)
84.00	ACTIVE	1.680E-02	27960.22
73.00	ACTIVE	9.395E-03	27968.45
62.00	ACTIVE	4.157E-03	27977.43
51.00	ACTIVE	2.785E-03	28102.44

IV.--COMPLETE RESULTS

ELEV. (FT)	DEFLECTION (FT)	SHEAR FORCE (LB)	BENDING MOMENT (LB-FT)	<-SOIL PRESS. LEFT	(PSF)-> RIGHT
89.00	2.045E-02	0.00	0.00	0.00	2077.27
88.00	1.971E-02	2182.16	1088.10	0.00	2283.26
87.00	1.897E-02	4558.90	4459.45	0.00	2466.43
86.00	1.823E-02	7107.22	10297.24	0.00	2626.41
85.00	1.751E-02	9803.55	18761.44	0.00	2762.41
84.00	1.680E-02	12623.28	29988.05	0.00	2873.16
83.00	1.612E-02	-12419.69	16127.60	0.00	2957.36
82.00	1.545E-02	-9430.94	5224.50	0.00	3016.11
81.00	1.479E-02	-6395.17	-2662.48	0.00	3051.43
80.00	1.412E-02	-3334.74	-7498.03	0.00	3065.50
79.00	1.345E-02	-269.79	-9268.09	0.00	3060.54
78.00	1.277E-02	2781.71	-7977.60	0.00	3038.71
77.00	1.209E-02	5803.83	-3648.40	0.00	3001.93
76.00	1.140E-02	8782.41	3682.73	0.00	2951.75
75.00	1.071E-02	11704.56	13965.61	0.00	2889.21
74.00	1.004E-02	14558.07	27137.71	0.00	2814.59
73.00	9.395E-03	17330.55	43124.40	0.00	2727.24
72.00	8.785E-03	-7959.46	33869.88	0.00	2626.61
71.00	8.208E-03	-5386.51	27241.96	0.00	2516.36
70.00	7.656E-03	-2926.43	23130.40	0.00	2401.03
69.00	7.127E-03	-582.27	21419.88	0.00	2284.72
68.00	6.618E-03	1646.71	21994.07	0.00	2170.88
67.00	6.129E-03	3764.31	24739.14	0.00	2062.18

66.00	5.665E-03	5776.52	29546.39	0.00	1960.32
65.00	5.228E-03	7690.49	36313.97	0.00	1865.90
64.00	4.825E-03	9513.35	44947.44	0.00	1778.27
63.00	4.465E-03	11250.89	55359.19	0.00	1695.40
62.00	4.157E-03	-15071.33	67466.34	0.00	1613.74
61.00	3.907E-03	-13479.50	53209.79	0.00	1569.04
60.00	3.708E-03	-11844.70	40522.28	0.00	1701.38
59.00	3.547E-03	-10083.66	29536.15	0.00	1821.34
58.00	3.414E-03	-8207.12	20371.36	0.00	1932.30
57.00	3.300E-03	-6222.55	13138.87	0.00	2037.29
56.00	3.199E-03	-4134.66	7943.67	0.00	2138.91
55.00	3.106E-03	-1945.83	4887.38	0.00	2239.12
54.00	3.017E-03	343.09	4070.21	0.00	2339.08
53.00	2.933E-03	2731.97	5592.12	0.00	2439.02
52.00	2.854E-03	5220.32	9553.05	0.00	2538.00
51.00	2.785E-03	7806.07	16051.98	0.00	2633.78
51.00	2.785E-03	-20296.37	16051.98	0.00	2633.78
50.00	2.726E-03	-17617.07	-2917.74	0.00	2725.06
49.00	2.666E-03	-14845.13	-19162.41	0.00	2819.08
48.00	2.587E-03	-11973.34	-32588.00	0.00	2924.82
47.00	2.479E-03	-8985.95	-43088.78	0.00	3050.41
46.00	2.331E-03	-5859.57	-50539.14	0.00	3202.94
45.00	2.137E-03	-2564.41	-54786.57	0.00	3388.18
44.00	1.891E-03	934.35	-55645.81	0.00	3610.35
43.00	1.594E-03	4674.82	-52894.72	0.00	3871.79
42.00	1.248E-03	8696.35	-46271.83	0.00	4172.68
41.00	8.599E-04	13037.21	-35476.26	0.00	4510.61
40.00	4.390E-04	17731.76	-20170.08	0.00	4880.21
39.00	0.000E+00	22760.16	0.00	92.50	5272.67

3.5 Winkler 1–FEM Study Comparisons

Wall moments from the Winkler 1 analysis for each excavation stage are shown in Figure 3.14. These are compared to wall moments obtained from a nonlinear finite element method (NLFEM) analysis. The results from the NLFEM analysis are reported in Mosher and Knowles (1990). Flexural behavior based on the Winkler 1 analysis differs somewhat from that determined in the NLFEM study, especially for the final two excavation stages. However, the Winkler 1 and NLFEM studies indicate that the wall moments for the various excavation stages are similar in magnitude.

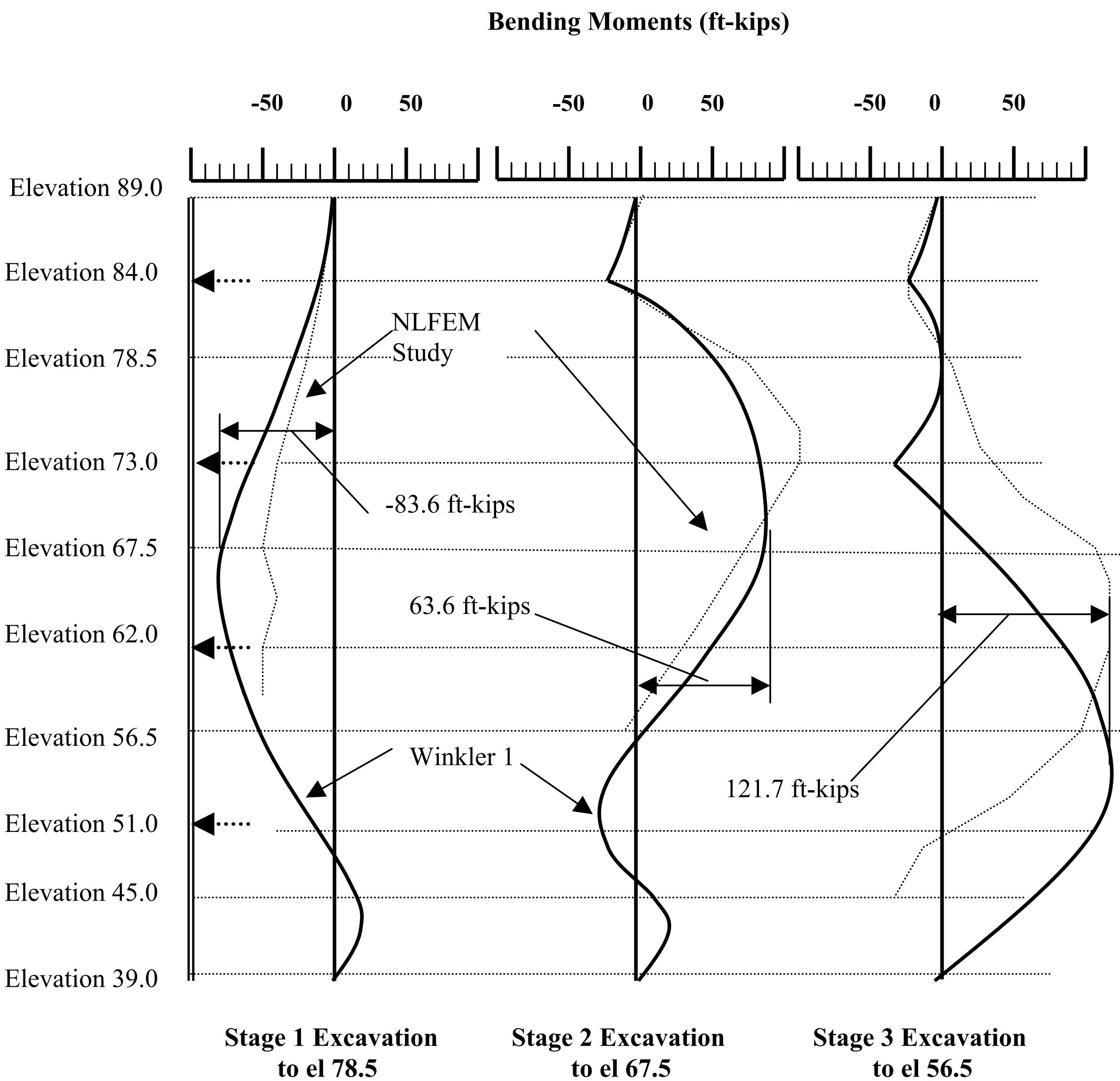

Figure 3.14. Wall moments (Continued)

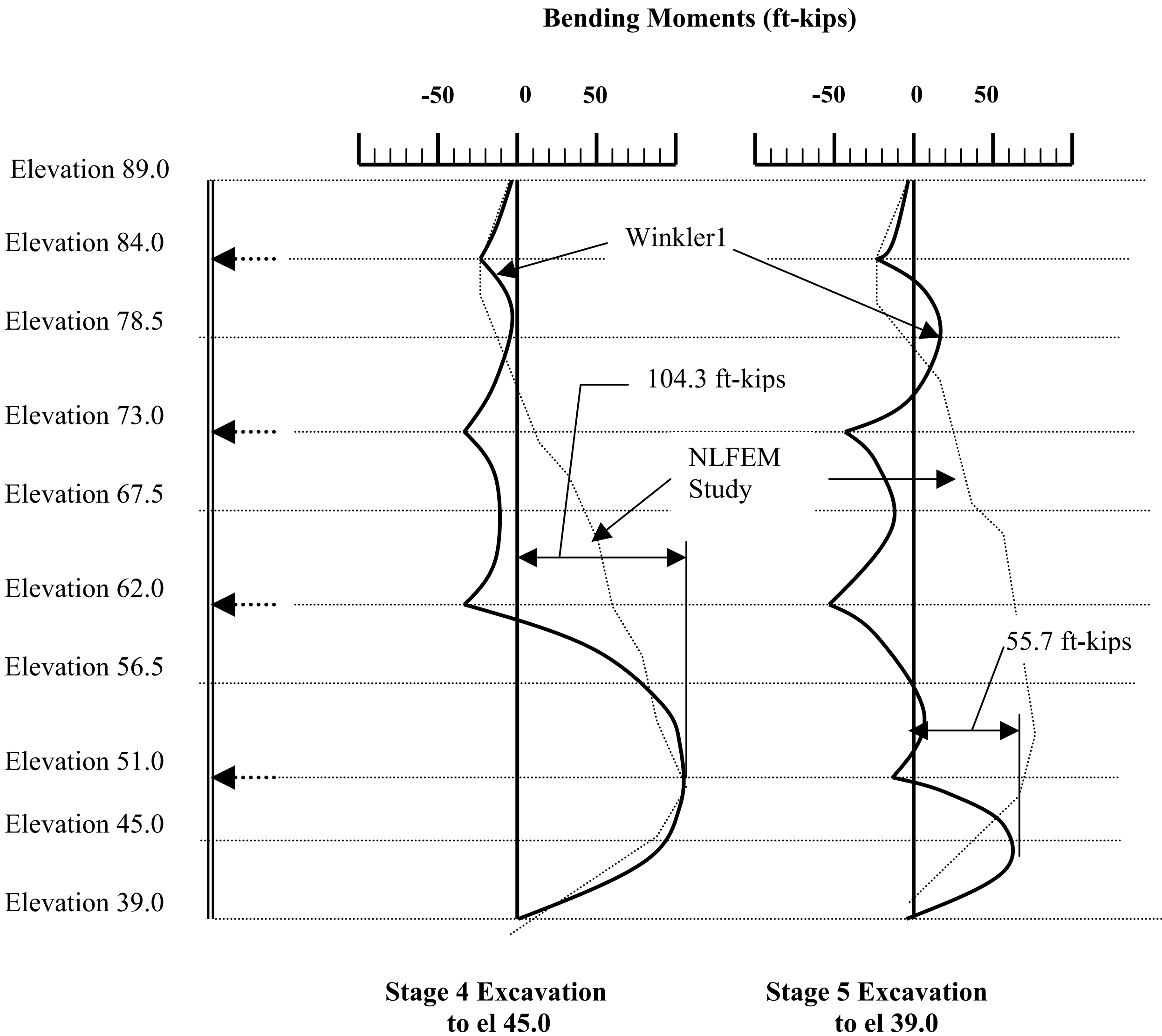

Figure 3.14. (Concluded)

Earth pressures for the final excavation stage are shown in Figure 3.15. Note that earth pressures computed by the soil-structure interaction analyses at the upper anchor location are significantly above at-rest earth pressures after the first anchor is installed and prestressed. The pressures at the upper anchor location remain at high levels throughout construction and after wall completion. At final excavation, the earth pressures in the lower half of the wall are nearer to at-rest conditions.

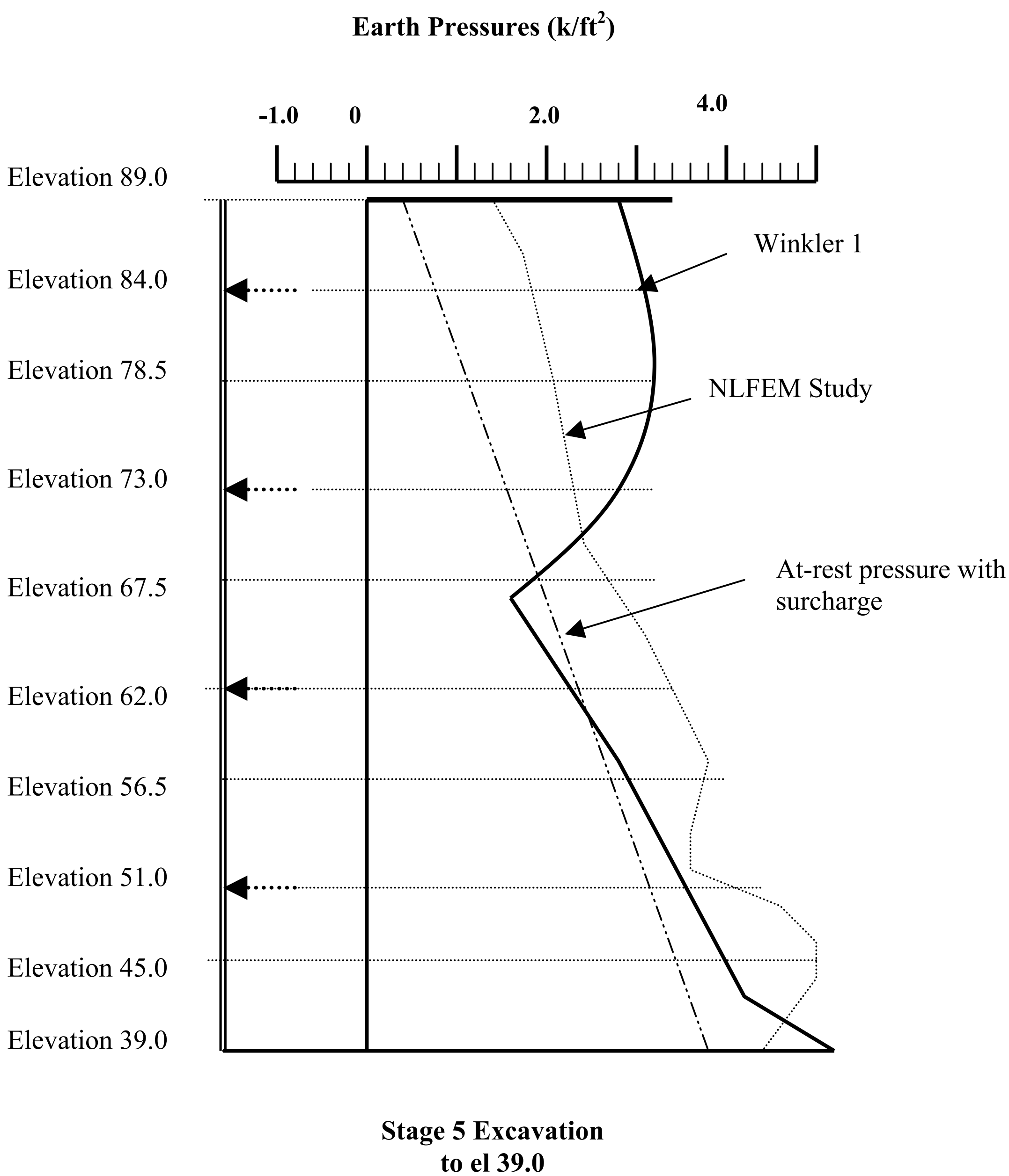

Figure 3.15. Earth pressures-final excavation

Computed wall displacements from the Winkler 1 analyses for each stage of excavation are shown in Figure 3.16. These are compared with wall displacements obtained from the NLFEM analyses. The NLFEM analyses indicate that, upon installation and prestressing of the upper anchor, the wall is pulled into the retained soil (positive displacement). Displacements remain positive throughout the other stages of construction. The Winkler analyses indicate the wall will tend to displace toward the excavation, although the displacements may be small. Displacement for the final excavation stage measured in the field indicate the wall displacement is somewhere between that predicted by the NLFEM study and that computed by the Winkler 1 analysis (see Figure 3.16b).

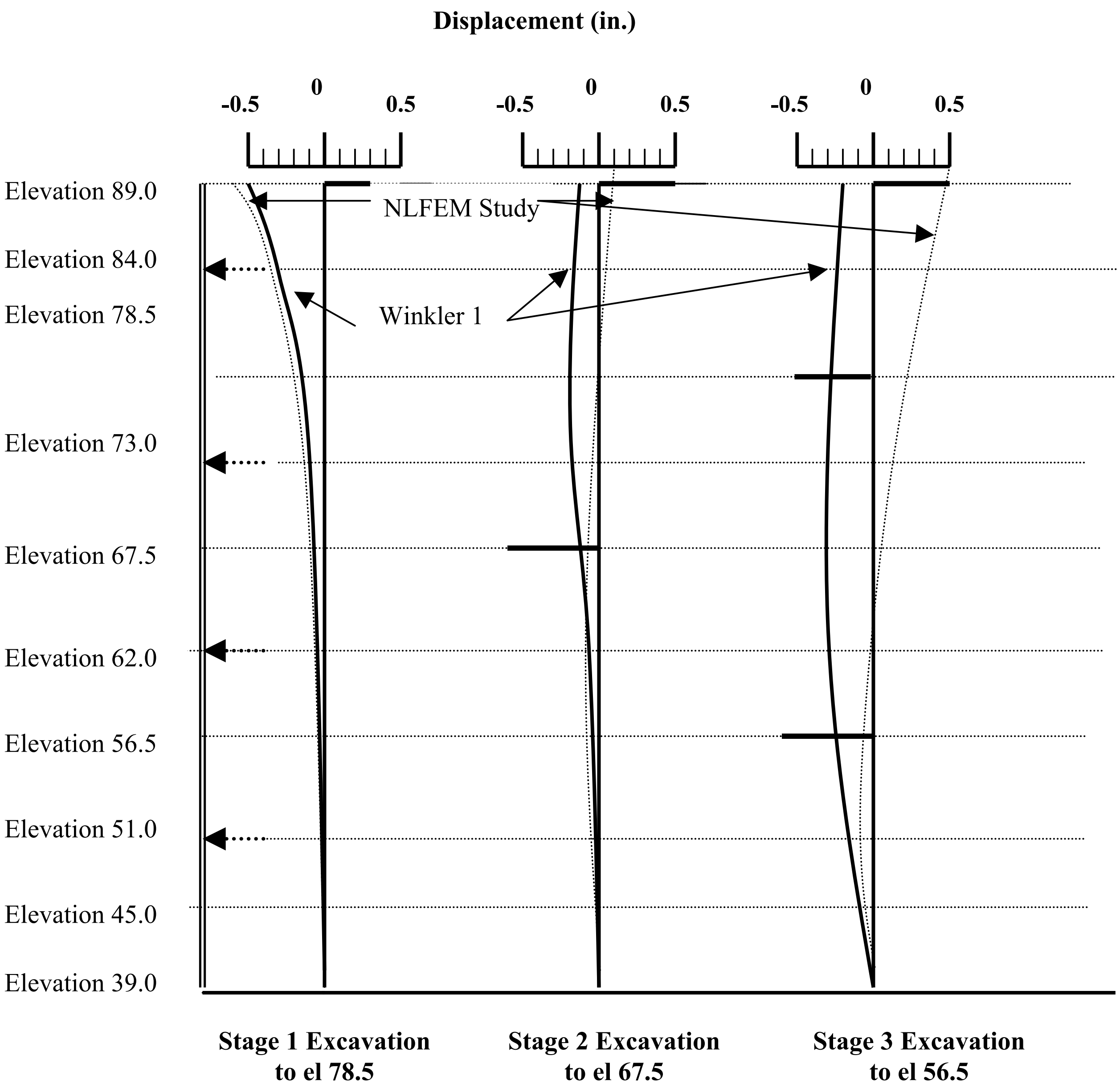

Figure 3.16. Deflections (all stages) (Continued)

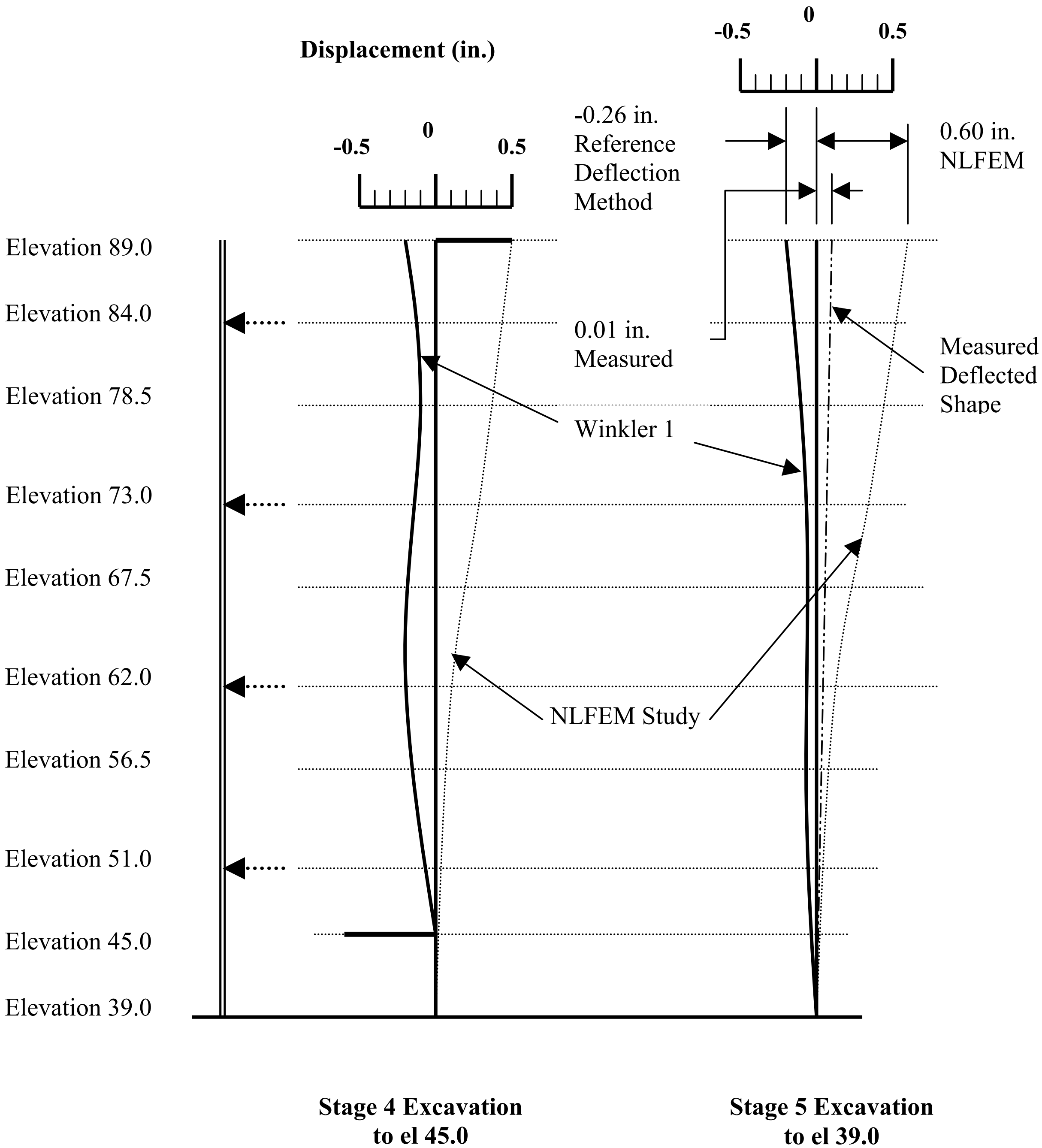

Figure 3.16. (Concluded)

3.6 Discussion of Results–Example 2

Various methods were demonstrated for potential application with respect to the design and evaluation of a ***stiff*** tieback wall with a single anchor. Except for the apparent pressure method (i.e., Rigid 1 analysis), all the methods used involve a construction-sequencing type analysis. Soil arching tends to develop both horizontally and vertically with flexible wall systems, resulting in earth pressure concentration at tieback locations. This reduces the moment and shear demands on the wall system. The Rigid 1 approach, since it is based on the measured response of systems, produces a reliable design for flexible wall systems. With stiff wall systems, soil arching is less pronounced, and soil pressures at the facing tend to be more uniform with little tendency to concentrate at tieback locations. The design of stiff wall systems, therefore, should more closely follow classical earth pressure theory, and should consider construction-sequencing effects (Note: construction-sequencing effects are inherently included in the apparent pressure method.) The Rigid 2 and Winkler 1 analysis methods, which were demonstrated with respect to Example 2, are methods that are available to evaluate the performance of stiff tieback wall systems. Until additional research is conducted to evaluate the validity of these construction-sequencing methods, the designer is required to determine which method is most suitable and applicable to his or her particular wall system and site conditions.

The stiff wall design with high anchor prestress and close anchor spacing was chosen for this particular wall system because of stringent displacement controls placed on this project to ensure that settlement and lateral movement did not occur in the retained soil. Measured wall deflections indicated the wall was pulled back into the ground as the tieback anchors were stressed. This action was also captured by the nonlinear finite element soil-structure interaction (NLFEM) analysis. The Winkler 1 analysis indicated a very slight displacement of the wall toward the excavation. Although the Winkler 1 analysis did a reasonable job of predicting wall displacement, this (as indicated by FHWA-RD-98-066) may have occurred because of the large anchor loads, which prevented appreciable plastic soil movement from occurring. Also, although the Winkler 1 analysis provided computed displacements that are in reasonable agreement with measured displacements, it must be remembered that, until further research is conducted, the authors of this report do not recommend that this type of analysis be used to predict wall displacements.

In the introduction to this report it was pointed out that many designers feel that the apparent pressure diagram approach used for flexible tieback wall systems is ill advised for use in the design of stiff tieback wall systems (Kerr and Tamaro 1990). These investigators have also indicated that the apparent pressure approach for stiff wall systems will underpredict loads in the lower tiebacks and underpredict negative moments at the tieback anchor locations. It can be seen that this is true for this particular multiple-tieback anchor example by comparing the final excavation stage results for the apparent pressure method (i.e., Rigid 1 analysis) with the final excavation stage results from the Rigid 2 and Winkler 1 analyses. However, for the upper anchors, the anchor forces determined by the Rigid 1 analysis are higher that those determined by the Rigid 2 analysis method.

The apparent pressure diagram approach (i.e., Rigid 1 analysis) may be needed for stiff wall systems to ensure that the upper anchor design loads, as determined by construction-sequencing analyses, are adequate to meet “safety with economy” and “stringent displacement control” performance objectives.

Until additional research is conducted to evaluate the validity of these construction-sequencing methods, the designer is required to determine which method is most suitable and applicable to his or her particular wall system and site conditions.

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Appendix A
Drained Shear Strength Parameters for Stiff Clay Sites

A.1 Introduction

Although permanent ground anchor walls are seldom constructed in normally consolidated clay deposits, they are routinely built in overconsolidated clays. The apparent earth pressure design approach for tieback walls constructed at stiff clay sites for undrained (short-term) and drained (long-term) conditions is described in FHWA-RD-97-130 and in Strom and Ebeling (2001). The development of R-y curves for stiff clay sites by the reference deflection method is described in FHWA-RD-98-066. This appendix is intended to present information required to develop the drained shear strength parameters (i.e., drained friction angle) for overconsolidated clays, since the drained friction angle for a normally consolidated clay and intact overconsolidated clay are not the same. This information is taken from FHWA-RD-97-130 and is presented to facilitate the development of earth pressures and R-y curves for use in the construction-sequencing analyses illustrated in the main text of this report. Terms used below in describing and developing drained shear strength parameters for stiff clay sites are as follows:

s = drained shear strength

σ' = effective normal stress

ϕ' = drained friction angle

c = cohesion intercept

OCR = overconsolidation ratio

m = factor defining the extent of fissures in the soil

A.2 Drained Shear Strength of Overconsolidated Clay (FHWA-RD-97-130)

The drained strength of a normally consolidated cohesive soil depends on the drained friction angle (ϕ') and the effective normal stress (σ') and is expressed by the relationship

$$s = \sigma' \tan \phi' \tag{A.1}$$

The effective normal stress (σ') on the shear plane is the total normal stress on the plane less the pore-water pressure after equilibrium is reached. Friction angle (ϕ') depends on the clay content of the soil, clay mineralogy, and arrangement of clay particles. Figure A.1 (from Terzaghi, Peck, and Mesri 1996) shows how ϕ' varies with the plasticity index for normally consolidated clays.

Data points far above the line represent soils that have an effective normal stress less than 1,000 psf and a clay content less than 20 percent, and data points well below the line represent soils having effective normal stresses greater than 8,350 psf and clay contents greater than 50 percent.

The drained shear strength of overconsolidated clay should be greater than the drained shear strength of a similar soil in a normally consolidated state. The drained shear strength of saturated overconsolidated clay is called the intact shear strength, and is defined with respect to the cohesion intercept (c') and the friction angle (ϕ') of a Mohr failure envelope by Equation A.2.

$$s = c' + \sigma' \tan \phi' \tag{A.2}$$

Friction angles for the intact overconsolidated clay are higher at effective stresses lower than the preconsolidation pressure, and trend toward the normally consolidated friction angle at high effective normal stresses. Terzaghi, Peck, and Mesri (1996) used Equation A.3 to express the drained strength of overconsolidated clay in terms of the drained strength of the same soil in its normally consolidated state, the overconsolidation ratio (OCR), and term *m*, which depends on the fissures in the soil.

$$s = \sigma' \tan \phi' OCR^{1-m} \tag{A.3}$$

The preconsolidation pressure used to determine the OCR in Equation A.2 is the effective normal stress where the Mohr diagram failure envelope for the overconsolidated clay joins the failure envelope for the normally consolidated clay. The exponent *m* for clays and shales is given in Table A-1.

Table A.1 Values of *m* in Equation A.3		
	m	
Soil Description	**Intact Soil**	**Destructured Soil**
Stiff clays and shales	0.5 – 0.6	0.6 – 0.8
Soft clays	0.6 – 0.7	0.6 – 0.9
Source: Terzaghi, Peck, and Mesri (1996).		

Terzaghi, Peck, and Mesri (1996) defined intact soils as soils that are undisturbed and unfissured, and destructured soils as slightly fissured stiff clays and shales and soft clays sheared to a large-strain condition. Destructured soils are stronger than fully strained softened stiff clays or shales or completely remolded soft clays. Fully strained softened or remolded clays will have an *m* of 1 (approximately), and their drained shear strength will approximately equal the normally consolidated shear strength.

Drained shear strength of heavily overconsolidated clay depends upon the condition of the clay after unloading and swelling. A badly fissured and jointed clay's drained shear strength may be reduced to its fully softened shear strength (strength in its normally consolidated state). If large displacements have occurred within heavily overconsolidated stiff clay in the geologic past, the drained friction angle may be reduced to a residual value along planes where the displacements occurred. These planes must be continuous for a considerable distance for the shear strength to be reduced to a residual value. The residual friction angle is equal to or lower than the drained friction angle of a normally consolidated clay (fully strain softened). When the displacements occur, the clay particles are reoriented parallel to the direction of shearing. The magnitude of the friction angle reduction depends upon the clay content and the shape of the clay particles. The residual friction angle will be low for soils that have a high percentage of plate-shaped clay minerals. For an anchored wall, residual shear strength is mobilized only when displacements occur along pre-existing shear surfaces. These surfaces have to be oriented in a direction that will affect the stability of the anchored wall, or the behavior of the wall will not be dependent upon the residual shear strength of the soil. Figure A.2 (from Patton and Henderson 1974) gives drained residual friction angles for rock gouge material as a function of plasticity index.

Terzaghi, Peck, and Mesri (1996) present the residual friction angle as a function of the friction angle of normally consolidated clays (Figure A.3).

Both figures illustrate the strength reduction that can occur when a stiff, heavily overconsolidated clay is sheared, reducing the strength to a residual value.

Figure A.4 combines previously described relationships and serves as a guide for estimating the drained friction angle for fine-grained soils in different states of stress or disturbance. The line representing the normally consolidated state is the trend line from Figure A.1. Lines representing the overconsolidated soils were determined by setting Equation A.1 equal to Equation A.3 and solving for ϕ' in Equation A.1. Values selected for *m* in Equation A.3 are presented in Figure A.4. Curves representing intact and destructured soils were drawn for clays with an OCR of 2. The range for the residual friction angles was developed from Figures A.2 and A.3.

It should be noted that the short-term (undrained) apparent earth pressures could be greater than the pressures computed using the drained shear strength parameters.

Atterberg limits for the clay, the OCR, the extent of fissuring, and the nature and orientation of joints or shears are needed to use Figure A.4 for estimating the drained friction angle. After estimating the drained friction angle, one should determine the earth pressures associated with the drained condition and the pore-water pressures, and compare them with the earth pressures associated with the undrained shear strength. The pressures that give the greatest demands with respect to the tieback wall structural component of interest should be used for design of that component. Demands associated with the undrained earth pressure condition may be greater than those associated with drained earth pressures plus water pressure. When the wall is going to be built in a heavily overconsolidated deposit, local experience should guide in determining the degree of disturbance and the soil strength. Laboratory tests can be used to determine drained shear strength parameters, but tests done on samples recovered from the deposit may not accurately represent the strength of a fissured soil. In addition to testing, local experience, and understanding of the geologic events that have affected soils at the site, the relationships in Figure A.3 should be considered when estimating the drained friction angle.

Stress relief in heavily overconsolidated fine-grained soils may result in a strength reduction. How this reduction affects anchored walls is not clear. Sills, Burland, and Czechowski (1977) reported that stress relief in a 26-ft-deep excavation in London clay resulted in deep-seated movements behind ground anchors that were twice the height of the wall but no increase in anchor load. If there is a concern that wall movements will cause stress relief in the ground, the measured drained strength can be reduced. If stress relief occurs, the strengths will likely be greater than the normally consolidated drained shear strength (see Figure A.3). Drained shear strengths should not be reduced below the normally consolidated strengths unless deposit has been sheared in the geologic past and the discontinuities are oriented in a direction that affects the stability of the wall.

Poor drilling techniques using air or water to clean the drill hole may fracture the soil and reduce the soil's shear strength or pressurize the drilling fluid in open fractures. The strength reduction or the effect of pressurizing the drilling fluid is not considered in the design. Fracturing the ground is controlled by preventing collaring of the hole when drilling with air or water. A collar occurs when the hole becomes blocked and cuttings no longer return up the drill hole to the surface. If a collar occurs, the pressurized drilling fluid (air or water) is forced into the ground, disrupting the formation. Auger drilling methods will not disrupt the soil where collaring is likely.

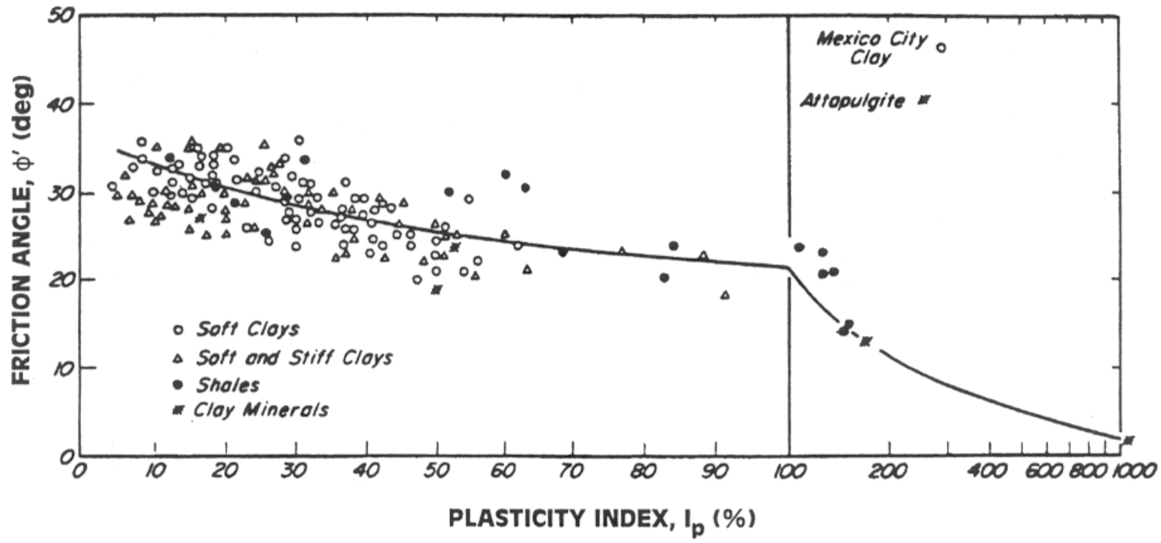

Figure A.1. Undrained friction angle ϕ' for normally consolidated clays

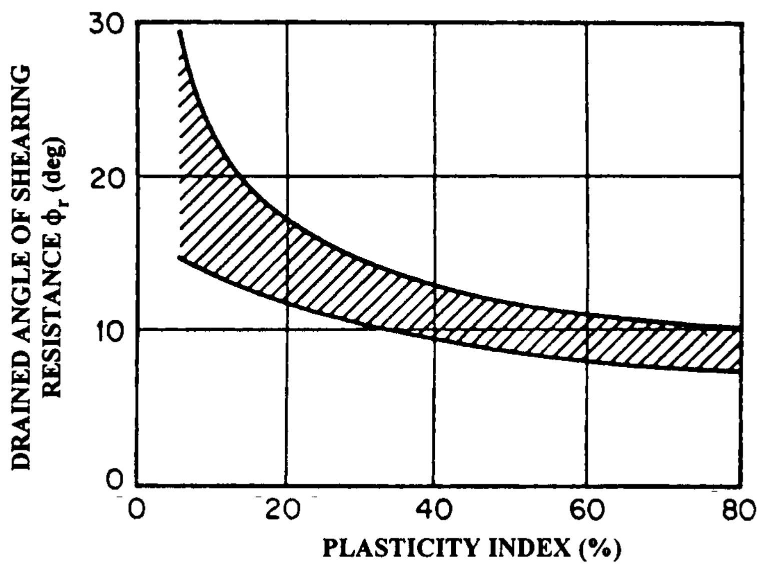

Figure A.2. Approximate relationship between the drained residual friction angle and plasticity index for rock (after Figure 36, FHWA-RD-97-130)

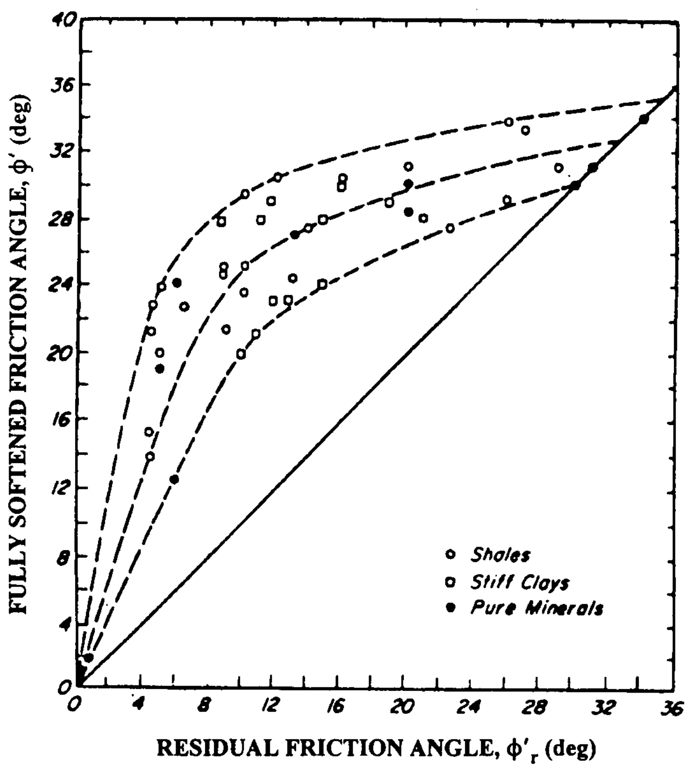

Figure A.3. Relationship between fully softened ϕ' and residual ϕ' (after Figure 37, FHWA-RD-97-130)

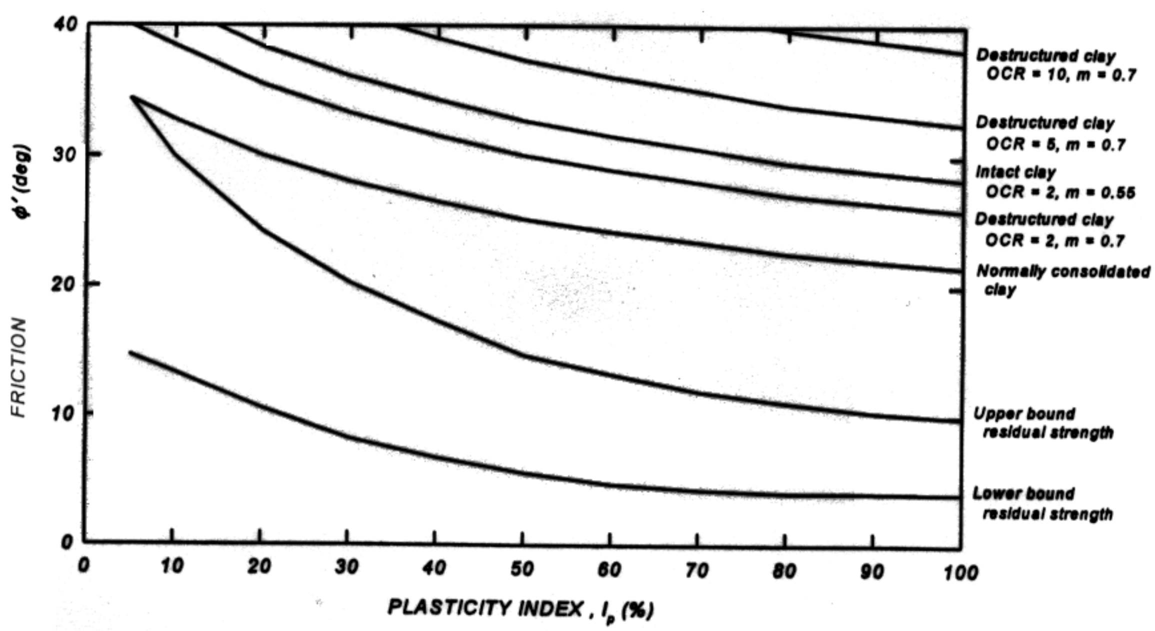

Figure A.4. Friction angle ϕ' for clays in different states as a function of plasticity index (after Figure 37, FHWA-RD-97-130)

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14. ABSTRACT

Methods used in the design of flexible and stiff tieback walls are described. Methods applicable to the design of stiff tieback wall systems are illustrated by example. Important in the design of stiff tieback wall systems is the consideration of construction sequencing effects. Illustrated by example are the equivalent beam on rigid supports method and the equivalent beam on inelastic supports method.

Both the equivalent beam on rigid supports and the equivalent beam on inelastic supports analysis methods consider construction sequencing effects. The equivalent beam on rigid supports method uses soil pressure distributions based on classical methods. The equivalent beam on inelastic supports method uses soil springs (nonlinear) to determine earth-pressure loadings and preloaded concentrated springs (nonlinear) to determine tieback forces. Soil springs are in accordance with the reference deflection method proposed in the Federal Highway Administration's "Summary report of research on permanent ground anchor walls; Vol II, Full-scale tests and soil structure interaction model" (FHWA-RD-98-066).

Soil springs are shifted after each excavation stage to account for the plastic soil movements that occur during excavation. The software program CMULTIANC, newly developed to facilitate the equivalent beam on inelastic supports construction-sequencing analysis, is illustrated in the report.

(Continued)

15. SUBJECT TERMS

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The results from the equivalent beam on rigid supports and equivalent beam on inelastic supports analyses are compared with each other and to the results obtained from other tieback wall analyses. The results are also compared with those obtained from apparent pressure diagram analyses. The apparent pressure diagram approach is common to the design of flexible wall systems.