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UNIVERSITY OF COLORADO AT DENVER

ANALYSIS AND DESIGN OF GEOTEXTILE-REINFORCED EARTH WALLS

PARAMETRIC STUDY AND PRELIMINARY DESIGN METHOD

by

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16. Abstract

A parametric study was undertaken to investigate the effects of material properties, geosynthetic configurations, underlying foundations, and facings on the performance of geosynthetic reinforced retaining walls. The parametric study was conducted by using a finite element program SSCOMP. The program had been validated through comparisons with field measurement of many different earth structures. The effects of the following parameters on the performance of geosynthetic-reinforced retaining walls were investigated: the backfill stiffness and strength; the geosynthetic stiffness, the geosynthetic length; the geosynthetic layer spacing; the foundation stiffness and strength; the foundation depth; the surcharge pressure; and the facing rigidity.

In view of the complexity of the performance of the walls as affected by the factors investigated, and increasing availability of computers to design engineers, a computer-based design procedure is proposed. The design method involves execution of two computer programs, GSWALL and SSCOMP. The program GSWALL is used to obtain a trial design. The program SSCOMP, which is capable of accommodating all of the above-mentioned factors, is used subsequently to calculate the detailed response of the wall, including wall movements and tensile forces induced in the geosynthetic. The trial design should be modified, using the findings of the parametric study as a guide, until an acceptable design is secured.

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Chapter 1
INTRODUCTION

In actual construction, geosynthetic-reinforced retaining walls have repeatedly demonstrated excellent performance characteristics. They have exhibited many distinct advantages over conventional retaining walls (and, to a lesser extent, over other MSE walls). Among the advantages are:

1) Geosynthetic walls are inherently flexible; therefore, are capable of withstanding large foundation settlement. In the Glenwood Canyon geofabric test wall, for example, little distress can be detected although up to 3 ft of settlement, primarily occurs in its foundation soil, has occurred. This superior feature makes geofabric walls suitable for any foundation soils including soft clay foundations.

2) If properly constructed, geosynthetic walls are remarkably stable. In spite of many attempts by practicing engineers and researchers to load geosynthetic walls to failure (in order to examine their ultimate load carrying capacities and safety margins), no one has succeeded in bringing about a "major" failure of any geosynthetic walls, even for

those designed with a safety factor less than one.

3) Geosynthetic walls, especially geofabric walls, are low in total cost. Typically the total cost of geofabric walls, when granular soils are readily available, is between 1/2 to 1/3 of that of comparable conventional retaining walls.

4) Construction of geosynthetic walls is rapid and requires minimum excavation and no heavy equipment.

5) Geofabric walls have no drainage problems. In fact, the geofabric sheets can facilitate drainage and accelerate soil consolidation when backfills of low permeabilities are used.

6) Geosynthetic reinforcements have strong resistance to corrosion and bacterial action, compared with metallic reinforcements.

The basic design criteria for geosynthetic-reinforced retaining walls involve satisfying external stability and internal stability. The external stability is generally evaluated by considering the reinforced soil mass as a semi-rigid gravity retaining wall with active pressure acting behind the wall. The wall is then checked, using methods similar to those for conventional stability analysis of earth retaining structures, for the stability criteria of (1) overturning, (2) sliding, (3) foundation bearing capacity, and (4) overall slope failure.

The internal stability criteria for geosynthetic-reinforced retaining walls require an evaluation of adequate stability against (1) tensile rupture failure, (2) pullout failure, and (3) long-term creep failure. Various methods have been proposed for designing geosynthetic walls against internal failure. A comparison of the existing design methods presented in Volume I of this report has revealed that, while there are significant differences in the design concept of the methods, the greatest discrepancy among the various design methods stems from the safety factors assigned in each method. In a typical wall examined in that study, the combined factor of safety (in terms of the quantity of geosynthetic needed) ranged from 3 to 23 depending on the method used. Apparently the safety factors used in the existing design methods are somewhat arbitrarily assigned and not based on experiences learned from years of actual construction.

The principal investigator is convinced that a rational design method for geosynthetic-reinforced retaining walls should be based on deformation criteria for internal stability evaluation, i.e., based on specified deformation limits. This is necessary because the deformation (strain) associated with tensile rupture and creep failure of geosynthetics is often greater than 100% and because geosynthetics of similar rupture strengths may have very different tensile stiffnesses.

Moreover, none of the existing design methods address the effect of the foundation settlement. They simply assume that the walls are to be constructed over a rigid foundation. However, geosynthetic walls are at times constructed over a soft foundation and the effect of the foundation has been known to be important. In fact, one of the most important advantages of geosynthetic walls is that they are capable of withstanding large deformation (redistributing the resulting stresses) due to foundation settlement.

In this report, the result of a parametric study is presented. The parametric study was conducted by using a finite element program SSCOMP. The program had been validated through comparisons with field measurement of many earth structures. The program is capable of accommodating various wall/foundation geometries (including soft foundations of different depths), soil and geosynthetic properties, construction operations, and external loading conditions. Using the program SSCOMP, the effects of the following parameters on the performance of geosynthetic-reinforced retaining walls were investigated: the backfill stiffness and strength, the geosynthetic stiffness, the geosynthetic length, the geosynthetic layer spacing, the foundation stiffness and strength, the foundation depth, the magnitude of surcharge, and the rigidity of facing.

In addition, a preliminary design method for geosynthetic-reinforced retaining walls is also presented. The design method involves execution of two computer programs: GSWALL and SSCOMP. The program GSWALL is based on the Geoservice design method and is used to obtain a "trial design." The program SSCOMP calculates the stresses, strains and displacements of the backfill and the foundation, and the internal forces and displacements in the geosynthetic layers. The trial design should be modified until a satisfactory design, verified by the result of the program SSCOMP, is obtained.

Chapter 2

THE ANALYTICAL MODEL--SSCOMP

The design approach of geosynthetic-reinforced retaining walls that has been used for actual construction uses a simple limiting equilibrium type of analysis (see Volume 1 of this Report). In this approach, the classical earth pressure theory is assumed to be applicable and the presence of the geosynthetic reinforcement protruding beyond an assumed failure plane is simply considered to provide additional horizontal forces that increase the stability of the earth mass.

A number of theoretical difficulties exist with analyzing geosynthetic-reinforced retaining walls using such a limiting equilibrium method. For one thing, the method assumes rigid-plastic stress-strain behavior for the soil and ignores the changes in geosynthetic extensibility (and strength) resulting from soil-geosynthetic interaction. Another difficulty is that the method does not consider any redistribution of stresses in the earth mass due to the presence of the geosynthetic reinforcement, although it has been well recognized that the stress distribution in the backfill and the failure surface may be very different from those derived from the classical earth pressure theory.

In addition, field construction of geosynthetic-reinforced retaining walls has clearly shown that construction sequence affects the performance of the walls significantly. The construction sequence effect, however, cannot be properly accounted for in limiting equilibrium methods.

It is to be noted that the ultimate state of geosynthetic-reinforced retaining wall is typically associated with very large deformation. When a limiting equilibrium method is used in design, factors of safety are employed to limit the deformation to an acceptable amount. However, unlike conventional earth structure, there is not adequate practical experience with geosynthetic walls; consequently, the factors of safety are somewhat arbitrary. This is evidenced by the wide disparity in the factors of safety suggested by various design methods (see Vol. 1 of this Report).

Among various analytical methods, the finite element method is best suited for analysis of the performance of geosynthetic walls. This is because (1) it is capable of simulating non-linear, stress-dependent behavior of the backfill and the foundation soil, (2) it can accommodate the stress-strain-strength properties of geosynthetics and the interactive behavior between the geosynthetic and the confining soil, (3) it permits description of practically any geometric configuration of geosynthetic walls, and (4) it is well suited for simulating incremental construction procedure.

2.1 Description of SSCOMP Program

The finite element analyses performed in this study were conducted by the computer program SSCOMP. The program is a general, plane strain soil-structure program for static analysis of earth structures including the consideration of compaction induced stresses and deformations. It calculates stresses, strains, and displacements in soil elements and internal forces and displacements in structural elements. The original program containing only soil analysis was coded by Ozawa in 1973 and was named ISBILD. Dicken added structural elements in the program to allow for inclusion of structures in the analysis and changed the name to SSTIP. In 1980, Wong implemented interface elements and a new soil model (the modified Duncan model--using bulk modulus formulation) and renamed it SSTIPN. The program SSCOMP improved on SSTIPN by incorporating a bilinear model for analyzing compaction induced stresses. The finite element programs SSCOMP and its predecessors have been successfully applied to several soil engineering problems including calculation of stresses and movements in embankments and slopes, buried culverts, various earth retaining structures, and mechanically stabilized earth structures.

Detail descriptions of SSCOMP program are given by Seed and Duncan (1984). A brief summary of its main features is presented herein.

<u>Element Types.</u> SSCOMP program has five types of elements as follows:

(a) Soil Elements - Soil elements are four node, two-dimensional, isoparametric elements with compatible modes of displacement.

(b) Bar Elements - Bar elements are two node elements with axial stiffness only (i.e., can only resist axial forces).

(c) Beam-Column Elements - Straight beam-column elements are two node elements with axial, shear and bending stiffness.

(d) Nodal Links - The nodal link is made up of two linear elastic springs (a normal and a shear spring) that control the relative displacements between two nodes.

(e) Interface Elements - The interface element is made up of two nodal links. It is used to model soil-structure interface movement or shear plane within a soil mass.

<u>Soil Model.</u> SSCOMP program employs two soil behavior models. The first soil model is a nonlinear (hyperbolic) stress-strain and bulk moduli model, which is used to calculate soil element material properties during any solution increment. The second

soil behavior model is a hysteretic loading-unloading model for stresses resulting from soil compaction.

Structural Model. The stress-strain relationship of bar and beam-column elements is assumed to be linear elastic.

Nonlinear Solution Technique. SSCOMP program adopts a "two-iteration" solution procedure for each increment of analysis, which may be placement of a soil layer, compaction of a soil layer, placement of a structure, or application of an external load. Each increment is analyzed twice; the first time using soil moduli values based on the stresses before the increment, and the second time using soil moduli values based on the average stresses during the increment.

2.2 Characteristics of the Finite Element Analysis Procedure

The principal characteristics of the finite element analysis of the performance of geosynthetic walls are:

(1) Both the soil (backfill, retained soil, and foundation) and the geosynthetic are divided into a number of two-dimensional elements for purposes of analysis. Any configuration of geosynthetic arrangement and any backfill condition may be represented.

(2) The analyses are conducted step-by-step, each step representing a construction lift (placement of fill and compaction) or application of a live load. Stresses

and displacements in the soil (backfill and foundation) and forces and displacements along the geosynthetic layers are determined at each step of the analysis, thus providing a complete picture of the wall performance during construction and under operative condition.

(3) The nonlinear stress-dependent stress-strain behavior of the soil is simulated in the analyses, using the most widely used modified Duncan hyperbolic model (Duncan, et al., 1980). The instantaneous value of Young's modulus and the secant value of bulk modulus are related to the stresses in each element by means of empirical parameters, which may be determined from the results of laboratory triaxial tests.

(4) The deformations of the geosynthetic and the soil are calculated in an integrated manner by incorporating the combined stiffness of both the geosynthetic and the contacting soil. Therefore, the soil-geosynthetic interaction effect is fully accounted for in the analysis.

2.3 Modified Hyperbolic Duncan Soil Model

The nonlinear, stress-dependent soil behavior model employed in the program SSCOMP is the modified hyperbolic Duncan model. The model assumes that the stress-strain curves for soils can be approximated as hyperbolas shown in Figure 2.1. The

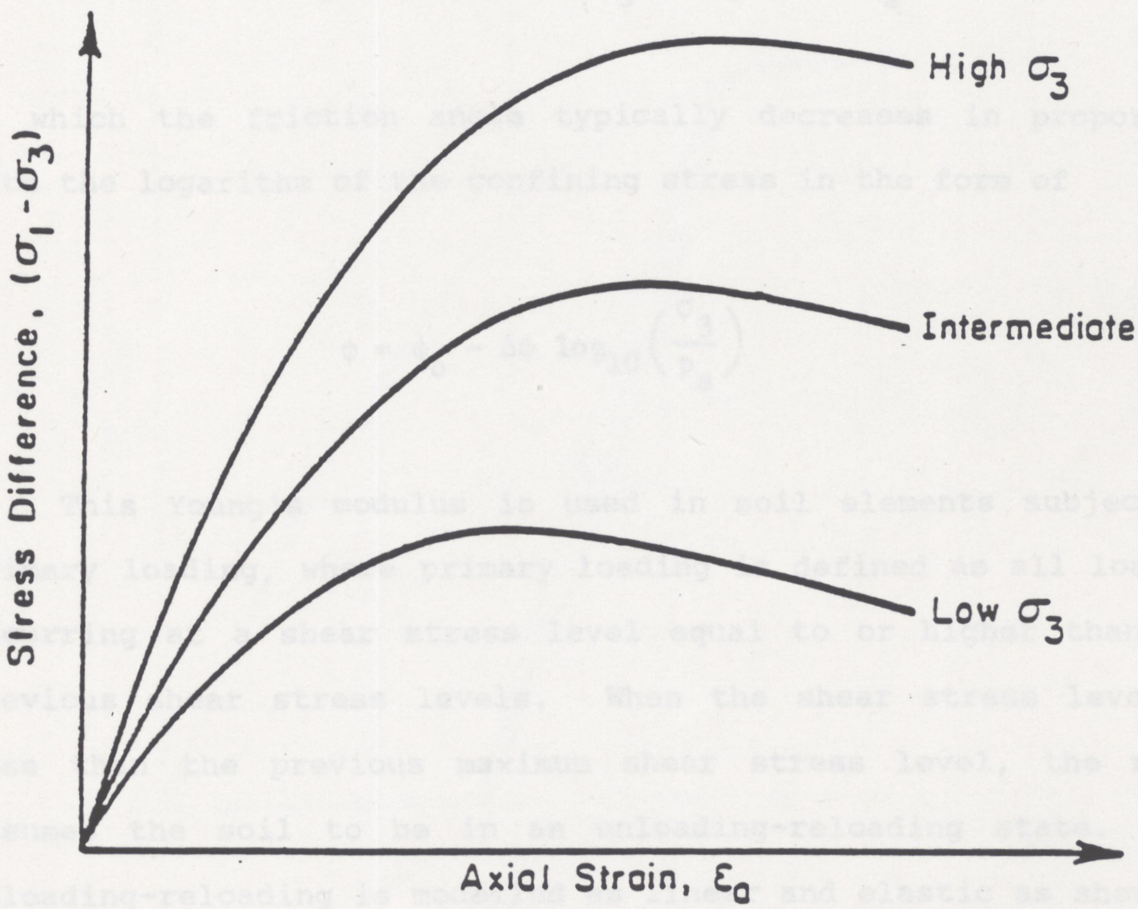


Figure 2.1 Hyperbolic Stress-Strain Curves of Soil

instantaneous slope of the hyperbolic stress-strain curve is the Young's modulus, E_t, which is a function of confining stress and shear stress level, and can be expressed as:

$$E_t = \left[1 - \frac{R_f(1-\sin\phi)(\sigma_1-\sigma_3)}{2c\cos\phi + 2\sigma_3\sin\phi}\right]^2 Kp_a\left(\frac{\sigma_3}{p_a}\right)^n$$

in which the friction angle typically decreases in proportion with the logarithm of the confining stress in the form of

$$\phi = \phi_o - \Delta\phi \log_{10}\left(\frac{\sigma_3}{p_a}\right)$$

This Young's modulus is used in soil elements subject to primary loading, where primary loading is defined as all loading occurring at a shear stress level equal to or higher than all previous shear stress levels. When the shear stress level is less than the previous maximum shear stress level, the model assumes the soil to be in an unloading-reloading state. The unloading-reloading is modelled as linear and elastic as shown in Figure 2.2. The unloading-reloading modulus is a function only of confining stress as:

$$E_{ur} = K_{ur}\, p_a\left(\frac{\sigma_3}{p_a}\right)^n$$

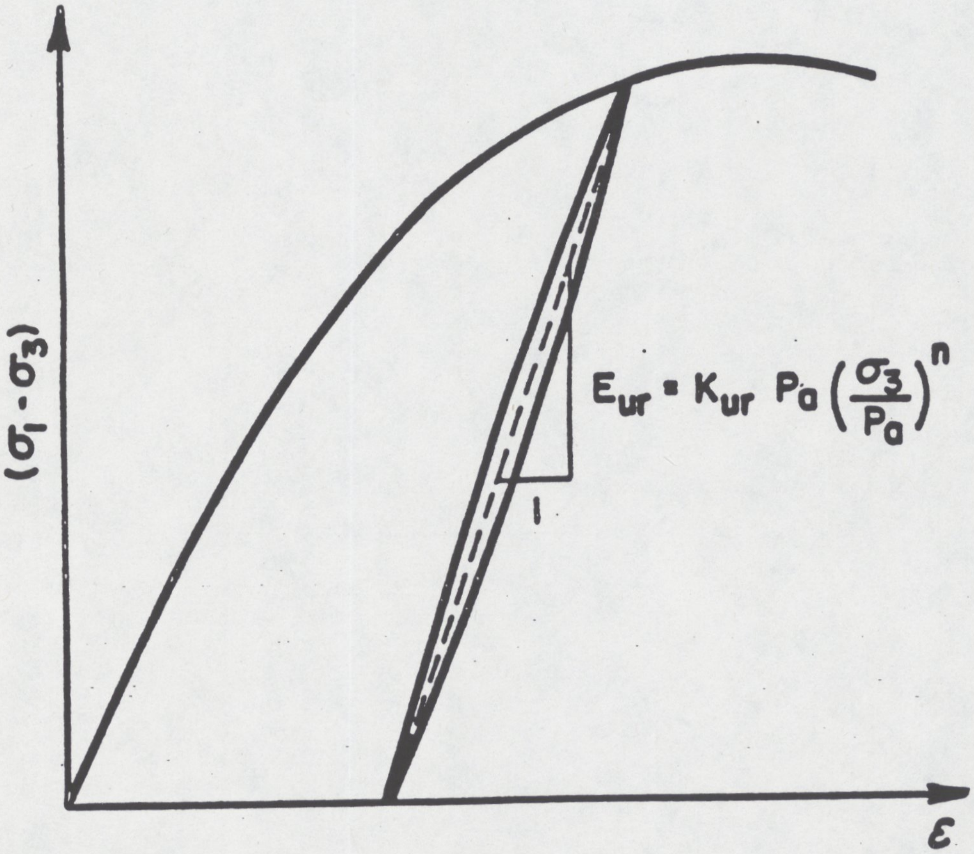


Figure 2.2 Unloading-Reloading Curve of Soil

The model also assumes that the bulk modulus of soil is dependent on the confining stress as:

$$B = K_b \, Pa \left(\frac{\sigma_3}{Pa} \right)^m$$

There are nine material parameters involved in the modified hyperbolic Duncan model. Table 2.1 summarizes the role of each of these parameters. Detail descriptions of the model and the procedure for determining the material parameters are given by Duncan, et al. (1980).

It should be noted that Duncan, et al. (1980) has compiled the values of the material parameters for more than one hundred different soils tested under drained and undrained conditions. This wide data base can be used to estimate reasonable values of the parameters in cases where the available information on the soil is restricted to descriptive classification. The data base is also useful for assessing whether parameter values derived from laboratory test results with past experience. Representative parameter values for soils tested under drained conditions are presented in Table 2.2.

Table 2.1 Summary of the Modified Hyperbolic Duncan Model Soil Parameters

Parameter	Name	Function
K, K_{ur}	Modulus number	Relate E_i and E_{ur} to σ_3
n	Modulus exponent	
c	Cohesion intercept	Relate $(\sigma_1-\sigma_3)_f$ to σ_3
ϕ, $\Delta\phi$	Friction angle parameters	
R_f	Failure ratio	Relates $(\sigma_1-\sigma_3)_{ult}$ to $(\sigma_1-\sigma_3)_f$
K_b	Bulk modulus number	Value of B/P_a at $\sigma_3 = P_a$
m	Bulk modulus exponent	Change in B/P_a for ten-fold increase in σ_3

Table 2.2 Representative Parameter Values of the Modified Hyperbolic Duncan Soil Model

Unified Soil Classification	RC* Stand. AASHTO	γ_m k/ft³	ϕ_o deg	$\Delta\phi$ deg	c k/ft²	K	n	R_f	k_b	m
GW, GP	105	0.150	42	9	0	600	0.4	0.7	175	0.2
SW, SP	100	0.145	39	7	0	450	0.4	0.7	125	0.2
	95	0.140	36	5	0	300	0.4	0.7	75	0.2
	90	0.135	33	3	0	200	0.4	0.7	50	0.2
SM	100	0.135	36	8	0	600	0.25	0.7	450	0.0
	95	0.130	34	6	0	450	0.25	0.7	350	0.0
	90	0.125	32	4	0	300	0.25	0.7	250	0.0
	85	0.120	30	2	0	150	0.25	0.7	150	0.0
SM-SC	100	0.135	33	0	0.5	400	0.6	0.7	200	0.5
	95	0.130	33	0	0.4	200	0.6	0.7	100	0.5
	90	0.125	33	0	0.3	150	0.6	0.7	75	0.5
	85	0.120	33	0	0.2	100	0.6	0.7	50	0.5
CL	100	0.135	30	0	0.4	150	0.45	0.7	140	0.2
	95	0.130	30	0	0.3	120	0.45	0.7	110	0.2
	90	0.125	30	0	0.2	90	0.45	0.7	80	0.2
	85	0.120	30	0	0.1	60	0.45	0.7	50	0.2

* RC = relative compaction, in percent

Chapter 3

PARAMETRIC STUDY

In this chapter, a parametric study of the performance of geosynthetic walls is presented. The parametric study was conducted by using the finite element program SSCOMP to investigate the effects of material properties, reinforcement configuration, surcharge loading, foundation soil, and facing rigidity on the wall performance. For the purposes of comparison, two "control walls" were selected for the analyses. The first control wall, referred to as Control Wall A, was for a wall constructed over a rigid foundation. The second wall, referred to as Control Wall B, was for a wall constructed over a flexible foundation. The conditions of the two control walls are:

(A) Control Wall A:

<u>Geometry</u>:
wall height H = 12 ft
reinforcement spacing S = 1 ft
reinforcement length (Uniform) L = 9 ft
rigid foundation
horizontal crest
vertical facing

<u>Materials</u>:
backfill:
a uniform, medium-dense GP soil compacted to 95% standard Proctor, with the modified hyperbolic Duncan model parameters shown in Table 3.1.

TABLE 3.1 MODIFIED HYPERBOLIC DUNCAN MODEL PARAMETERS FOR SOILS OF THE CONTROL WALLS

	γ (pcf)	K	n	R_f	K_B	m	C (psf)	ϕ	$\Delta\phi$	Ko	Kur
Backfill A	125	600	0.6	0.7	175	0.2	0	39°	7°	0.37	600
Foundation (of (Control Wall B)	120	120	0.45	0.7	110	0.2	300	30°	0°	0.5	120

geosynthetic:
linear elastic, $E = 1.2 \times 10^6$ psf/ft
$A = 0.0125$ ft^2/ft

no slippage at the soil-geosynthetic interface

Loading: uniform surcharge q
$q = 0.2 \times$ (soil unit weight) $\times H$

(B) Control Wall B:

The same as the Control Wall A, except

Foundation: a soft clay with its parameters for the modified hyperbolic Duncan soil model shown in Table 3.1.
foundation depth D = 14 ft

Figure 3.1 shows the configuration of the Control Wall B. The same configuration applies to the Control Wall A except that the foundation is rigid. In the parametric study, two categories of analyses were performed: one on a rigid foundation (associated with the Control Wall A), the other on a soft foundation (associated with the Control Wall B). While a certain factor (e.g., the effect of geosynthetic stiffness) is being examined, all the conditions were kept the same as the respective control wall except for the particular factor under investigation.

3.1 Finite Element Discretization

The basic finite element mesh used in this parametric study is shown in Figure 3.2. For the analyses associated with the Control Wall A, the backfill was simulated by 144 soil elements, the geosynthetic layers by 72 bar elements, and the facing by 12 bar elements. For the analyses associated with the Control Wall

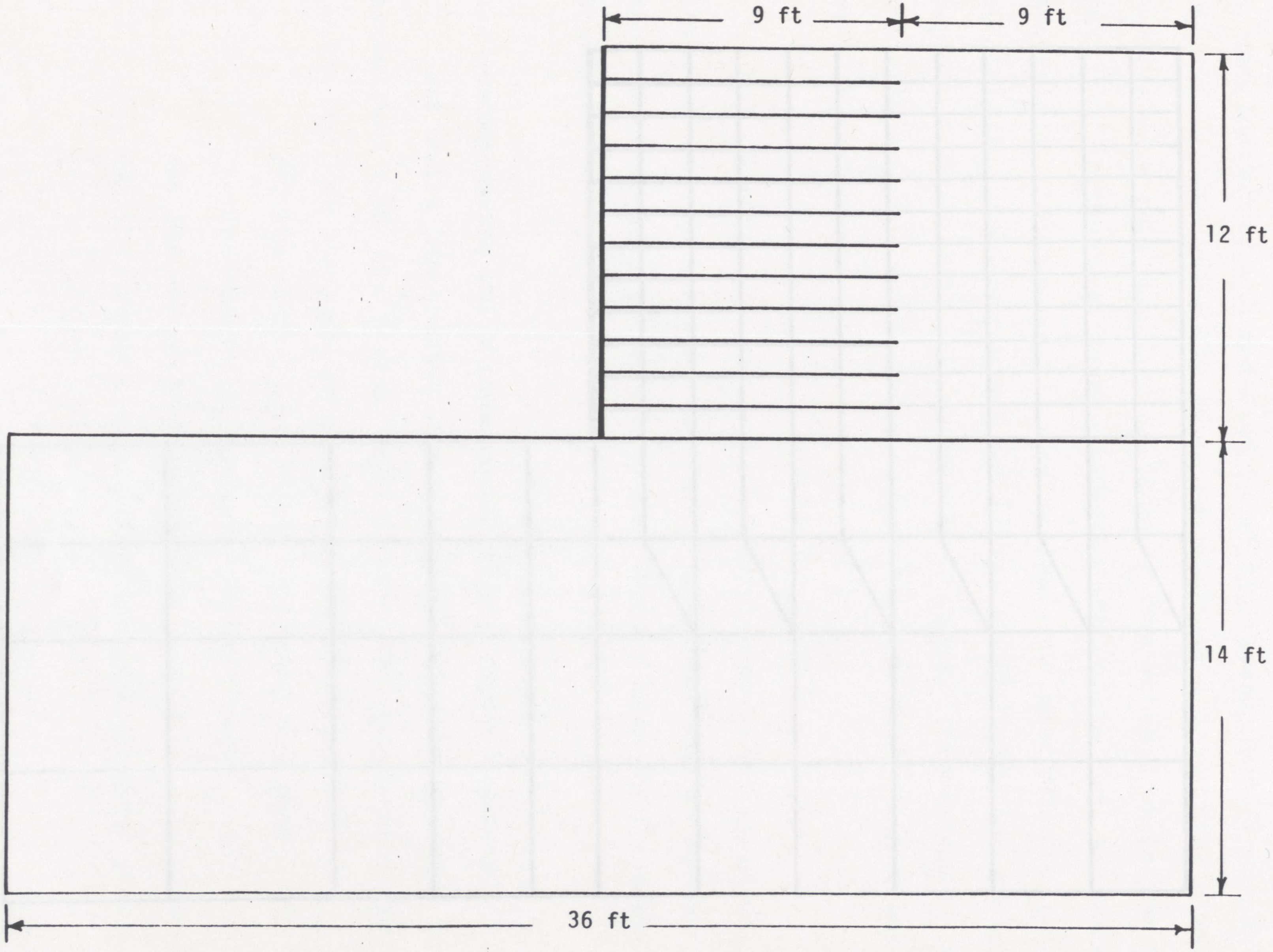


Figure 3.1 Configuration of the Control Wall B

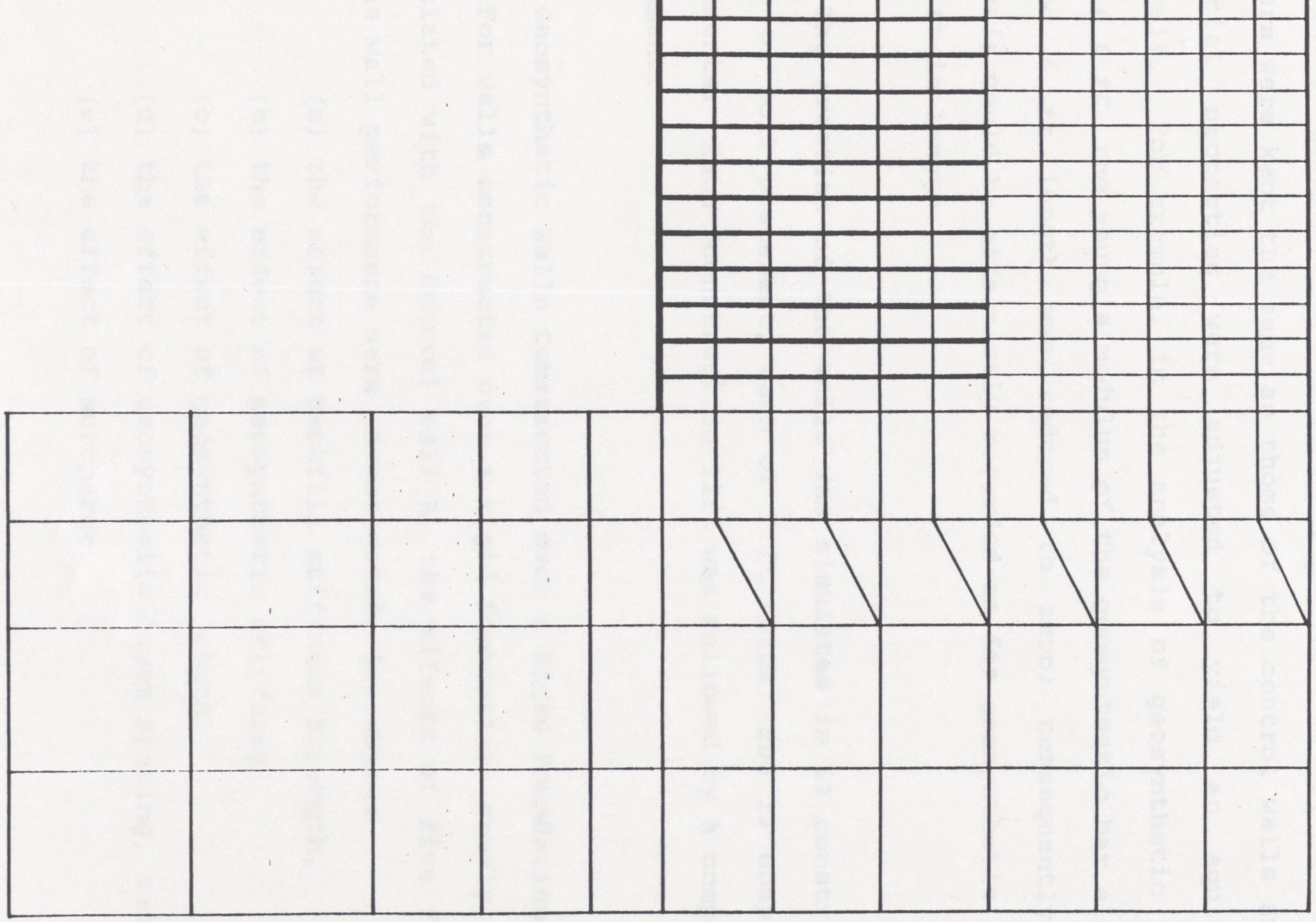


Figure 3.2 Finite Element Mesh for the Parametric Study

B, the foundation was modelled by an additional 56 soil elements. When the geometry was varied in the parametric study, the element numbers were kept the same as those of the control walls and the material properties were adjusted to yield an equivalent analysis. For example, in the analysis of geosynthetic length equals 6 ft, the young's modulus of the geosynthetic bar elements beyond 6 ft length was reduced to zero; consequently, the analysis could be effectively regarded as for geosynthetic layers of 6 ft in length.

The erection of the walls was simulated in 12 construction lifts of soil placement, each of 1 ft thick, and 12 compaction increments. Each construction lift was followed by a compaction increment.

3.2 Geosynthetic Walls Constructed over a Rigid Foundation

For walls constructed over a rigid foundation, namely, those associated with the Control Wall A, the effects of five factors on the wall performance were investigated, including:

(a) the effect of backfill stiffness/strength,

(b) the effect of geosynthetic stiffness,

(c) the effect of geosynthetic length,

(d) the effect of geosynthetic layer spacing, and

(e) the effect of surcharge.

The following sections present the results of the analyses and discussions of the results.

3.2.1 Effect of Backfill Stiffness/Strength

The effect of using backfills of different stress-strain-strength behavior were examined. The model parameters of three different backfills, designated as Backfills A, B, and C are listed in Table 3.2. It is to be noted that Backfills A, B, and C were selected to represent granular backfills of medium, medium-dense, and dense densities, respectively.

Figures 3.3, 3.4, and 3.5 depict the horizontal wall displacements, the tensile force distribution in the geosynthetic layers at depths of 2 ft, 6 ft, and 10 ft, and the lateral earth pressures on the facing for the three backfills. It is seen that the backfill stress-strain-strength behavior has significant effects on the wall movement and the forces induced in the geosynthetic reinforcement. The stiffer the backfill give rise to smaller wall movement and geosynthetic forces. The maximum lateral movement of the wall occurs approximately at the mid-height of the wall for all three backfills.

The tensile forces in the geosynthetic reinforcement are very different, both in terms of the magnitude and the shape of distribution, at different depth. Near the top surface, the forces are fairly uniform and are relatively small. At the mid-height, the forces assume a distribution resembles what is

TABLE 3.2 MODIFIED HYPERBOLIC DUNCAN MODEL PARAMETERS OF BACKFILLS A, B, AND C, FOUNDATION D, AND THE COHESIVE BACKFILL

	γ (pcf)	K	n	R_f	K_B	m	C (psf)	ϕ	$\Delta\phi$	Ko	Kur
Backfill A	125	400	0.6	0.7	175	0.2	0	36^o	5^o	0.41	400
Backfill B	125	600	0.6	0.7	175	0.2	0	39^o	7^o	0.37	600
Backfill C	125	850	0.6	0.7	175	0.2	0	42^o	7^o	0.33	850

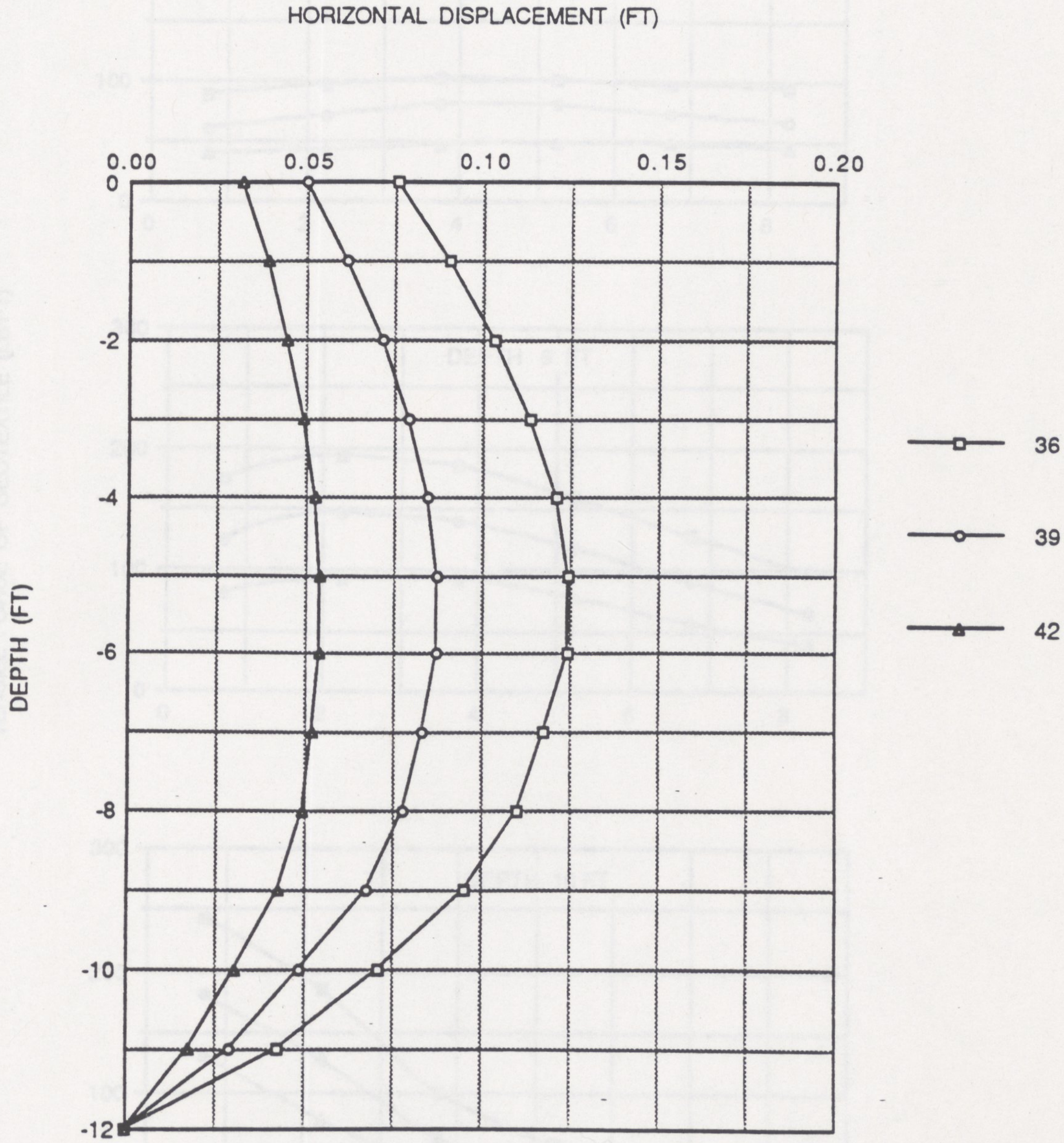


Figure 3.3 Effect of Backfill Stiffness/Strength on Horizontal Wall Displacement--Rigid Foundation

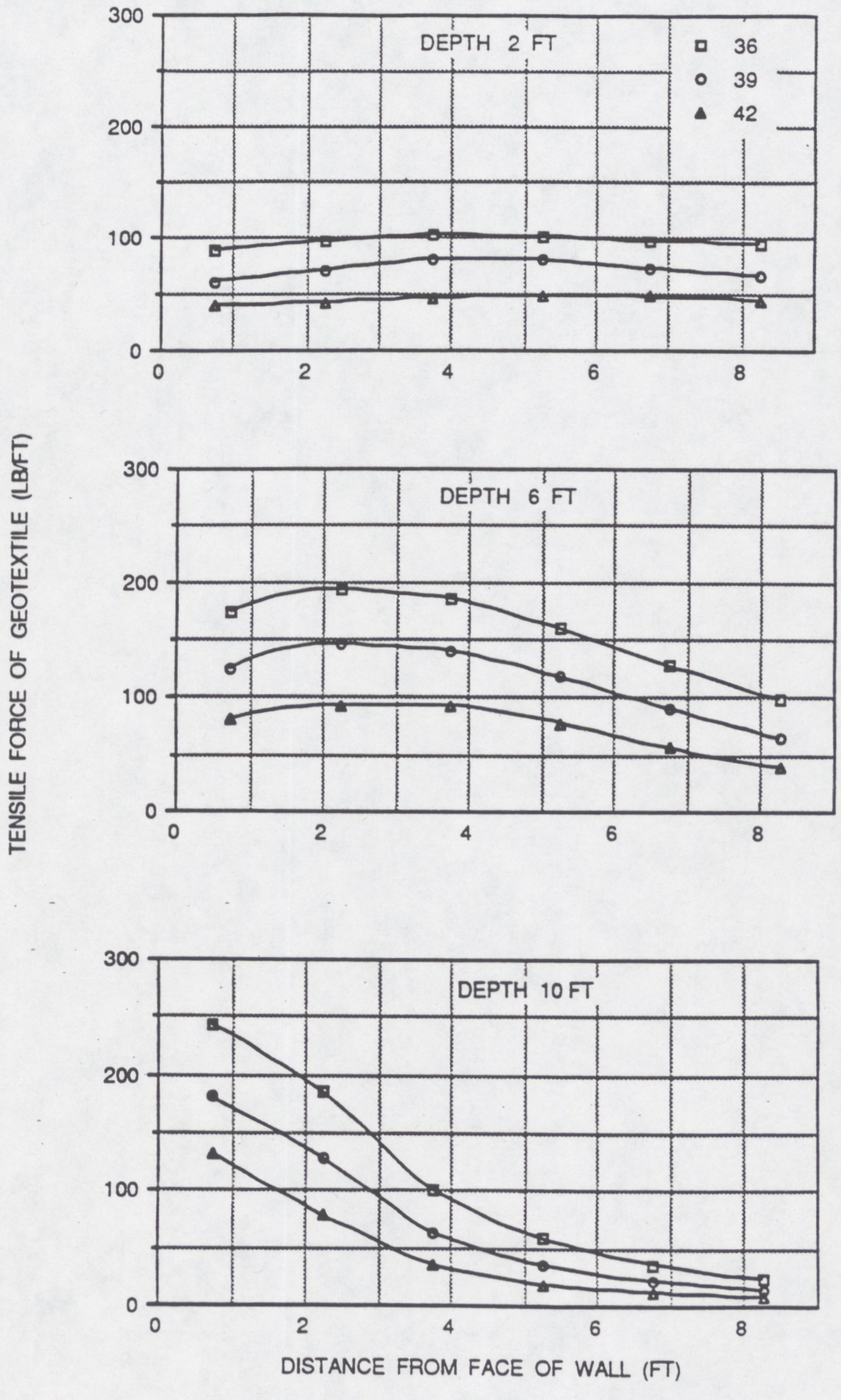


Figure 3.4 **Effect of Backfill Stiffness/Strength on Geosynthetic Tensile Force--Rigid Foundation**

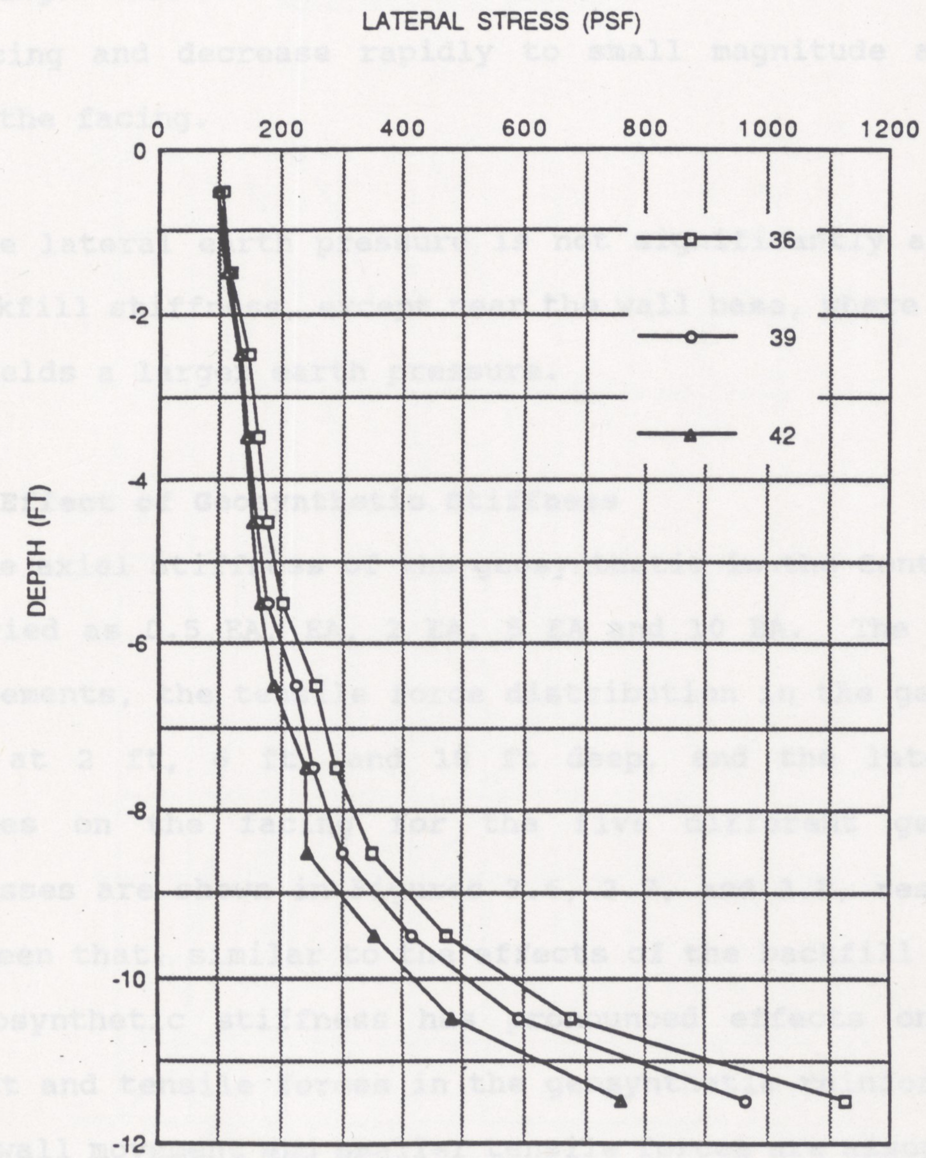


Figure 3.5 Effect of Backfill Stiffness/Strength on Lateral Earth Pressure--Rigid Foundation

assumed in the Geoservice design method, i.e., a triangular distribution, with the largest forces occurring about 2 ft behind the facing. Near the base of the wall, the maximum forces occur at the facing and decrease rapidly to small magnitude about 5 ft behind the facing.

The lateral earth pressure is not significantly affected by the backfill stiffness, except near the wall base, where the softer soil yields a larger earth pressure.

3.2.2 Effect of Geosynthetic Stiffness

The axial stiffness of the geosynthetic in the Control Wall A was varied as 0.5 EA, EA, 2 EA, 5 EA and 10 EA. The horizontal displacements, the tensile force distribution in the geosynthetic layers at 2 ft, 6 ft, and 10 ft deep, and the lateral earth pressures on the facing for the five different geosynthetic stiffnesses are shown in Figures 3.6, 3.7, and 3.8, respectively. It is seen that, similar to the effects of the backfill stiffness, the geosynthetic stiffness has pronounced effects on the wall movement and tensile forces in the geosynthetic reinforcement. A larger wall movement and smaller tensile forces are associated with a smaller geosynthetic stiffness.

The effect of geosynthetic stiffness on the lateral earth pressure is modest. The larger the geosynthetic stiffness, the larger the lateral earth pressure.

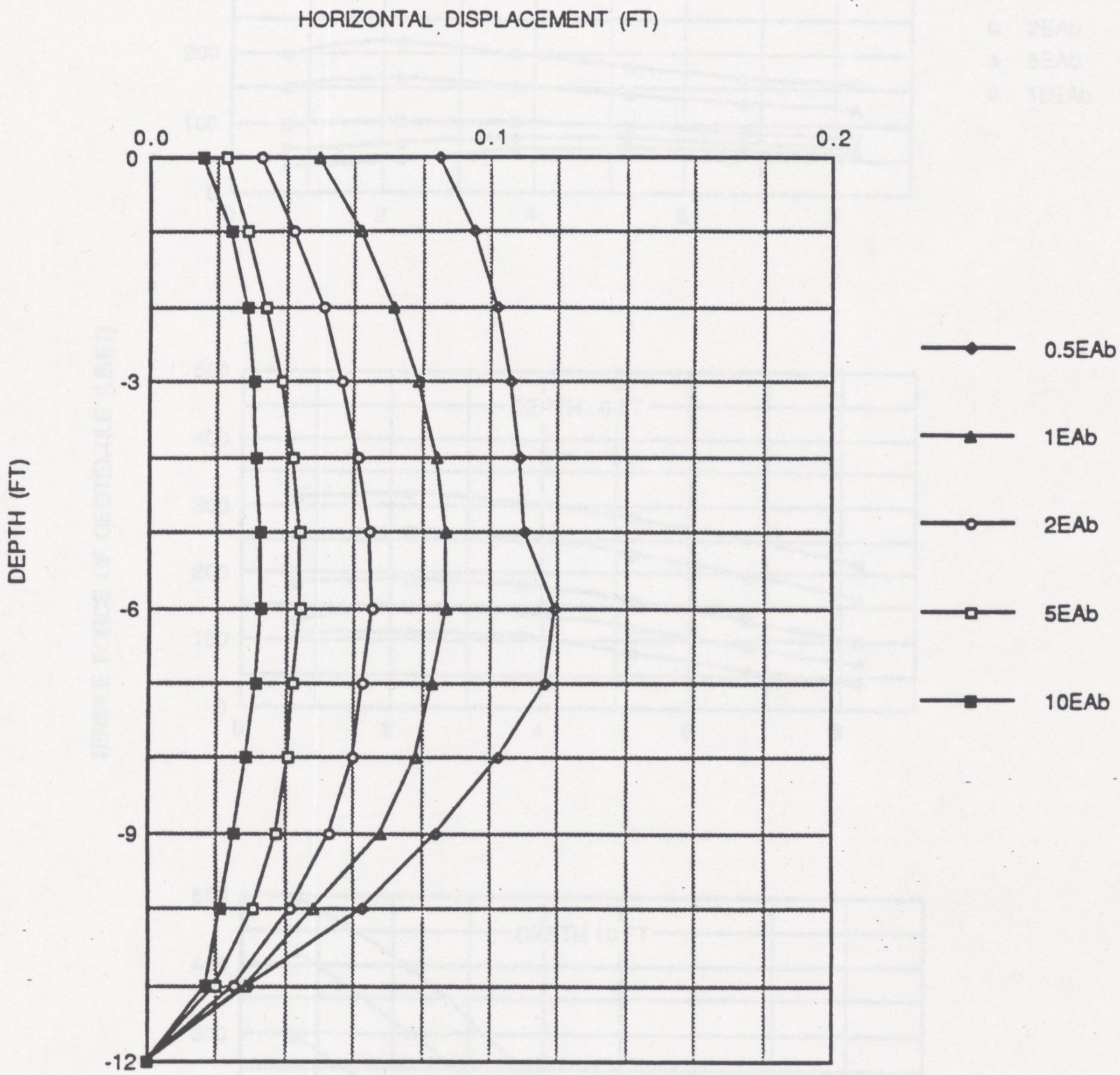


Figure 3.6 **Effect of Geosynthetic Stiffness on Horizontal Wall Displacement--Rigid Foundation**

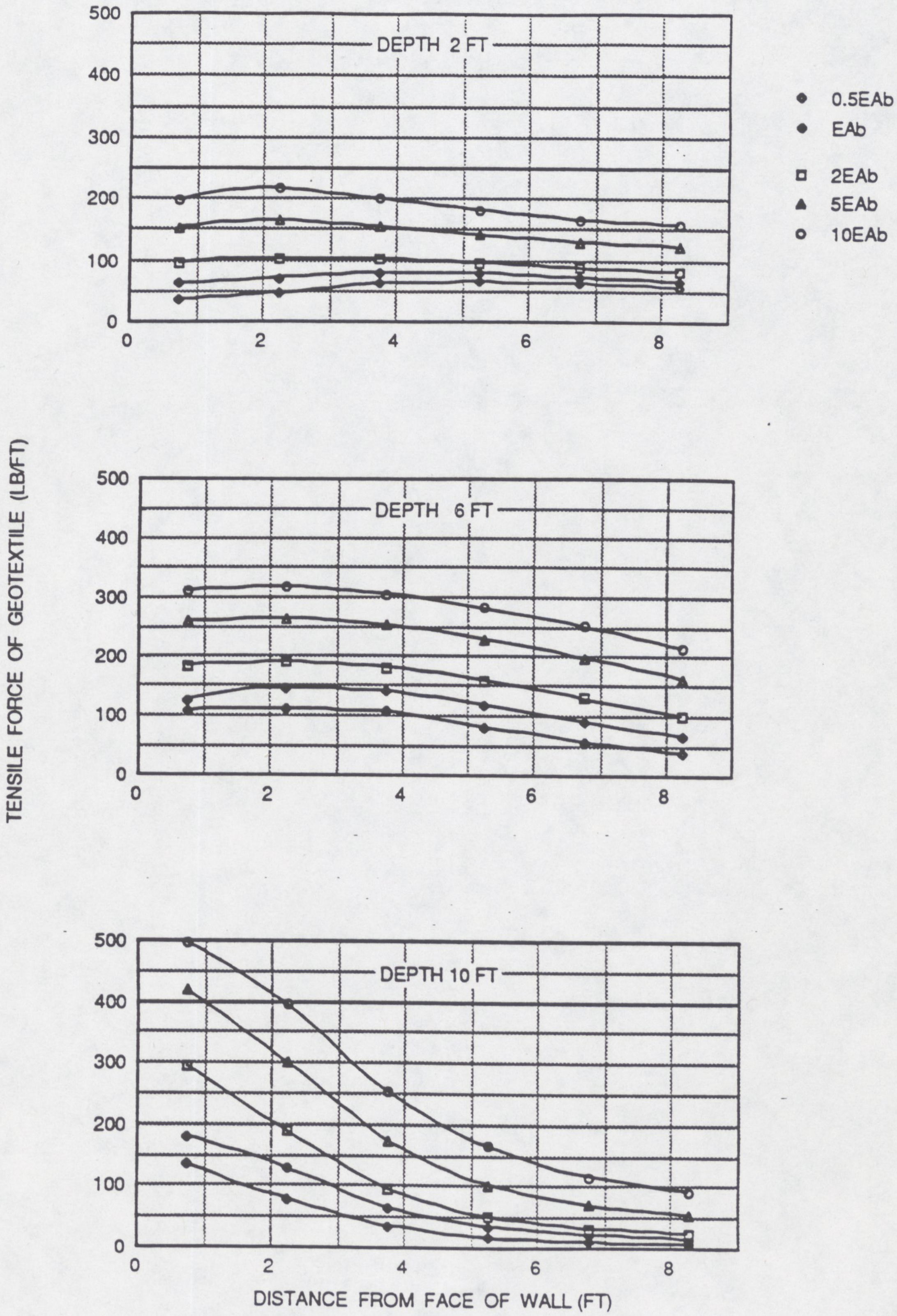


Figure 3.7 Effect of Geosynthetic Stiffness on Geosynthetic Tensile Force--Rigid Foundation

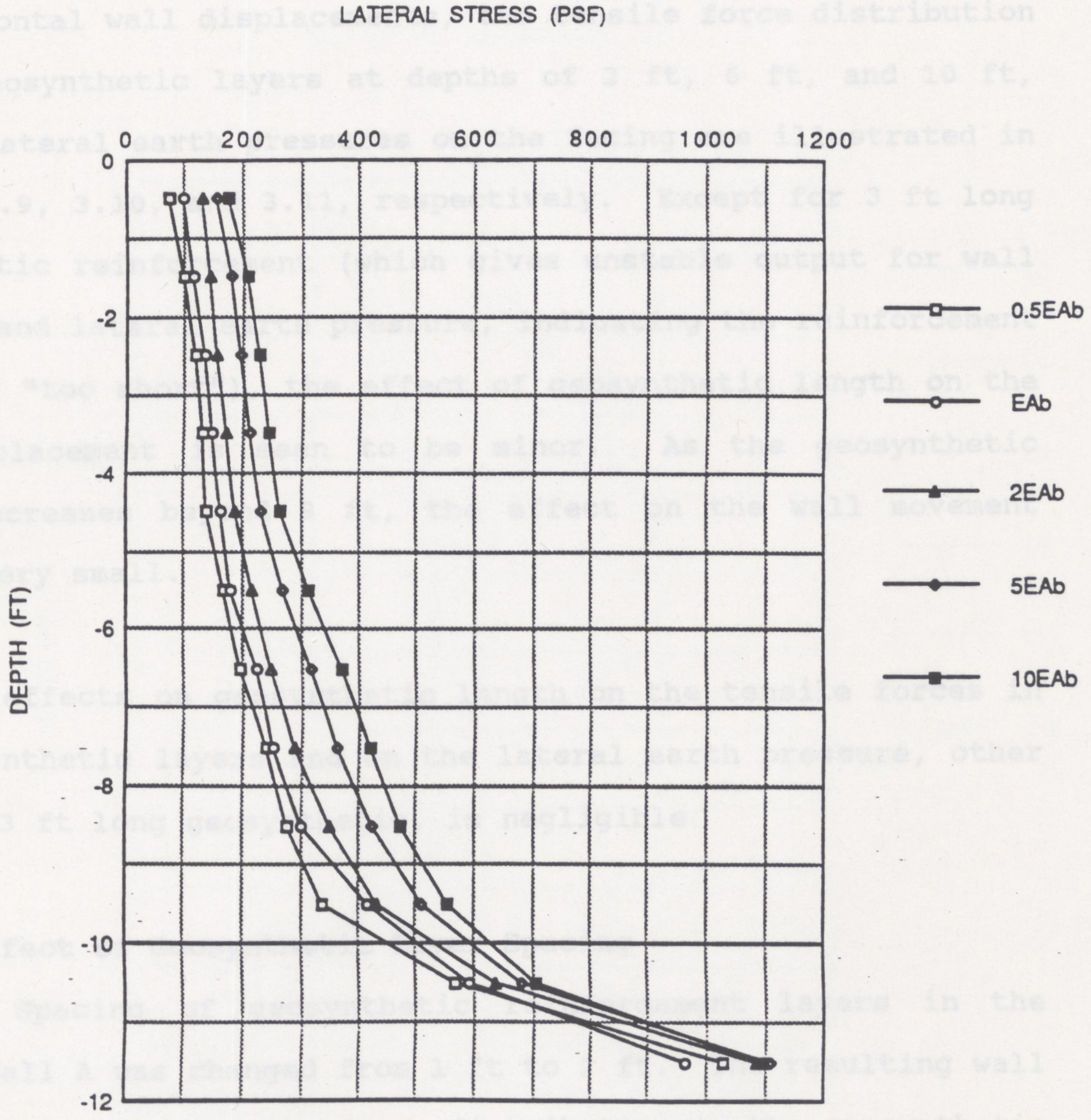


Figure 3.8 Effect of Geosynthetic Stiffness on Lateral Earth Pressure--Rigid Foundation

3.2.3 Effect of Geosynthetic Length

The geosynthetic length in the Control Wall A was varied from 9 ft to 3 ft, 4.5 ft, 6 ft, 12 ft and 18 ft. The change in the horizontal wall displacements, the tensile force distribution in the geosynthetic layers at depths of 2 ft, 6 ft, and 10 ft, and the lateral earth pressures on the facing are illustrated in Figures 3.9, 3.10, and 3.11, respectively. Except for 3 ft long geosynthetic reinforcement (which gives unstable output for wall movement and lateral earth pressure, indicating the reinforcement length is "too short"), the effect of geosynthetic length on the wall displacement is seen to be minor. As the geosynthetic length increases beyond 9 ft, the effect on the wall movement becomes very small.

The effects on geosynthetic length on the tensile forces in the geosynthetic layers and on the lateral earth pressure, other than for 3 ft long geosynthetic, is negligible

3.2.4 Effect of Geosynthetic Layer Spacing

The Spacing of geosynthetic reinforcement layers in the Control Wall A was changed from 1 ft to 2 ft. The resulting wall displacements, the tensile force distribution in the geosynthetic at different depths, and the lateral earth pressure on the facing are shown in Figures 3.12, 3.13, and 3.14, respectively. The output of wall movement for 2-ft spacing suffers from numerical instability in the bottom half of the wall, indicating that 2-ft

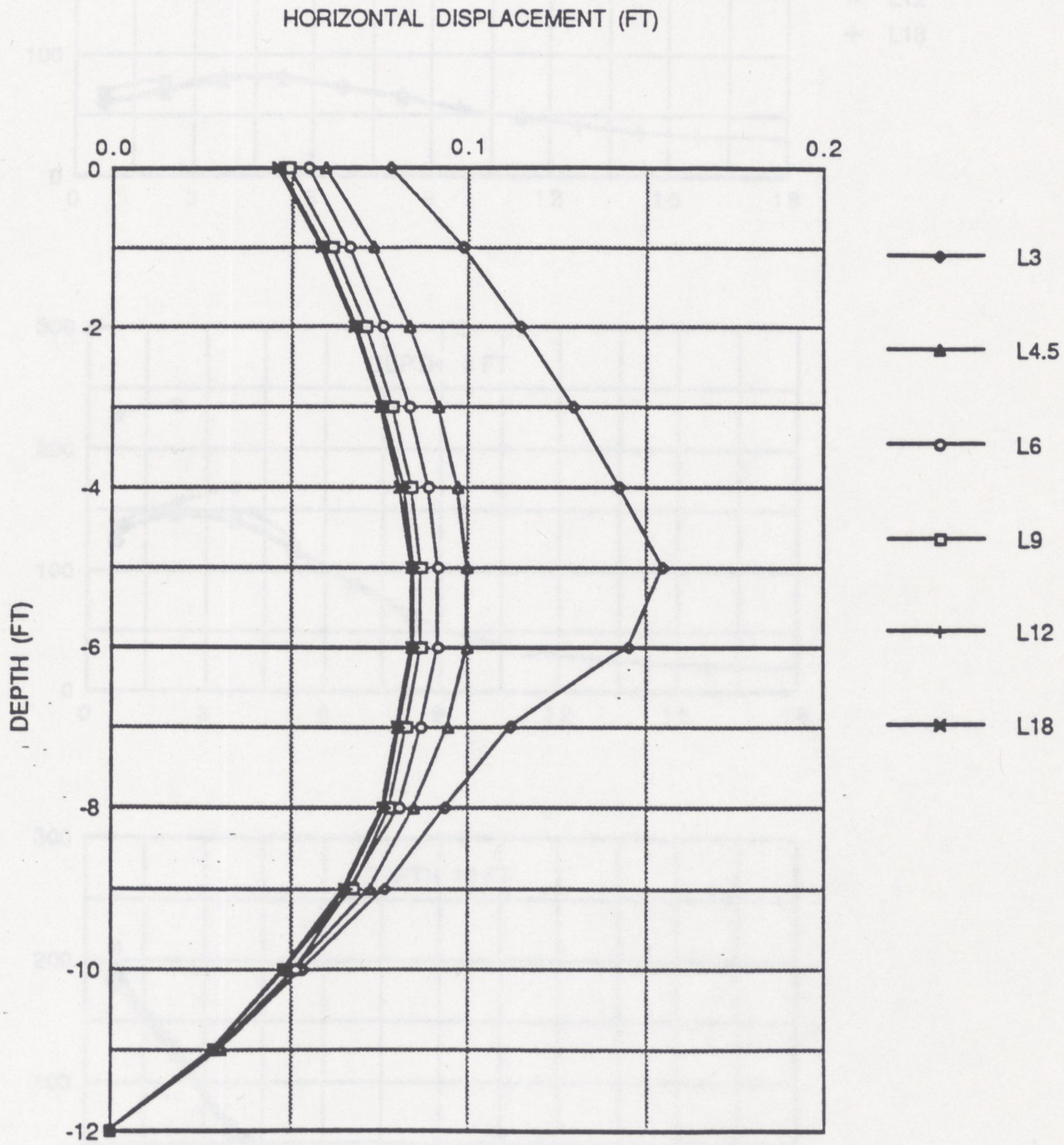


Figure 3.9 **Effect of Geosynthetic Length on Horizontal Wall Displacement--Rigid Foundation**

Fig : F-2

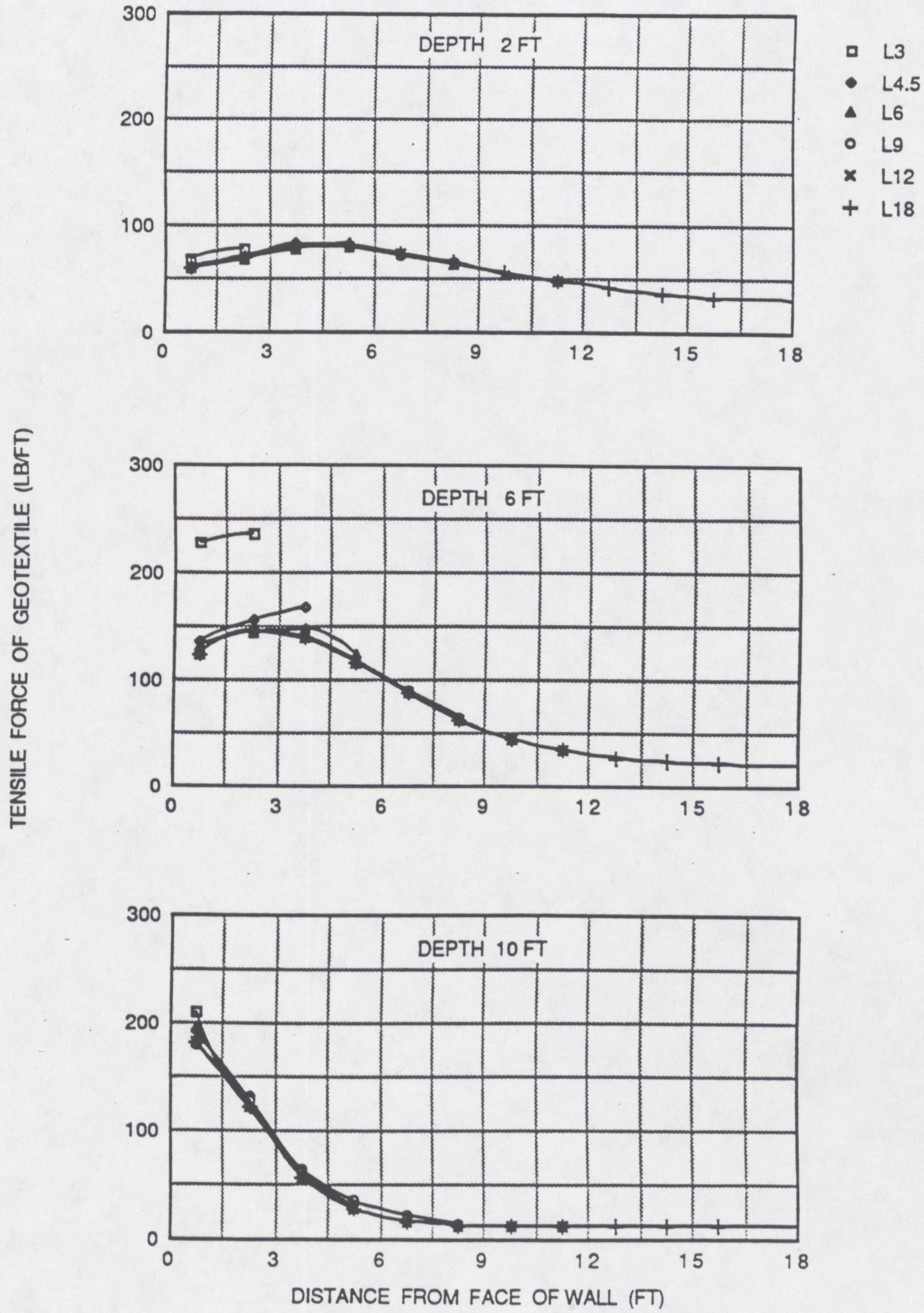


Figure 3.10 **Effect of Geosynthetic Length on Geosynthetic Tensile Force--Rigid Foundation**

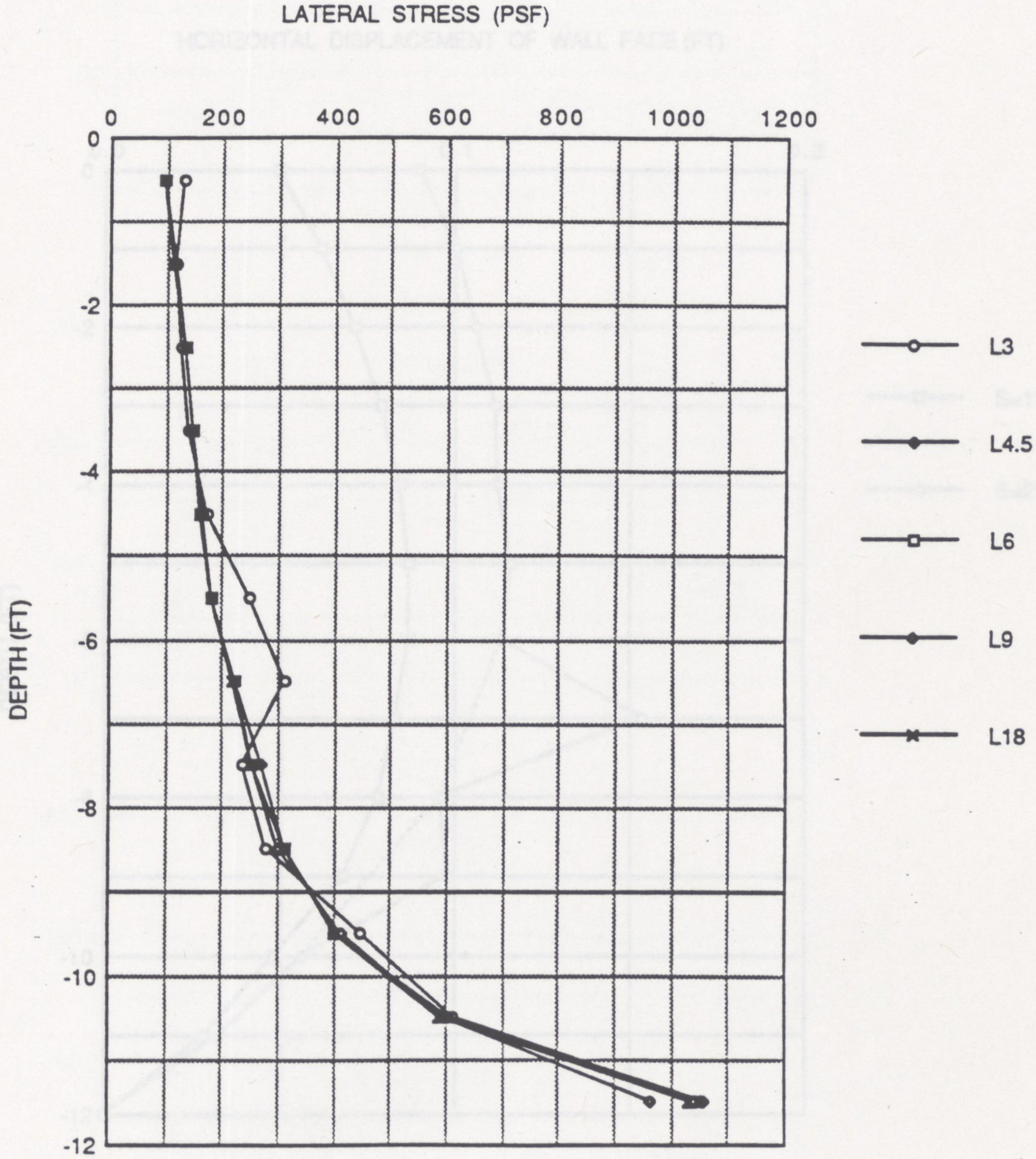


Figure 3.11 Effect of Geosynthetic Length on Lateral Earth Pressure--Rigid Foundation

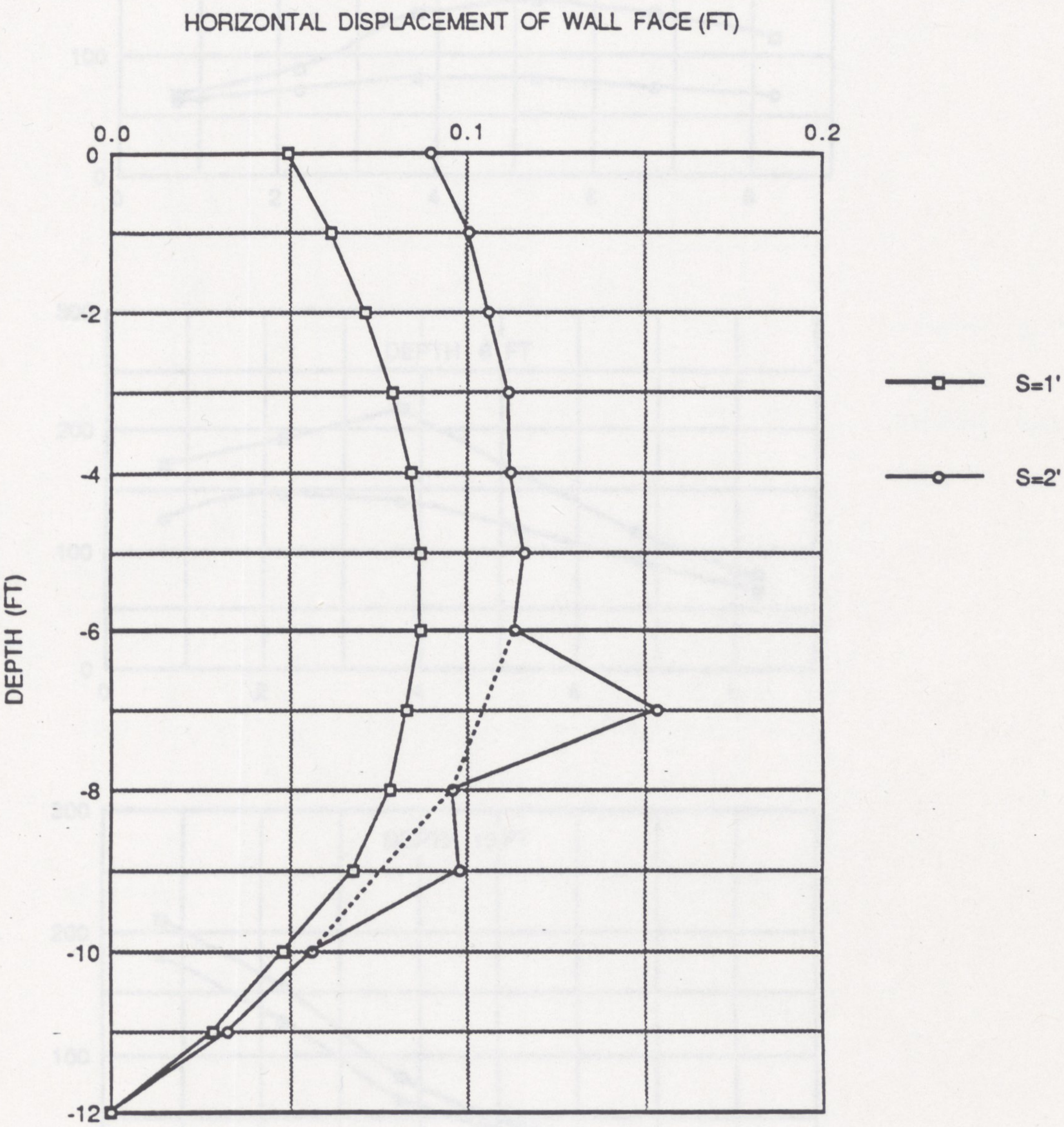


Figure 3.12 Effect of Geosynthetic Layer Spacing on Horizontal Wall Displacement--Rigid Foundation

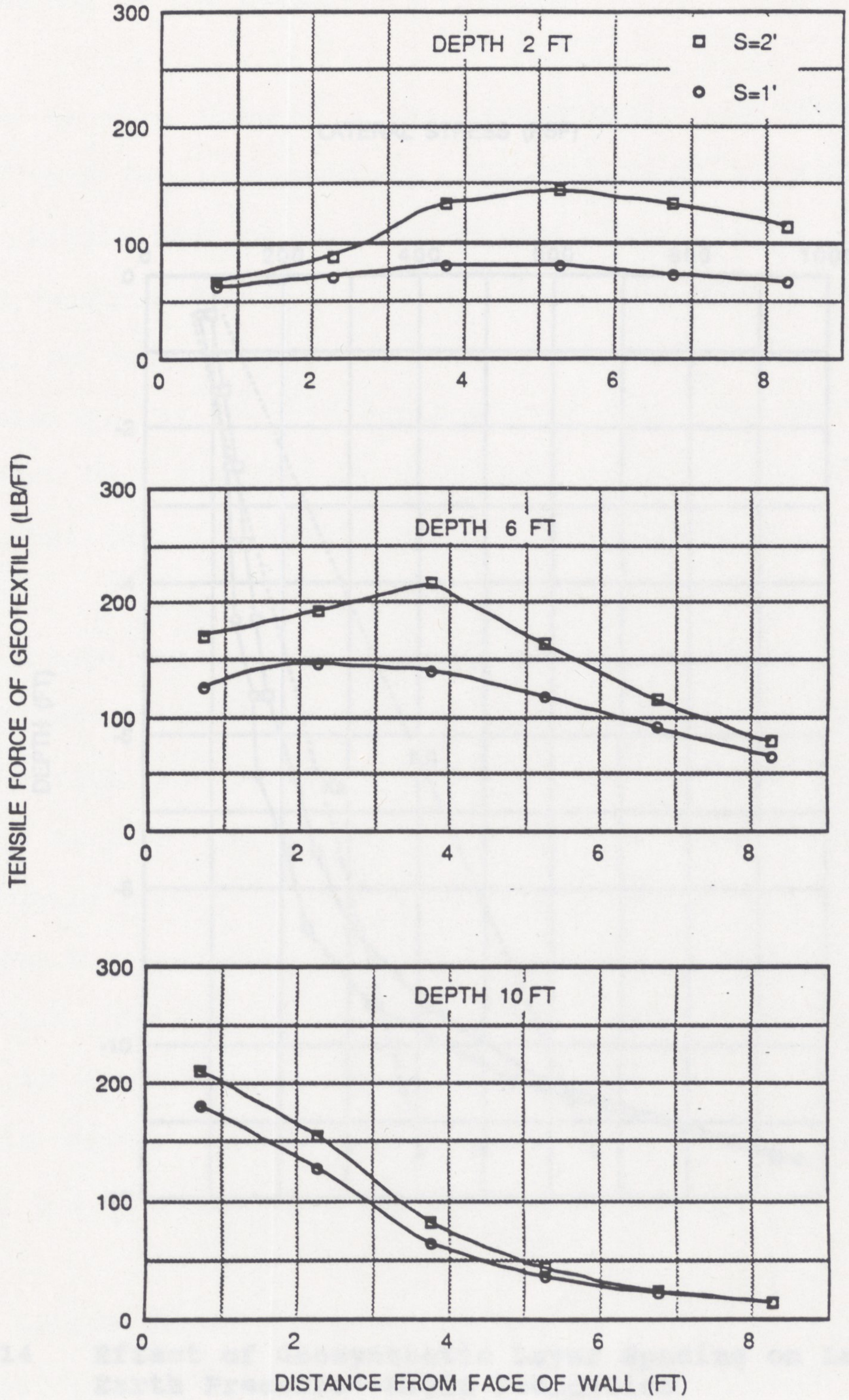


Figure 3.13 Effect of Geosynthetic Layer Spacing on Geosynthetic Tensile Force--Rigid Foundation

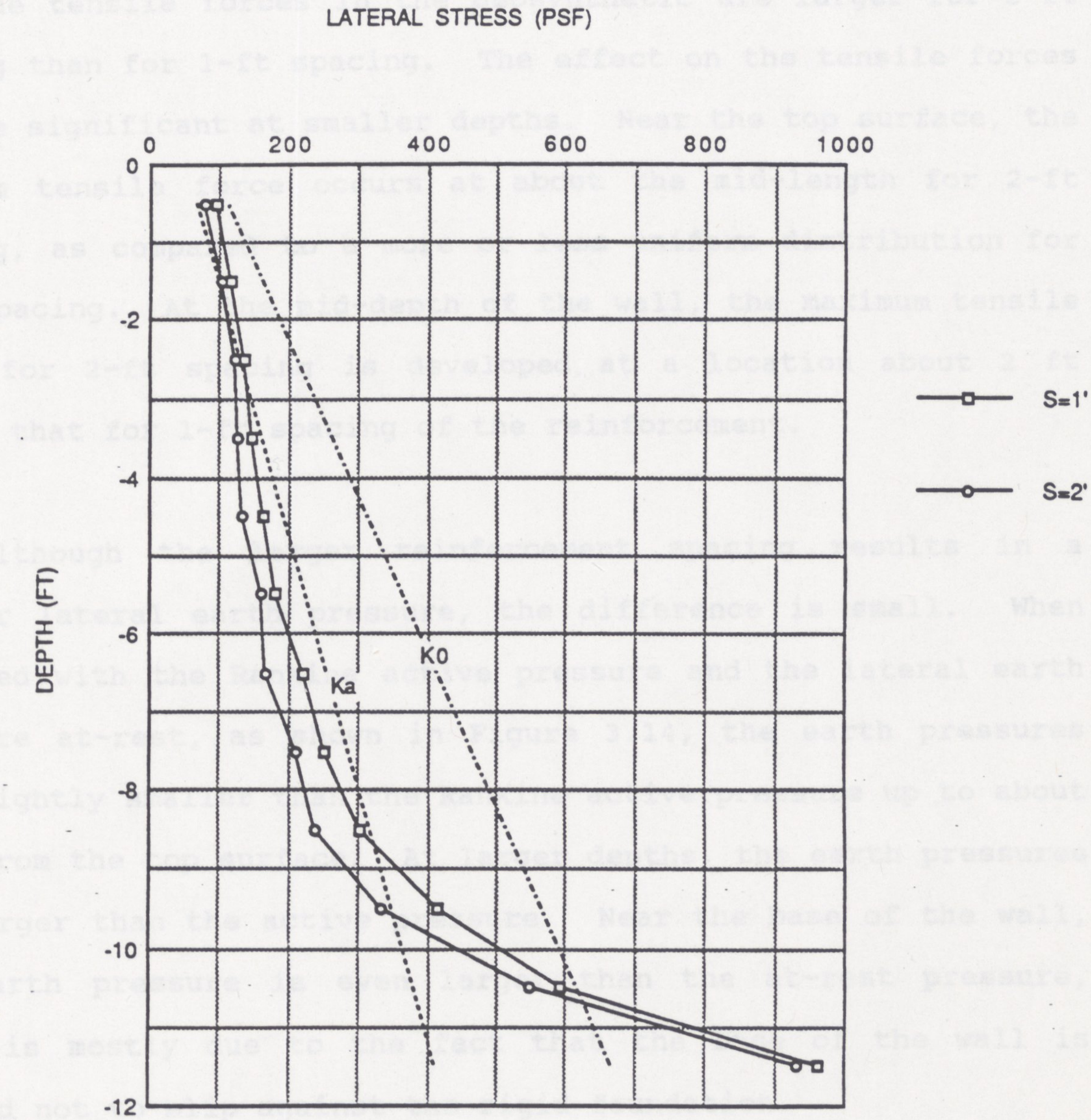


Figure 3.14 Effect of Geosynthetic Layer Spacing on Lateral Earth Pressure--Rigid Foundation

wall spacing is too large.

The tensile forces in the geosynthetic are larger for 2-ft spacing than for 1-ft spacing. The effect on the tensile forces is more significant at smaller depths. Near the top surface, the maximum tensile force occurs at about the mid-length for 2-ft spacing, as compared to a more or less uniform distribution for 1-ft spacing. At the mid-depth of the wall, the maximum tensile force for 2-ft spacing is developed at a location about 2 ft behind that for 1-ft spacing of the reinforcement.

Although the larger reinforcement spacing results in a smaller lateral earth pressure, the difference is small. When compared with the Rankine active pressure and the lateral earth pressure at-rest, as shown in Figure 3.14, the earth pressures are slightly smaller than the Rankine active pressure up to about 8 ft from the top surface. At larger depths, the earth pressures are larger than the active pressure. Near the base of the wall, the earth pressure is even larger than the at-rest pressure, which is mostly due to the fact that the base of the wall is assumed not to slip against the rigid foundation.

3.2.5 Effect of Surcharge

Figures 3.15, 3.16, and 3.17 show, respectively, the horizontal wall displacements, the tensile force distribution in the geosynthetic layers at 2 ft, 6 ft, and 10 ft deep, and the

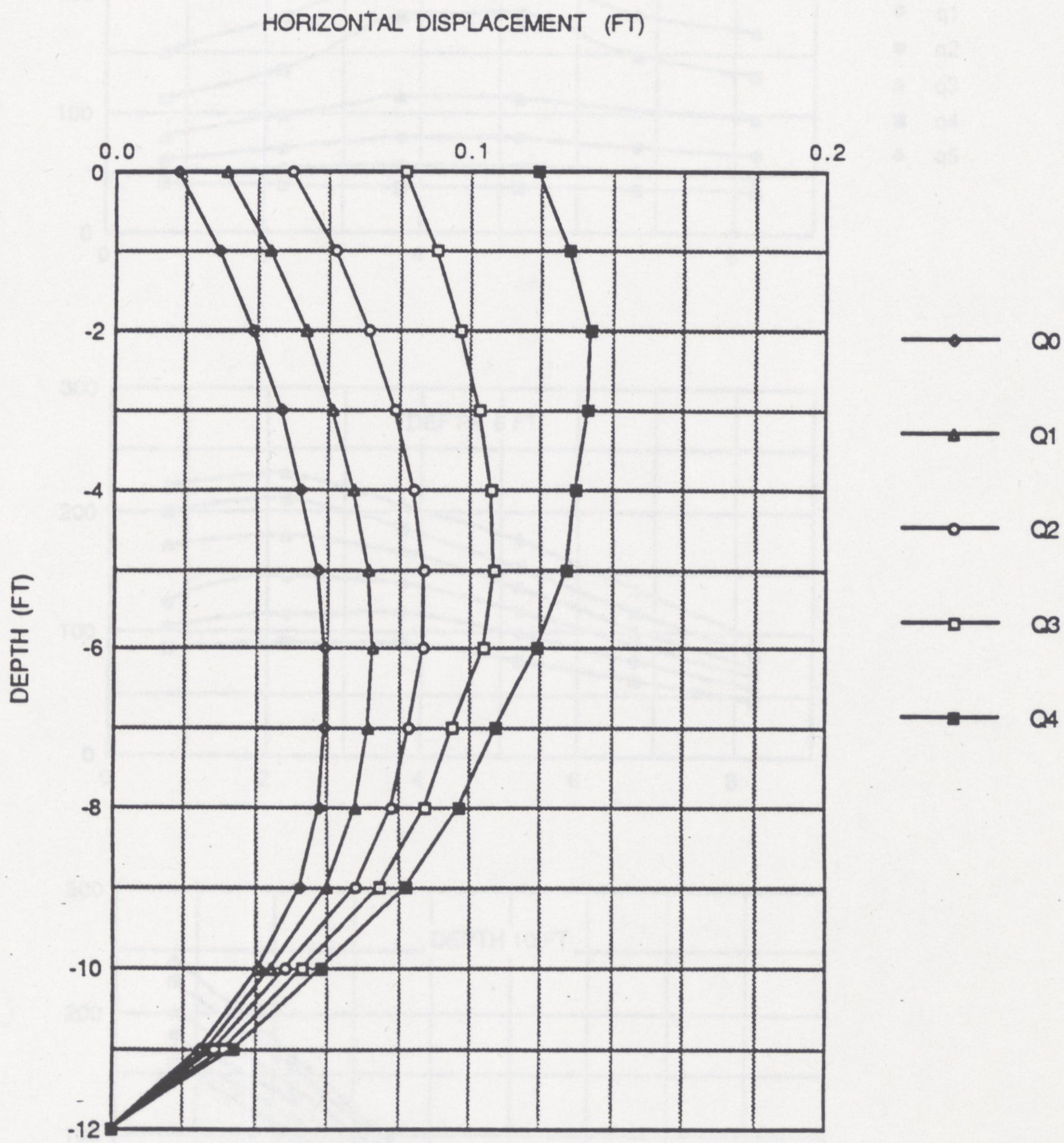


Figure 3.15 Effect of Surcharge on Horizontal Wall Displacement--Rigid Foundation

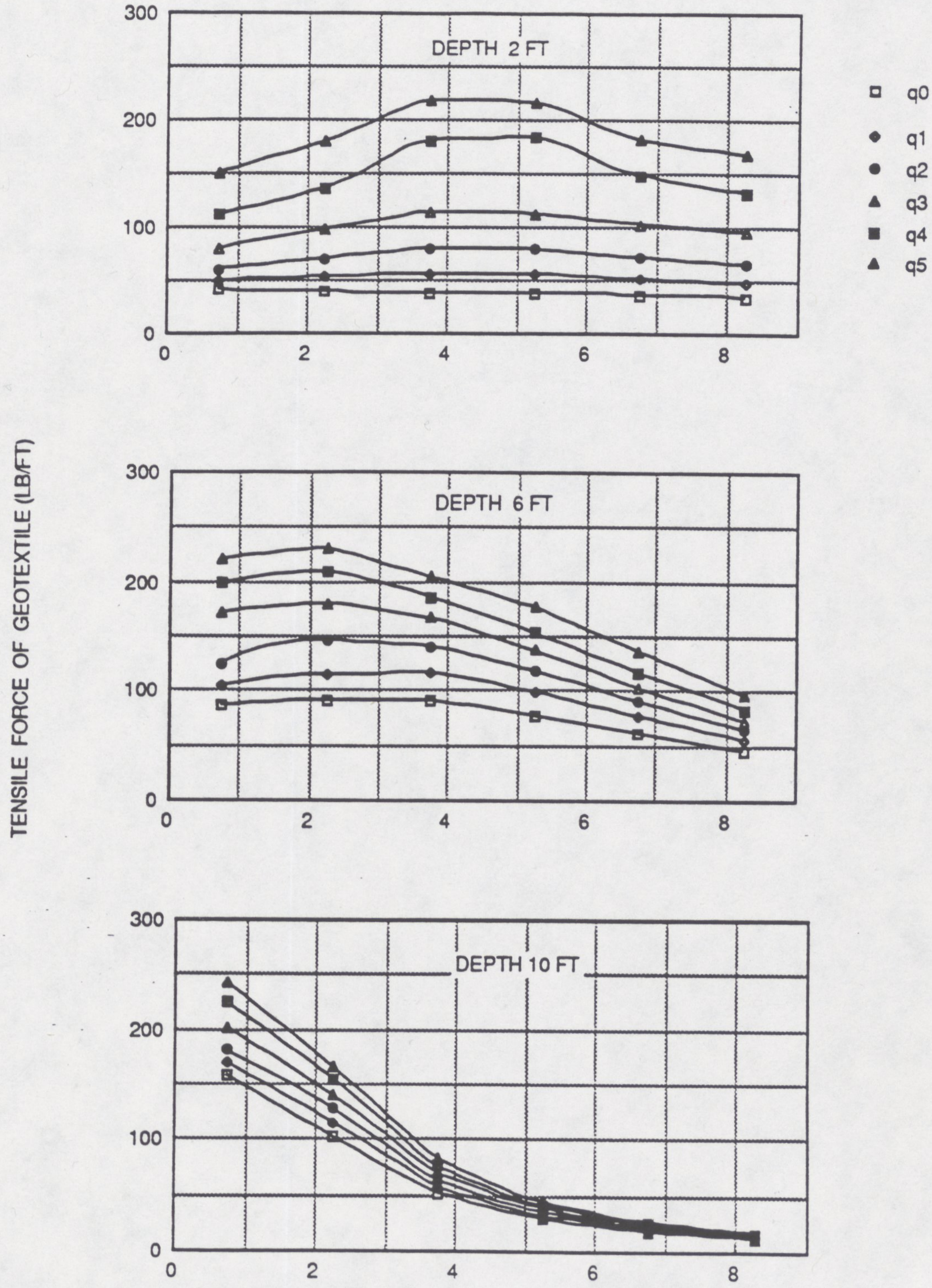


Figure 3.16 Effect of Surcharge on Geosynthetic Tensile Force--Rigid Foundation

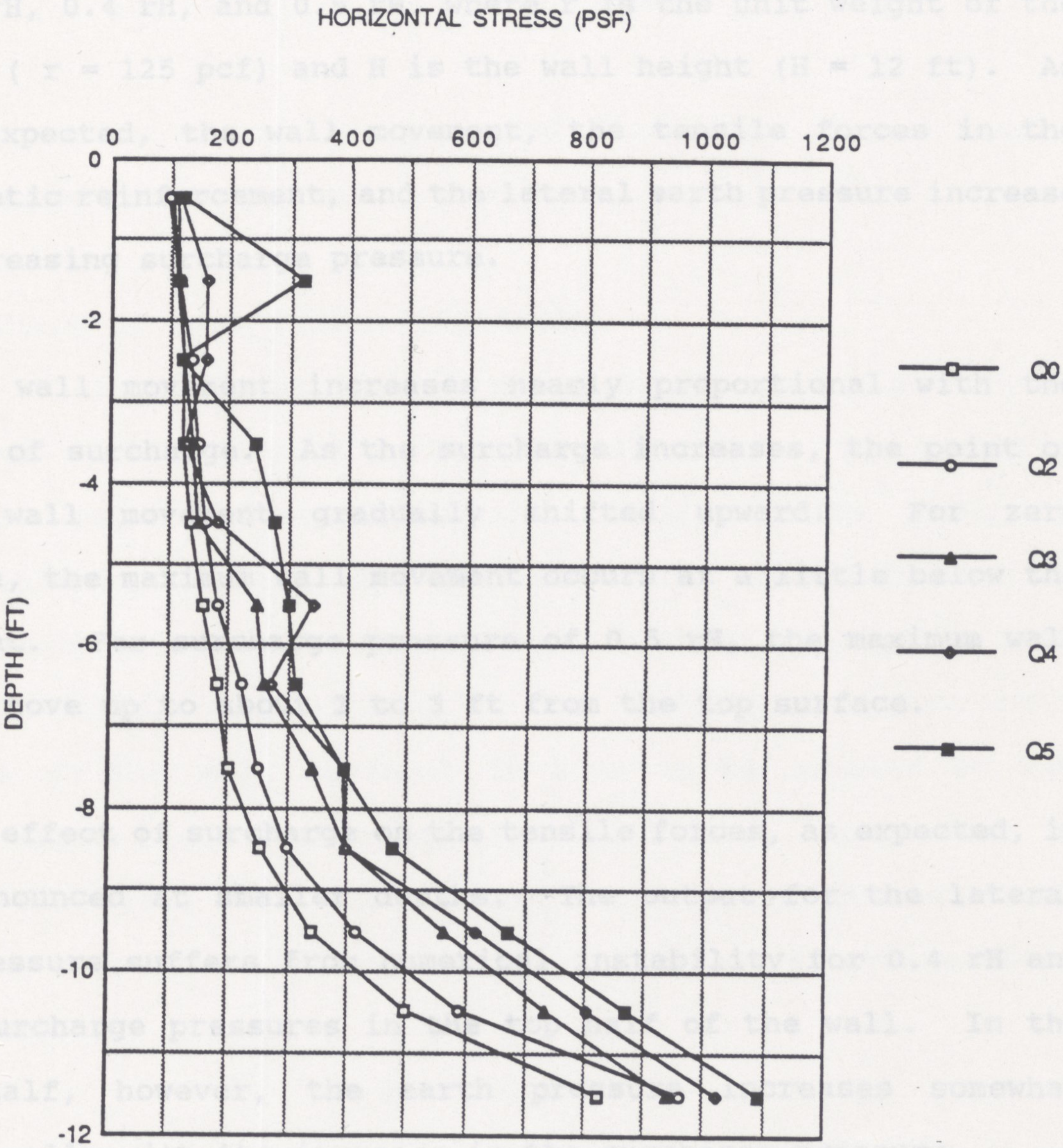


Figure 3.17 Effect of Surcharge on Lateral Earth Pressure--Rigid Foundation

lateral earth pressure distribution on the wall, as the uniform surcharge pressure on the wall surface varies from 0 to 0.1 rH, 0.2 rH, 0.3 rH, 0.4 rH, and 0.5 rH, where r is the unit weight of the backfill (r = 125 pcf) and H is the wall height (H = 12 ft). As may be expected, the wall movement, the tensile forces in the geosynthetic reinforcement, and the lateral earth pressure increase with increasing surcharge pressure.

The wall movement increases nearly proportional with the increase of surcharge. As the surcharge increases, the point of maximum wall movement gradually shifted upward. For zero surcharge, the maximum wall movement occurs at a little below the mid-height. For surcharge pressure of 0.5 rH, the maximum wall movement move up to about 2 to 3 ft from the top surface.

The effect of surcharge on the tensile forces, as expected, is more pronounced at smaller depths. The output for the lateral earth pressure suffers from numerical instability for 0.4 rH and 0.5 rH surcharge pressures in the top half of the wall. In the bottom half, however, the earth pressure increases somewhat proportionally with the increase in the surcharge pressure.

3.3 Geosynthetic Walls Constructed over Flexible Foundations

For walls constructed over a flexible foundation, namely, those associated with the Control Wall B, the wall performance as affected by the following three factors were investigated:

(a) the effect of geosynthetic stiffness,

(b) the effect of geosynthetic length, and

(c) the effect of foundation depth.

The results of the analyses and discussion of the results are presented in the following sections.

3.3.1 Effect of Geosynthetic Stiffness

The axial stiffness (EA) of the geosynthetic in the Control Wall B was varied to 0.5EA, 2EA, 5EA, and 10EA. The resulting horizontal wall displacements, the tensile forces along the geosynthetic layers at depths of 2 ft, 6 ft, and 10 ft, and the lateral earth pressure on the facing are shown in Figures 3.18, 3.19, and 3.20, respectively. The effect of geosynthetic stiffness on the wall movement is seen to be similar to that obtained for a rigid foundation (Section 3.2.2), except that the base of the wall for the Control Wall B deforms with the underlying foundation and results in significant lateral movement at the base.

The effect of geosynthetic stiffness on the tensile forces in the reinforcement is also similar to that for a rigid foundation, except that the forces in the geosynthetic near the base do not reduce to as small magnitudes as those for the rigid foundation.

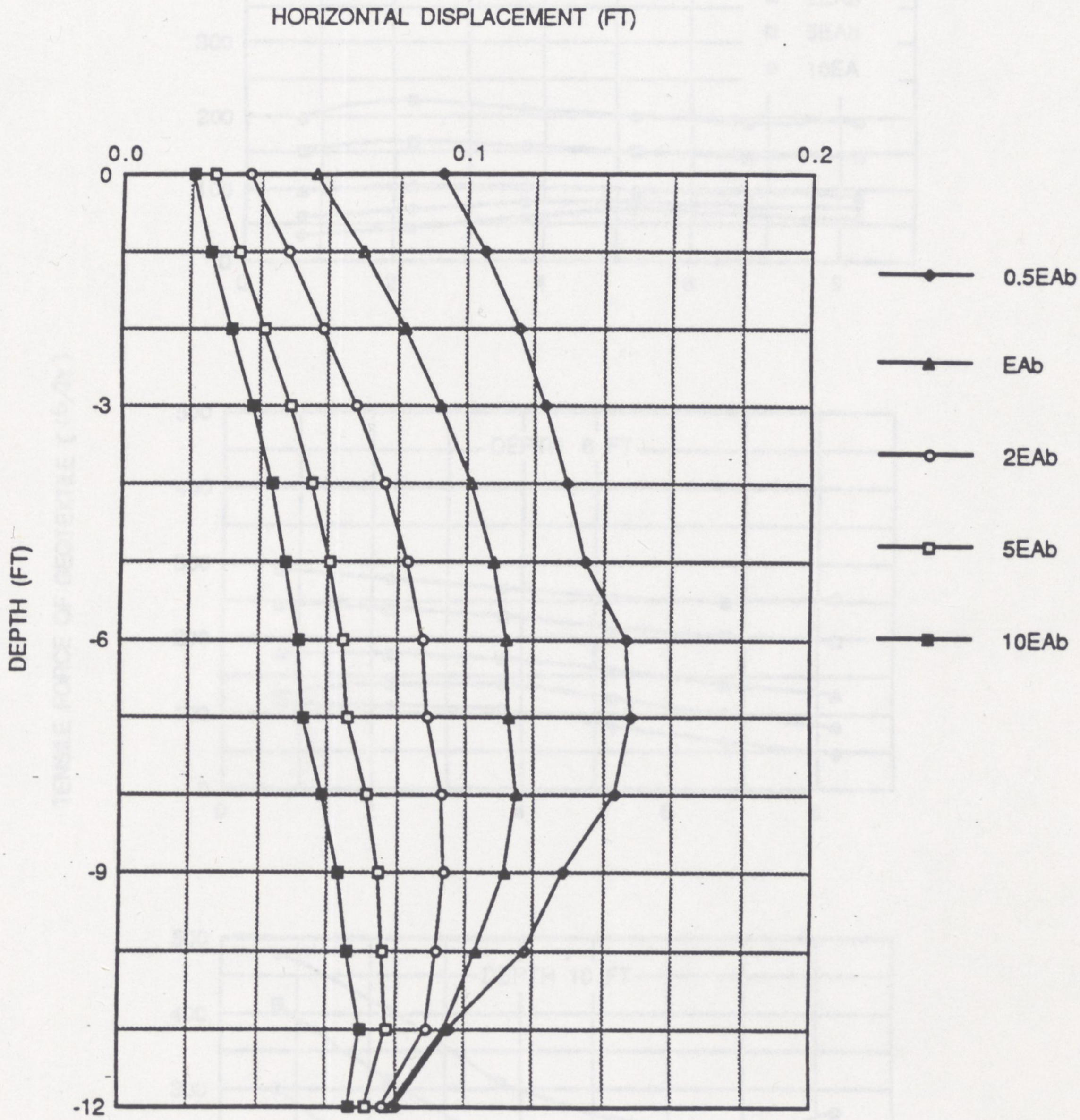


Figure 3.18 **Effect of Geosynthetic Stiffness on Horizontal Wall Displacement--Flexible Foundation**

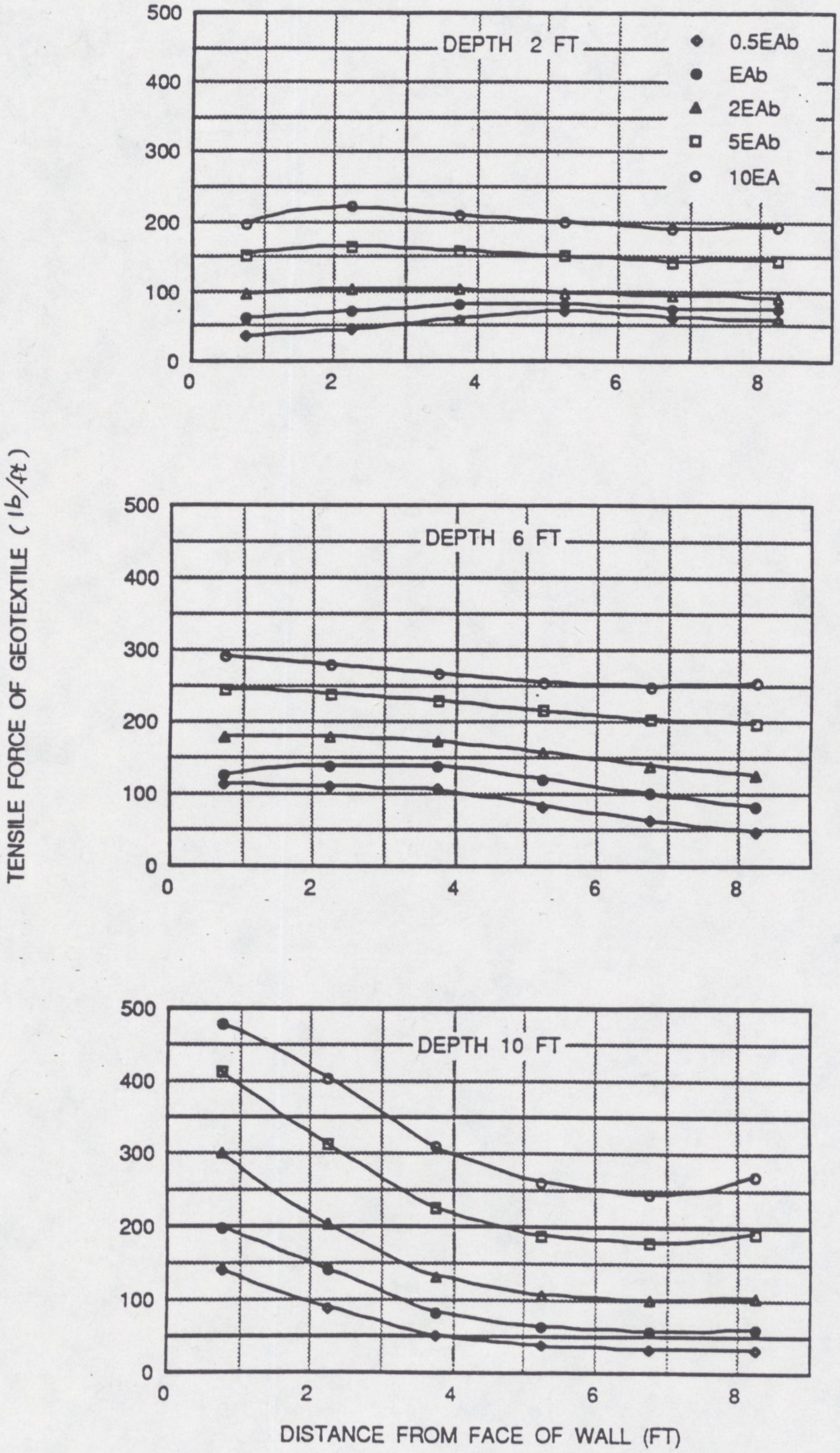


Figure 3.19 Effect of Geosynthetic Stiffness on Geosynthetic Tensile Force--Flexible Foundation

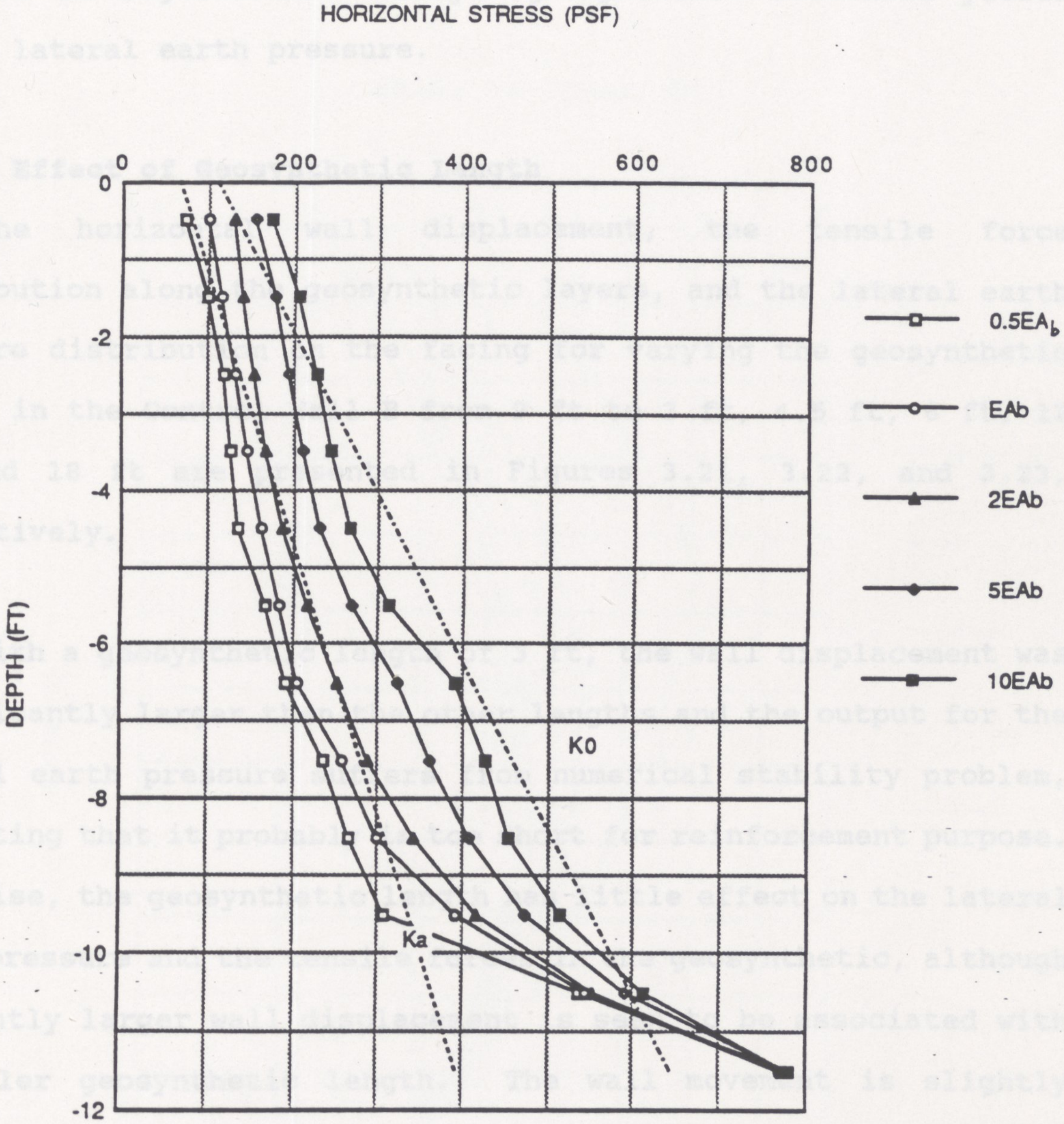


Figure 3.20 Effect of Geosynthetic Stiffness on Lateral Earth Pressure--Flexible Foundation

The effect of geosynthetic stiffness on the lateral earth pressure is significant. Larger geosynthetic stiffness yields larger lateral earth pressure.

3.3.2 Effect of Geosynthetic Length

The horizontal wall displacement, the tensile force distribution along the geosynthetic layers, and the lateral earth pressure distribution on the facing for varying the geosynthetic length in the Control Wall B from 9 ft to 3 ft, 4.5 ft, 6 ft, 12 ft, and 18 ft are presented in Figures 3.21, 3.22, and 3.23, respectively.

With a geosynthetic length of 3 ft, the wall displacement was significantly larger than the other lengths and the output for the lateral earth pressure suffers from numerical stability problem, indicating that it probably is too short for reinforcement purpose. Otherwise, the geosynthetic length has little effect on the lateral earth pressure and the tensile forces in the geosynthetic, although a slightly larger wall displacement is seen to be associated with a smaller geosynthetic length. The wall movement is slightly larger than that associated with a rigid foundation (Figure 3.9).

3.3.3 Effect of Foundation Depth

The depth of the foundation in the Control Wall B was

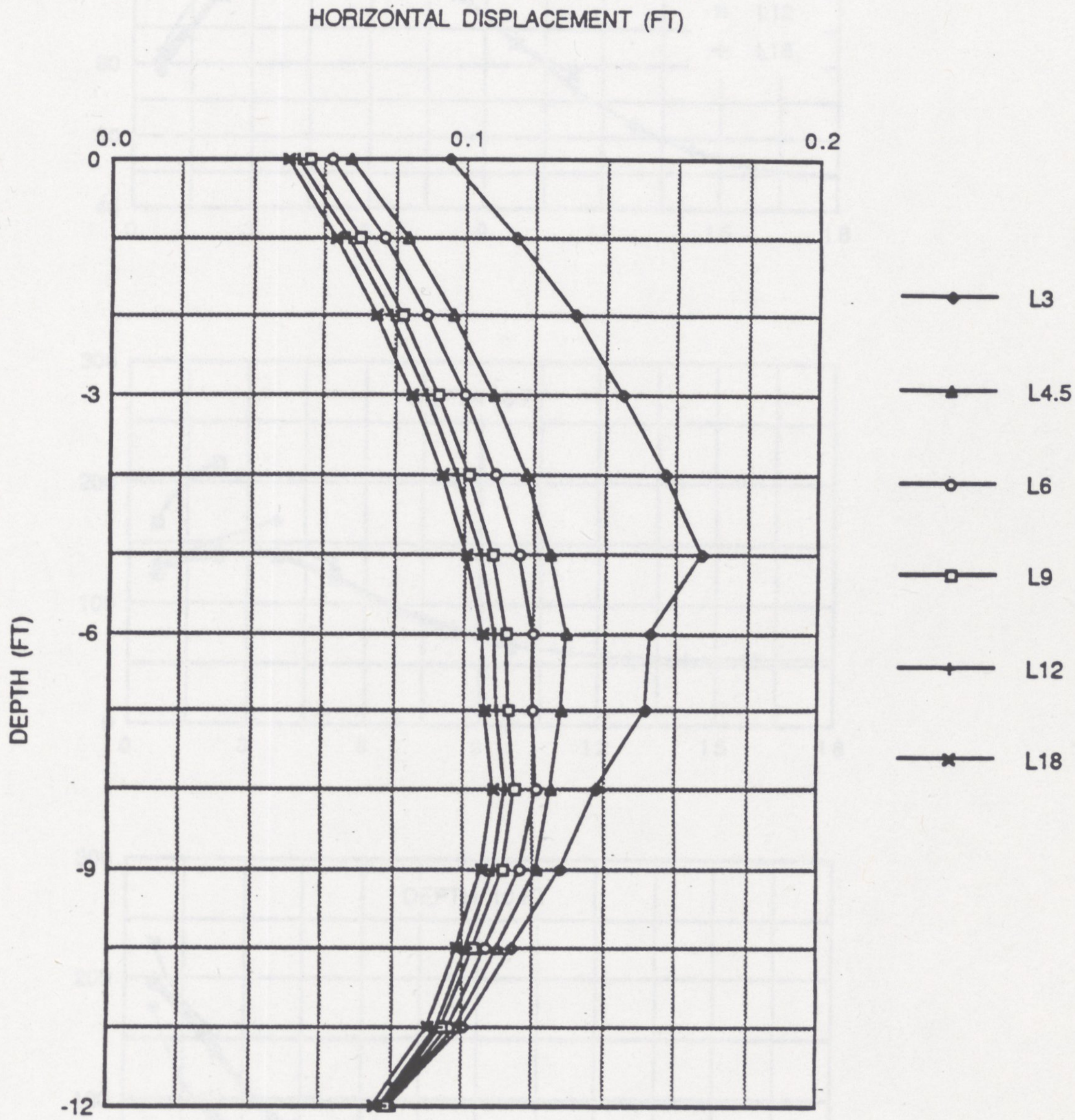


Figure 3.21 Effect of Geosynthetic Length on Horizontal Wall Displacement--Flexible Foundation

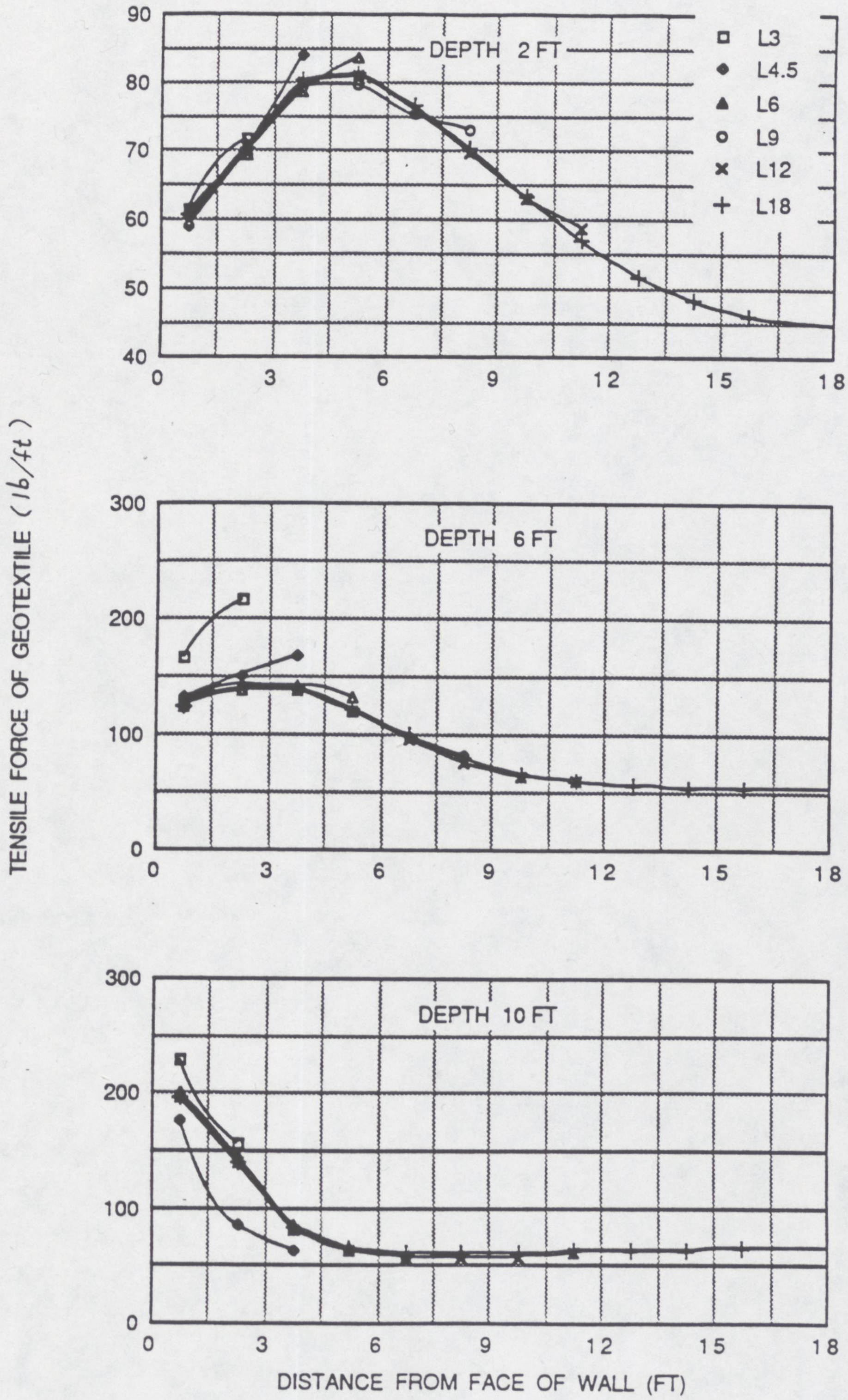


Figure 3.22 Effect of Geosynthetic Length on Geosynthetic Tensile Force--Flexible Foundation

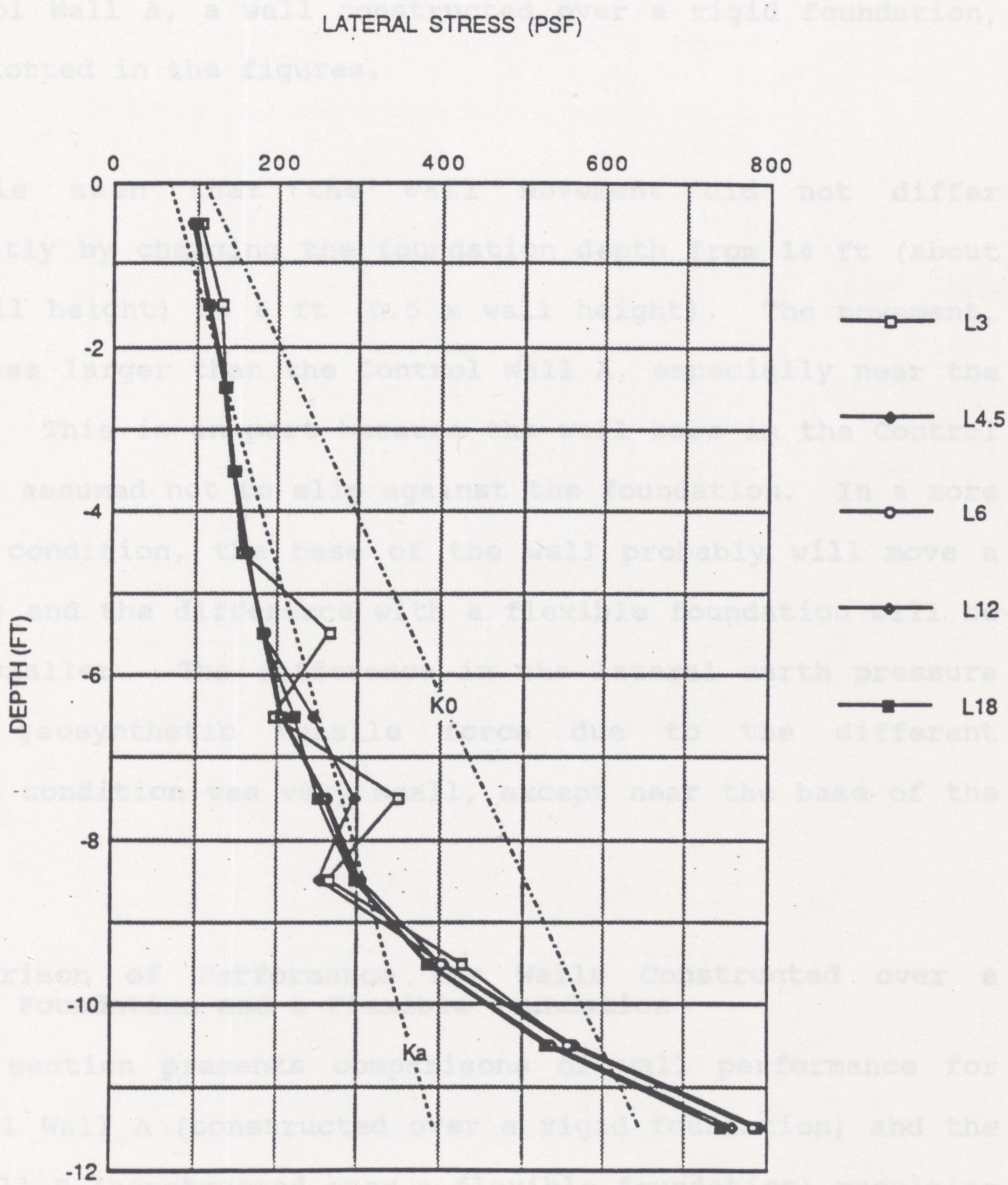


Figure 3.23 Effect of Geosynthetic Length on Lateral Earth Pressure--Flexible Foundation

changed from 14 ft to 6 ft and the results are shown in Figures 3.24, 3.25, and 3.26. For comparison purposes, the results of the Control Wall A, a wall constructed over a rigid foundation, is also plotted in the figures.

It is seen that the wall movement did not differ significantly by changing the foundation depth from 14 ft (about 1.15 x wall height) to 6 ft (0.5 x wall height). The movement, however, was larger than the Control Wall A, especially near the wall base. This is in part because the wall base in the Control Wall A was assumed not to slip against the foundation. In a more realistic condition, the base of the wall probably will move a few inches and the difference with a flexible foundation will be slightly smaller. The difference in the lateral earth pressure and the geosynthetic tensile force due to the different foundation condition was very small, except near the base of the walls.

3.4 Comparison of Performance for Walls Constructed over a Rigid Foundation and a Flexible Foundation

This section presents comparisons of wall performance for the Control Wall A (constructed over a rigid foundation) and the Control Wall B (constructed over a flexible foundation) resulting from differences in (1) the geosynthetic stiffness and (2) the geosynthetic length.

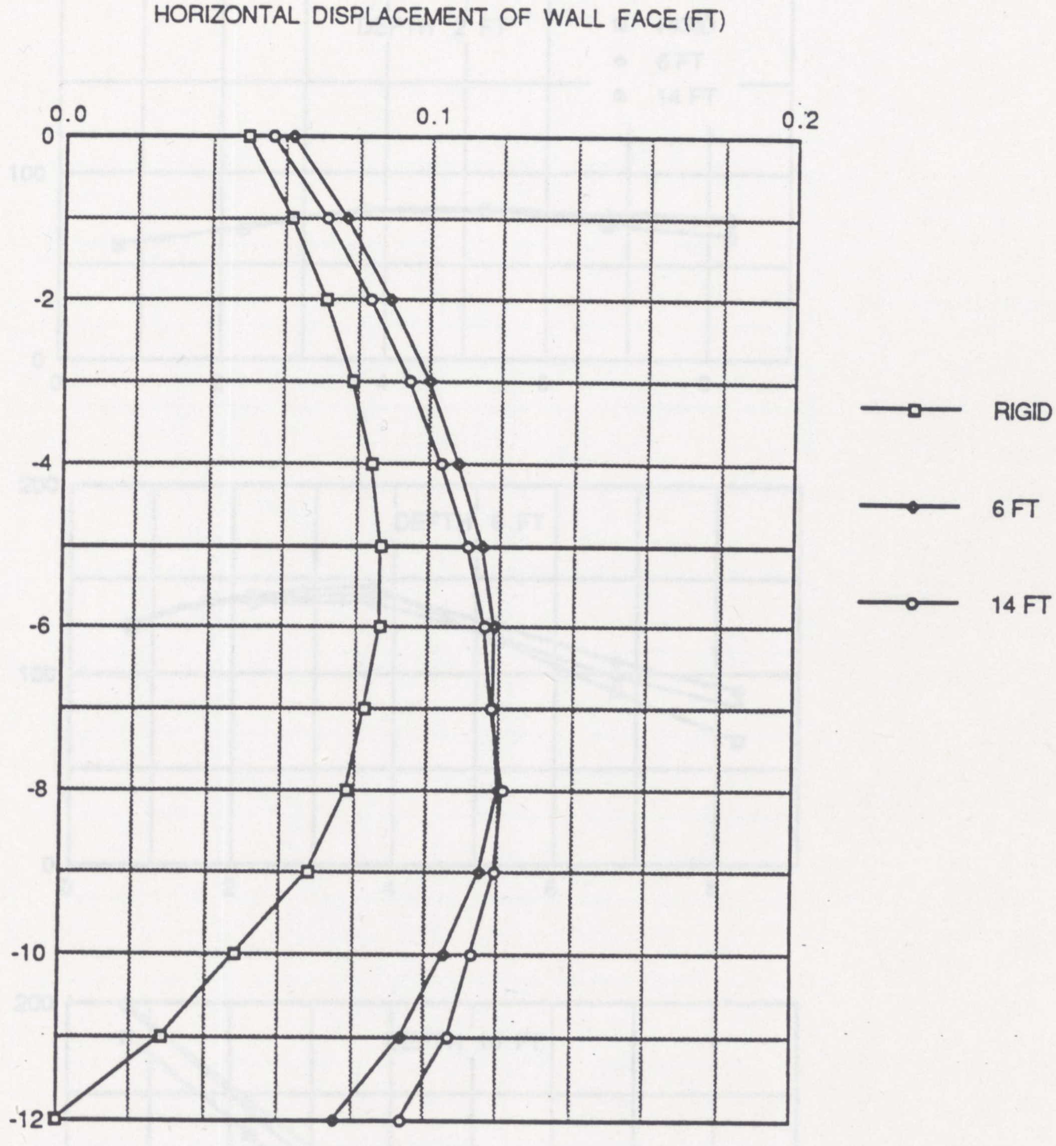


Figure 3.24 Effect of Foundation Depth on Horizontal Wall Displacement--Flexible Foundation

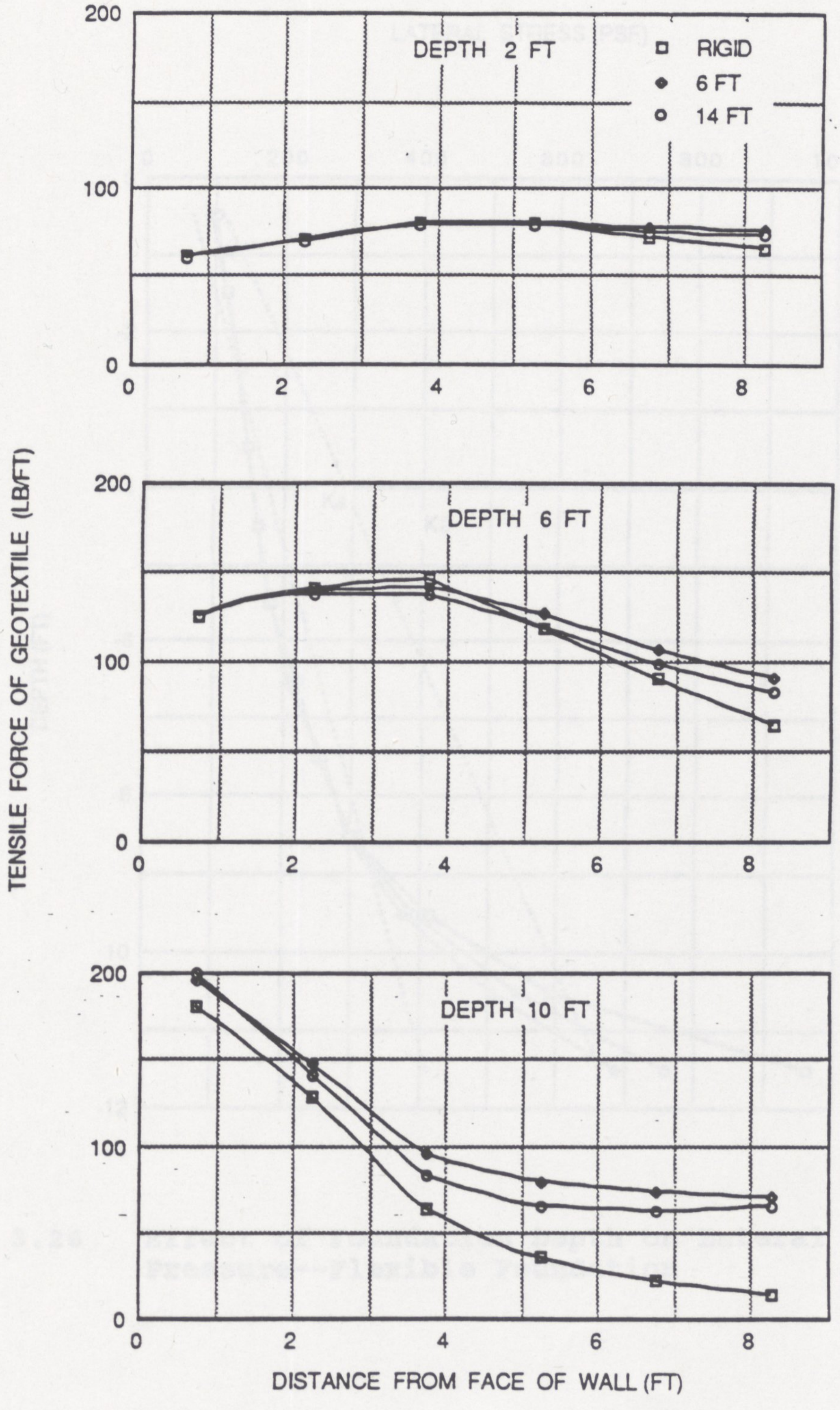


Figure 3.25 Effect of Foundation Depth on Geosynthetic Tensile Force--Flexible Foundation

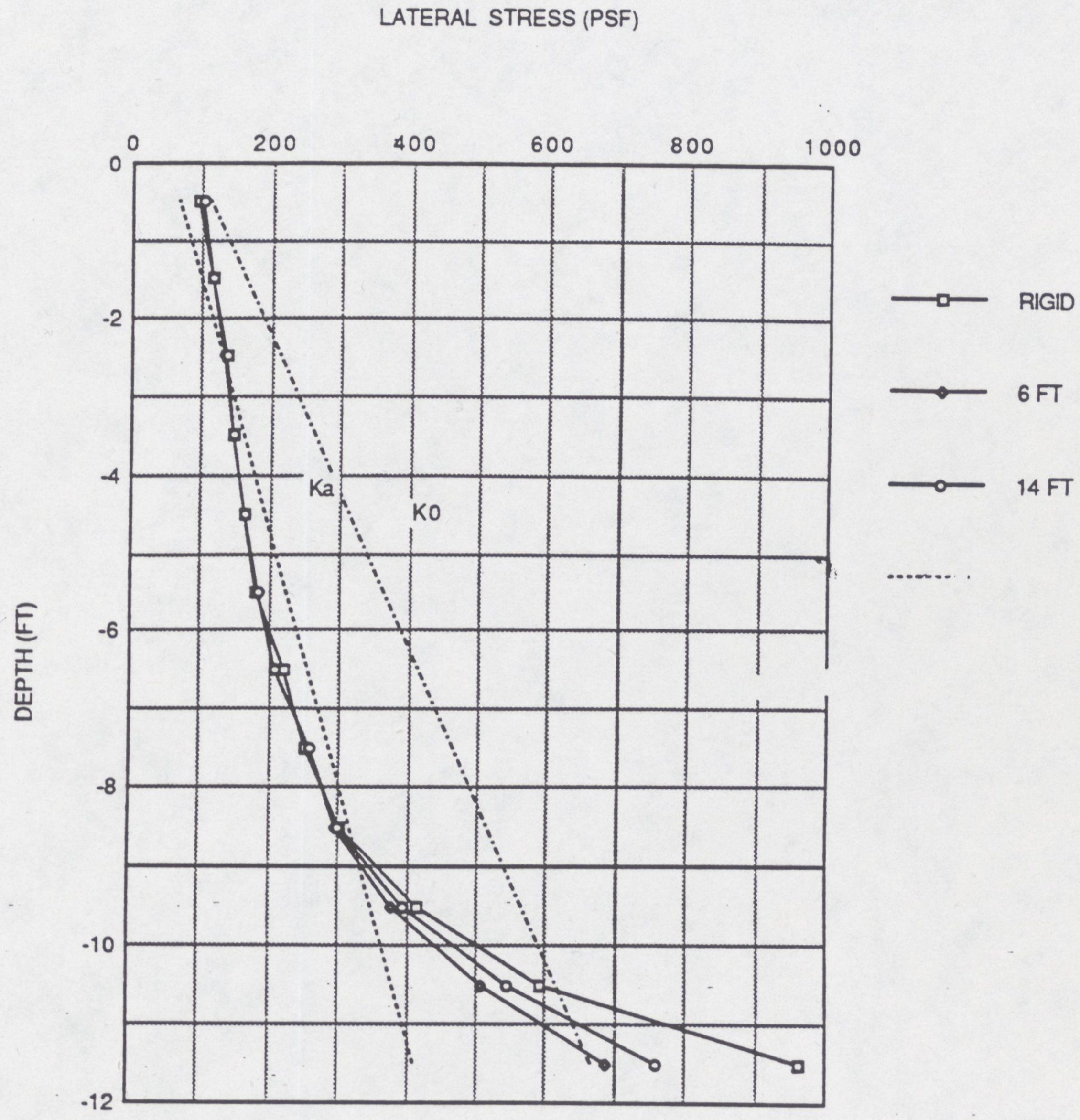


Figure 3.26 Effect of Foundation Depth on Lateral Earth Pressure--Flexible Foundation

3.4.1 Effect of Geosynthetic Stiffness

The axial stiffness of the geosynthetic in the Control Walls A and B was varied from 0.5 EA to 10 EA, where EA = 1.607×10^4 lb/ft. The effects on the following quantities for the two control walls are plotted:

(a) the horizontal wall displacement, see Figure 3.27,

(b) the tensile force distribution in the geosynthetic layers at the depths of 2 ft, 6 ft, and 10 ft, see Figure 3.28,

(c) the lateral earth pressure distribution, see Figure 3.29,

(d) the maximum horizontal wall displacement, see Figure 3.30,

(e) the maximum tensile force in the geotextile, see Figure 3.31.

As seen in Figure 3.27, the Control Wall B induced larger wall movements than the Control Wall A for all the geosynthetic axial stiffnesses investigated. Both walls show an increase of wall movement with decreasing axial stiffness of the geosynthetic. The increases in the horizontal wall movement for the two control walls as shown in Figure 3.27 were similar. The increase of the axial stiffness from 0.5 EA to EA, however, was more "effective," compared to the increase from EA to 10 EA.

Figure 3.28 indicates that the tensile forces developed in

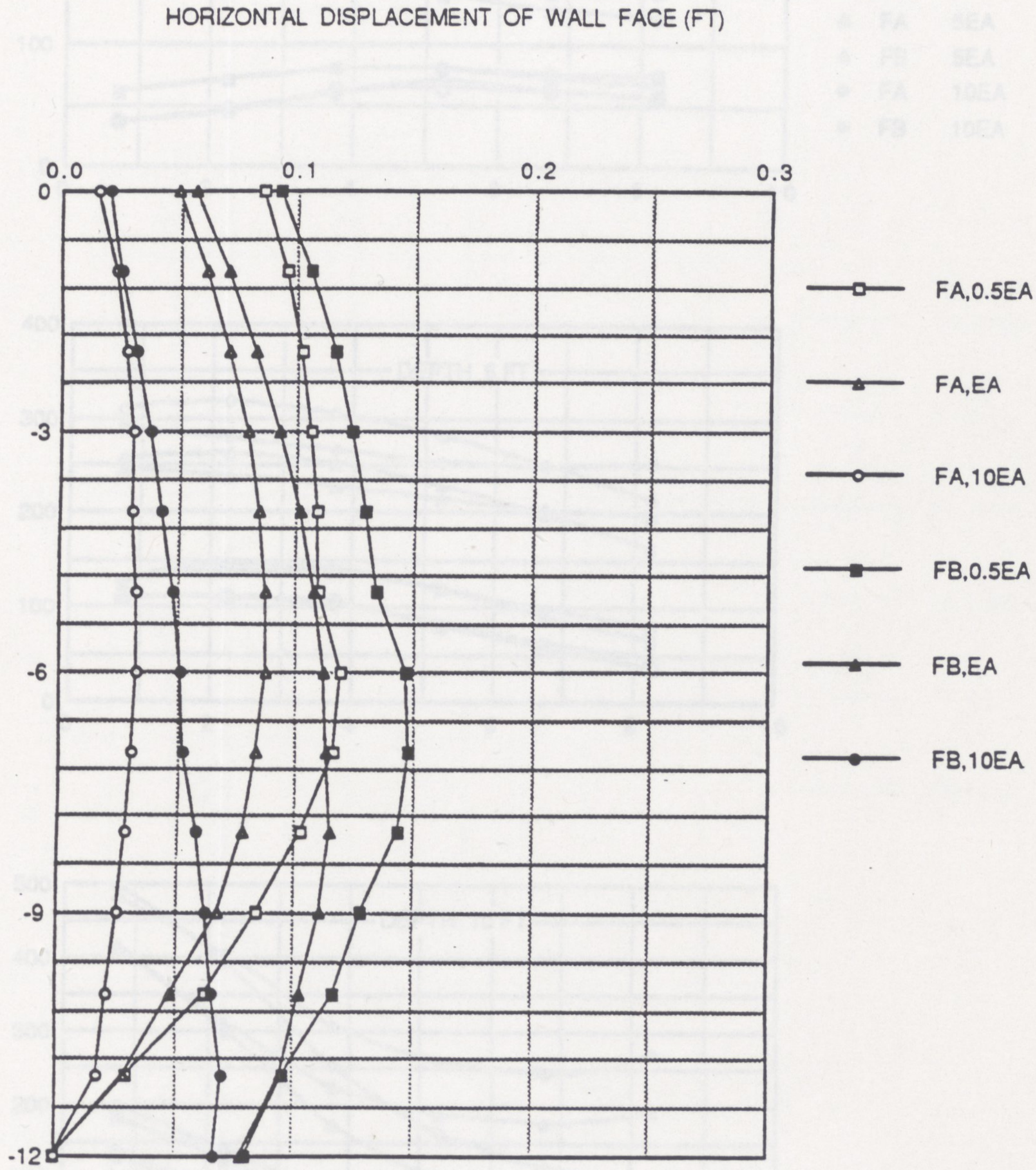


Figure 3.27 Effect of Geosynthetic Stiffness on Horizontal Wall Displacement

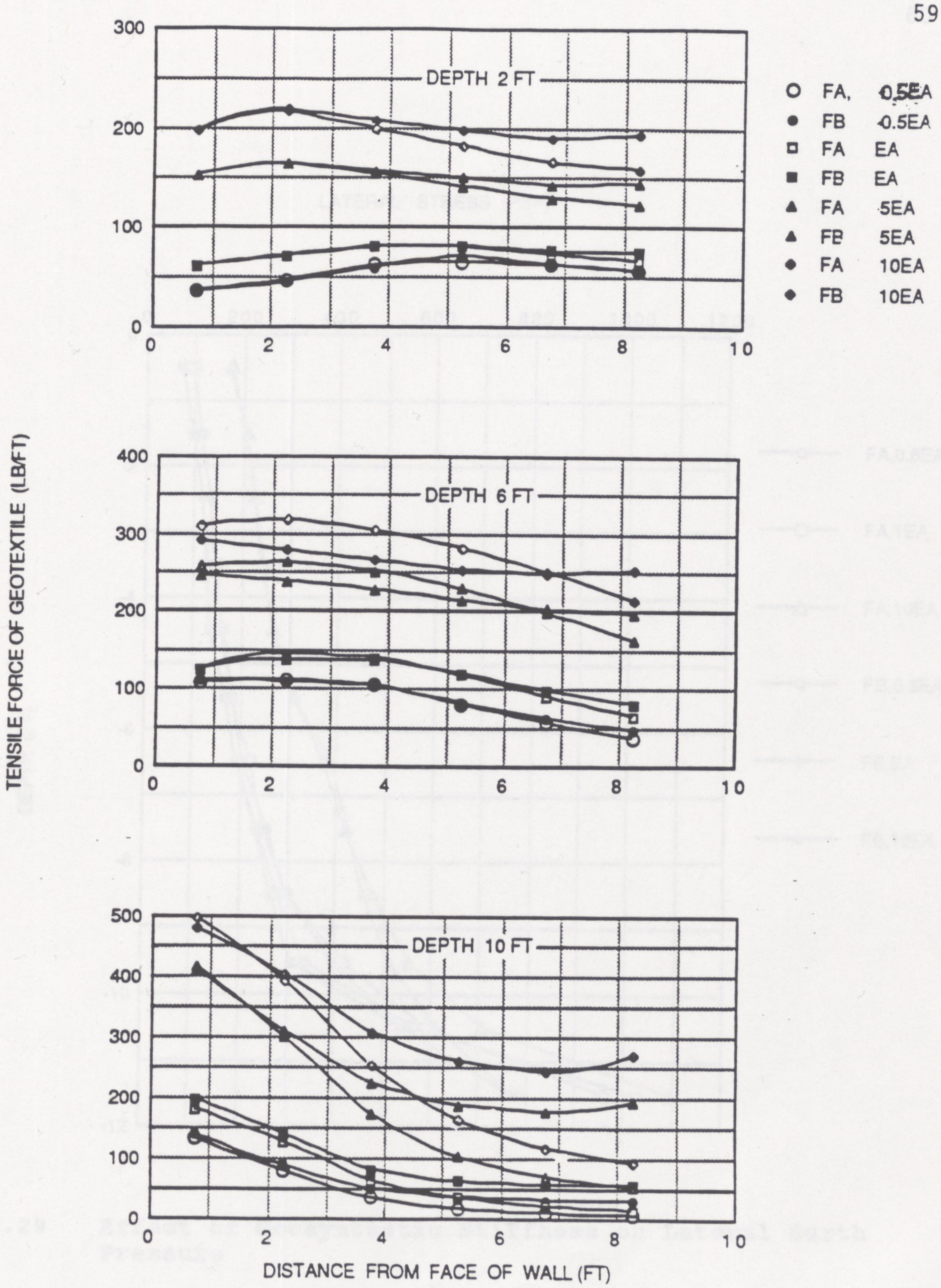


Figure 3.28 Effect of Geosynthetic Stiffness on Geosynthetic Tensile Force

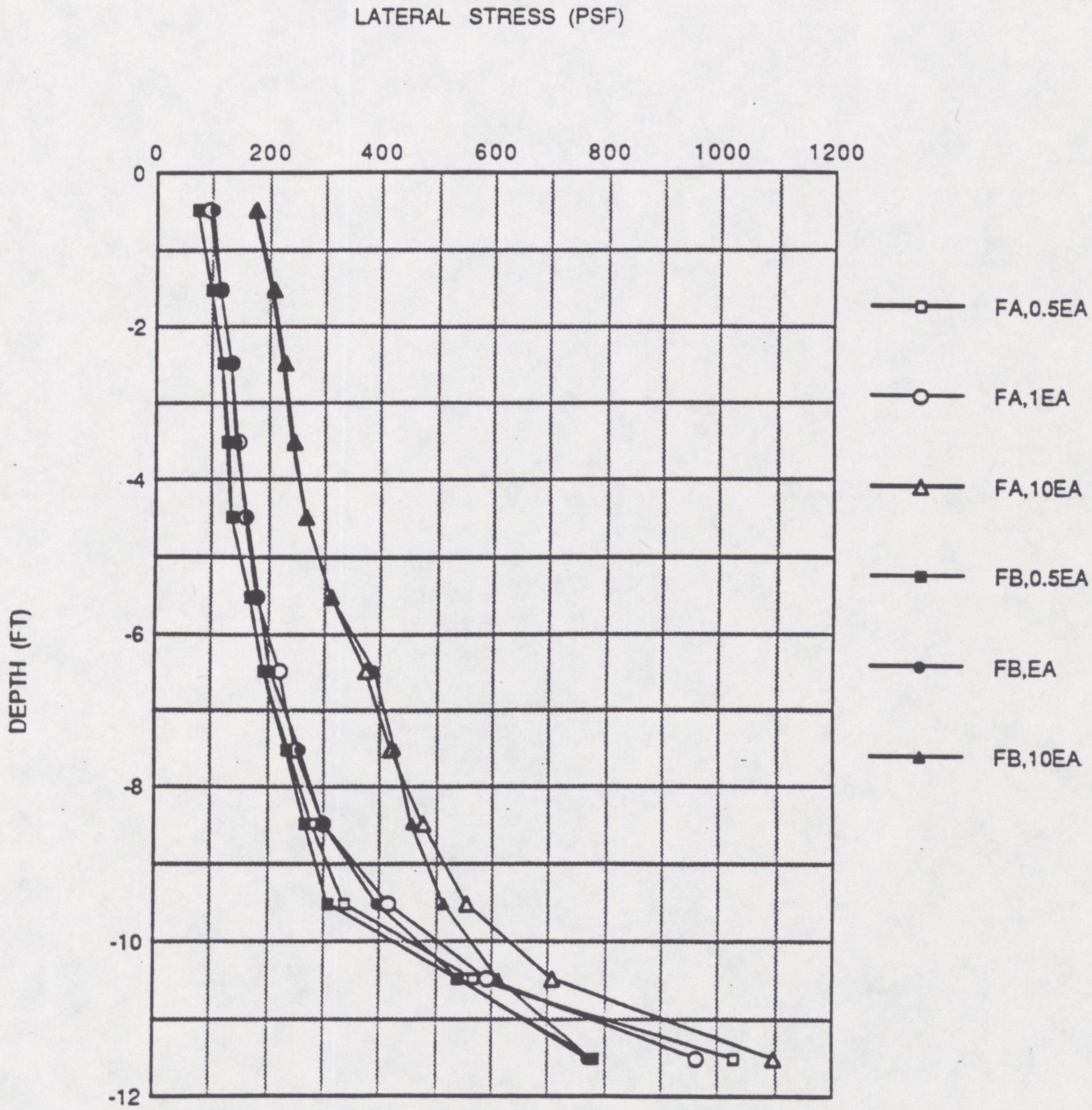


Figure 3.29 Effect of Geosynthetic Stiffness on Lateral Earth Pressure

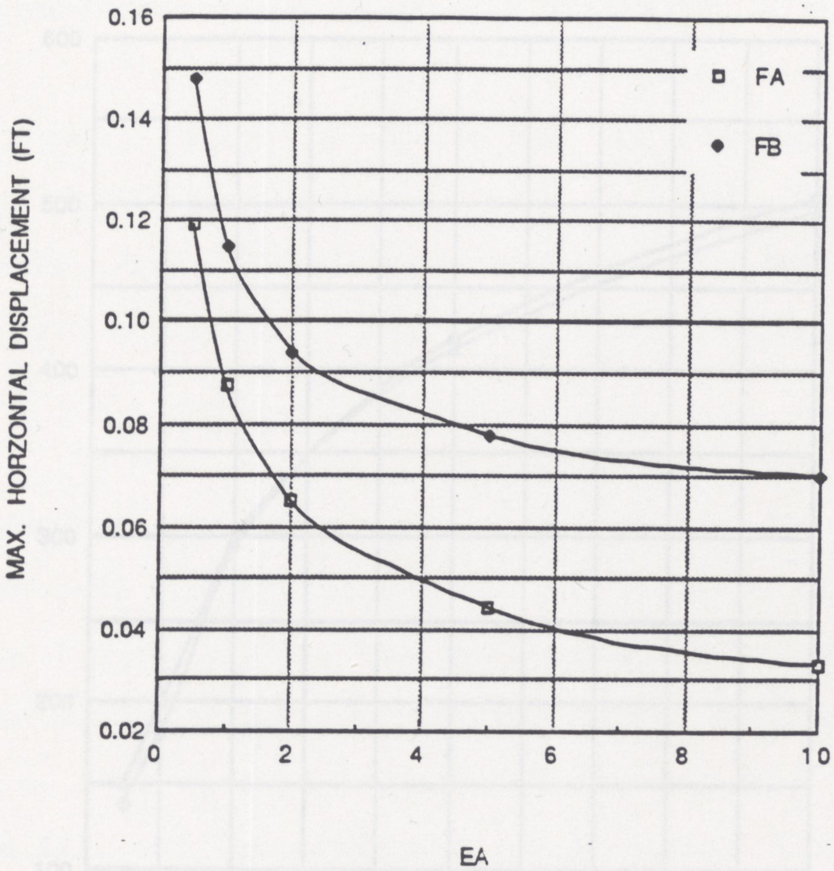


Figure 3.30 Effect of Geosynthetic Stiffness on Maximum Horizontal Wall Displacement

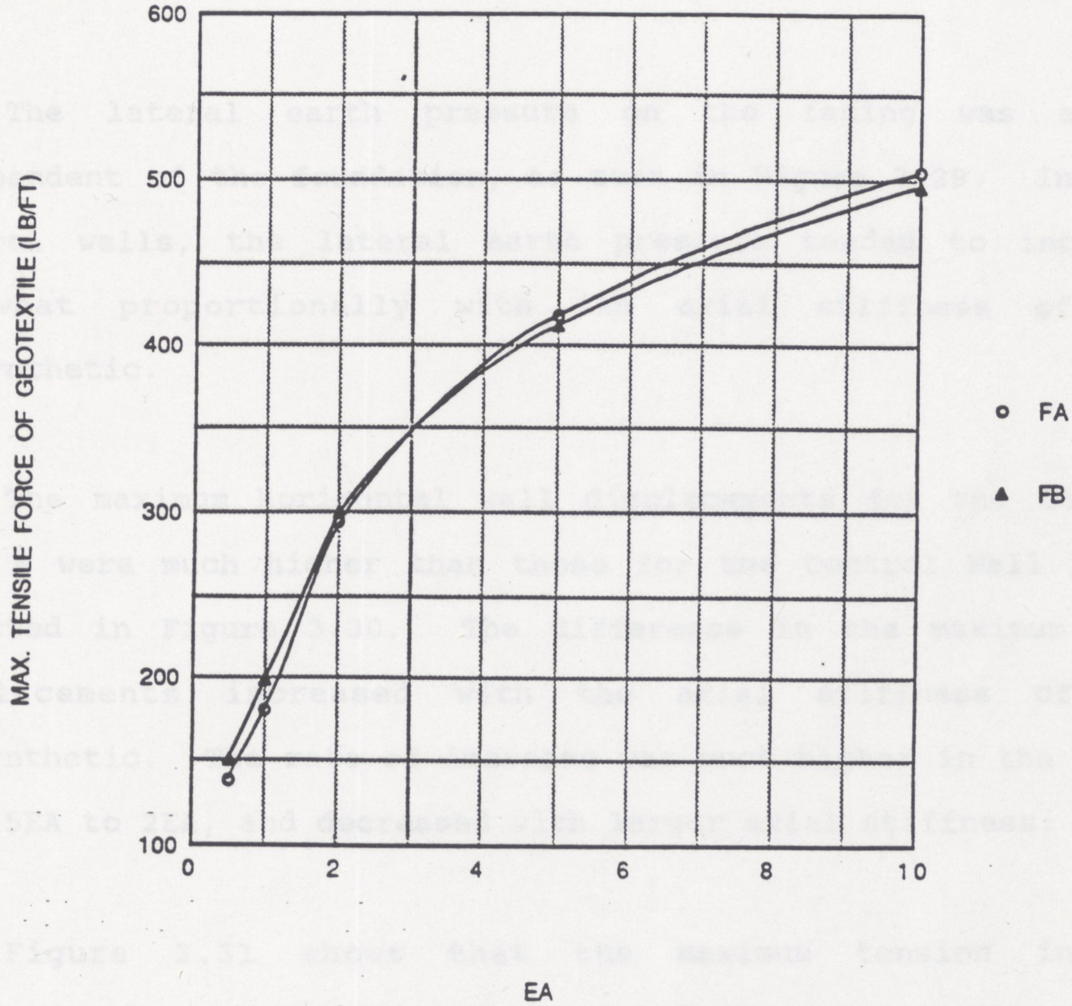


Figure 3.31 Effect of Geosynthetic Stiffness on Maximum Tensile Force in the Geosynthetic

the two control walls are not affected by the foundation except when the axial stiffness is larger (5EA and 10EA), wherein the effect was more pronounced at greater depths.

The lateral earth pressure on the facing was almost independent of the foundation, as seen in Figure 3.29. In both control walls, the lateral earth pressure tended to increase somewhat proportionally with the axial stiffness of the geosynthetic.

The maximum horizontal wall displacements for the Control Wall B were much higher than those for the Control Wall A, as depicted in Figure 3.30. The difference in the maximum wall displacements increased with the axial stiffness of the geosynthetic. The rate of increase was much higher in the range of 0.5EA to 2EA, and decreased with larger axial stiffness.

Figure 3.31 shows that the maximum tension in the geosynthetic was not affected by the foundation. As the axial stiffness of the geosynthetic increased, the maximum tension for both control walls increased in a nonlinear manner that resembles a hyperbolic function.

3.4.2 Effect of Geosynthetic Length

The length of the geosynthetic reinforcement in Control Walls A and B was varied from 3 ft to 18 ft. The effects on the following quantities for the two control walls were plotted:

(a) the horizontal wall displacement, see Figure 3.32,

(b) the tensile force distribution in the geosynthetic layers at the depths of 2 ft, 6 ft, and 10 ft, see Figure 3.33,

(c) the lateral earth pressure distribution, see Figure 3.34,

(d) the maximum horizontal wall displacement, see Figure 3.35,

(e) the maximum tensile force in the geotextile, see Figure 3.36.

Figure 3.32 shows that the wall movement was significantly affected by the foundation for all the geosynthetic lengths. Longer geosynthetic resulted in smaller wall movement. The manner by which the wall movement was affected by the geosynthetic length for the two control walls was similar.

It is seen from Figure 3.33 that the effect of foundation on the tensile force in the geosynthetic was more apparent at larger depths. The effect of geosynthetic length was very small.

The lateral earth pressures plotted in Figure 3.34 are seen to be somewhat independent of the foundation and the geosynthetic length, except near the base of the walls where longer geosynthetic resulted in smaller earth pressure.

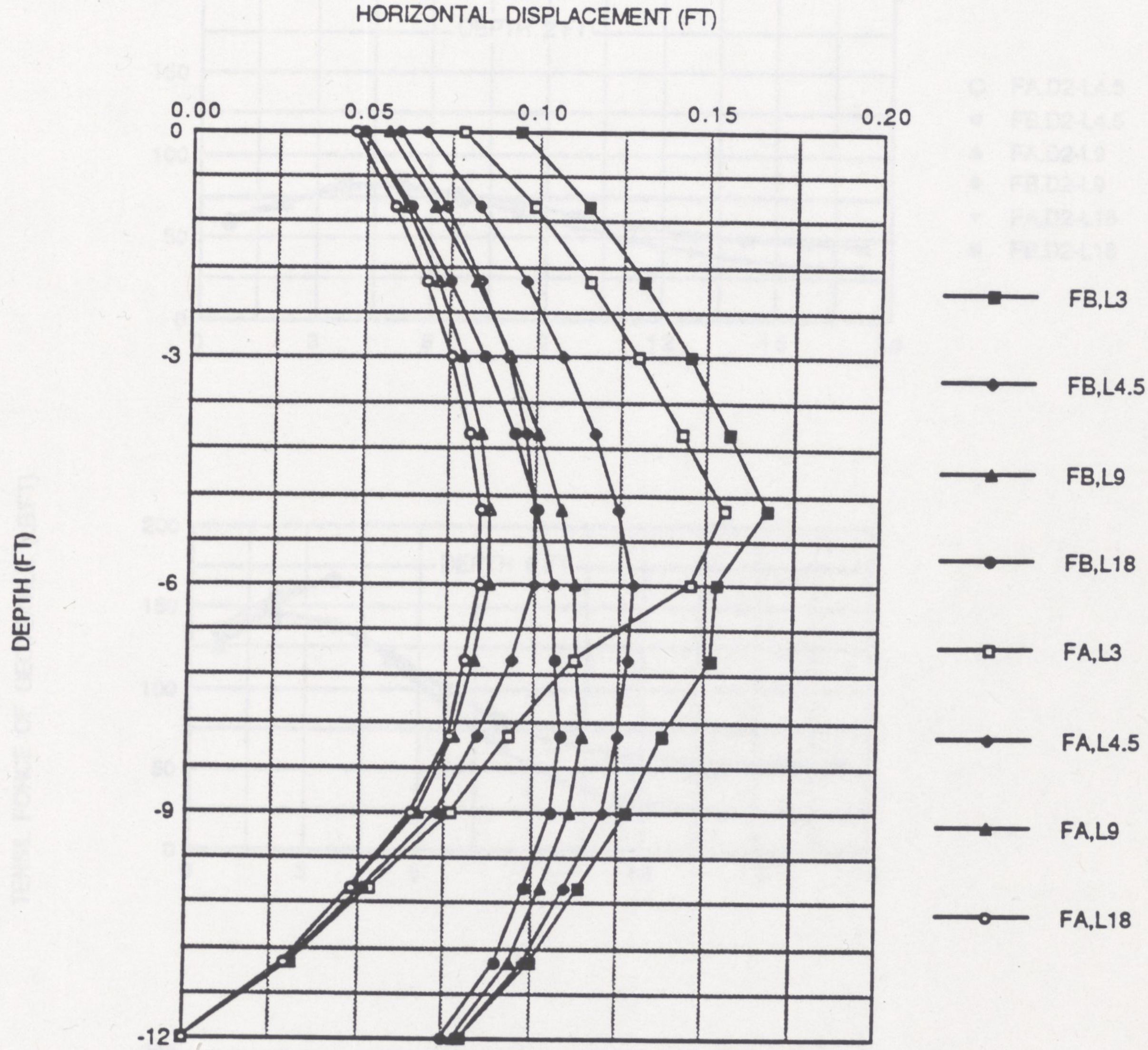


Figure 3.32 **Effect of Geosynthetic Length on Horizontal Wall Displacement**

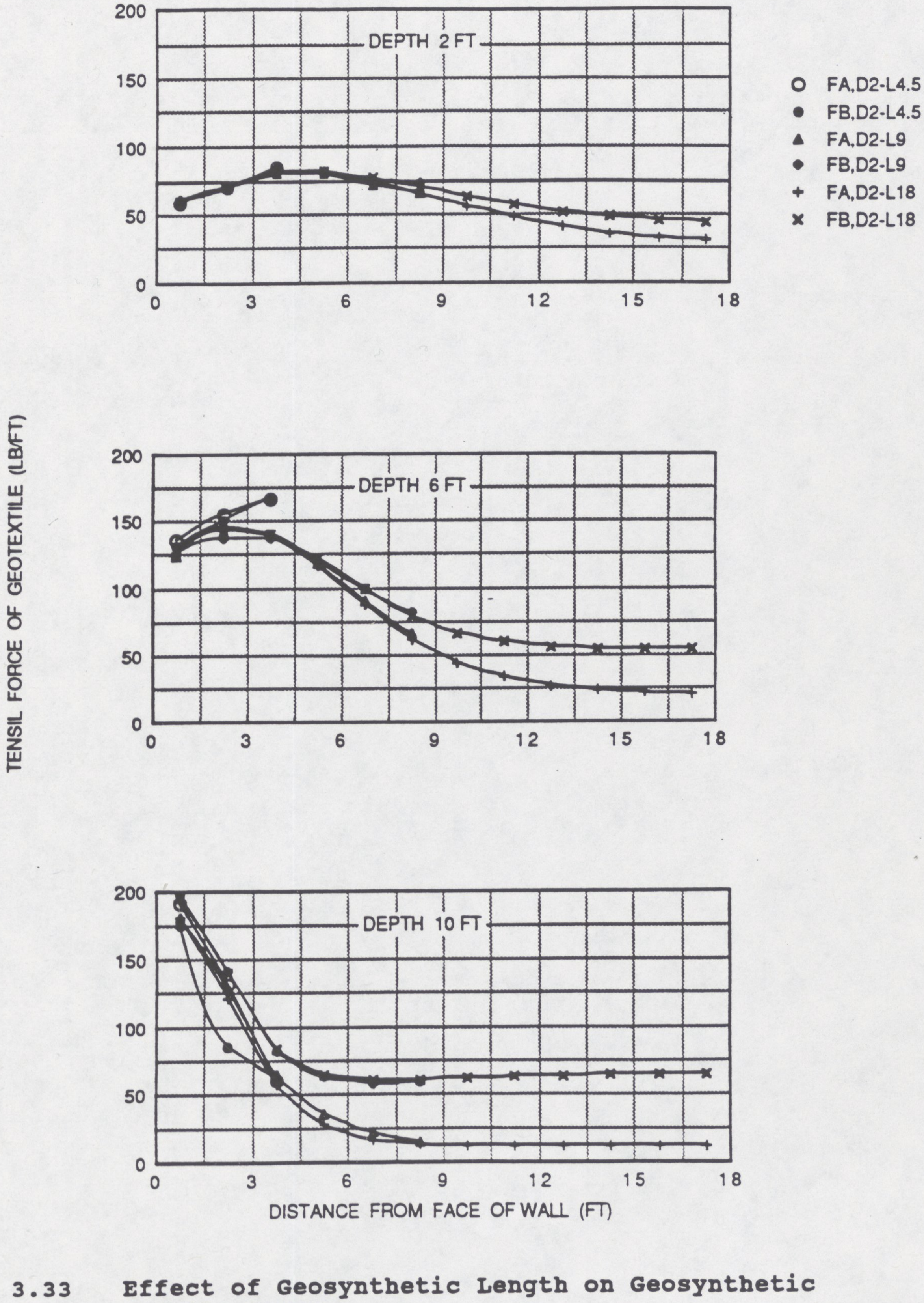


Figure 3.33 Effect of Geosynthetic Length on Geosynthetic Tensile Force

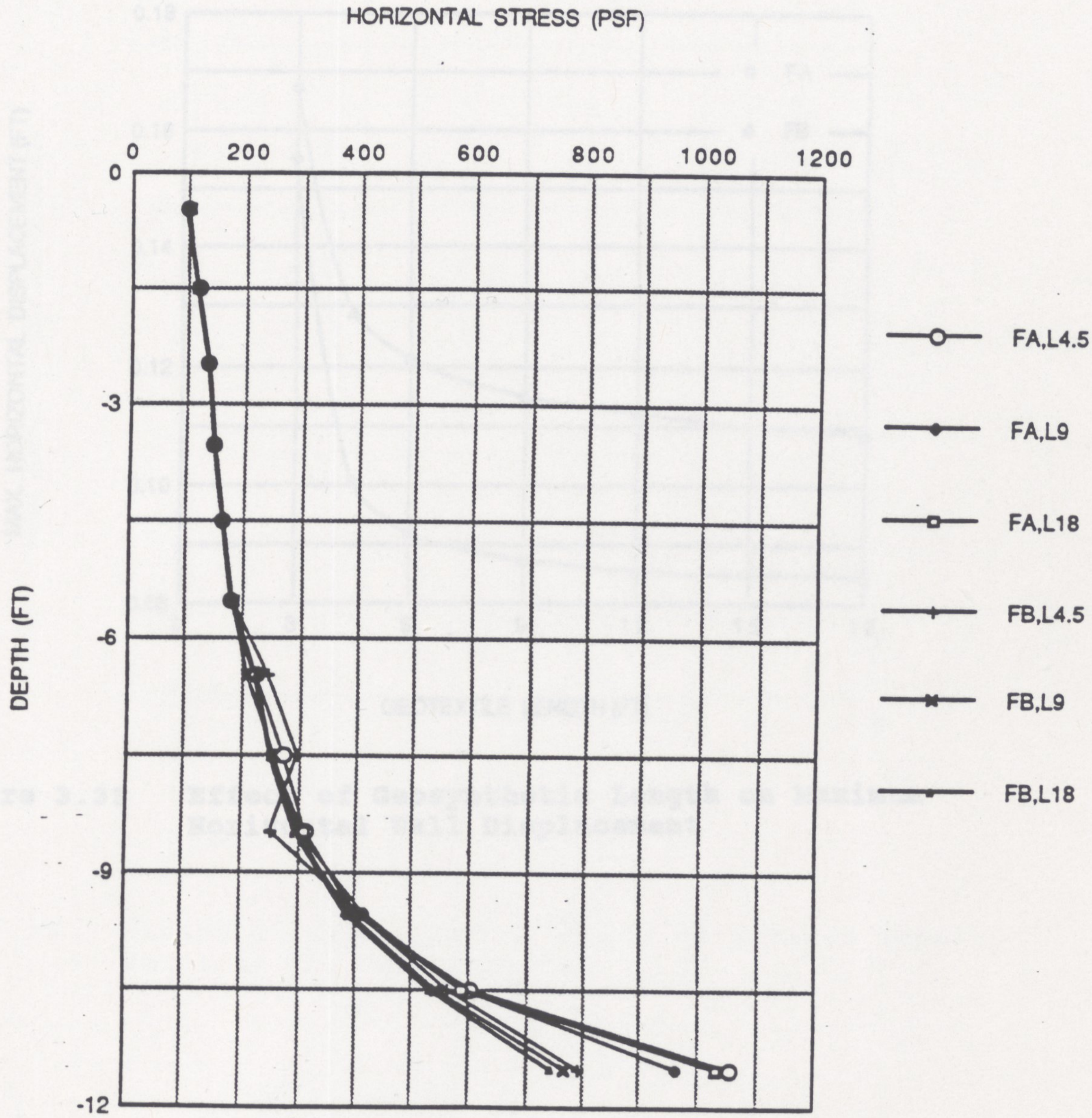


Figure 3.34 Effect of Geosynthetic Length on Lateral Earth Pressure

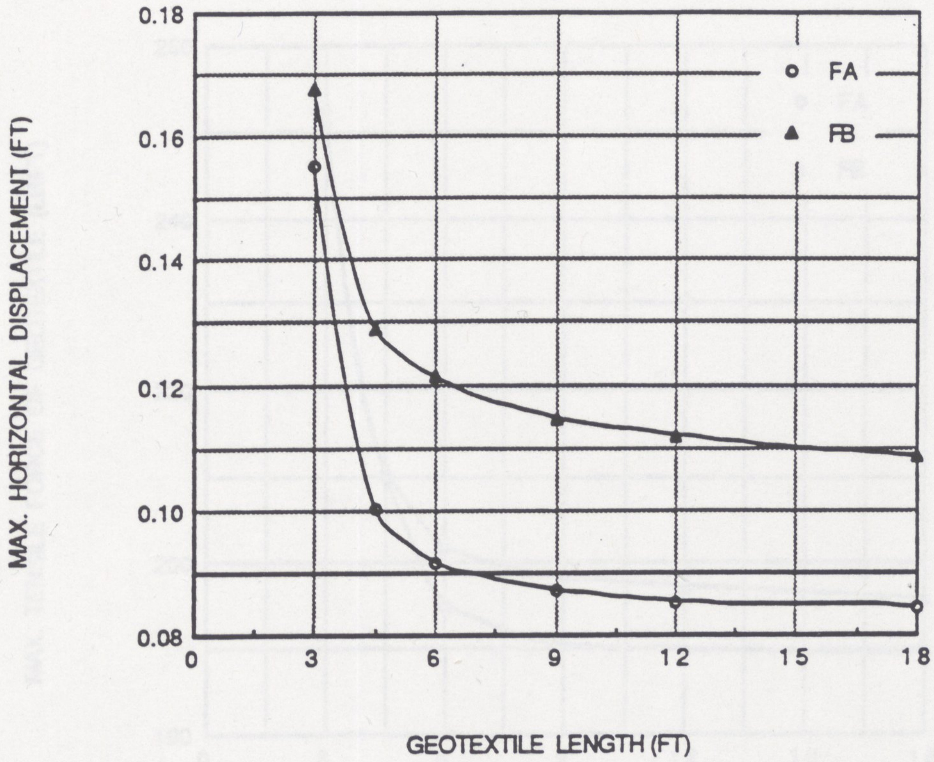


Figure 3.35 Effect of Geosynthetic Length on Maximum Horizontal Wall Displacement

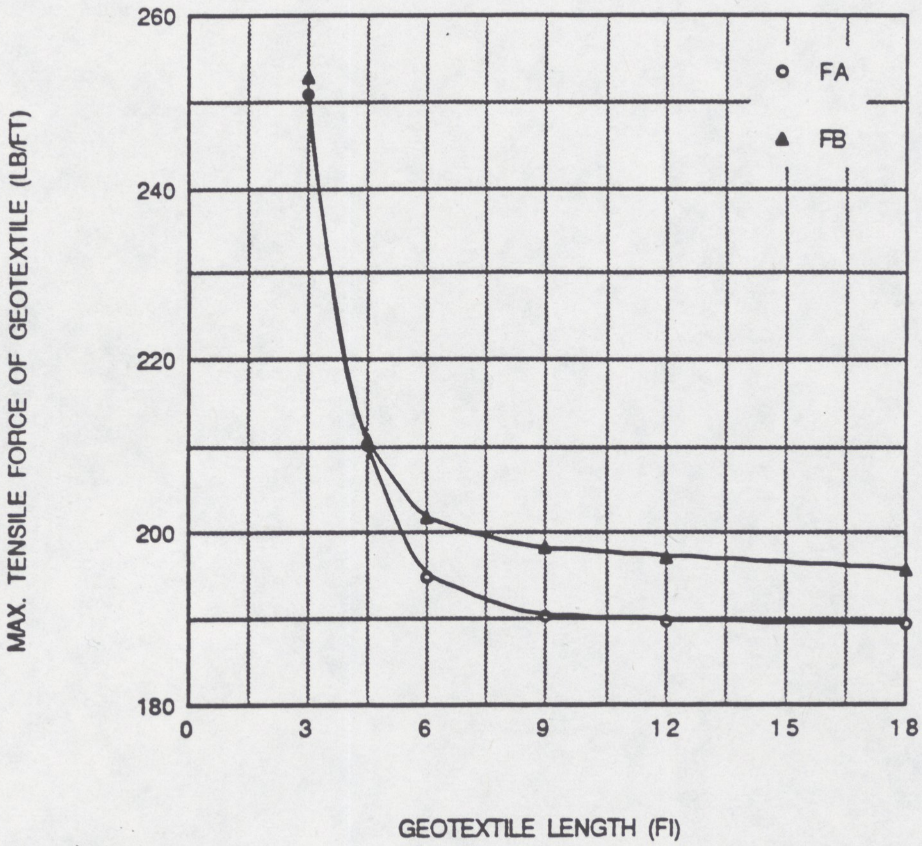


Figure 3.36 Effect of Geosynthetic Length on Maximum Tensile Force in the Geosynthetic

Figure 3.35 indicates that the maximum wall displacements were higher in the Control Wall B than in the Control Wall A for all the geosynthetic lengths. The differences are nearly constant for all the geosynthetic lengths except for the 3 ft long reinforcement, which as mentioned earlier, would most likely be in a unstable condition.

The effect of geosynthetic length on the maximum tensile force in the geosynthetic, as illustrated in Figure 3.36, was small for both control walls, again, except when 3 ft long geosynthetic was used. The difference due to the foundation was also small.

3.5 Comparison of Wall Performance Due to Different Foundations

In this section, the performances of geosynthetic walls due to different foundations are compared. A clayey foundation material softer than that of the Control Wall B foundation was included in the comparisons, and is referred to as Foundation D, whereas the foundation for the Control Wall B is referred to as Foundation B. Also included in the comparison was the walls on a rigid foundation, as in the Control Wall A. The modified hyperbolic Duncan model parameters for Foundation D are listed in Table 3.3.

The comparisons include the effects of the foundations on the following:

TABLE 3.3 MODIFIED HYPERBOLIC DUNCAN MODEL PARAMETERS OF FOUNDATION D AND THE COHESIVE BACKFILL

	γ (pcf)	K	n	R_f	K_B	m	C (psf)	ϕ	$\Delta\phi$	Ko	Kur
Foundation D	120	60	0.45	0.7	50	0.2	120	30°	0°	0.5	60
The Cohesive Backfill	120	120	0.45	0.7	110	0.2	300	30°	0°	0.5	120

(a) The horizontal wall displacements under different surcharge pressures, see Figure 3.37 (a) and (b).

(b) The tensile force distribution in the geosynthetic layers at different depths, see Figure 3.38.

(c) the lateral earth pressure on the facing, see Figure 3.39.

(d) the maximum lateral deflection of the wall under different surcharge pressures, see Figure 3.40.

(e) the maximum tensile force in the geosynthetic as the wall is subjected to different surcharge pressures, see Figure 3.41.

Figure 3.37 (a) indicates that the softer foundation (Foundation D) resulted in larger wall movements, especially near the base of the wall, and the effect was slightly more pronounced as the surcharge pressure became higher. It is to be noted that the location at which the maximum movement occurred was also affected by the foundation material. This is also true when comparing the wall movements with those of rigid foundation (Figure 3.37 (b)).

The tensile force distribution shown in Figure 3.38 was hardly affected by the foundation material except near the base where the tensile forces were higher for softer foundation materials. The change in the tensile force is more pronounced toward the free end of the geosynthetic.

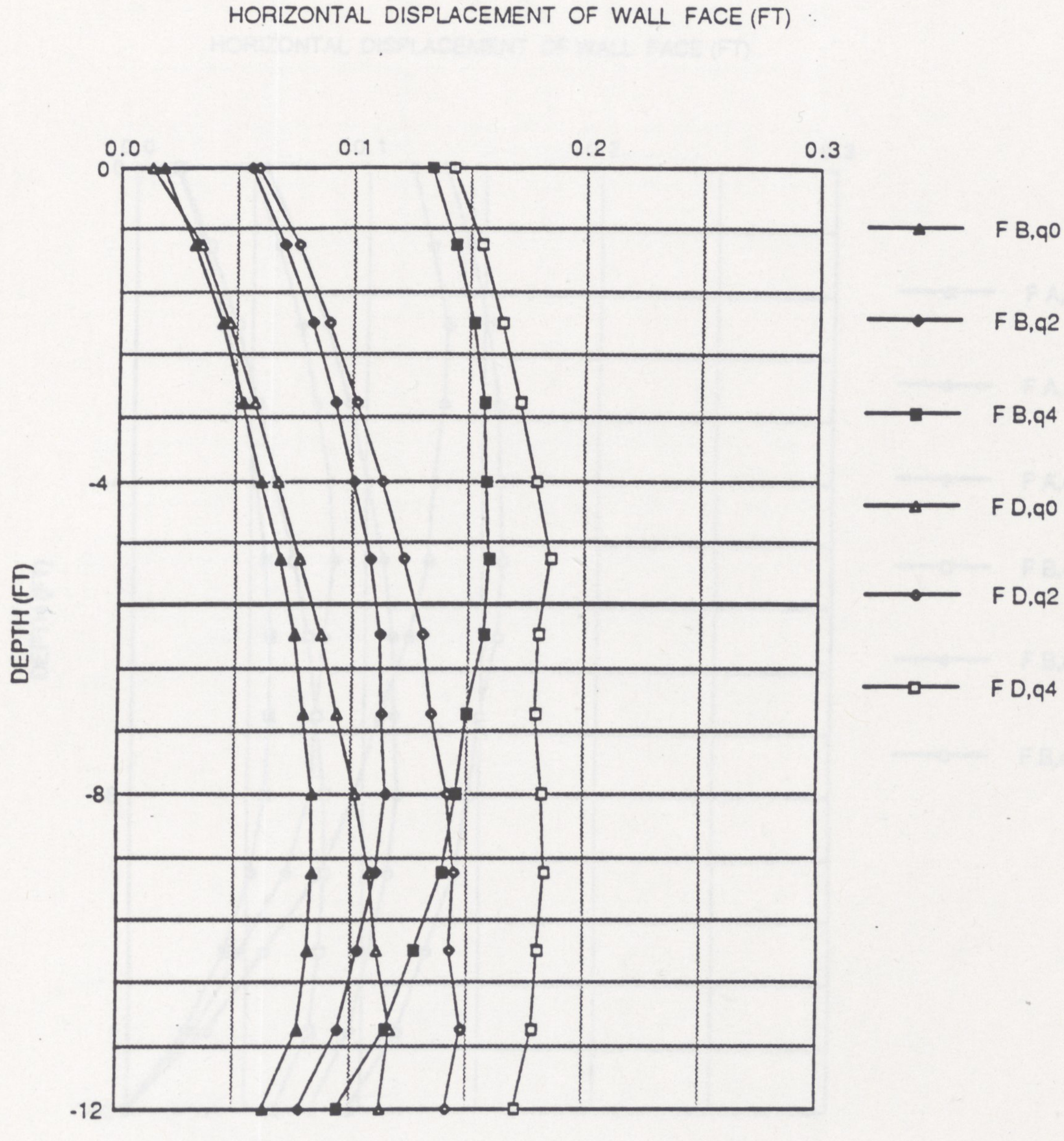


Figure 3.37(a) **Effect of Foundation on Horizontal Wall Displacement under Different Surcharge Pressures--Foundations B and D**

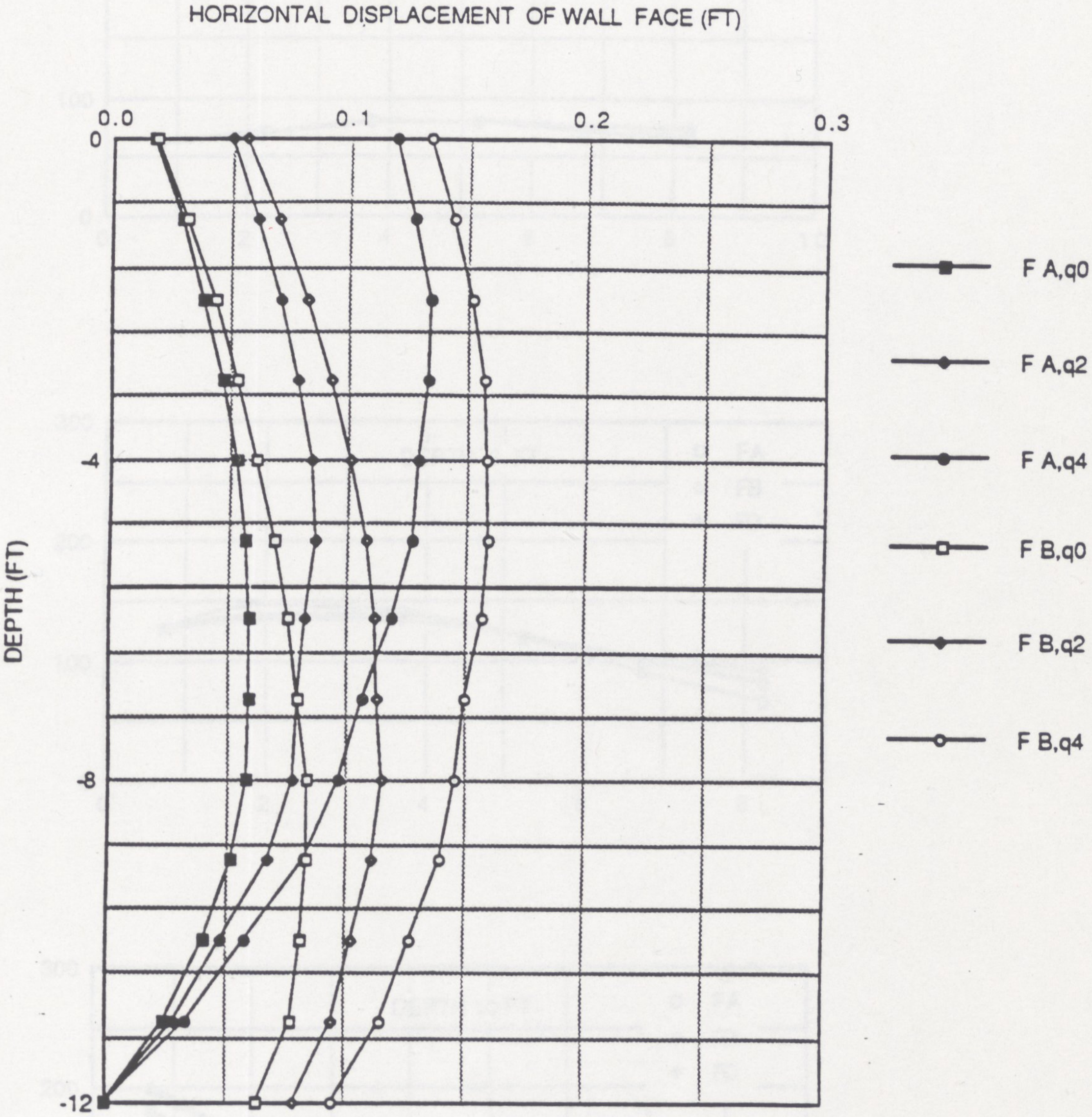


Figure 3.37(b) Effect of Foundation on Horizontal Wall Displacement under Different Surcharge Pressures--Foundations A and B

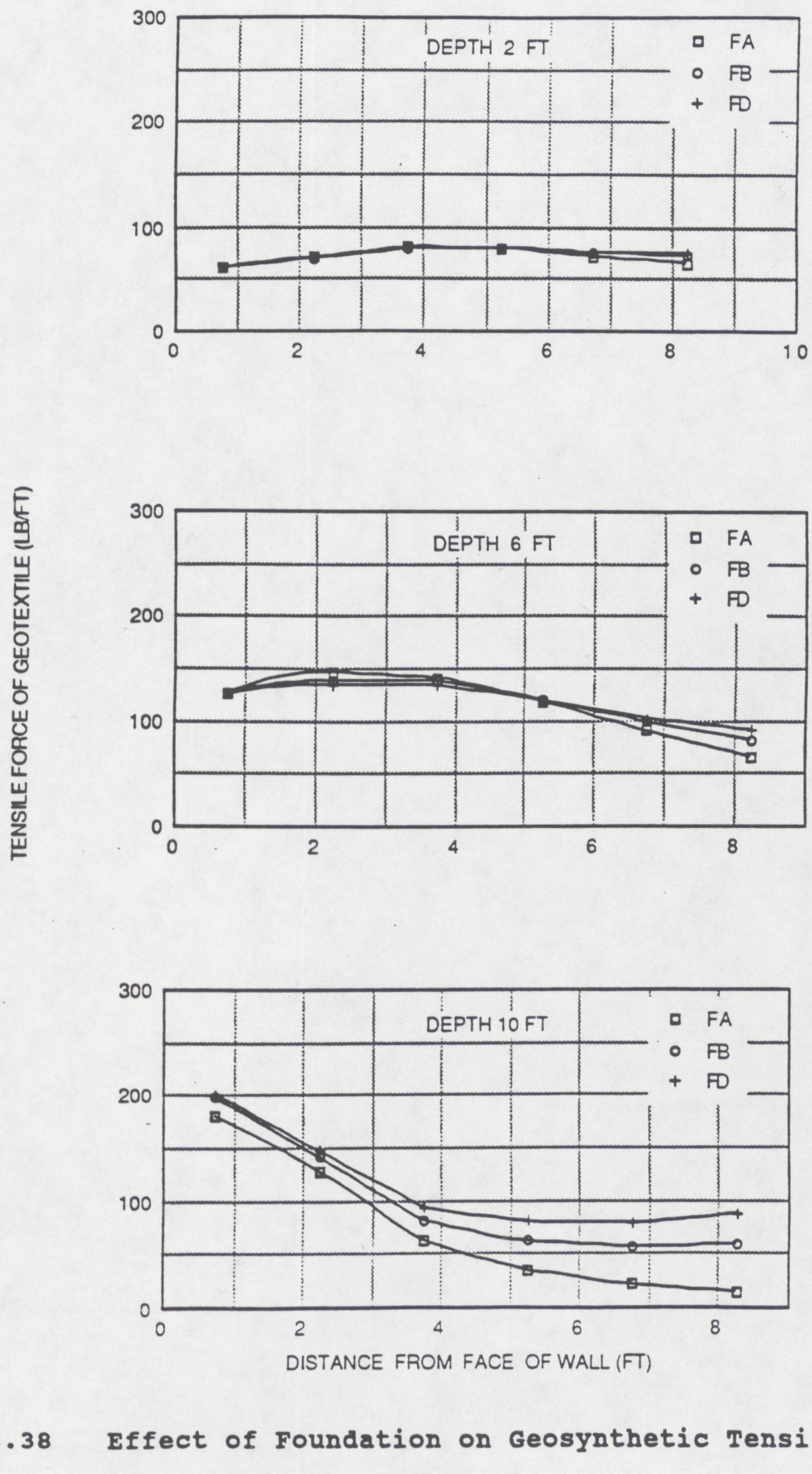


Figure 3.38 Effect of Foundation on Geosynthetic Tensile Force

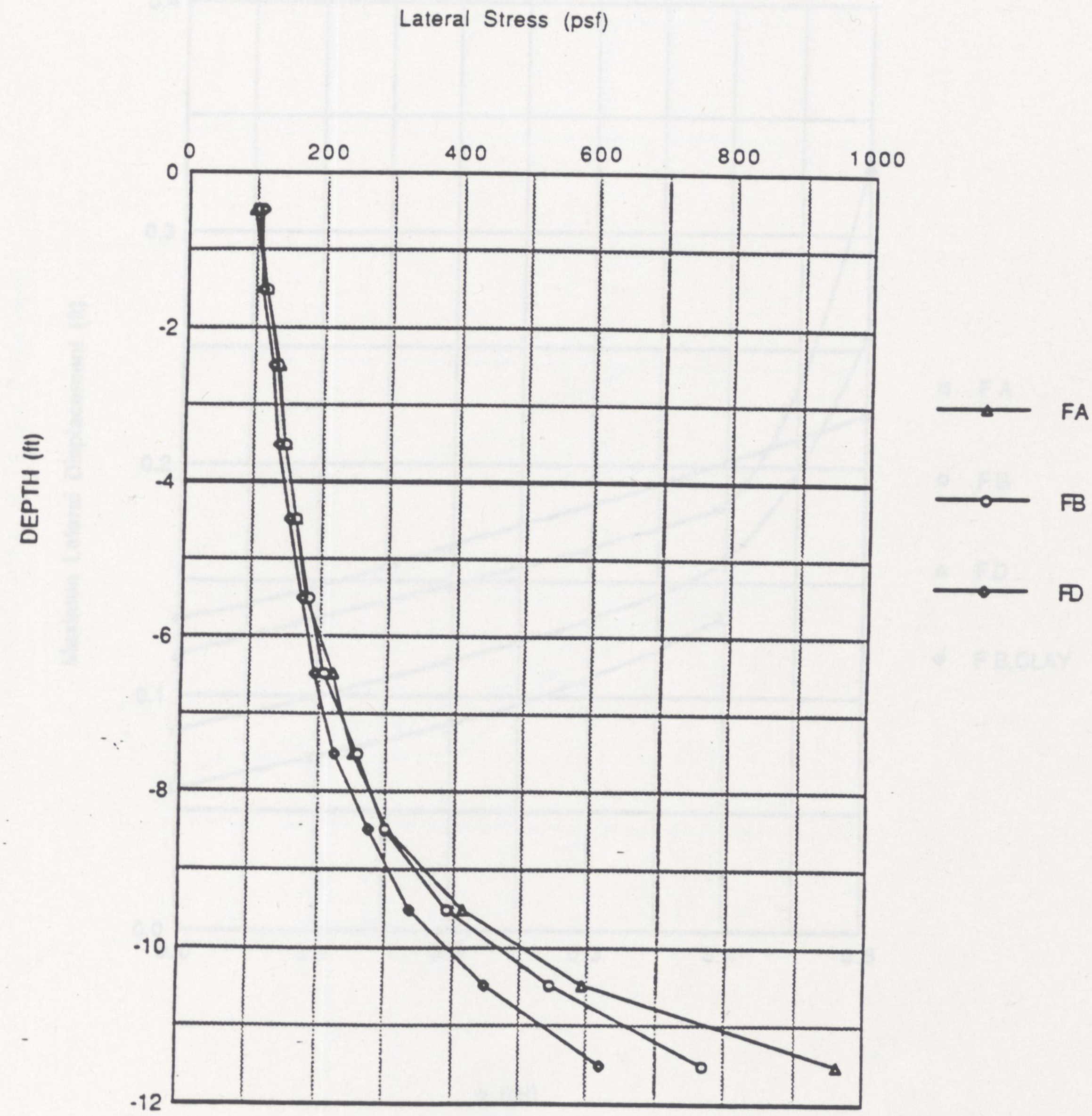


Figure 3.39 Effect of Foundation on Lateral Earth Pressure

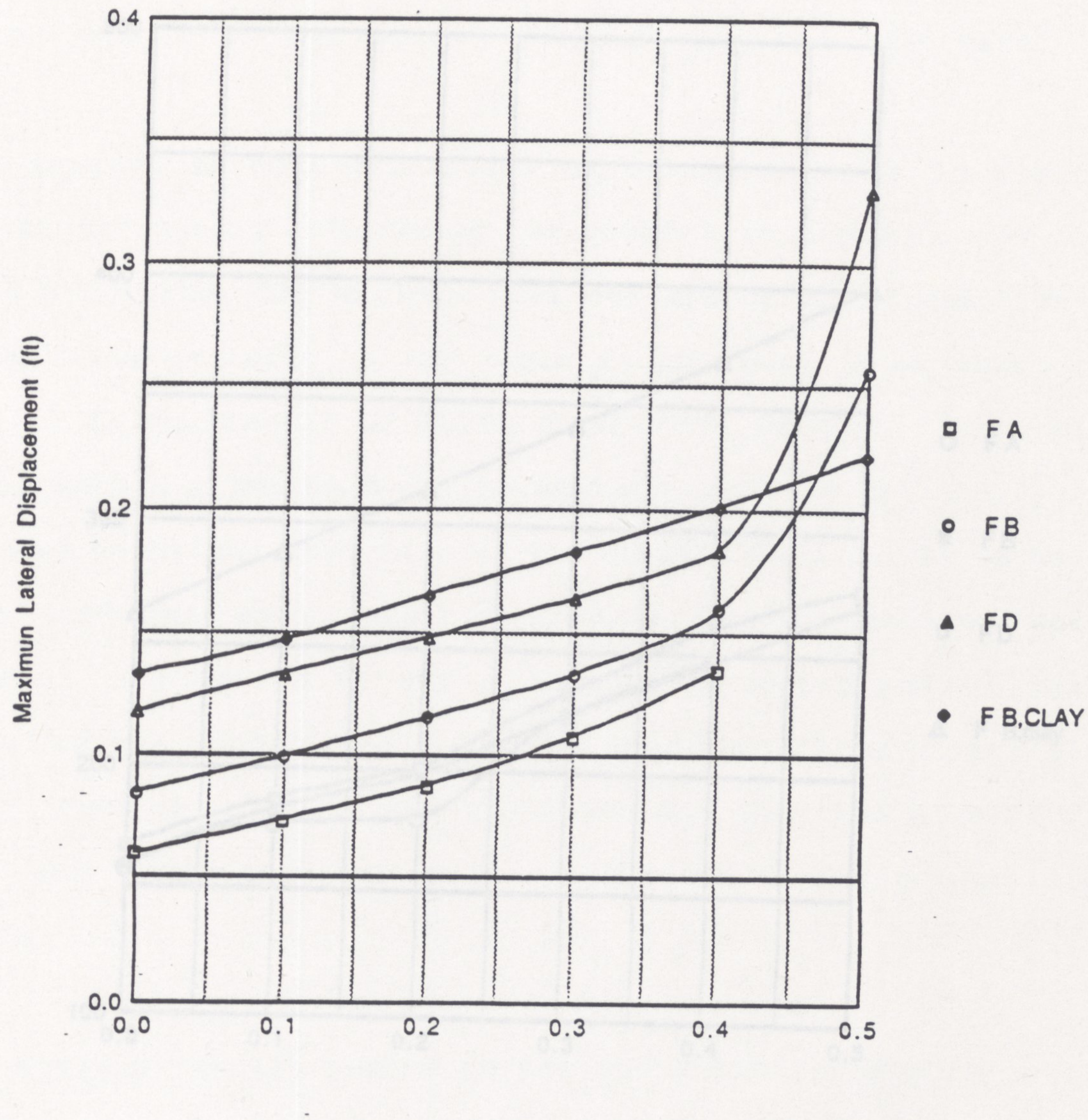


Figure 3.40 Effect of Foundation on Maximum Horizontal Wall Displacement under Different Surcharge Pressures

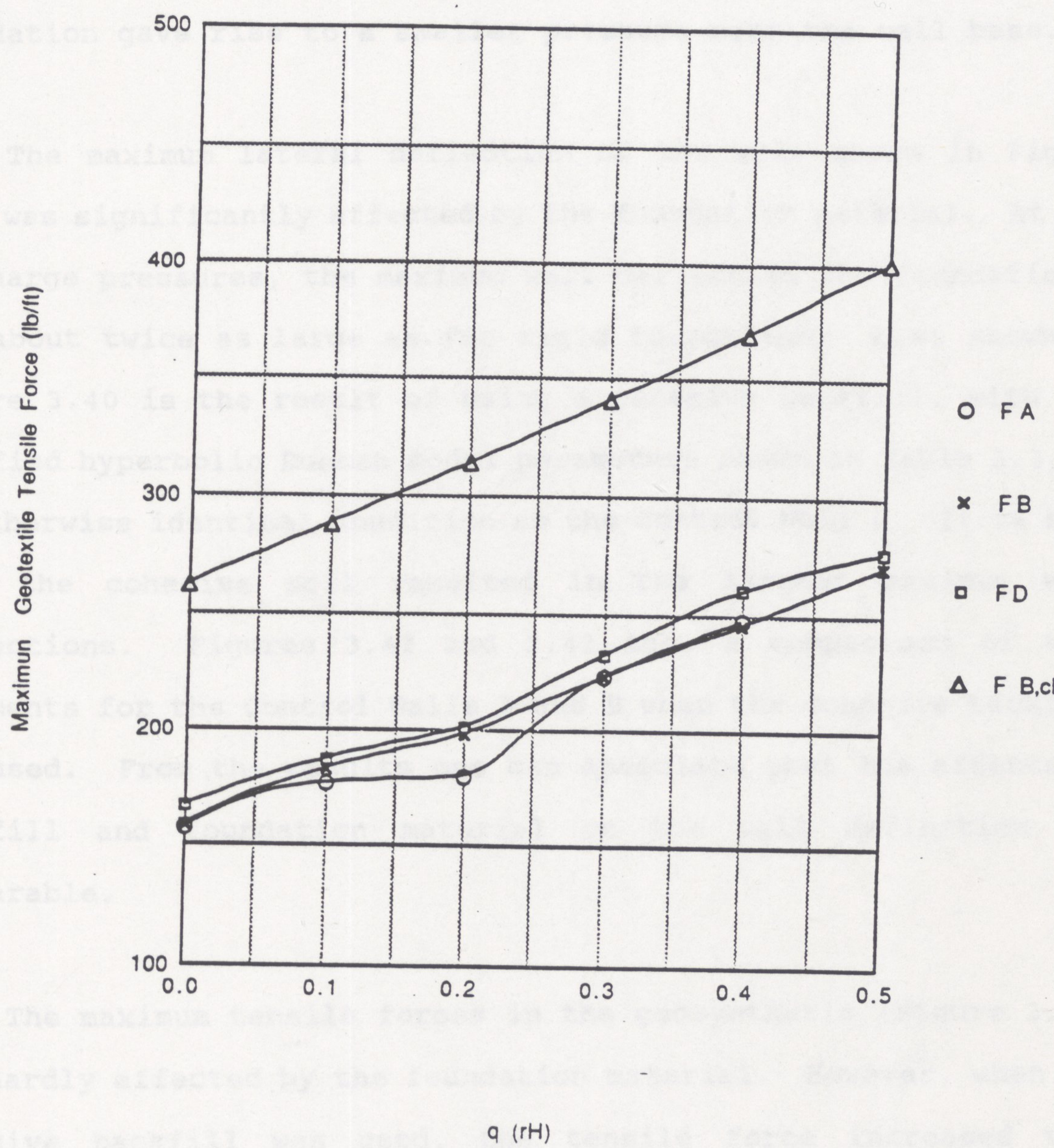


Figure 3.41 Effect of Foundation on Maximum Tensile Force in the Geosynthetic under Different Surcharge Pressures

It is seen from Figure 3.39 that the lateral earth pressure was not affected by the foundation material except near the wall base, where the largest earth pressure developed. The softer foundation gave rise to a smaller pressure near the wall base.

The maximum lateral deflection of the wall shown in Figure 3.40 was significantly affected by the foundation material. At low surcharge pressures, the maximum wall deflection for Foundation D was about twice as large as for rigid foundation. Also shown in Figure 3.40 is the result of using a cohesive backfill, with its modified hyperbolic Duncan model parameters shown in Table 3.3, in an otherwise identical condition as the Control Wall B. It is seen that the cohesive soil resulted in the largest maximum wall deflections. Figures 3.42 and 3.43 show a comparison of wall movements for the Control Walls A and B when the cohesive backfill was used. From the results one can speculate that the effects of backfill and foundation material on the wall deflection are comparable.

The maximum tensile forces in the geosynthetic (Figure 3.41) are hardly affected by the foundation material. However, when the cohesive backfill was used, the tensile force increased very significantly.

3.6 **Effect of Facing Rigidity**

To examine the effect of facing rigidity on the wall

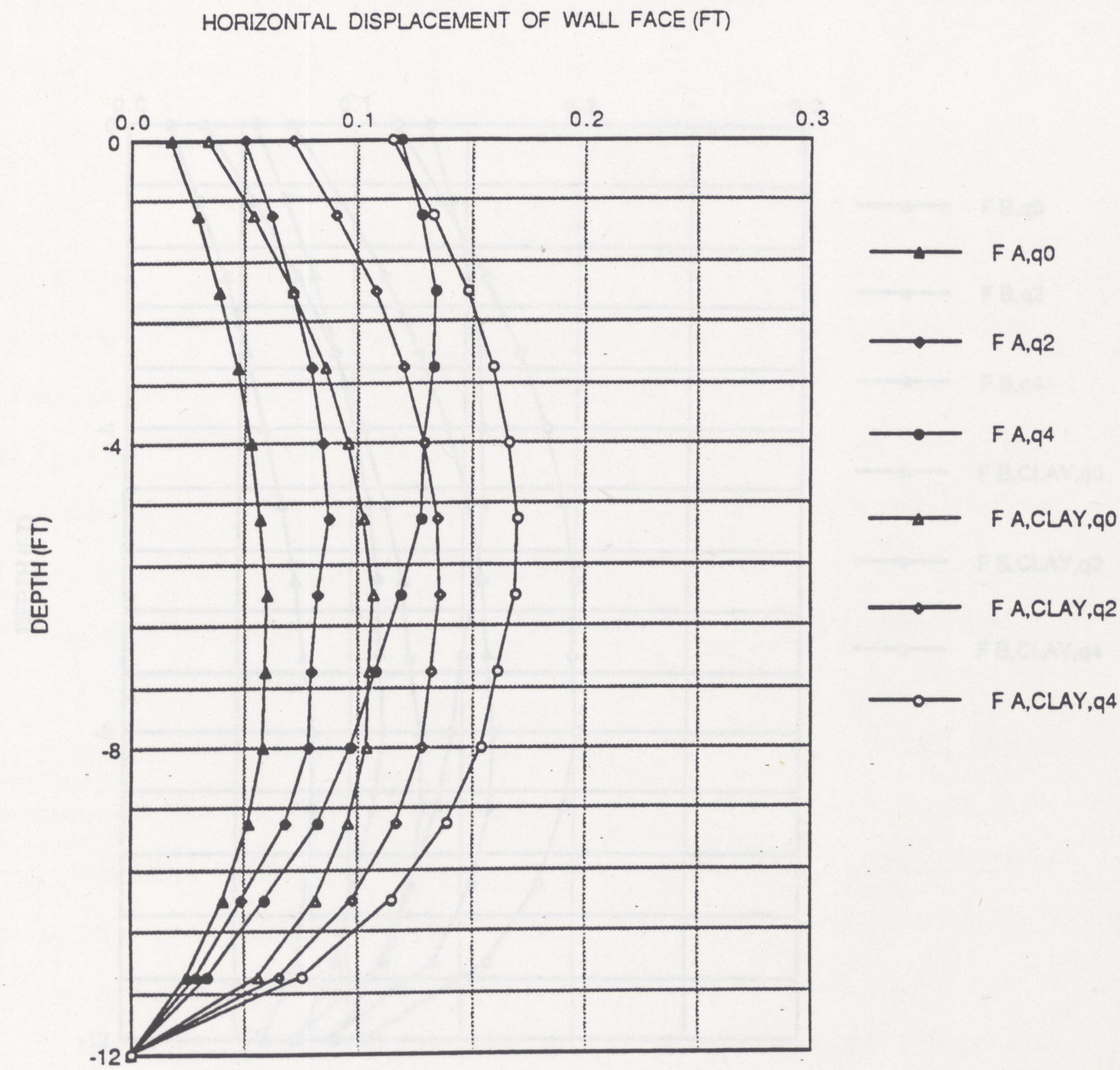


Figure 3.42 **Wall Movemant of the Control Wall A, with granular and Cohesive backfills, under Different Surcharge Pressures**

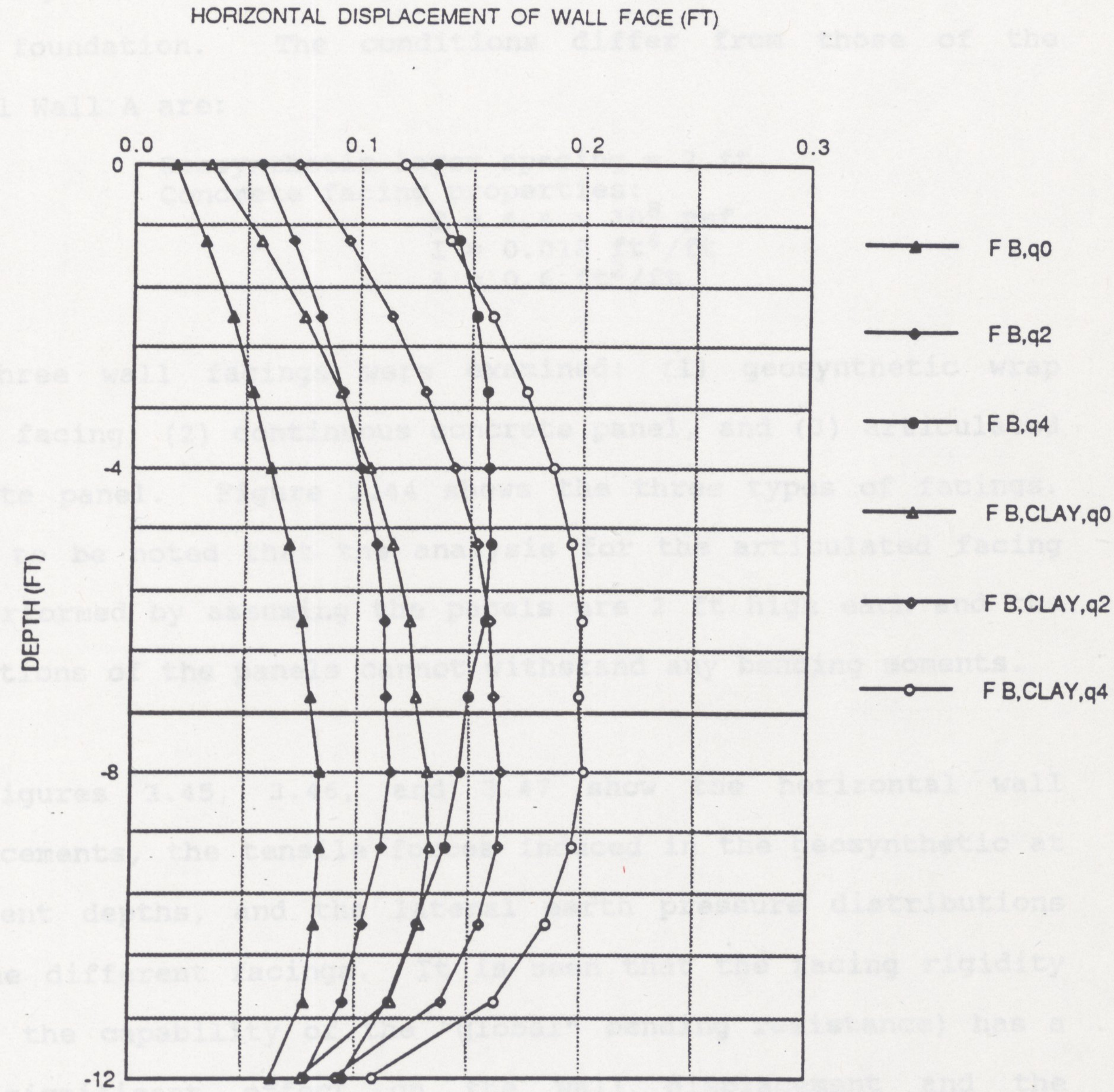


Figure 3.43 Wall Movemant of the Control Wall A, with granular and Cohesive backfills, under Different Surcharge Pressures

performance, a geosynthetic wall similar to the Control wall A was analyzed. The wall was assumed to be constructed over a rigid foundation. The conditions differ from those of the Control Wall A are:

Geosynthetic layer spacing = 2 ft
Concrete facing properties:

$$E = 4.4 \times 10^8 \text{ psf}$$
$$I = 0.018 \text{ ft}^4/\text{ft}$$
$$A = 0.6 \text{ ft}^2/\text{ft}$$

Three wall facings were examined: (1) geosynthetic wrap around facing, (2) continuous concrete panel, and (3) articulated concrete panel. Figure 3.44 shows the three types of facings. It is to be noted that the analysis for the articulated facing was performed by assuming the panels are 2 ft high each and the connections of the panels cannot withstand any bending moments.

Figures 3.45, 3.46, and 3.47 show the horizontal wall displacements, the tensile forces induced in the geosynthetic at different depths, and the lateral earth pressure distributions for the different facings. It is seen that the facing rigidity (i.e., the capability of the "global" bending resistance) has a very significant effect on the wall displacement and the geosynthetic forces. The wall with the continuous concrete facing experiences much smaller wall movement and geosynthetic forces than the other two facing types. This finding agrees with the results of the model tests and full-scale tests conducted by the University of Tokyo (Tatsuoka, et al., 1987).

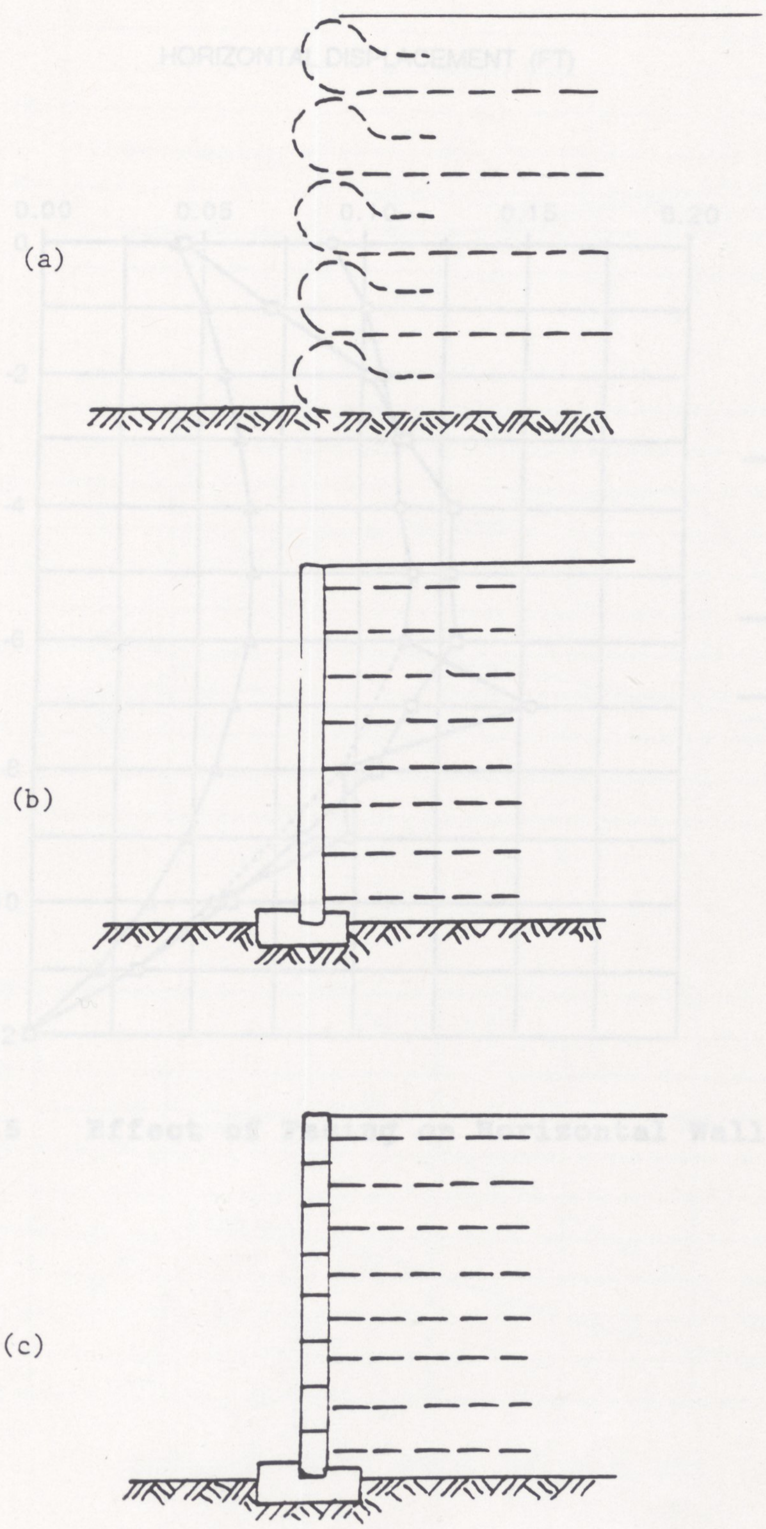


Figure 3.44 Types of Wall Facings: (a) Geosynthetic Wrap Around Facing; (b) Continuous Concrete Panels; (c) Articulated Concrete Panels

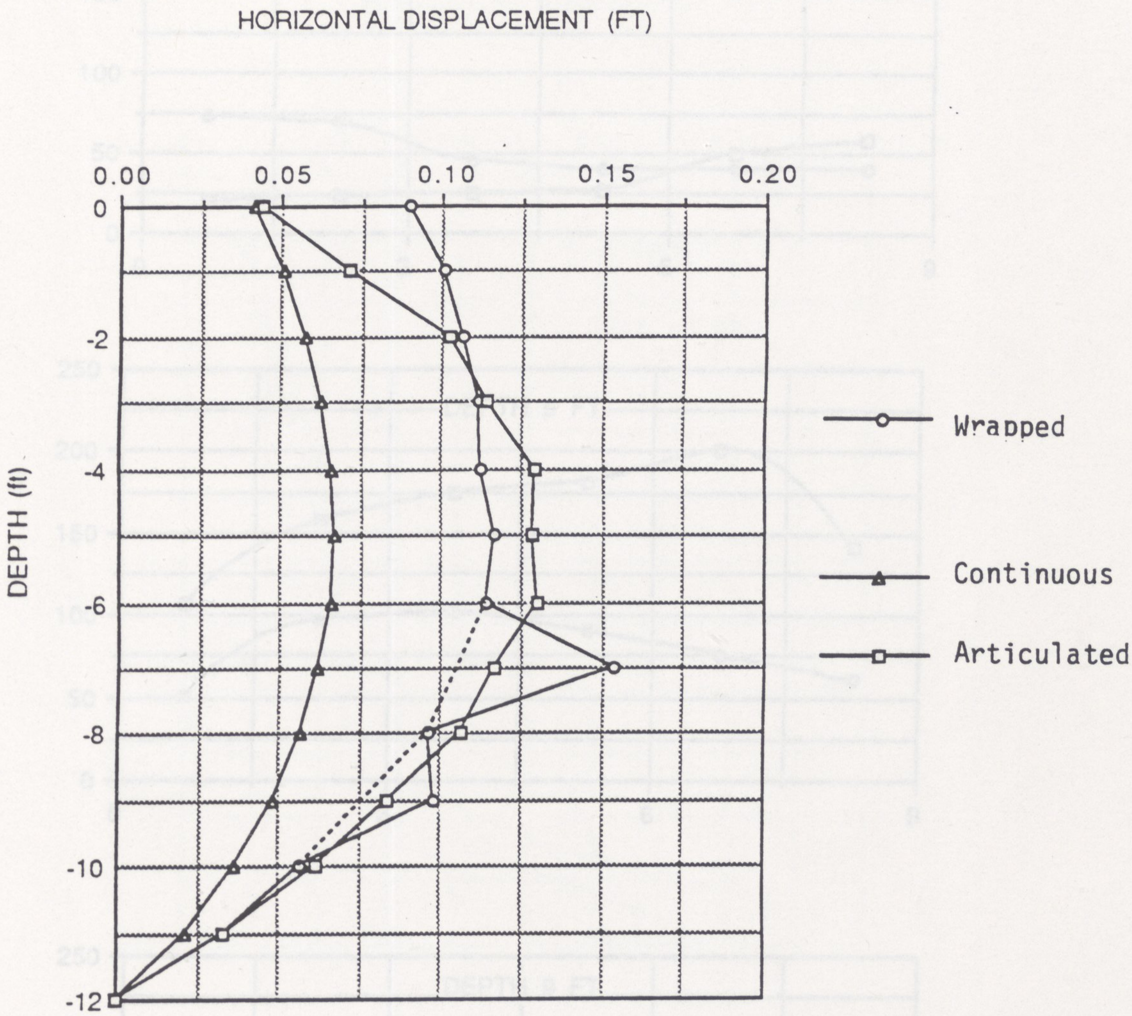


Figure 3.45 Effect of Facing on Horizontal Wall Displacement

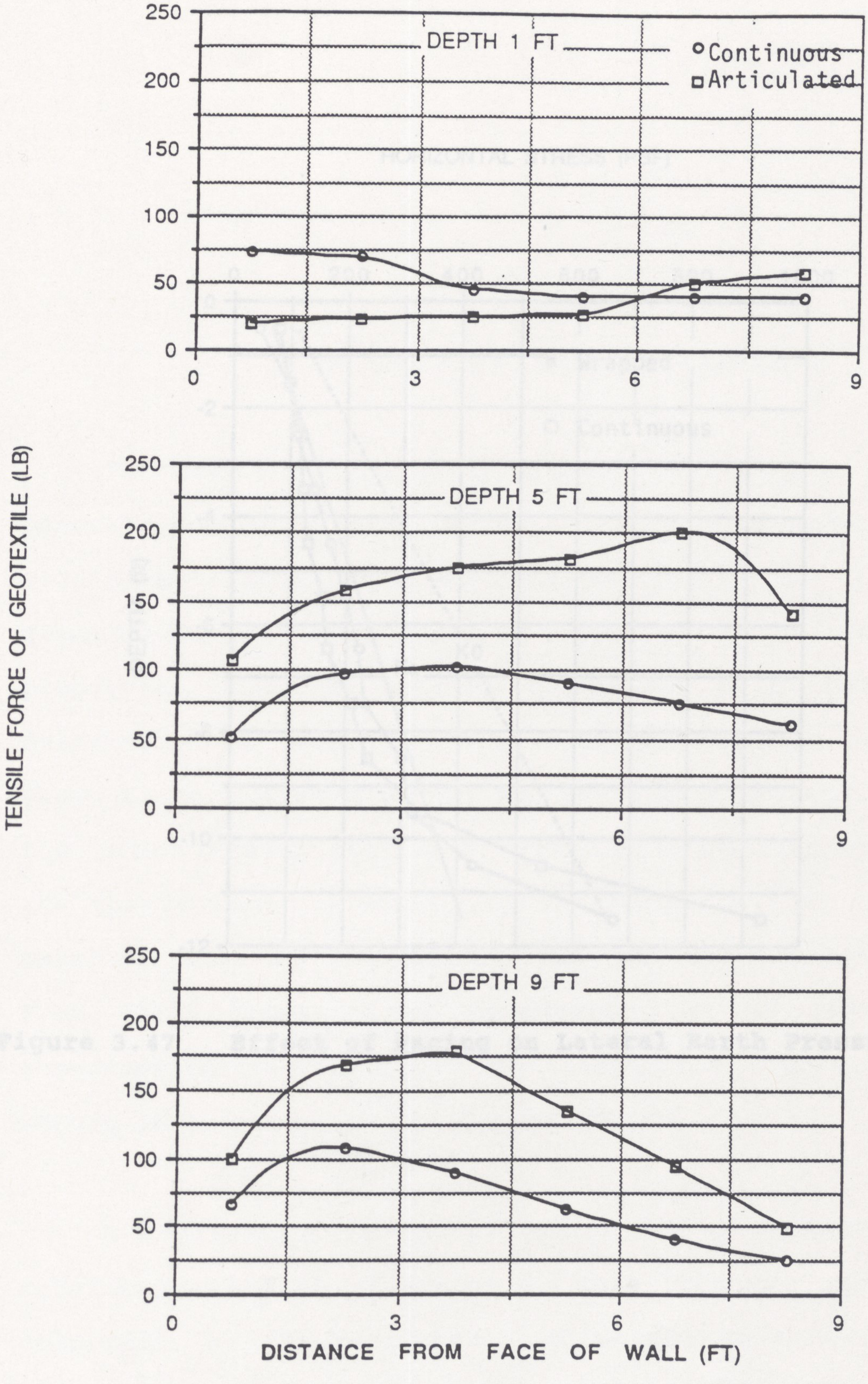


Figure 3.46 Effect of Facing on Geosynthetic Tensile Force

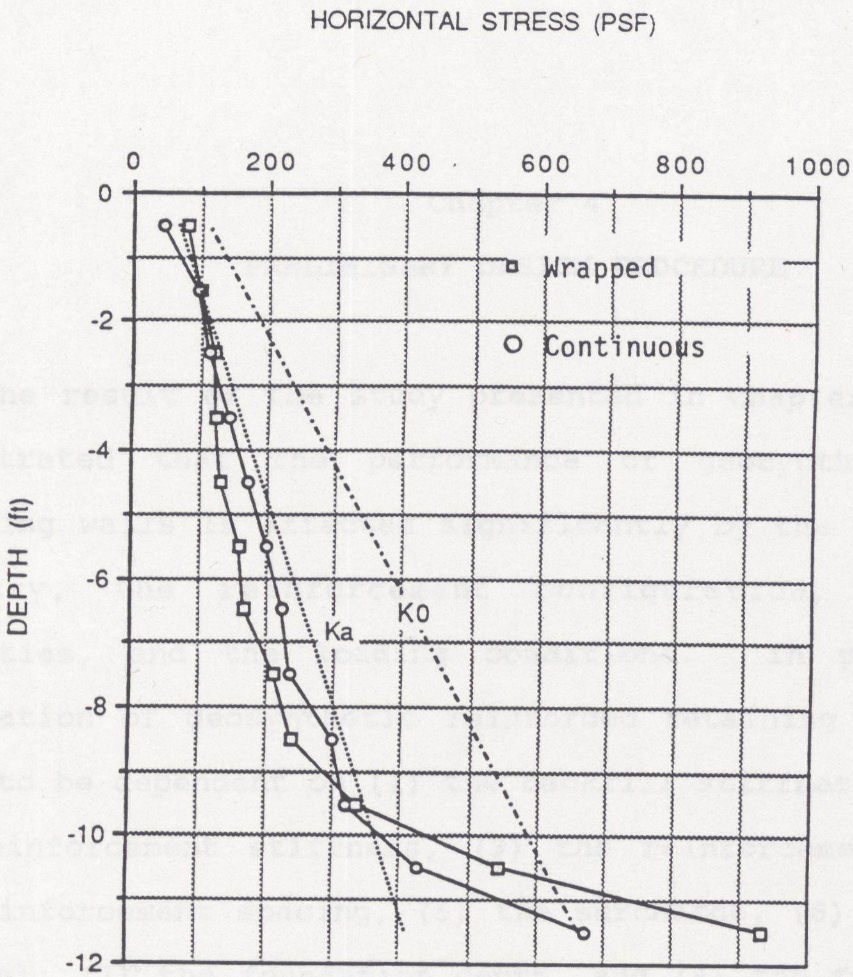


Figure 3.47 Effect of Facing on Lateral Earth Pressure

Chapter 4

PRELIMINARY DESIGN PROCEDURE

The result of the study presented in Chapter 3 has clearly demonstrated that the performance of geosynthetic-reinforced retaining walls is affected significantly by the wall/foundation geometry, the reinforcement configuration, the material properties, and the loading conditions. In particular, the deformation of geosynthetic reinforced retaining walls has been shown to be dependent on (1) the backfill stiffness/strength, (2) the reinforcement stiffness, (3) the reinforcement length, (4) the reinforcement spacing, (5) the surcharge, (6) the foundation material, (7) the foundation depth, and (8) the facing rigidity. From field construction experiences, it is learnt that the construction sequence and the compaction operation also have strong influence on the wall performance.

A preliminary design procedure for geosynthetic-reinforced retaining walls is presented in this Chapter. In view of the complexity of the performance of the walls as affected by the above-mentioned factors, and increasing availability of computers to design engineers, the preliminary design procedure is computer based.

4.1 Preliminary Design Procedure

The preliminary design procedure involves execution of two computer programs. The first program, GSWALL, is used to obtain a trial design for a geosynthetic-reinforced retaining wall, and should be executed first (Step 1). The second program, SSCOMP, which is the same computer program employed in the parametric study, is used to check the wall performance for the trial design (step 2). If the wall performance is satisfactory, the trial design can be accepted. Should the wall performance be unacceptable, the trial design should be modified, and additional analyses should be performed until a design with satisfactory performance is obtained (Step 3).

4.1.1 Step 1--Obtain a Trial Design by Executing the Program GSWALL

The program GSWALL is based on the design method proposed by the Geoservice Inc. (generally referred to as the Geoservice method). Detail description of the design method and a design example are presented in an FHWA report (FHWA-HI-89-002). A brief summary of the Geoservice method is given herein.

The Geoservice design method is the only comprehensive design method for geosynthetic-reinforced retaining walls that considers both external stability (sliding, overturning, bearing capacity) and internal stability (pullout and rupture) of the walls. In addition, the design method has two unique features:

(1) it accounts for the limit strain of the geosynthetic reinforcement in the design, and (2) it calculates the maximum displacement of the wall. Although the wall displacement and the maximum geosynthetic force calculated by the method are typically much larger than the actual values, it is considered a good "first trial" method.

The Geoservice method has 18 design steps and involves a trial and error process. The use of the design method can be cumbersome and error-prone. Therefore, the computer program, GSWALL, was developed. The program is written using the Lotus spread sheet. The use of this program requires fundamental knowledge of the Lotus spread sheet, otherwise the program is very "user friendly."

It is to be noted that the method is limited to the following conditions:

(1) The backfill is cohesionless.

(2) Soil properties are uniform throughout the reinforced zone.

(3) The wall face is vertical.

(4) The crest of the wall is horizontal.

(5) The wall is constructed over a very rigid foundation, although the bearing capacity of the foundation is evaluated.

(6) The surcharge load on the top surface is uniformly

distributed.

(7) The surcharge load is less than 20% of the total weight of the soil fill.

(8) All geosynthetic layers have the same length.

When field conditions deviate from these limitations, it is suggested that this method be used by simplifying the field condition to conform to the limitations in order to obtain a "trial" design. For example, the foundation should be treated as very rigid in using the program GSWALL even if the foundation is soft.

4.1.2 Step 2--Check Wall Performance by Executing the Program SSCOMP

Upon obtaining the trial design from GSWALL program, it is suggested that the designer use the trial geometry to perform an analysis using the program SSCOMP. The program SSCOMP employs the finite element method. A description of the program is given in Chapter 2. The use of the program requires a working knowledge of application of the finite element method in geotechnical engineering.

The analysis will provide detailed response of the geosynthetic-reinforced retaining wall with any prescribed construction sequences (including compaction operation) under any loading conditions. If the analysis indicates that the wall will perform satisfactorily, an adequate design is obtained. If otherwise, the designer will need to proceed with the step 3 analysis.

4.1.3 Step 3--Modify the Trial Design to Obtain an Acceptable Design

If the result of the Step 2 analysis indicates that the design is not acceptable, either too conservative or does not have adequate safety margin (for examples, the wall displacement is excessive or the forces in the geosynthetic are too large), the design should be modified. The findings of the parametric study presented in Chapter 2 may be used as a guide to modify the design. The modified design should be checked again, using the SSCOMP program, until an acceptable design is obtained.

4.2 Input and Output of the Program GSWALL

The information required for executing the program GSWALL include the following:

(A) Geometry:
- the wall height.
- the geosynthetic reinforcement spacing.

(B) Loading:
- the uniform surcharge pressure.

(C) Material Properties:
- the unit weight of the soil.
- the angle of internal friction (at peak of stress-strain curve) of the soil and the corresponding strain.
- the coefficient of friction at soil-geosynthetic interface.
- the angle of internal friction (at residual stress) of the soil and the cohesion at the

- residual stress.
- the bearing capacity coefficient.
- the foundation soil properties (unit weight, angle of internal friction, and cohesion).

(D) Design Limits:
- the design limit strain.
- the factor of safety against sliding.
- the factor of safety against overturning.
- the factor of safety against bearing capacity failure.
- the factor of safety against rupture (and creep) of the geosynthetic.

Upon executing the program GSWALL, the following information will be obtained:

- the minimum reinforcement length and a Table showing the required reinforcement anchored length for each layer.
- the maximum horizontal wall displacement and the maximum allowable wall displacement according to the prescribed design limit strain.
- the required geosynthetic tension.

A sample example for using the program GSWALL is given in Appendix A.

4.3 Input and Output of the Program SSCOMP

The information required to execute the program SSCOMP includes the following:

(A) Finite Element Mesh
- the nodal number and nodal coordinates for each node.
- the element number and element sequence for each element and its material number.

- the boundary conditions.

(B) Construction Sequence, Compaction Operation, and External Loads
- the sequence of wall construction (i.e., sequence of placement for soil layers, compaction of soil layers, and placement of geosynthetic layers).
- the loads applied to a partially constructed wall or a completed wall.
- the peak compaction pressure profile.

(C) Material Properties
- the properties of the geosynthetic, namely, its Young's modulus, cross sectional area, and moment of inertia (if non-zero).
- the properties of the facing.
- the soil-geosynthetic interface properties.
- the soil parameters for the modified Duncan hyperbolic model (see Chapter 2).

(D) Preexisting Quantities
- the preexisting stresses, strains, and displacements in the soil mass.
- the preexisting forces (and moments, if non-zero) in the geosynthetic layers.

The output of the program SSCOMP include the following results for every construction layer:

- the stresses, strains, of each soil element.
- the forces (and moments, if non-zero) of each bar element (geosynthetic) and beam element (facing).
- the displacements at each nodal point (of the soil elements, geosynthetic elements, and facing elements).

A sample example for illustrating the use of the program SSCOMP is given in Appendix B.

Chapter 5

SUMMARY AND CONCLUSIONS

A comparison of the existing design methods for geosynthetic-reinforced retaining walls presented in Volume I of this report has clearly revealed that, while there are significant differences in the design concept of the methods, the greatest discrepancy among the various design methods stems from the internal stability safety factors adopted in the methods. In a typical wall evaluated in that study, the combined safety factors (in terms of the quantity of geosynthetic needed) against internal stability ranged from 3 to 23 depending on the method used.

The widely varied safety factors adopted in these design methods indicate that the safety factors are somewhat arbitrary. This is in part due to the fact that the construction of geosynthetic-reinforced retaining walls has only gained popularity in recent years; as a result, the knowledge concerning the safety margin associated with the design methods is not founded on reliable empiricism.

The loading test presented in Volume II of this report has demonstrated that the most commonly used design method, the Forest Service method, is overly conservative. The test wall was loaded at a surcharge load more than three times the failure load predicted by the Forest Service method; however, failure did not occur.

The principal investigator is convinced that a rational design method for internal stability (tensile rupture failure, pullout failure, and long-term creep failure) of geosynthetic-reinforced retaining walls should be deformation-criteria based, i.e., based on specified deformation limits. This is necessary because:

(1) There has not been adequate case histories documenting short-term and long-term behavior of geosynthetic walls; consequently there is not adequate empiricism based on which proper and reliable safety factors against internal failure of reinforced soil mass can be established.

(2) The deformation (strain) associated with tensile rupture and creep failure of geosynthetics is often greater than 100% and geosynthetics of similar rupture strengths may have very different tensile stiffnesses.

(3) Methods based on safety factors cannot address the effect of the foundation settlement and facing rigidity in a rational manner. However, both factors

have been known to have important influence on the performance of geosynthetic-reinforced retaining walls.

In this report, the results of a parametric study are presented. The parametric study was conducted by using a finite element program SSCOMP. The program has been validated through comparisons with field measurement of many different earth structures. The program is capable of accommodating various wall/foundation geometries (including soft foundations of different depths), soil and geosynthetic properties, construction operations, facing rigidity, and external loading conditions. Using the program SSCOMP, the effects of the following parameters on the performance of geosynthetic-reinforced retaining walls were investigated:

- the backfill stiffness and strength
- the geosynthetic stiffness
- the geosynthetic length
- the geosynthetic layer spacing
- the foundation stiffness and strength
- the foundation depth
- the surcharge pressure
- the facing rigidity

The parametric study indicated that:

(1) Each of the factors investigated showed significant

effects on the performance of geosynthetic-reinforced retaining walls.

(2) The geosynthetic wall performance (including the horizontal wall displacement, the forces induced in the geosynthetic layers, and the lateral earth pressure) as affected by each factor showed a definite "trend." This trend, together with the degree to which each factor affects the wall performance as revealed in this parametric study, can serve as a very useful guide for refining trial designs to obtain an acceptable design.

(3) The effects of multiple factors (e.g., due to different geosynthetic stiffness and foundation depth) on geosynthetic wall performance are complicated and the combinations are too many that it would be very difficult to achieve an exhaustive rational design method that requires only hand computations.

In view of the complexity of the performance of the walls as affected by the factors investigated in the parametric study, and increasing availability of computers to design engineers, a computer-based design procedure is proposed. The design method involves execution of two computer programs, GSWALL and SSCOMP, and includes the following three steps:

<u>Step 1</u>: Execute the program GSWALL, which is based on the

Geoservice design method, to obtain a "trial design."

<u>Step 2:</u> Perform a finite element analysis of the trial design using the program SSCOMP. If the results of the finite element analysis are satisfactory (i.e., the wall displacement is acceptable and the maximum force in the geosynthetic layers is smaller than the design limit), an adequate design is obtained.

<u>Step 3:</u> If the design is found too conservative, modifications may be made before accepting the design. On the other hand, if the trial design is found unsafe (e.g., the maximum wall displacement is excessive or the forces in the geosynthetic are too large), modifications must be made. In either case, the results of the parametric study can be used as a guide to obtain the modified trial design. Analyses using the program SSCOMP should be conducted until an acceptable design is secured.

BIBLIOGRAPHY

Duncan, J. M., Byrne, P., Wong, K. S., and Mabry, P., "Strength, Stress-Strain and Bulk Modulus Parameters for Finite Element Analyses of Stresses and Movements in Soil Masses," Report No., UCB/GT/80-01, Department of Civil Engineering, University of California, Berkeley, August 1980, 77 pp.

Seed, R. B. and Duncan, J. M., "SSCOMP: A Finite Element Analysis Program for Evaluation of Soil-Structure Interaction and Compaction Effects," Report No. UCB/GT/84-02, Department of Civil Engineering, University of California, Berkeley, February, 1984, 127 pp.

Tatsuoka, F., Nakamura, K., Iwasaki, K., and Yamauchi, H., "Behavior of Steep Clay Embankments Reinforced with a Nonwoven Geotextile Having Various Face Structures," Proceedings of Post Vienna Conference on Geotextiles, Singapore, October 1987, p.387-403.

APPENDIX A

Sample Example for the Program GSWALL

GSWALL -- Geotextile-Reinforced Retaining Wall Design

Based on the Geoservice Method (Jan. 1989)
Coded by: J.C. Lin & J.T.H. Wu (Feb. 1990)
Modified by: P. Macklin (Sept. 1990)

08:15 PM

(1) WALL CHARACTERISTICS:		
Height (in feet)==================================>	10	H
Geosynthetic Spacing (in feet)====================>	1	s
(2) LOADING CONDITION:		
Uniformly Distributed Surcharge (in psf)==========>	0	q
(3) SOIL PROPERTIES FOR THE REINFORCED AND RETAINED ZONES:		
Moist Unit Weight (pcf)===========================>	107	r
Friction Angle at peak stress (degrees)===========>	37	ø'p
Strain at peak stress=============================>	0.1	E
Friction Angle at residual stress (in deg.)=======>	35	ø'r
Cohesion at residual stress (psf)=================>	0	c'r
Bearing Capacity Coefficient (center load)========>	20	Nr
(4) SOIL PROPERTIES FOR THE FOUNDATION SOIL:		
Moist Unit Weight (in pcf)========================>	120	rf
Friction Angle (in degrees)=======================>	99	ø'f
Cohesion (in psf)=================================>	0	c'f
Coef. of friction of the soil-geosyn. interaction=>	0.5	u

Step 1. Check that the basic assumption is true.

$q_{max} = 0.2*r*H = 214.0$ psf >= $q = 0.0$ psf

Step 2. What design limit strain do you want?============> 0.1

Step 3. The retained earth friction angle = the 'residual' value.

Step 4. Calculate the backfill active earth pressure coef.

$Ka = (Tan(45 - ø'/2))**2 = 0.25$

Step 5. Calc. a min. reinforcement length for SLIDING.

What is the depth (z) from the crest to the sliding plane? (in feet) =======================> 10

Driving force :

$Pa = 0.5*Ka*rb*z^2 + Ka*q*z = 1329.92$ lb/ft

Resisting force:

$Pr = u * Tan(ø') * (r*z*Ls + q*Ls)$
Ls = length of min. reinforcement (sliding plane).

What safety factor (sliding) do you want?================> 1.2

$FSs = Pr / Pa = 1.20$

The length required to obtain the safety factor (sliding) at depth z is determined by:

L (sliding) = 4.26 ft

Step 6. Calculate a minimum reinforcement length for OVERTURNING.

Driving Moment :

$Md = (Ka*(H**2)/6)*(r*H+3*q) = 4433.07$ lb

What safety factor (overturning) do you want?============> 2

$FSo = Mr / Md = 2.00$

The length required to obtain the factor of safety (overturning) is determined by:

L (overturning) = 4.07 ft

Step 7. Calc. a min. reinforcement length for BEARING CAPACITY.

What safety factor (bearing capacity) do you want?=======> 2

Check that the following is true:

```
IF  :  L(assumed) = 4.66 ft = L(calculated) = 4.66 ft
THEN:  L (bearing capacity) = 4.66 ft
```

Step 8. Calculate a minimum reinforcement length for ECCENTRICITY.

L (eccentricity) >= 4.99 ft

Step 9. Calc. a min. reinforcement length for RANKINE ZONE EXTENSION

L (rankine zone extension) = 4.99 ft

Step 10. Select the minimum reinforcement length:

```
L (sliding)------------------------->    4.26
L (overturning)--------------------->    4.07
L (bearing capacity)---------------->    4.66
L (eccentricity)-------------------->    4.99
L (rankine zone extension)---------->    4.99

L (selected) ======================>    4.99
```

Step 11. Calculate the maximum horizontal displacement.

D = 2.99 inches; and should be less than 2.0 inches!

Step 12. Calculate the maximum vertical stress.

maximum vertical stress = 1605.0 psf

Step 13. Determine the maximum horizontal stress.

maximum horizontal stress = 434.9 psf

Step 14. Calculate the required design geosynthetic tension.

What safety factor (geofabric tensile strength)
do you want?====================> 1.5

t (required) = 652.4 lb/ft; at 0.10 strain
for a 1.0 ft spacing between fabric layers

Step 15. Summary of Geosynthetic Requirements:

At what depth (ft) from the top of the wall does
the top geosynthetic layer occur?=============> 0.5

What safety factor (for pullout) do you want?============> 1.5

FABRIC LAYER	DISTANCE FROM TOP OF WALL Z (ft)	MAX. VERT. STRESS (psf)	MAX. HORZ. STRESS (psf)	TENTATIVE ANCHORED FABRIC LENGTH (ft)	REQUIRED ANCHORED FABRIC LENGTH (ft)
1	0.5	53.5	14.5	0.0	0.6
2	1.5	161.7	43.8	0.6	0.6
3	2.5	273.2	74.0	1.1	0.6
4	3.5	390.4	105.8	1.6	0.6
5	4.5	516.4	139.9	2.1	0.6
6	5.5	654.5	177.4	2.6	0.6
7	6.5	809.5	219.4	3.2	0.7
8	7.5	987.7	267.7	3.7	0.7
9	8.5	1198.0	324.7	4.2	0.8
10	9.5	1453.9	394.0	4.7	0.8

L (sliding)-----------------------> 4.26
L (overturning)-------------------> 4.07
L (bearing capacity)--------------> 4.66
L (eccentricity)------------------> 4.99
L (rankine zone extension)---------> 4.99

L (selected) =====================> 4.99

t (required) = 652.4 lb/ft; at 0.10 strain
for a 1.0 ft spacing between fabric layers

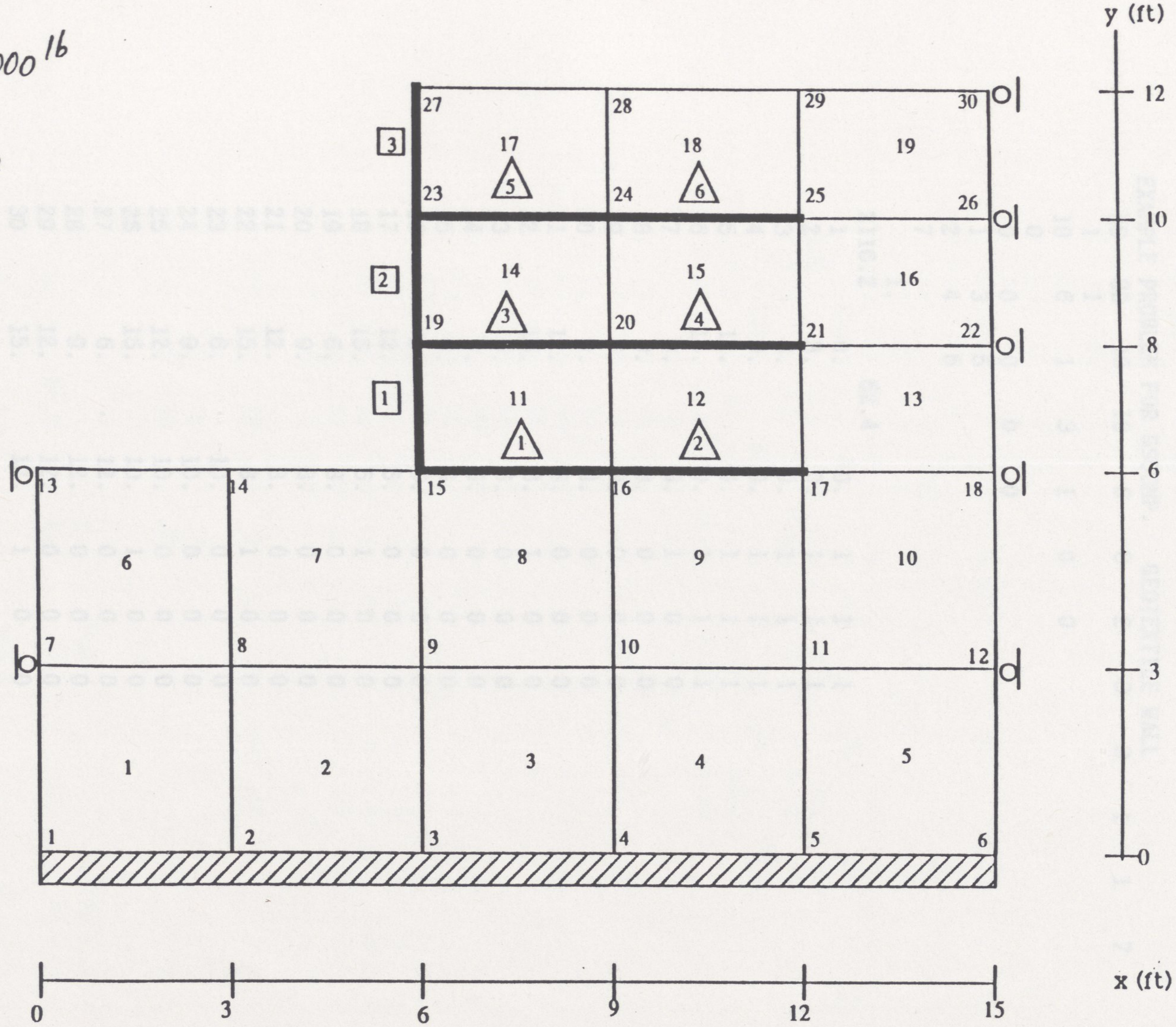
D = 2.99 inches; and should be less

APPENDIX B

Sample Example for the Program SSCOMP

Geosynthetic: EA = 4000 lb

Soil	Backfill	Foundation
γ (pcf)	125	115
c (psf)	0	200
ϕ	39°	30°
$\Delta\phi$	7°	0
K	600	90
n	0.6	0.45
R_f	0.7	0.7
K_b	175	80
m	0.2	0.2
K_o	0.37	0.5
K_{ur}	360	90



INPUT FILE :

```
EXAMPLE PROBLEM FOR SSCOMP.   GEOTEXTILE WALL
  19   30   10   18    0    0    2    3    3    1    1    7
   1    1
  10    6    1    3    1    0    0
   0
   0    0    0    0    0
   1    3    5
   2    4    6
   7
        1.
2116.2      62.4
   1       0.        0.    1    1    1
   2       3.        0.    1    1    1
   3       6.        0.    1    1    1
   4       9.        0.    1    1    1
   5      12.        0.    1    1    1
   6      15.        0.    1    1    1
   7       0.        3.    1    0    0
   8       3.        3.    0    0    0
   9       6.        3.    0    0    0
  10       9.        3.    0    0    0
  11      12.        3.    0    0    0
  12      15.        3.    1    0    0
  13       0.        6.    0    0    0
  14       3.        6.    0    0    0
  15       6.        6.    0    0    0
  16       9.        6.    0    0    0
  17      12.        6.    0    0    0
  18      15.        6.    1    0    0
  19       6.        8.    0    0    0
  20       9.        8.    0    0    0
  21      12.        8.    0    0    0
  22      15.        8.    1    0    0
  23       6.       10.    0    0    0
  24       9.       10.    0    0    0
  25      12.       10.    0    0    0
  26      15.       10.    1    0    0
  27       6.       12.    0    0    0
  28       9.       12.    0    0    0
  29      12.       12.    0    0    0
  30      15.       12.    1    0    0
```

```
 1   320000.    0.0125        0.
 1   15   16    1
 2   16   17    1
 3   19   20    1
 4   20   21    1
 5   23   24    1
 6   24   25    1
 1   320000.        0.    0.0125         0.        0.        0.
   0.           0.
 1   15   19    1    1    1
 2   19   23    1    1    1
 3   23   27    1    1    1
 1      115.0       90.       .45        .70       80.       .20
  200.0        30.         0.        0.5       90.
   2.0        160.       0.52        0.7       0.52
 2      125.0      600.       .60        .70      175.        .2
   0.0         39.        7.0       0.37      360.
  2.93        40.0       0.15       0.57      0.15
 1    1    2    8    7    1
 5    5    6   12   11    1
 6    7    8   14   13    1
10   11   12   18   17    1
11   15   16   20   19    2
13   17   18   22   21    2
14   19   20   24   23    2
16   21   22   26   25    2
17   23   24   28   27    2
19   25   26   30   29    2
 1   11   13   19   22   19   22    0    0    0    0    0    0    0    0
 2   14   16   23   26   23   26    0    0    0    0    0    0    0    0
 3   17   19   27   30   27   30    0    0    0    0    0    0    0    0
 1    0    0    0    0    0    1
 0    3
15   16   17   19   20   21   23   24   25   27
 2        0.
 1    1    1    5         3.
 2    1    6   10         6.
 2
 1    8
      0.          0.        0.5     302.4        2.     230.4       3.0  216.0
      5.       144.0        8.0      72.0       12.      28.8       16.   16.0
 2    9
      0.          0.        0.5     360.0        1.     374.4        2.  187.2
     3.0        57.6         5.      28.8        8.      14.4       12.   11.5
     16.         7.2
```

```
 1    2     0     1     2     0
 1          7.    2          8.0
11   19    15
19   22
 2    2     0     1     2     0
 1          7.    2          8.0
14   23    19
23   26
 3    2     0     1     2     0
 1          7.    2          8.0
17   27    23
27   30
 1   -1
 2   -1
 3   -1
 4   -1
 5   -1
 6   -1
 7   -1
 0    3
27   28    150.0     150.0    0
28   29    150.0     150.0    0
29   30    150.0     150.0    0
```

EXAMPLE PROBLEM FOR SSCOMP. GEOTEXTILE WALL

```
**************************************************
**************************************************
*                                    *
* LOAD CASE ********************************* 1    *
*                                    *
**************************************************
**************************************************
```

LARGEST ELE. NO. IN THIS INCREMENT 19

LARGEST N.P. NO. IN THIS INCREMENT 30

\f LOAD CASE = 1 ITERATION = 2

```
******************************************************
```

NP	DELTA-X	DELTA-Y	DELTA-ZZ	X-DISP	Y-DISP	ZZ-ROTAT	TOTAL	NP
1	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	1
2	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	2
3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	3
4	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	4
5	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	5
6	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	6
7	0.00000	-.00018	0.00000	0.00000	-.00040	0.00000	.00040	7
8	-.00207	.00014	0.00000	-.01315	.00081	0.00000	.01317	8
9	-.00291	-.00175	0.00000	-.01818	-.01108	0.00000	.02129	9
10	-.00232	-.00248	0.00000	-.01417	-.01654	0.00000	.02178	10
11	-.00122	-.00282	0.00000	-.00706	-.01833	0.00000	.01964	11
12	0.00000	-.00290	0.00000	0.00000	-.01845	0.00000	.01845	12
13	-.00400	.00007	0.00000	-.02241	.00119	0.00000	.0224	13
14	-.00428	.00133	0.00000	-.02361	.00790	0.0000	.02489	14
15	-.00529	-.00406	0.00000	-.02910	-.02670	0.00000	.03949	15
16	-.00388	-.00597	0.00000	-.02033	-.03993	0.00000	.04481	16
17	-.00199	-.00675	0.00000	-.01036	-.04309	0.00000	.04431	17
18	0.00000	-.00693	0.00000	0.00000	-.04344	0.00000	.04344	18
19	-.00764	-.00675	0.00000	-.03802	-.04409	0.00000	.05822	19
20	-.00418	-.00818	0.00000	-.02013	-.05124	0.00000	.05505	20
21	-.00182	-.00908	0.00000	-.01011	-.05471	0.00000	.05564	21
22	0.00000	-.00913	0.00000	0.00000	-.03749	0.00000	.03749	22
23	-.00723	-.01023	0.00000	-.03203	-.04596	0.00000	.05602	23
24	-.00390	-.01123	0.00000	-.01710	-.04511	0.00000	.04825	24
25	-.00172	-.01136	0.00000	-.00768	-.04498	0.00000	.04563	25
26	0.00000	-.01122	0.00000	0.00000	-.02769	0.00000	.02769	26
27	-.00592	-.01294	0.00000	-.02198	-.03693	0.00000	.04298	27
28	-.00363	-.01341	0.00000	-.00797	-.01499	0.00000	.01698	28
29	-.00190	-.01309	0.00000	-.00382	-.01352	0.00000	.01405	29
30	0.00000	-.01305	0.00000	0.00000	-.01318	0.00000	.01318	30

\f STRUCTURAL ELEMENTS - LINEAR ELASTIC

BAR ELEMENTS--INTERNAL MEMBER FORCES

ELEMENT NO.	AXIAL FORCE
INCREMENTAL VALUES	
1	1.8842
2	2.5180
3	4.6049
4	3.1491
5	4.4489
6	2.8980
TOTAL VALUES	
1	11.6945
2	13.2923
3	23.8539
4	13.3683
5	19.9036
6	12.5616

BEAM ELEMENTS-INTERNAL MEMBER FORCES

		NODE I		NODE J	
ELEMENT NO.	AXIAL FORCE	SHEAR FORCE	MOMENT	SHEAR FORCE	MOMENT
INCREMENTAL VALUES					
1	-5.3697	0.0000	0.0000	0.0000	0.0000
2	-6.9539	0.0000	0.0000	0.0000	0.0000
3	-5.4326	0.0000	0.0000	0.0000	0.0000
TOTAL VALUES					
1	-34.7811	0.0000	0.0000	0.0000	0.0000
2	-28.1315	0.0000	0.0000	0.0000	0.0000
3	-16.1295	0.0000	0.0000	0.0000	0.0000

\f FOUR NODE SOLID ELEMENTS - MODULI AND STRAINS (STRAINS IN PERCENT)

LE	ELAS MOD	BULK MOD	SHEAR MOD	POIS	EPS-X	EPS-Y	GAM-XY	EPS-1	EPS-3	GAMMAX	ELE
1	110546.6	132936.7	40600.2	.361	.219	-.007	.199	.257	-.044	.301	1
2	72902.6	128735.7	25932.6	.406	.084	.171	.720	.490	-.235	.726	2
3	73957.1	133171.5	26273.6	.407	-.067	.460	.630	.608	-.214	.822	3
4	75793.2	135286.7	26941.5	.407	-.119	.581	.384	.630	-.168	.798	4
5	79624.3	137033.8	28373.2	.403	-.118	.613	.120	.618	-.123	.740	5
6	46318.6	103802.3	16245.0	.426	.239	-.145	.416	.330	-.236	.566	6
7	53053.3	117465.1	18618.8	.425	.175	.142	1.131	.725	-.407	1.132	7
8	65651.7	124791.6	23242.5	.412	-.213	.650	.596	.743	-.306	1.049	8
9	65498.2	126691.7	23163.3	.414	-.285	.802	.240	.815	-.298	1.113	9
10	66861.5	127766.2	23663.1	.413	-.290	.829	.063	.830	-.291	1.121	10
11	86490.9	231995.2	30076.1	.438	-.371	.548	.443	.598	-.422	1.020	11
12	70449.6	220952.9	24345.7	.447	-.273	.415	.123	.421	-.278	.699	12
13	71758.2	220930.0	24814.9	.446	-.268	.411	.020	.411	-.269	.680	13
14	38739.5	191710.0	13209.8	.466	-.372	.437	.153	.444	-.379	.82	14
15	60027.3	204404.4	20684.0	.451	-.236	.317	.085	.320	-.240	.56	15

16	84636.1	206415.2	29558.7	.432	-.207	.270	.021	.270	-.207	.477	16
17	102945.1	185219.2	36573.7	.407	-.186	.178	.005	.178	-.186	.365	17
18	168467.3	210622.3	61633.3	.367	-.132	.136	.013	.137	-.132	.26	18
19	155974.6	218269.4	56475.7	.381	-.114	.118	.007	.118	-.114	.232	19

\f FOUR NODE SOLID ELEMENTS - STRESSES

ELE	SIG-X	SIG-Y	TAU-XY	SIG-1	SIG-3	TAU-MAX	THETA	SIG1/SIG3	LEVEL	SLCRIT	SIG-XC
1	706.801	735.610	64.540	787.334	655.077	66.128	38.709	1.202	.066	.140	86.920
2	654.940	876.102	181.404	977.972	553.070	212.451	29.317	1.768	.236	.236	84.819
3	710.402	1167.161	150.572	1212.331	665.232	273.550	16.699	1.822	.270	.270	56.754
4	738.767	1299.683	90.355	1313.878	724.572	294.653	8.929	1.813	.275	.275	27.708
5	777.172	1362.757	29.186	1364.208	775.721	294.243	2.846	1.759	.262	.262	28.602
6	395.675	207.857	71.397	419.734	183.798	117.968	71.377	2.284	.222	.222	143.544
7	609.169	532.930	214.969	789.372	352.727	218.323	50.028	2.238	.312	.312	150.214
8	520.809	877.350	127.465	918.232	479.927	219.152	17.783	1.913	.265	.265	91.369
9	532.462	1015.152	51.230	1020.529	527.085	246.722	5.992	1.936	.282	.282	42.797
10	552.714	1056.645	14.046	1057.036	552.323	252.356	1.595	1.914	.281	.281	42.495
11	248.834	855.260	153.782	892.028	212.066	339.981	13.447	4.206	.731	.731	25.404
12	191.316	757.004	53.334	761.988	186.332	287.828	5.339	4.089	.683	.683	0.000
13	191.338	760.252	10.392	760.442	191.148	284.647	1.046	3.978	.663	.663	0.000
14	94.044	499.434	40.941	503.527	89.950	206.789	5.710	5.598	.803	.803	21.781
15	129.889	577.499	36.211	580.409	126.979	226.715	4.595	4.571	.708	.708	33.479
16	129.107	516.421	12.645	516.833	128.695	194.069	1.868	4.016	.600	.600	51.440
17	53.727	251.095	4.290	251.188	53.634	98.777	1.245	4.683	.504	.504	43.030
18	106.719	298.904	7.422	299.190	106.433	96.379	2.208	2.811	.338	.338	87.711
19	133.597	271.002	4.578	271.155	133.445	68.855	1.906	2.032	.208	.208	109.801

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Division of Transportation Planning

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91-5 *Reflective Sheeting (Final)
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*Reports soon to be published

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90-4 Pavement Marking Materials
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90-14 Study of Urban Interchange Performance

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87-06 Bridge Approach Settlement
87-07
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