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A literature review and a computer study are conducted to identify the requirements of finite element programs for the specific use of discrete analyses of reinforced soil structures. The utility of six commercially available programs are appraised with respect to the identified requirements. The results of the study are directed to engineers and engineering students who wish to analyze reinforced soil structures and choose to utilize an available program for the analyses, instead of writing a special finite element program.

Three structures--a retaining wall, an embankment, and a strip footing are analyzed utilizing the SAP V, NONSAP, and ANSYS computer programs to study the effects of slippage and compression of the reinforcement. Also, effects of material models used and program differences on analyses of reinforced soil structures are considered. Special modeling techniques for slippage, sequential construction, and limitation of the tensile strength of soils for use in programs that do not offer direct methods for these factors are also investigated. Capabilities of the ANSYS, NONSAP, REA, SAP V, STARDYNE, and STRUDL programs, which may be desired for analyses of reinforced soil structures, are tabulated and an example is presented to aid in the selection of a program for analysis of a particular structure.

Conclusions of this study indicate that specific requirements of reinforced soil structures, such as slippage and the three-dimensional effects of strip reinforcing, greatly hinder the utility of generally available programs for use in analyses.

Evaluation of Finite Element Techniques for
Soil Reinforcement Applications

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CHAPTER I. INTRODUCTION

Background

The practice of reinforcing soil with inclusions of foreign material is centuries old. Examples of previous applications include the use of small tree trunks and branches in swampy areas to construct corduroy roads and the use of bundles of fibers or branches called "faggots" to stabilize soil along river banks (Lee et al., 1973). Modern reinforced earth structures were introduced in the 1960's by Henri Vidal (1967). Thousands of reinforced soil structures have been completed to date. Applications include retaining structures, mat foundations, and foundation for embankments.

Many of the design techniques presently utilized do not model the soil-reinforcement interaction. Only the finite element method lends itself to modeling the soil-reinforcement interaction.

The finite element method originated in the aerospace industry, and was first employed as a method of structural analysis (Turner et al., 1956). Today, uses of the method have expanded to other applications such as heat flow, fluid mechanics, rock mechanics, and soil mechanics. Applications of the finite element method in soil mechanics include analyses of shallow foundations, deep foundations, excavations, embankments, retaining structures, soil consolidation, and dynamic responses. Recently, the finite element method has been employed for analyses of reinforced soil structures (Al-Hussaini and Johnson, 1977; Chang, 1974).

Purpose

The purpose of this study is to identify the capabilities of finite element programs which may be required for the analysis of reinforced soil structures and to evaluate some available finite element programs.

Scope

The study includes a literature review of reinforced soil and finite element techniques. Previous studies of finite element analyses of reinforced soil and the capabilities of various computer programs are also reviewed. Criteria for selecting a finite element program, for use in the analysis of a reinforced soil structure, are identified and summarized. The results of the literature review are analyzed and limited computer analyses are made to evaluate the consequences of utilizing a program which does not meet certain criterion. The computer analyses are conducted with the SAP V (1977), NONSAP (1974), and ANSYS (1979) computer programs.

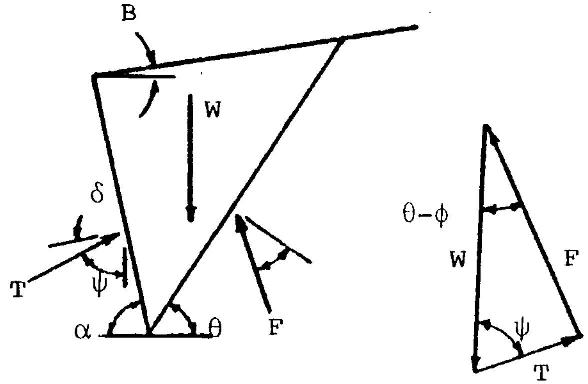
CHAPTER II. LITERATURE REVIEW

Soil Reinforcement

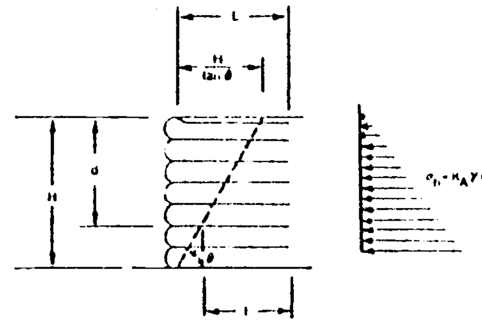
Reinforced soil has been defined as a material formed by the association of earth, cohesionless as well as slightly cohesive soil, and reinforcements, linear elements which are capable of sustaining significant tensile stresses (Vidal, 1967). Thin metal strips, typically three to four inches (7.6 to 10.2 cm) wide, and geotextiles are commonly used as reinforcing elements. Retaining structures also have facing elements and a mechanical connection between the reinforcing and facing element.

Lee et al. (1973) and Whitcomb and Bell (1979) have presented analysis techniques for reinforced soil retaining walls based on Rankine and Coulomb classical earth pressure theories. Wager (1976) has described the analysis of reinforced embankments based on plane strain circular arc stability. Figure 1 illustrates the models analyzed in which the magnitude and distribution of soil pressure and the failure surface are defined on the basis of classical soil mechanics theories, and the distribution of tensile force to the reinforcements and the effective length of the reinforcements are determined on empirical bases. The design procedures formulated with classical theories and on empirical criteria are typically based on maximum loads and stresses and do not model the interaction of the foundation, backfill or embankment, and reinforcing.

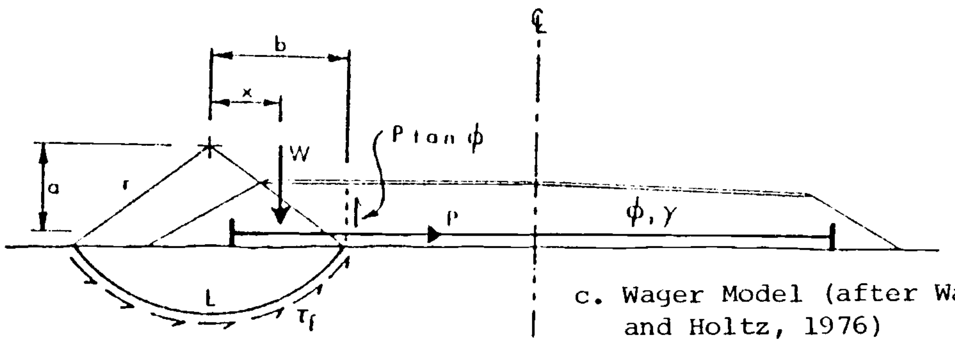
Schlosser and Long (1975) have studied the effect of soil-reinforcing interaction on the strength of a cohesionless soil, with triaxial test equipment, as reported by Hausmann (1978). This study revealed that the reinforcement clearly increases the strength and stiffness of the soil. Soil-reinforcing interaction has also been studied by Bassett and Last (1978). They concluded that the introduction of reinforcement apparently modifies the possible dilation of the soil mass and the alignment of the failure or slip field.



a. Coulomb Model (after Whitcomb and Bell, 1979)



b. Rankine Model (after Lee et al., 1973)



c. Wager Model (after Wager and Holtz, 1976)

$$FS = \frac{\tau_f L_r + P_a + P(\tan \phi)\, b}{W x} \geqq 1.5$$

FIGURE 1. Analysis Procedures.

It is evident that the strength of a soil structure and the locations of the failure planes through the structure are affected by the introduction of reinforcement into the system. It follows that the strength of the structure and the location of the failure plane may not be correctly defined when the analysis of a reinforced soil structure does not account for soil-reinforcement interaction. Therefore, it is desirable to model the soil-reinforcement interaction in an analysis. The finite element method is an analysis technique which offers the capability of modeling the soil-reinforcement interaction. The finite element method is based on working loads, therefore expected stresses and deformations of the actual structure are calculated in an analysis.

Finite Element Method

The finite element method may be defined as a process in which a continuum is approximated with a model of discrete subregions, with a finite number of unknowns, to study the unknowns of the continuum (Desai and Christian, 1977). Thus it is possible to model a reinforced soil structure with elements which represent the individual components of the structure and the interfaces between components. Unknowns, such as stresses and strains, may then be determined for a model with a given applied load. Typically, the finite element method consists of six basic steps or aspects (Desai and Abel, 1972). The six steps are:

1. Discretization of the continuum.
2. Selection of the displacement models.
3. Derivation of the element stiffness matrix using a variational principle.
4. Assembly of the algebraic equations for the overall discretized continuum.
5. Solutions for the unknown displacements.
6. Computation of the element strains.

The process of discretization consists of dividing the continuum into elements and labeling the elements and node points, which define the boundaries of the elements. General guidelines for discretization may be found in references by Desai and Abel (1972) and Segerlind (1976). However, the division of the continuum is basically dependent on the judgment of the engineer. Boundary conditions, which are prescribed dependent variables on the boundary of the region being analyzed, are a concern when defining the continuum (Desai and Christian, 1977). In soil-structure interaction analyses artificial boundaries may be located by trial and error, so as to have a negligible effect on the results of the analysis.

When appropriate, the model should make use of symmetry to decrease the required amount of computer storage and computation time. Likewise, a two-dimensional model is favored over a three-dimensional when appropriate.

Definition of interface elements between boundaries of the soil and structure are typically required in soil-structure interaction analyses. There are particular considerations when defining boundary conditions for dynamic analyses. Thorough discussions of boundary conditions for dynamic analyses have been presented by Desai and Christian (1977) and Valliappan et al. (1976).

Many different types of elements are used in the finite element method (Desai and Abel, 1972). One-dimensional elements are represented with lines. Two-dimensional elements are commonly represented with triangles or quadrilaterals. Tetrahedron and hexahedron elements are typically utilized for three-dimensional analyses.

Material nonlinearities and geometric nonlinearities are important when formulating a numerical analysis. Material nonlinearity is substantial when a material is not elastic or when a material is stressed beyond its yield point. Geometric nonlinearities should be considered when a structure is subjected to large deformations, such as a structure on a soft clay. Incremental and/or iterative techniques are commonly employed for nonlinear analyses (Desai and Abel, 1972).

The finite element technique is well-suited for use in soil dynamics. A discussion on the applications of finite element analyses in soil dynamics has been presented by Seed and Lysmer (1972) and by Roesset and Kausel (1976).

A weak spot in finite element method application is material analysis, the sophistication of the method demands that materials be modeled to be truly representative of prototype conditions (Anderson et al., 1972). Some of the structural or material parameters required for a finite element analysis may be physically difficult to determine. Finite element analyses are only as accurate as the parameters which are incorporated into the analyses.

Constitutive Laws and Modeling Techniques for Reinforced Soil

A popular approach for representing the nonlinear behavior of a soil is to approximate its stress-strain curve as a hyperbola (Duncan and Chang, 1970). Other techniques, such as bilinear and multilinear models, are also available for depicting the stress-strain behavior of a soil (Desai and Christian, 1977). Another desirable modeling characteristic is a material tension cut-off or limit. Clean granular materials, which are commonly used as backfills in reinforced soil structures, should ideally be modeled as materials with zero or near-zero tensile strengths. If a finite element analysis of an embankment or excavation is to be made, the analysis should model the construction sequence. Modeling of the construction sequence will more realistically simulate the stress path of the soil; therefore, resulting in more accurate and realistic evaluation of the soil stresses and strains.

A special problem which arises when analyzing a composite material such as reinforced soil is modeling the slippage between the soil and the reinforcement. Herrmann (1977) identified two possible slippage models. The first model was applied to reinforced concrete (Ngo and Scordelis, 1967) and introduced artificial springs between the reinforcement and soil. One normal and one tangential spring, which model the frictional bond between soil and reinforcing, are used to connect the soil nodal point to the reinforcing nodal point, as shown in Figure 2. Although this model is easily adaptable to many finite element programs, it has some disadvantages. The first is that the normal spring introduces an additional gobal unknown. The second disadvantage is that an inaccurate description of the tangential spring stiffness may lead to inadequate numerical characteristics. An additional disadvantage is that the "large" stiffness of the springs may overwhelm the "small" stiffness of the soil (Peterson, 1977). This problem may be avoided if one nodal displacement is treated as a respective displacement and the second nodal displacement is relative to the first nodal displacement.

The second slippage model also utilizes springs but differs from the first model in that the springs are released once slippage occurs (Herrmann, 1977, 1978). This method is not discussed further as it is not applicable to most available finite element programs.

Previous Studies of Finite Element Analysis of Reinforced Soil

Within this section the reinforced soil structures and finite element modeling techniques, utilized in previous research work, are described. The source of the finite element programs are also presented. Some of the results of these analyses will be further discussed in Chapter V.

Al-Hussaini and Johnson (1977, 1978), of the U.S. Army Engineer Waterways Experiment Station in Vicksburg, Mississippi, made an analysis of a reinforced soil wall in 1977. A full-scale test wall was constructed, instrumented, and loaded to failure. The 12-foot (3.6 m) high wall was founded on Vicksburg Loess. The wall was reinforced with 4-inch (10.2 cm) wide and 10 feet (3.0 m) long galvanized steel strips, spaced at intervals of 2 feet (0.61 m) vertically and 2.5 feet (0.76 m) horizontally, and had aluminum facing panels.

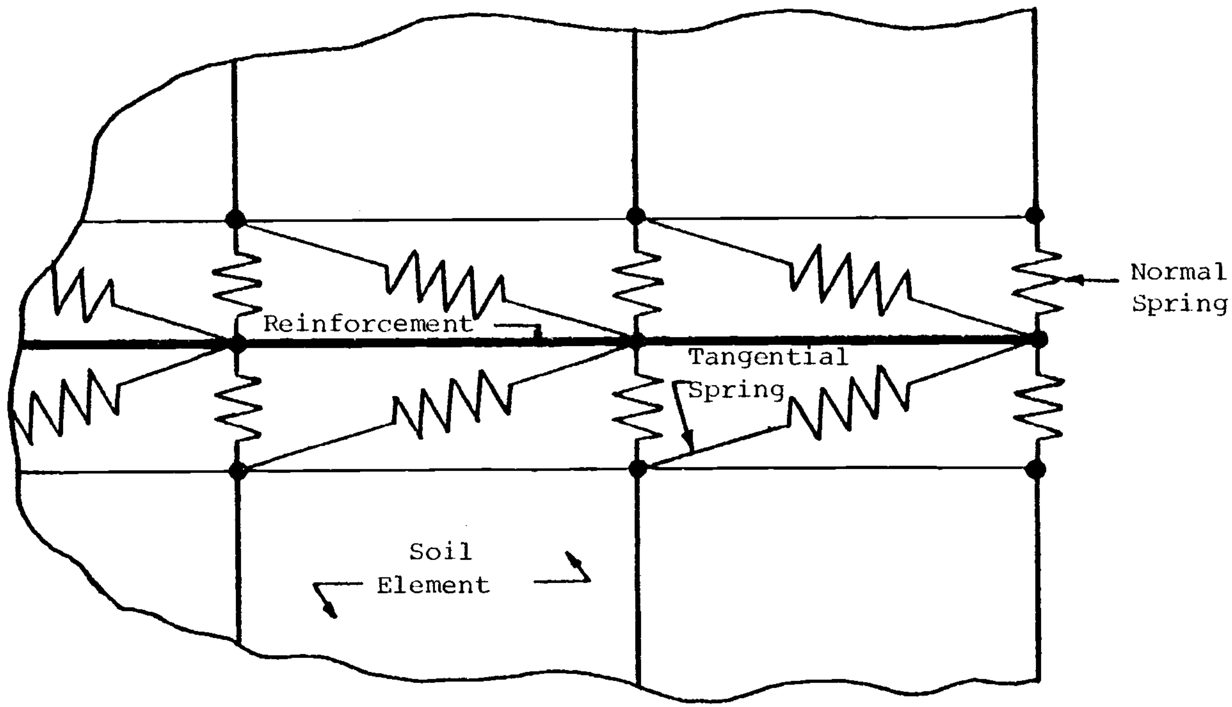


FIGURE 2. Slippage Model

A two-dimensional analysis was performed. The analysis included the effects of sequential construction, material nonlinearities, and slippage between and separation of the components. A modified version of a program written by Clough and Duncan (1969) for analyses of Port Allen and Old Rivers Locks, was utilized for the analysis. The techniques employed in the finite element analysis were evaluated by comparison to the field and laboratory recorded values. The writers concluded the following:

1. Interface elements must be inserted between the foundation soil and the back of the reinforced earth wall to permit separation and thereby allowing the reinforcing tensile stresses to decrease to zero to correlate with field observations.
2. Interface elements between soil and reinforcing are required to model friction and slippage.
3. Assuming that the three-dimensional reinforcing strip may be represented in two dimensions by increasing the area of the strip, to cover the entire length of wall, and decreasing the shear resistance between the soil and reinforcing by the same proportion is an appropriate and satisfactory assumption.
4. Boundary conditions of the wall play a significant role in the finite element results. Two analyses, one with fixed-end skin elements and the other with free-end skin elements, were necessary to obtain a good correlation with the field data.

The writers made the following recommendations, based upon their results, which are pertinent to this study:

1. Better constitutive equations for the behavior and interaction of the materials are required.
2. More studies are needed to improve the interface element between the soil and reinforcing.

Al-Hussani and Johnson (1977) also performed a parametric study which was concerned with the tensile stress distribution along the reinforcing. Results of this study are presented in their report. Conclusions drawn from this full-scale field test will be discussed further under the parametric studies section.

The first Reinforced Earth wall in the United States was constructed in California in 1972 (Chang et al., 1974). A large slide occurred along California State Highway 39 and a Reinforced Earth wall was utilized to reopen the route. The Reinforced Earth wall was selected as the best suited for foundation conditions and as the most economical alternative (Walkinshaw, 1975).

The design, construction, and instrumentation of the Highway 39 wall has been the basis of numerous papers. Chang, Forsythe, and Smith presented a report on the embankment in 1972; they followed up with a report on the performance of the embankment in 1974 which compared field measured values to design values. In 1974, Chang, of the California Department of Transportation, completed a very thorough study of the Highway 39 Reinforced Earth fill. The report covers the basic theory, design, construction, instrumentation and field data for the reinforced earth fill. A good correlation was found between the field data and design equation values. The Reinforced Earth fill was also evaluated with a finite element analysis.

The finite element analysis was made with a program entitled "Plane Strain Finite Element Incremental Construction Program for Embankment and/or Reinforced Earth Analysis with Beam Element and Material Property Options," developed by Professors L. R. Herrmann and K. M. Romstad of the University of California at Davis. This program incorporates the elastic properties of the soil and reinforcing into a composite material for analysis. After analysis, the composite stresses and strains are correlated to stresses and strains in the soil and reinforcements. The program is not capable of incorporating nonlinearities into the analysis, nor of modeling possible slippage between the reinforcement and soil. Capabilities of the program include modeling of the construction sequence and use of overburden dependent and orthotropic materials. A user manual, listing, and examples of input and output are appended to the Chang report. Since the publication of the Chang report, Herrmann (1978) has developed another finite element program, entitled REA.

The finite element analysis of the Highway 39 Reinforced Earth fill yielded results which were comparable to the field measured stress and strain values. The stress distributions were in good agreement. However, the magnitude of stresses were different. Chang hypothesized that the discrepancies could be attributed to:

1. Two dimensional idealization of a three-dimensional structure.
2. Not including a time function in the analysis to account for foundation settlement.
3. Improper simulation of the construction sequence.
4. Not accounting for the edge effect of the skin plate in the composite analysis.

The researchers were satisfied that the equivalent composite material approach to the analysis was suitable and yielded reasonable results. To improve the finite element analysis, it was proposed that the following be incorporated into the analysis:

1. Slippage of the reinforcement.
2. Elasto-plastic behavior of the soil and reinforcement.
3. Edge effects of the skin elements.
4. Improved techniques to deal with the overburden and time dependent effects of the composite materials of the elastic properties.

Romstad, Herrmann, and Shen (1976) presented two additional papers which were based on the Highway 39 Reinforced Earth fill. Their study was also based on composite material properties. Chang and Forsythe presented an additional paper in 1977 which dealt with this fill. These three papers were extrapolated from material previously presented and discussed.

Bell, Greenway, and Vischer (1977) have described a field test of fabric reinforced roads across muskeg and present the results of a finite element analysis of the road. The program NONSAP (1974) was utilized for the analysis. The two-dimensional, plane strain analysis modeled the material and geometric nonlinearities. However, the analysis did not model possible slip of the reinforcement. The purpose of this study was to evaluate the tension in the fabric reinforcement, and not to study finite element techniques. However, the analysis results are comparable to actual field values. Therefore, results of the study will be used elsewhere in this thesis when appropriate.

Additional studies on finite element analyses of reinforced soil, based on full-scale reinforced soil structures, have been reported. Simons, Frank, and Krüger (1979) of Germany used field measurements of pressures and friction, and finite element modeling to predict pressures and deformations due to different construction procedures. The writers developed their own finite element program, which accounted for the nonlinearity of the soil. Computed results were comparable to the measured values, with the measured values numerically greater.

Corté (1977) reports the use of the Rosalie finite element program for comparison of field values of two French Reinforced Earth structures. The analyses assume linear elastic soil behavior and do not model slippage of the reinforcement. The writer reports that the results of the analyses are qualitatively in very good agreement with field values.

Finite element analyses have also been used in conjunction with model tests to study reinforced soil and analysis techniques. Richardson and Lee (1974) utilized finite element analyses and results of model reinforced soil walls on a shaking table to study seismic design aspects. A modified version of the Berkeley computer program QUAD-4 (Idriss et al., 1973) was employed for the analyses. QUAD-4 includes strain dependent modulus and damping. The program was modified to include elastic tension-compression bar elements, which were used to model the reinforcing. Results of the study indicate that the finite element analysis can predict fairly accurate tie forces. However, the tie forces are very sensitive to the inputted soil properties. Richardson and Lee (1974), therefore, concluded that a dynamic finite element analysis, which uses bar elements for ties and appropriate non-linear strain dependent modulus and damping in the soil, led to calculated tie forces which were in reasonable agreement with the tie forces determined from the proposed seismic design envelope.

Jones (1978) has also reported the use of finite element analysis for reinforced soil applications. Jones states that his analysis is similar to that of Al-Hussaini and Johnson (1977), which has previously been discussed. Other finite element analyses have been utilized to study specific aspects of reinforced soil, such as slippage. One such study is that of Naylor and Richards (1978), in which an equivalent composite finite element model, which allows slippage, is developed and the importance of slippage is assessed. This study, along with others, will be referred to in the parametric studies chapter when appropriate.

Summary

A finite element analysis of a reinforced soil structure appears advantageous to an analysis procedure which was formulated with classical soil mechanic theories and on empirical data. The design equations are based on maximum loads and stresses and do not model the interaction of the structural components. A finite element analysis does model interaction of the components and is based on working loads and stresses. An additional, and usually desirable, feature of a finite element analysis is that the analysis yields the strains or deformation of the structure for a given loading.

As illustrated within the literature, finite element analyses are accurate when the model utilized for the analyses accurately describe the structure and loading. Modeling of a reinforced soil structure and loading includes describing the following parameters:

1. Material stress-strain characteristics;
2. Boundary conditions;
3. Interface conditions;
4. Loading sequence;
5. Structural response of individual components.

Some of the structural parameters required for a finite element analysis may be physically difficult to determine. Other parameters may be difficult or impossible to incorporate into a finite element analysis with a particular computer program. The effects of not modeling, or incorrectly describing, various parameters on the finite element analysis of a reinforced soil structure are studied in the parametric studies chapter of this thesis.

CHAPTER III. SUMMARY OF COMPUTER PROGRAM CAPABILITIES

Within this chapter the capabilities of a variety of commercially available finite element computer programs are summarized. The features or program capabilities included are those which may be desirable to incorporate into an analysis of a reinforced soil structure as indicated in the literature.

The following commercially available (except REA) programs are considered: NONSAP (Bathe et al., 1974); SAP V (1977); STRUDL (Logcher et al., 1968); ANSYS (DeSalvo and Swanson, 1979); STARDYNE (1977); REA (Herrmann, 1978). The coding system used for summarizing program capability is as follows:

0 - Not a feature of the program.

1 - Not a feature, but possible to compensate for with modeling techniques. However, undesirable numerical characteristics may result.

2 - Not a feature, but possible to compensate for with modeling techniques resulting in a high confidence in numerical results.

3 - A feature of the program.

Although only six programs are summarized in Tables 1 through 7, it is obvious that, with the proper documentation, the capabilities of additional programs may easily be analyzed by the reader. Utilizing the suggestions in Chapter IV along with Tables 1 through 7, it is possible to determine the suitability of a program for an analysis of a specific reinforced soil structure.

The significance of the various computer features on finite element analyses of reinforced soil are discussed in Chapter V.

TABLE 1. ANSYS Capabilities

MATERIAL CHARACTERISTICS \ ELEMENTS	2-D Truss	3-D Truss	2-D Beam	3-D Beam	2-D Plane Strain	3-D Solid	2-D Frictional and/or Gap	3-D Frictional and/or Gap	2-D Plate and/or Shell	3-D Plate and/or Shell
LINEAR ELASTIC	3	3	3	3	3	3	3	3	3	3
1. Isotropic	3	3	3	3	3	3	3	3	3	3
2. Anisotropic	0	0	0	0	3	3	0	0	0	3
3. Small strain	3	3	3	3	3	3	3	3	3	3
NONLINEAR	3	3	3	0	3	3	3	3	3	0
1. Small strain	3	3	3	0	3	3	3	3	3	0
2. Material	3	3	3	0	3	3	3	3	3	0
a. Strain softening	2	2	2	0	2	3	0	0	3	0
b. Perfectly plastic	3	3	3	0	3	3	3	3	3	0
c. Strain hardening	3	3	3	0	3	3	3	3	3	0
3. Geometric	3	3	3	0	3	3	3	3	3	0
4. Creep	3	3	3	0	3	3	0	0	3	0
5. Stress-strain curve description	3	3	3	0	3	3	0	0	3	0
6. Hyperbolic curve	0	0	0	0	0	0	0	0	0	0
7. Isotropic	3	3	3	3	3	3	3	3	3	3
8. Anisotropic	0	0	0	0	0	0	0	0	0	0
TENSION CUTOFF	2	3	0	0	2	2	3	3	2	0
COMPRESSION CUTOFF	2	3	0	0	2	2	0	0	2	0
PRESTRESS and/or PRESTRAIN	2	3	2	2	0	0	3	3	2	2

TABLE 2. NONSAP Capabilities

MATERIAL CHARACTERISTICS \ ELEMENTS	2-D Truss	3-D Truss	2-D Beam	3-D Beam	2-D Plane Strain	3-D Solid	2-D Frictional and/or Gap	3-D Frictional and/or Gap	2-D Plate and/or Shell	3-D Plate and/or Shell
LINEAR ELASTIC	3	3	0	0	3	3	0	0	0[a]	0
1. Isotropic	0	0	0	0	3	3	0	0	0	0
2. Anisotropic	0	0	0	0	3	0	0	0	0	0
3. Small strain	3	3	0	0	3	3	0	0	0	0
NONLINEAR	3	3	0	0	3	3	0	0	0	0
1. Small strain	3	3	0	0	3	3	0	0	0	0
2. Material	3	3	0	0	3	3	0	0	0	0
a. Strain softening	2	2	0	0	0	0	0	0	0	0
b. Perfectly plastic	2	2	0	0	3	2	0	0	0	0
c. Strain hardening	2	2	0	0	3	2	0	0	0	0
3. Geometric	3	3	0	0	3	0	0	0	0	0
4. Creep	0	0	0	0	0	0	0	0	0	0
5. Stress-strain curve description	3	3	0	0	3	3	0	0	0	0
6. Hyperbolic curve	0	0	0	0	0	0	0	0	0	0
7. Isotropic	0	0	0	0	3	3	0	0	0	0
8. Anisotropic	0	0	0	0	0	0	0	0	0	0
TENSION CUTOFF	2	2	0	0	0	0	0	0	0	0
COMPRESSION CUTOFF	2	2	0	0	0	0	0	0	0	0
PRESTRESS and/or PRESTRAIN	3	3	0	0	0	0	0	0	0	0

[a] 2-D plane strain element for axisymmetric 2-D analysis with axisymmetric loading available.

TABLE 3. REA Capabilities

MATERIAL CHARACTERISTICS \ ELEMENTS	2-D Truss	3-D Triss	2-D Beam	3-D Beam	2-D Plane Strain	3-D Solid	2-D Frictional and/or Gap	3-D Frictional and/or Gap	2-D Plate and/or Shell	3-D Plate and/or Shell
LINEAR ELASTIC	0[a]	0	3	0	3	0	3	0	0	0
1. Isotropic	0	0	3	0	3	0	3	0	0	0
2. Anisotropic	0	0	0	0	3	0	0	0	0	0
3. Small strain	0	0	3	0	3	0	3	0	0	0
NONLINEAR	0	0	3	0	3	0	0	0	0	0
1. Small strain	0	0	3	0	0	0	0	0	0	0
2. Material	0	0	3	0	0	0	0	0	0	0
a. Strain softening	0	0	0	0	0	0	0	0	0	0
b. Perfectly plastic	0	0	0	0	0	0	0	0	0	0
c. Strain hardening	0	0	3	0	0	0	0	0	0	0
3. Geometric	0	0	0	0	0	0	0	0	0	0
4. Creep	0	0	0	0	0	0	0	0	0	0
5. Stress-strain curve description	0	0	0	0	0	0	0	0	0	0
6. Hyperbolic curve	0	0	0	0	3	0	0	0	0	0
7. Isotropic	0	0	3	0	3	0	0	0	0	0
8. Anisotropic	0	0	0	0	0	0	0	0	0	0
TENSION CUTOFF	0	0	0	0	3	0	0	0	0	0
COMPRESSION CUTOFF	0	0	0	0	0	0	0	0	0	0
PRESTRESS and/or PRESTRAIN	0	0	3	0	0	0	3	0	0	0

[a]Reinforcing elements are represented with bending elements.

TABLE 4. SAP V Capabilities

MATERIAL CHARACTERISTICS \ ELEMENTS	2-D Truss	3-D Truss	2-D Beam	3-D Beam	2-D Plane Strain	3-D Solid	2-D Frictional and/or Gap	3-D Frictional and/or Gap	2-D Plate and/or Shell	3-D Plate and/or Shell
LINEAR ELASTIC	3	3	3	3	3	3	0	0	3	3[b]
1. Isotropic	3	3	3	3	3	3	0	0	3	3
2. Anisotropic	0	0	0	0	3	0	0	0	3	3
3. Small strain	3	3	3	3	3	3	0	0	3	3
NONLINEAR	0	0	0	0	0	0	0	0	0	0
1. Small strain	0	0	0	0	0	0	0	0	0	0
2. Material	0	0	0	0	0	0	0	0	0	0
a. Strain softening	0	0	0	0	0	0	0	0	0	0
b. Perfectly plastic	0	0	0	0	0	0	0	0	0	0
c. Strain hardening	0	0	0	0	0	0	0	0	0	0
3. Geometric	0	0	0	0	0	0	0	0	0	0
4. Creep	0	0	0	0	0	0	0	0	0	0
5. Stress-strain curve description	0	0	0	0	0	0	0	0	0	0
6. Hyperbolic curve	0	0	0	0	0	0	0	0	0	0
7. Isotropic	0	0	0	0	0	0	0	0	0	0
8. Anisotropic	0	0	0	0	0	0	0	0	0	0
TENSION CUTOFF	0	0	0	0	0	0	0	0	0	0
COMPRESSION CUTOFF	0	0	0	0	0	0	0	0	0	0
PRESTRESS and/or PRESTRAIN	2[a]	2[a]	0	0	0	0	0	0	0	0

[a]With temperature.

[b]Thick shell.

TABLE 5. STARDYNE Capabilities

MATERIAL CHARACTERISTICS \ ELEMENTS	2-D Truss	3-D Truss	2-D Beam	3-D Beam	2-D Plane Strain	3-D Solid	2-D Frictional and/or Gap	3-D Frictional and/or Gap	2-D Plate and/or Shell	3-D Plate and/or Shell
LINEAR ELASTIC	2	2	3	3	3	3	3[b]	0	2	0
1. Isotropic	3	3	3	3	3	3	0	0	3	0
2. Anisotropic	0	0	0	0	3	0	0	0	3	0
3. Small strain	3	3	3	3	3	3	0	0	3	0
NONLINEAR	0[a]	0[a]	0	0	0	0	0	0	0	0
1. Small strain	0	0	0	0	0	0	0	0	0	0
2. Material	0	0	0	0	0	0	0	0	0	0
a. Strain softening	0	0	0	0	0	0	0	0	0	0
b. Perfectly plastic	0	0	0	0	0	0	0	0	0	0
c. Strain hardening	0	0	0	0	0	0	0	0	0	0
3. Geometric	0	0	0	0	0	0	0	0	0	0
4. Creep	0	0	0	0	0	0	0	0	0	0
5. Stress-strain curve description	0	0	0	0	0	0	0	0	0	0
6. Hyperbolic curve	0	0	0	0	0	0	0	0	0	0
7. Isotropic	0	0	0	0	0	0	0	0	0	0
8. Anisotropic	0	0	0	0	0	0	0	0	0	0
TENSION CUTOFF	3	3	0	0	0	0	3	0	0	0
COMPRESSION CUTOFF	3	3	0	0	0	0	0	0	0	0
PRESTRESS and/or PRESTRAIN	2	2	2	0	0	0	0	0	0	0

[a] 1. Nonlinear in that compression or tension may be limited.
2. Nonlinear spring available for specific uses, such as dynamic analyses and as an exterior boundary element.

[b] Gap element.

TABLE 6. STRUDL[a] Capabilities

MATERIAL CHARACTERISTICS \ ELEMENTS	2-D Truss	3-D Truss	2-D Beam	3-D Beam	2-D Plane Strain	3-D Solid	2-D Frictional and/or Gap	3-D Frictional and/or Gap	2-D Plate and/or Shell	3-D Plate and/or Shell
LINEAR ELASTIC	3	3	3	3	3	3	0	0	3	0
1. Isotropic	3	3	3	3	3	3	0	0	3	0
2. Anisotropic	0	0	0	0	0	0	0	0	0	0
3. Small strain	3	3	3	3	3	3	0	0	3	0
NONLINEAR	3	3	3	3	3	0	0	0	0	0
1. Small strain	0	0	0	0	0	0	0	0	0	0
2. Material	0	0	0	0	0	0	0	0	0	0
a. Strain softening	0	0	0	0	0	0	0	0	0	0
b. Perfectly plastic	0	0	0	0	0	0	0	0	0	0
c. Strain hardening	0	0	0	0	0	0	0	0	0	0
3. Geometric	3	3	3	3	3	0	0	0	0	0
4. Creep	0	0	0	0	0	0	0	0	0	0
5. Stress strain curve description	0	0	0	0	0	0	0	0	0	0
6. Hyperbolic curve	0	0	0	0	0	0	0	0	0	0
7. Isotropic	0	0	0	0	0	0	0	0	0	0
8. Anistropic	0	0	0	0	0	0	0	0	0	0
TENSION CUTOFF	0	0	0	0	0	0	0	0	0	0
COMPRESSION CUTOFF	0	0	0	0	0	0	0	0	0	0
PRESTRESS and/or PRESTRAIN	2	2	2	2	0	0	0	0	0	0

[a]Updated through GTSTRUDL (Emkin et al., 1980).

TABLE 7. Additional Program Features

FEATURES \ PROGRAMS	ANSYS	NONSAP	REA	SAP V	STARDYNE	STRUDL
TYPES OF LOADING						
1. Gravity	3	2	3	3	3	3
2. Concentrated	3	3	3	3	3	3
3. Boundary pressure	3	2	3	3	3	3
4. Time varying	3	3	3	3	3	3
5. Dynamic	3	3	0	3	3	3
6. Frictional forces	3	0	3	0	0	0
OTHER DESIRABLE FEATURES						
1. Mix linear and nonlinear elements	3	3	3	0	0	0
2. Sequential addition of elements	0	0	3	0	0	0
3. Hinge element	2[a]	0	0	0	0	0
4. Element and/or node generator	3	3	3	3	3	3
5. Element and/or node renumbering	3	0	0	3	3	3
6. Master-slave node coupling	3	0	0	0	0	0
7. Restart capability	3	3	0	3	3	3
DOCUMENTATION MANUALS						
1. Data preparation	3	3	3	3	3	3
2. Programmer	0	3	3	0	3	3
3. Theoretical	3	3	0	3[b]	3	3
4. Sample problems	3	3	3	3	0	3

[a]With node coupling.

[b]With SAP IV (Bathe, Wilson and Peterson, 1974).

CHAPTER IV. FINITE ELEMENT METHOD PROGRAM SELECTION FOR REINFORCED SOIL PROBLEMS

Reinforced Soil Applications

Holtz (1978) has documented many reinforced soil applications. Three general areas of reinforced soil applications are considered in the following discussions: retaining structures, reinforced mats or slabs, and reinforcement between an embankment and its foundation. Retaining structures include reinforced walls, bridge abutments, wing walls, quay walls, and dams.

Program Requirements

The first step in a finite element analysis of a reinforced soil structure is deciding if a two-dimensional analysis, as opposed to three-dimensional, is adequate. Structures that vary with length, such as a bridge abutment or mat foundation, may require three-dimensional modeling for accurate analyses. Accurate modeling of strap reinforcement may also necessitate three-dimensional representation. The need of a three-dimensional analysis will eliminate the use of some computer programs immediately.

A two-dimensional analysis is desirable, when it is justifiable, as the cost of a two-dimensional analysis is much less than that of a three-dimensional analysis. Two-dimensional analysis is adequate if the structure, such as a retaining wall, embankment, or strip footing, and the loads acting on the structure can be considered as infinitely long. The use of equivalent two-dimensional systems to approximate three-dimensional problems has been discussed by Desai and Christian (1977).

Next to be considered are the program elements needed to model a specific reinforced soil structure. For example, it may be desirable to model a reinforced soil wall with two-dimensional plane strain elements representing the soil and truss elements and beam elements representing the reinforcing and facing panels, respectively. The engineer has the option of using a program which contains these elements in its library or substituting an available element in place of the desired element and accepting reduced accuracy. The application for which an element was intended is typically presented in the program documentation. The engineer must assess the suitability of the individual elements before incorporating them into an analysis.

Additional requirements which must be evaluated to make an intelligent choice of a finite element program are the material models, loading conditions, and deformations. The materials may be separated into two classes: the structural components and the soils. The reinforcing falls under structural components and may be divided into three groups: metal, geotextile and other reinforcing. A discussion of material models for metals and geotextiles follows; requirements of other reinforcing must be evaluated individually by the engineer.

A reasonable material model for the metal reinforcing is a linear elastic behavior until the yield stress is reached, then the metal fails plastically as described by Al-Hussaini and Johnson (1977). A linear model would be adequate if the metal is not stressed beyond its yield point.

Geotextiles are nonlinear materials and, therefore, should be modeled as such. The use of a nonlinear elastic material model, as presented by Bell, Greenway, and Vischer (1977), appears to be a satisfactory model. The behavior of some geotextiles, however, may permit the use of a linear model with the introduction of only small errors.

Other structural components, such as facing elements for a wall, may require additional elements for modeling. For example, concrete facing components should be modeled with beam elements, as opposed to axial or truss elements, to account for the bending stiffness of the panel. Concrete facing components may also be modeled with two- or three-dimensional elements. Facing components are typically constructed with compressible spacers between individual panels to compensate for wall settlement (Vidal, 1978). A hinge element may be used to model the compressible spacer between the structural elements. Additional structural components may require the use of other elements for appropriate modeling.

Soil is a nonlinear material and should typically be modeled as such. Further, cohesionless soils may exhibit little to no tensile strength and the response of a soil to a load is also a function of its confining pressure. Soils may respond elastically for very small strains. The strength of a dense, cohesionless soil may be defined with a modulus and Poisson's ratio, which apply for that particular loading. A dense, cohesionless soil will lose strength, or exhibit strain softening, when stressed beyond its peak strength (Lambe and Whitman, 1969). Clays may display strain hardening or may be represented as a perfectly plastic material.

Once the material stress-strain relationship is identified the capabilities of the computer programs to model the relationship may be evaluated from Tables 1 through 6. If a program was written for use in geotechnical engineering, it is likely that it will contain a hyperbolic relation to describe the soil behavior. Programs developed for structural analysis may or may not offer nonlinear analysis. If nonlinear analysis is possible, an individual program may offer different nonlinear material models. The engineer must evaluate the different models for their suitability in representing the actual soil. Some material models may not allow iteration, which will limit application of the model. Other materials may be differentiated by the failure criteria used. The effect of using a linear model to represent a nonlinear soil will be discussed in the parametric studies chapter.

Some programs also offer a tension cut-off option. This option may be necessary if a prior analysis (without the tension cut-off option) indicates high tensile stresses in the soil. Soils may also exhibit anisotropic or orthotropic material properties. Certainly it is desirable to model the soil as accurately as possible, therefore, anisotropic conditions should be modeled when necessary.

Loading conditions are another major area of requirements of finite element program for use in reinforced soil analyses. Loads on the actual structure must be defined and be applied to the computer model. Concentrated, gravity, and boundary pressure loads, which represent line, column, and surcharge loads, weight of the components, and water pressures and surcharge loads, respectively, are typically desired in an analysis. Most finite element programs will specify that concentrated loads be placed on nodal points only. Specifying the gravity load (unit weight) of a material may not be possible with some programs. In that case, the gravity load may only be applied as concentrated nodal points loads. This should have negligible affect on the analysis of properly discretized model. Boundary pressure loads available in some programs may be limited to only one side of an element. Whether or not this is adequate is another consideration in selecting a program.

Water pressure may or may not be a boundary load in a soil-structure interaction. A change in water pressure on one side of an impervious, structural material will result in a boundary pressure load on the structure. While a change in water pressure on a pervious soil element results in seepage through the material with the excess head being dissipated in the material. The load, from the change in water pressure on the soil, is a distributed load which may be represented as concentrated loads at the nodal points (Clough and Duncan, 1969).

As discussed in the Literature Review, it is often desirable to account for the sequential construction procedure in an analysis. In some programs it may not be possible to directly account for incremental construction, therefore, leading to use of various techniques to model the construction sequence. Specific modeling techniques are discussed in the parametric studies chapter.

Seismic loading is another type of loading which may be necessary to incorporate into an analysis. The program must have a dynamic analysis option to perform this analysis, unless an equivalent lateral load procedure is developed and utilized with a static analysis.

Deformations are the final major area of requirements which should be evaluated when selecting a program for a finite element analysis of a reinforced soil structure. The following types of deformations may be required in an analysis: small strain, large strain, material nonlinearity, large geometric displacement, creep and slippage. The engineer must determine the type(s) of deformations which must be considered and select a program which meets the requirements.

In Figure 3, an example checklist--or outline of considerations--for selecting a program for finite element analyses of a geotextile reinforced soil embankment is presented. Obviously, some other embankment structures may have a unique requirement, for which the program requirement may not appear in this example. Utilizing Tables 1 through 7, along with the example checklist, it is possible to determine the suitability of a program for analyses of the example embankment. The length of the embankment and reinforcement is much greater than the width, therefore a two-dimensional analysis will suffice. Truss and two-dimensional plane strain elements are desired to model the reinforcing and soil, respectively, with material models as noted in Figure 4. It is also desired to account for slippage, geometric nonlinearities, and sequential construction in the analyses; and apply unit weight and concentrated loads to the structure. If the suitability of the NONSAP program, for example, is being assessed, Tables 2 and 7 point out that it is not possible to account for slippage or sequential construction. The suitability of this program is not clear-cut because it meets most but not all of the desired features. The significance of the missing features on the results of the finite element analyses must be assessed to determine if the program is suitable for this particular application.

The significance of certain computer features on finite element analysis of reinforced soil are discussed in the parametric studies

Finite Element Analysis of a Reinforced Soil Embankment

Dimensional representation of structure: 2-D

Computer representation of structure:

Actual Component	Computer Element	Material Model
a. Reinforcement	Truss	Multilinear
b. Other structural components		
c. Embankment soil	2-D plane strain	Linear
d. Base or foundation soil	2-D plane strain	Nonlinear
e. Other		

Deformations:

a. Slippage	Yes, model with nonlinear springs
b. Geometric nonlinearities	Yes
c. Other	

Loading:

a. Dynamic	No
b. Sequential construction	No
c. Static	
1. Concentrated	Yes
2. Boundary pressure	No
3. Unit weight	Yes
4. Other	No

FIGURE 3. Example Program Selection

chapter. The significance is based on literature of previous researchers and/or computer analyses of example structures. These studies are typically based on one computer program and one particular type of structure, but the assessments will be extrapolated to other programs and types of structures if appropriate.

It should be noted that just the use of a suitable program does not guarantee satisfactory analyses. The constitutive laws for the materials and interfaces, which can be very difficult to determine, must also be appropriately defined to properly model the structure. Discussions on constitutive laws for geologic media and tension resistant inclusions in soils have been presented by Desai and Christian (1977) and Andrawes et al. (1980), respectively.

CHAPTER V. PARAMETRIC STUDIES

The effects of not incorporating various parameters, for example slippage of the reinforcement and sequential construction, into a finite element analysis of a reinforced soil structure are studied within this chapter. The studies are based on the literature and on computer analyses conducted by the author. A brief description of the computer models utilized precedes the studies, detailed descriptions are given in the Appendices. Conclusions are summarized in Chapter VI.

Computer Models

Three basic structures were studied: a retaining wall, an embankment, and a slab foundation as shown in Figure 4. A variety of analyses, incorporating different parameters and modeling techniques, were conducted. Analyses were performed utilizing the SAP V (1977), NONSAP (Bathe et al., 1974), or ANSYS (DeSalvo and Swanson, 1979) computer programs.

The retaining wall model was similar to the wall studied by Al-Hussaini and Johnson (1977) except concrete facing elements were assumed instead of aluminum skin elements as used by Al-Hussaini and Johnson. The 12 feet (3.6 m) high by 11 feet (3.4 m) deep retaining wall was reinforced with six galvanized steel strips; 4 inches (406 mm) wide, 0.024 inches (0.16 mm) thick, and 10 feet (3.0 m) long, spaced on 2.5 feet (0.76 m) centers horizontally and equally spaced vertically. The backfill was a dense, dry sand with Ø = 36°. A soft clay or a firm loess was assumed as the foundation soil. A surcharge load of up to 1500 lb/ft^2 (71.8 KPa) was placed on the backfill to approximate the failure load observed by Al-Hussaini and Johnson (1977).

The embankment studied was 4.5 feet (1.4 m) high, with a base width of 21 feet (6.4 m) and 1:1 side slopes (Bell, Greenway and Vischer, 1977). The granular embankment was separated from the saturated muskeg foundation by a continuous layer of geotextile reinforcement.

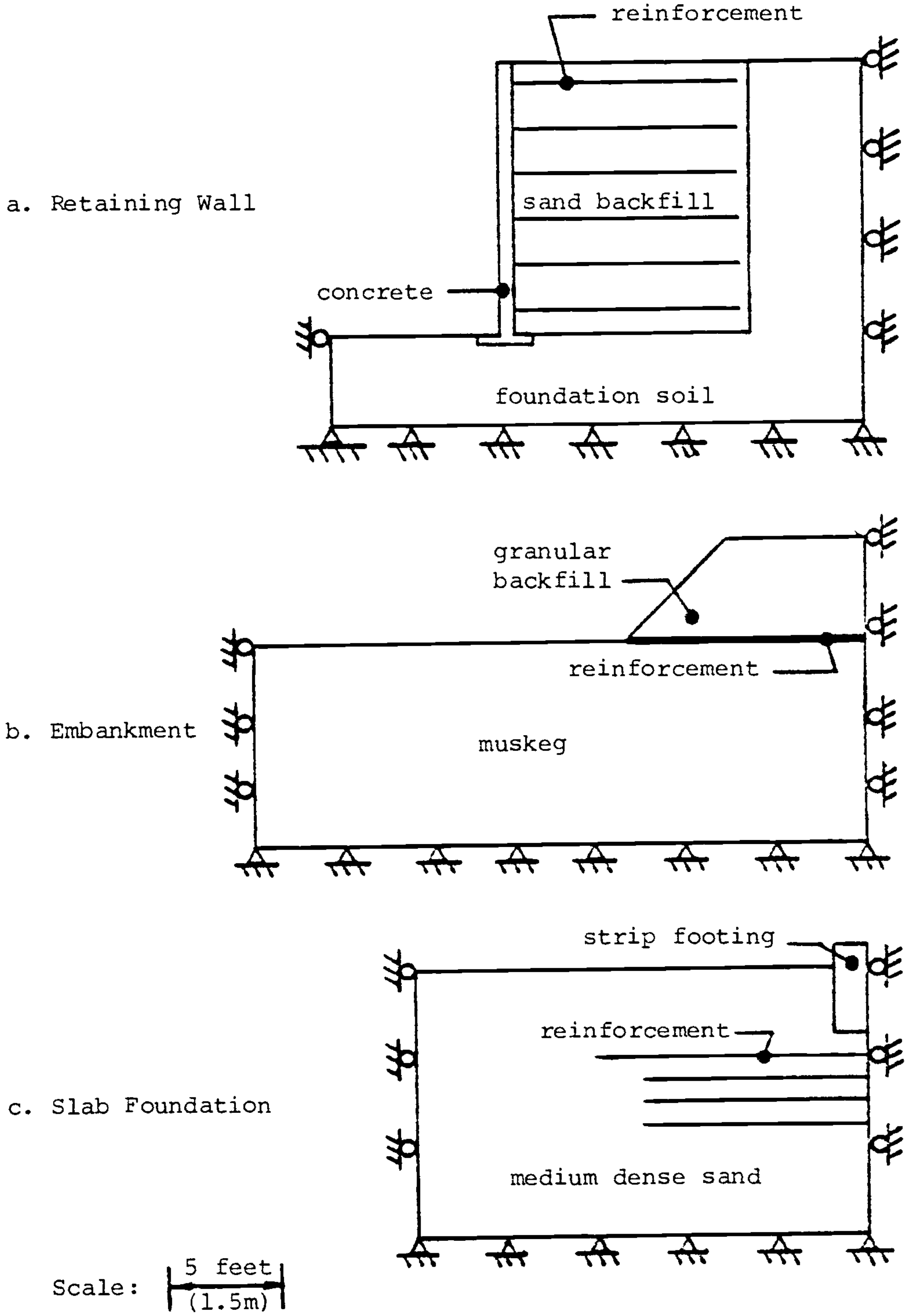


FIGURE 4. Reinforced Soil Structures

Live loads on the embankment were modeled by 1000 lb/ft (1.36 kN/m) line loads acting at points 1.5 feet (0.46 m) and 3.0 feet (0.91 m) in from the upper edges of the embankment.

The slab foundation consisted of four steel reinforcing strips, spaced at one-foot (0.30 m) centers horizontally, 0.06 inches (1.5 mm) thick, and 3.0 inches (76 mm) wide, embedded in dense granular soil. Three strips, 20 feet (6.1 m) long, were located at depths 5, 6, and 7 feet (1.5 m, 1.8 m, 2.1 m) below grade; the fourth strip was 24 feet (7.3 m) long and was located at a depth of 4 feet (1.2 m). The live load of 55 Kips/ft^2 (2.6 MPa) was applied through a 3-feet (0.91 (0.91 m) wide grade beam, centered above the reinforcing and located at a depth of 3 feet (0.91 m).

Slippage of Reinforcement

Based upon laboratory and field tests and on finite element analyses at the end of construction and just prior to failure, Al-Hussaini and Johnson (1977) concluded that interface elements between metal strip reinforcement and soil are necessary to model the slippage and friction forces between the components. Naylor and Richards (1978) also studied the significance of slipping with an idealized wall designed with a factor of safety of 1.5 and 2.0 against tie breakage and tie pullout, respectively. The study of Naylor and Richards revealed the following:

1. The slip analysis indicated slipping over a significant length of the strips.
2. The slip analysis predicted wall face displacements up to 17 percent greater than those predicted by the no-slip analyses for the example studied.
3. The locus of the point of maximum stress in the strips was significantly different between the two analyses.
4. The maximum tension in the strips was unaffected.

Naylor and Richards concluded that slipping is significant.

Bell, Greenway, and Vischer (1977), however, reported good agreement between actual field deflections of a geotextile reinforced embankment on muskeg and the deflections calculated with finite element analyses which did not account for possible slippage of the reinforcement.

Herrmann (1977) studied the effects of slippage and accounting for the edge effects of the facing elements on analyses of an experimental wall tested by Al-Hussaini and Perry (1976). The significance of slippage and edge effects are illustrated in Figure 5, where F is the force in one of the bottom reinforcing strips when the wall was at 83 percent of the final height and F_o is the force developed if the reinforcement provides complete confinement to the soil. Not accounting for slippage and edge effects resulted in predicted forces in the reinforcement which were more than 50% higher than experimental results. When slippage and edge effects were considered the predicted forces in the reinforcement were in very good agreement with the experimentally determined forces.

Analyses of the retaining wall and the embankment were conducted to study modeling techniques for and the significance of slippage. The retaining wall was analyzed with the ANSYS program and utilized a two-dimensional interface or gap element to model the frictional bond between the reinforcement and backfill soil. Analyses with the gap element resulted in reinforcement stresses that were equal to the stresses computed without gap elements and the computed deflections were less with the gap element included. Thus, the gap element added stiffness to the overall system rather than increase the flexibility of the system as desired. The influence of the gap element was not correct.

Kelley (1981) confirmed that the two-dimensional interface or gap element cannot model the possible slippage between reinforcement and soil as desired. The utilization of a gap element to model reinforcement slippage requires further study.

The embankment was analyzed with NONSAP and used normal and tangential springs to model possible slippage. A discretized model of

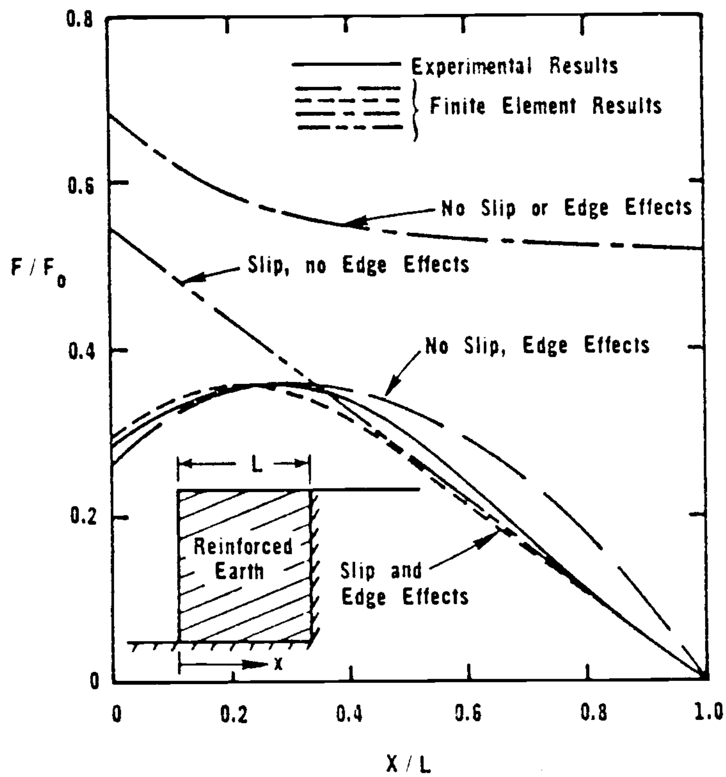


FIGURE 5. Effects of Slippage and Edge Effects (after Herrmann, 1977)

the embankment is shown in Figure 6. Distribution of calculated fabric tension is shown in Figure 7; the low stress in the first 1.5 feet (0.46 m) of geotextile in the slip model does not appear correct and may be a result of the arrangement of the slippage springs. The no-slip model resulted in a maximum geotextile stress that was 35 percent greater than the maximum stress in the slip model. The numerical values of geotextile stress are not in agreement with those presented by Bell, Greenway, and Vischer (1977) and may be a result of defining the geotextile material properties, however, it is the qualitative results that are of primary interest in this study. Ground surface deflections and horizontal shear stresses 3 feet (0.9 m) below the ground surface are shown in Figures 8 and 9 for the slip and no-slip cases. The distributions of deflections and shear stresses for the slip case are very similar to the distributions in the no-slip case. However, the values of maximum deflection and shear stress of the no-slip analysis are 39 percent and 26 percent greater than the values determined with the slip analysis, respectively. With the slippage model the reinforcement may only be stressed to a limited point before slippage occurs, thus the maximum stress in the reinforcement is limited and the inclusion of slippage should result in a more flexible model. Therefore, a decrease of reinforcement stresses with the inclusion of slippage into a model would be expected, however, the decrease in deflected shape with the inclusion of slippage would not be anticipated.

The decrease in deflections may be due to the rotations of the normal springs after loading occurs. A typical rotation of normal springs is shown in Figure 10. A maximum normal spring rotation of 89.54°, which results in a strain of 12530 percent in the normal spring, occurs in this example problem. The rotated normal springs add stiffness in the horizontal direction, which is not desired. A strain of 12530 percent in a normal spring alters the strain in the adjacent tangential springs by approximately eight percent. Rotation of the normal spring may possibly be limited by utilizing a beam element, with a large bending stiffness, rather than a truss element to represent the

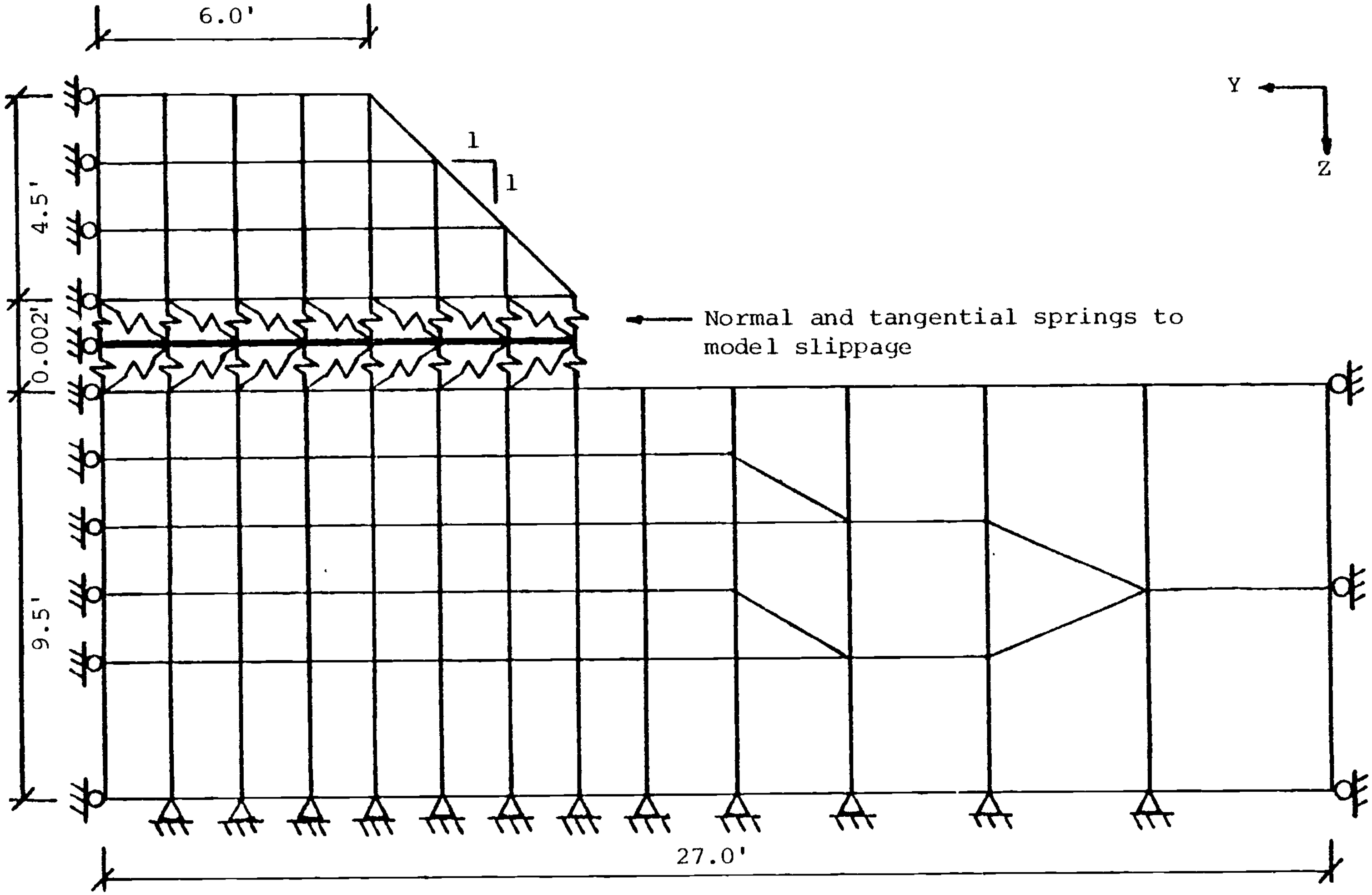


FIGURE 6. Discretized Model of Embankment

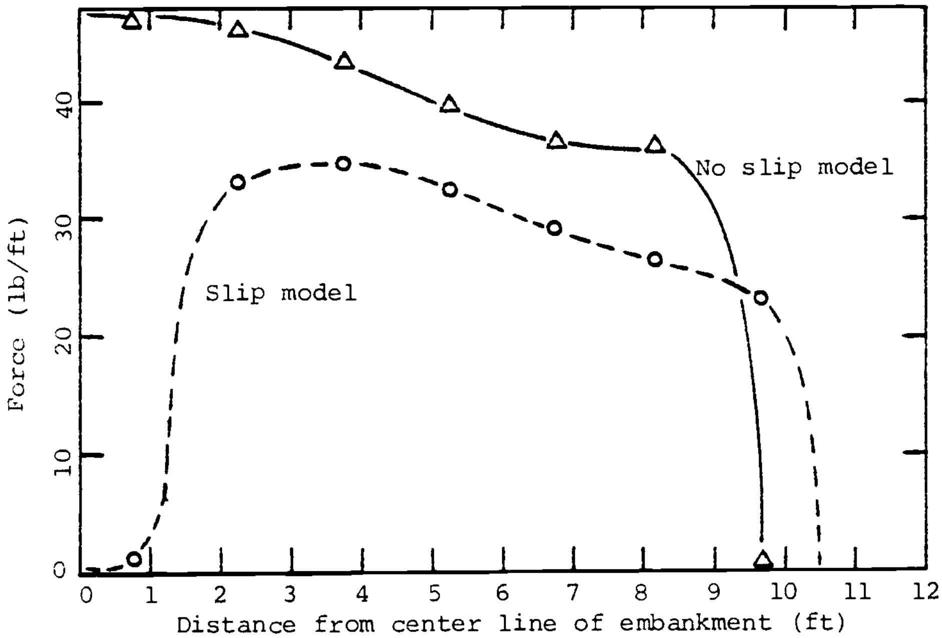


FIGURE 7. Distribution of Geotextile Stresses

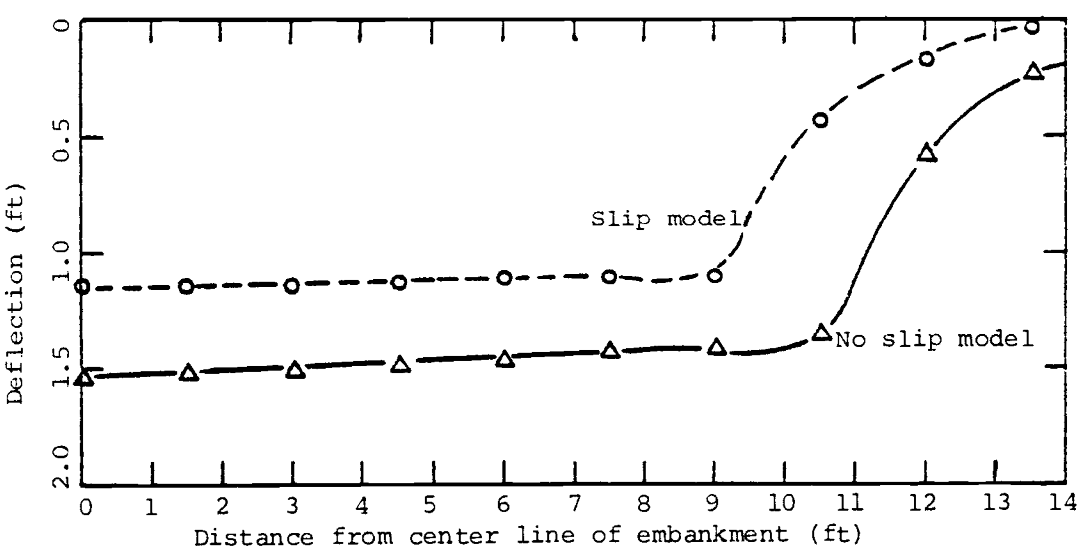


FIGURE 8. Ground Surface Deflections

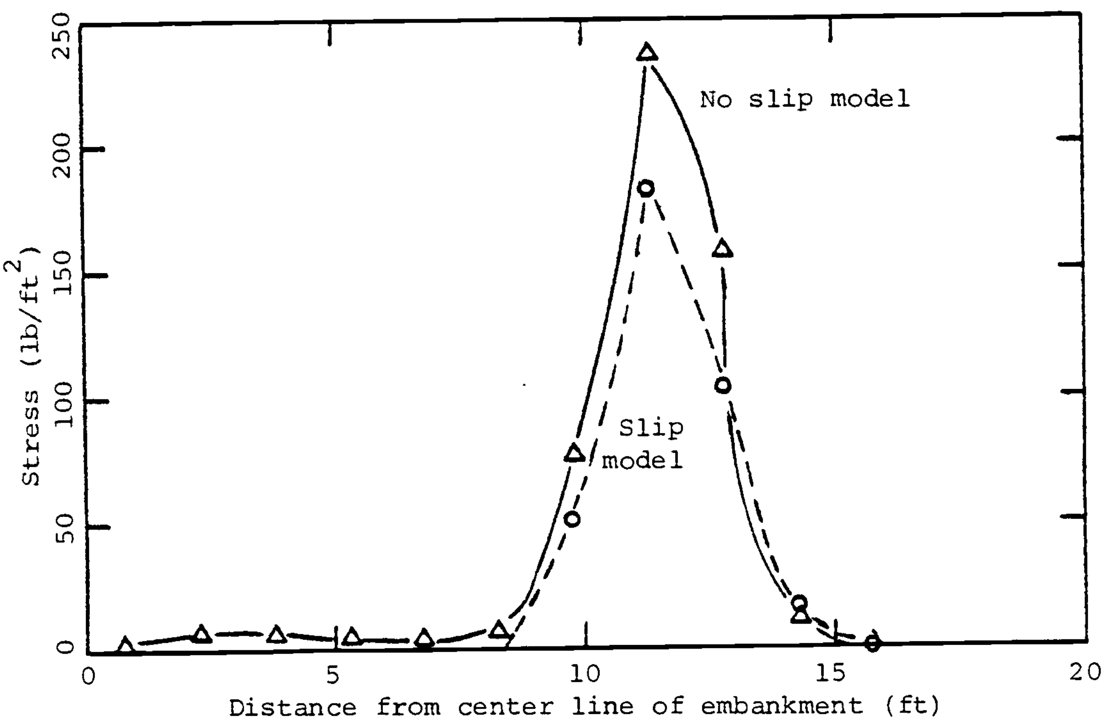


FIGURE 9. Shear Stresses 3 ft (0.9m) Below Ground Surface

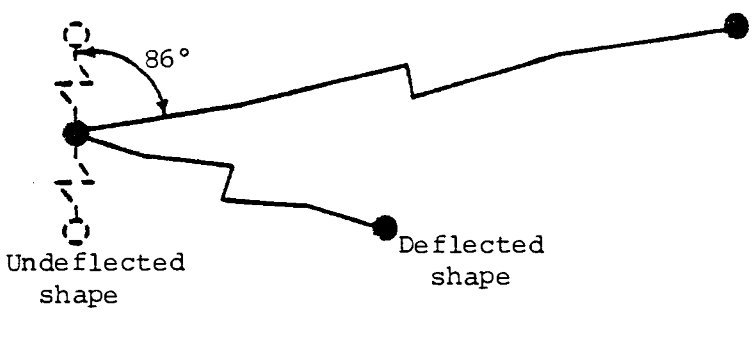


FIGURE 10. Typical Normal Spring Rotation

spring. The use of beam elements to represent normal springs and the resulting effects on analyses requires further study.

In summary, an analysis of a geotextile reinforced embankment, which utilized normal and tangential springs to model slippage of the reinforcement, has yielded questionable results. Rotation of the normal springs may have adversely affected the analysis. The effects of the rotation of normal springs may be studied with a computer program which allows master-slave node pairing, enabling the displacements of the two node points of the normal springs to be set equal.

Two-Dimensional Representation of Strip Reinforcement

Al-Hussaini and Johnson (1977) assumed that the metal strip reinforcements covered the entire length of the retaining wall, and that this could be compensated for by decreasing the shear resistance between the fill and reinforcement, to justify two-dimensional analyses of the structure. After completing their study, Al-Hussaini and Johnson concluded that this assumption was practical and satisfactory.

Naylor and Richards (1978) concluded that modeling strips as sheets extending over the entire plan area of the structure is not a sufficient representation of the structure. The frictional properties of the reinforcing-soil interface may be properly represented with a reduced value of friction, however this model causes serious error since it interrupts the vertical transfer of shear stress through the soil. Unfortunately these authors did not expand on the consequences of introducing this error into analyses. Two-dimensional representation of strip reinforcement may also lead to inaccurate transfer of surcharge loads to the reinforcement. Herrmann and Al-Yassin (1981) aggreed with Naylor and Richards that modeling of strips as sheets leads to a fictitious transfer of vertical shear and further stated that this fictitious shear could result in quite an erroneous prediction of reinforcement slippage. Pseudo-discrete two-dimensional analyses of strip reinforced soil structures, in which a composite representation is utilized in the horizontal direction, apparently offers no advantages, and may be disadvantageous, over complete composite representations and analyses (Herrmann and Al-Yassin, 1981).

Sequential Construction

A sequential, or incremental, construction analysis involves evaluation of stresses and deflections in a succession of structures, which correspond to construction lifts. The effect of sequential construction on finite element analyses of reinforced soil structures has not been specifically studied. Clough and Woodward (1967) studied the effect of incremental construction on finite element analyses of embankments and concluded that displacements during construction can be predicted only if the analysis is carried out sequentially following the construction history. These authors also concluded that stresses are affected to lesser extent by the sequential construction, but are more reliably predicted with the incremental analysis procedure. Al-Hussaini and Johnson (1977) accounted for sequential construction in their analyses which yielded deflection and stress values that were in good correlation with field measured values. Greenway (1976) utilized an incremental loading procedure in his analyses which yielded results that agreed with field data. Only one structure, the complete structure, is analyzed in the incremental loading procedure as opposed to a succession of structures being analyzed for sequential construction analyses. Accounting for sequential construction in analyses of reinforced soil structures leads to better modeling of the real structure and, therefore, increases the accuracy of the results. The effects of not modeling the sequential construction and the magnitude of errors in the numerical results are not well-defined.

Sequential analyses are not possible with many commercially available finite element programs. It is, therefore, desirable to determine if alternate procedures are available to totally or partially account for sequential construction in analyses. Boutrop (1981) has discussed one such procedure with the author. In this procedure the first construction sequence is modeled and analyzed, the model is then redefined as the second sequence added to the deflected shape of the first analysis and is reanalyzed, stresses are assumed cumulative. This process continues until the construction sequence is completed. The procedure is cumbersome and appears to be valid only for linear elastic materials.

A second procedure, incremental loading, has been studied with computer analyses of the slab foundation previously described. Two analyses were conducted with NONSAP, one of the entire structure and live load and a second analysis with the dead and live loads applied over a series of eight time steps. The results of the two analyses showed only minute discrepancies in calculated stresses and strains. It was concluded that if the materials are elastic and are not stressed beyond their proportional limit, and if geometric nonlinearities are negligible, the principle of superposition is applicable and no differences in stresses and strains occur.

An attempt was made to analyze the embankment, previously described, with incremental loads utilizing the NONSAP program. An analysis was completed with all loads being applied in one step, however convergence criteria could not be met with the same model and the loads being applied over four time steps. It was hypothesized that the failure to converge was due to the large geometric nonlinearities of this model.

Two modeling procedures to account for sequential construction in programs which are not capable of sequential analyses have been presented. The validity of the first procedure, in which the geometry is redefined for each analysis, appears limited to analyses where materials are not stressed beyond their linear elastic range. The second procedure, incremental loading, does not model the sequential construction of structures with negligible geometric nonlinearities. It has not been determined, in this study, whether or not the incremental loading procedure models sequential construction to any extent for structures with large geometric nonlinearities.

Tensile Capacity of Soil

The effects of modeling soils as being capable of withstanding tensile stresses on analyses are better evaluated on a case-by-case basis. The magnitude of errors is dependent on the type of soil--cohesive or cohesionless, the loading conditions, and the geometry of the structure. For example, a retaining structure would be more likely to develop high tensile stresses in its soil than an embankment would.

Three computer models are available which partially account for tensile limits of soils, the extent of which must again be evaluated on a case-by-case basis. For cohesionless soil, such as a clean sand, and a two-dimensional analysis, the soil may be modeled as an orthotropic linear elastic material with a realistic modulus of elasticity in the vertical direction, a very low modulus of elasticity in the horizontal direction, and a Poisson's ratio equal to zero (Bell, Greenway, and Vischer, 1977). A second model available only with nonlinear programs is to utilize a curve description option for defining the stress-strain characteristics of a material. If the program allows for separate descriptions in tension and in compression, the tensile portion may be represented with a very small modulus of elasticity to model the low tensile strength of the soil. The third model involves the use of interface or gap elements, which are inserted between soil elements and open when tension is developed across the soil elements.

Compression in Reinforcing Elements

Past researchers (Al-Hussaini and Johnson, 1977; Bell, Greenway, and Vischer, 1977; and Chang, 1974) have not disclosed any problems with compressive forces developing in the reinforcement during analyses. This may be attributed to the fact that only retaining walls and embankments were analyzed.

Analyses of the retaining wall and embankment in this study typically indicated only one computer element of reinforcing to be in compression, for each structure. However, analysis of the slab foundation resultedin eleven of twenty computer elements of reinforcing in compression, as shown in Figure 11.

Results of this study indicate that analyses of certain structures, such as slab foundations, as opposed to retaining walls and embankments, may result in high compressive stresses in segments of the reinforcing components. Typical reinforcements--geotextiles and metal strips--are not capable of carrying significant compressive loads; therefore, an analysis which indicates compression in the reinforcing does not model the behavior of the actual structure.

Linear Elastic Representation of Nonlinear Materials

As previously stated, many of the components of reinforced soil structures are nonlinear materials and should be represented as such when possible. Structural finite element programs, such as SAP V, STRUDL, and STARDYNE, are capable of linear analyses only. It is therefore appropriate to compare analyses of linear to nonlinear programs as part of this study.

A retaining wall on a soft clay foundation, with a 1500 lb/ft^2 (71.8 kPa) surcharge load was analyzed with the ANSYS and SAP V programs. The NONSAP and SAP V programs were also used to analyze the retaining wall on firm silt and soft clay foundations, with a 500 lb/ft^2 (23.9 kPa) surcharge. Slippage was not modeled in the nonlinear analyses as it was not possible to account for slippage in the linear analyses. Only the foundation materials were stressed beyond their linear elastic range in the three sets of analyses.

The stresses developed in a metal strip reinforcement for the three pairs of analyses are shown in Figure 13. The stresses in the central portion of the reinforcement are quantitatively in good agreement for the NONSAP and SAP V analyses; however, the stress quantities diverge at either end of the reinforcement. The large stresses calculated in the reinforcement near the front of the wall with the NONSAP analyses may be due to the material modeling of the soils. The backfill and foundation soils were modeled as elastic-plastic materials, with a Drucker-Prager yield condition, in the NONSAP analyses, as opposed to linear elastic, with von Mises yield criteria, modeling for the SAP V analyses and nonlinear elastic, with von Mises yield criteria, modeling for the ANSYS analysis.

The Drucker-Prager yield criteria is an approximation of the Mohr-Coulomb law which is frequently used for soils, concrete, and other frictional materials (Zienkiewicz, 1977). The Drucker-Prager and von Mises isotropic yield surfaces in principal stress space are shown in Figure 12. The effects of yield criteria on analyses of reinforced soil structures require further study.

The reinforcement stresses calculated with the ANSYS and SAP V programs, for 1500 lb/ft^2 (71.8 kPa) loading cases, are approximately equal. These analyses indicate that reinforcement stresses are affected by the material yield criteria to a greater extent than by material nonlinearities, for the specific cases analyzed. The tensile stress distributions of Figure 13 also indicate high stresses at the end of the reinforcement furthest from the skin elements. The actual stress at the end point is zero (Al-Hussaini and Johnson, 1977). Therefore, additional techniques or elements, such as the use of interface elements as reported by Al-Hussaini and Johnson (1977), are required to model the end-effects of the reinforcement.

Computed deflections at the front of the wall are presented in Table 8 and the nodal points are shown in Figure 14. In general, the analyses which were conducted indicate only small differences in computed deflections between the nonlinear and linear programs.

Specific conclusions on the effect of linear elastic representation of nonlinear materials on finite element analyses of reinforced soil structures cannot be drawn from the limited analyses conducted. For the specific cases analyzed, which did not model possible slippage of reinforcement, the effects of linear representation of nonlinear materials on deflections of the structure and tensile stresses in the reinforcement were minimal. Additional analyses of more structures, which vary with type, geometry, material composition, and loading, are required to develop specific conclusions on linear elastic

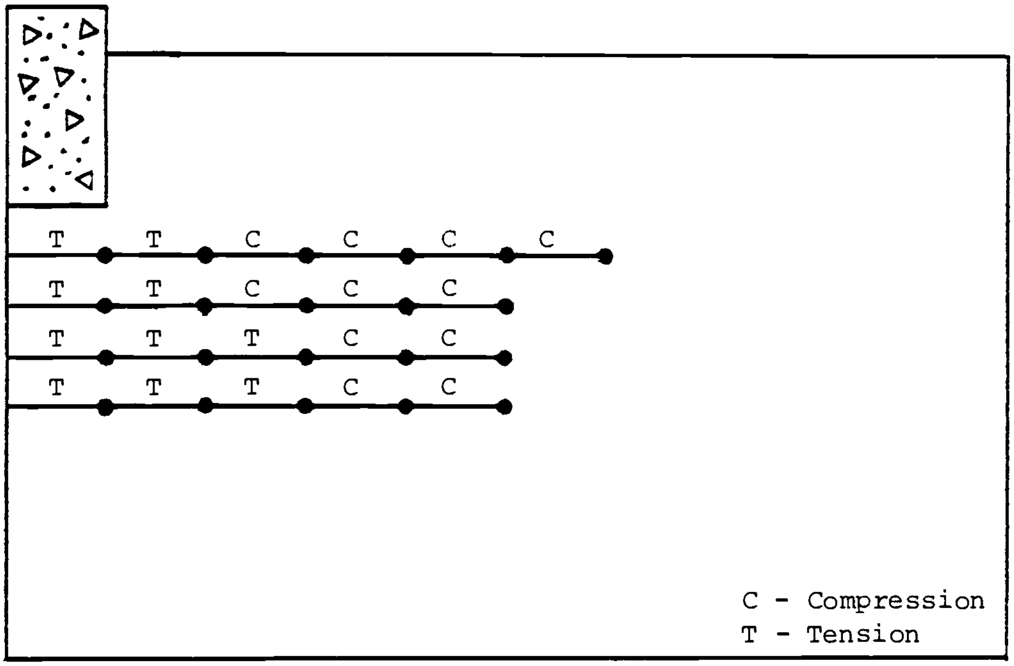


FIGURE 11. Slab Foundation Elements in Compression

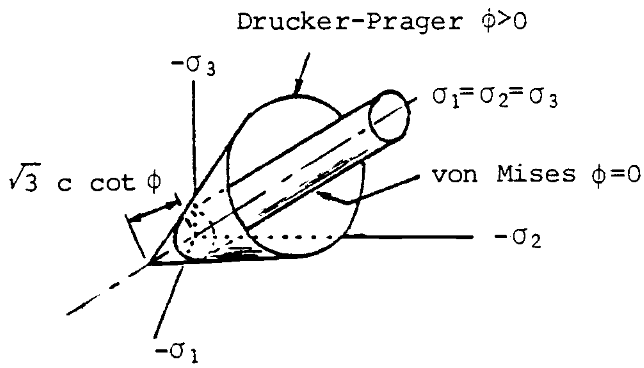
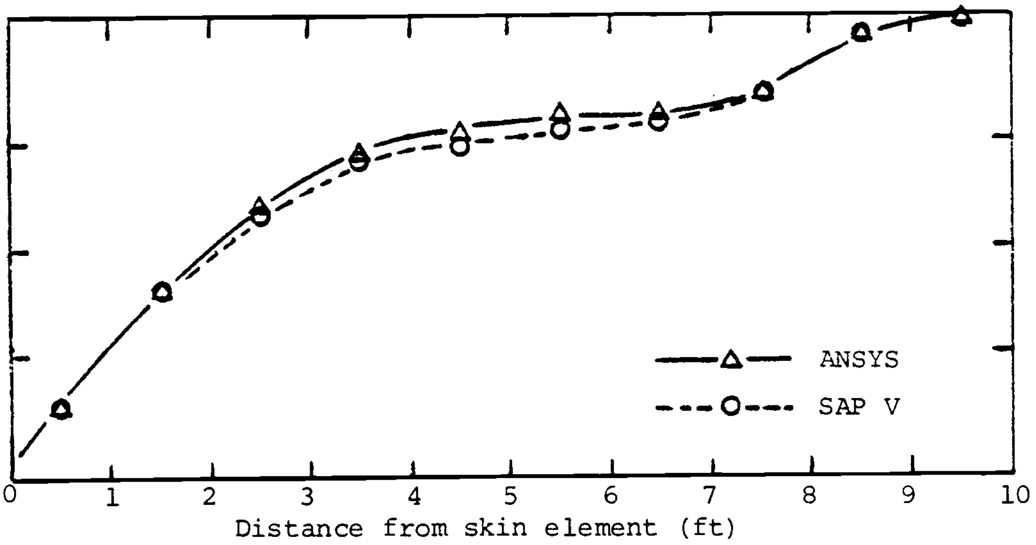
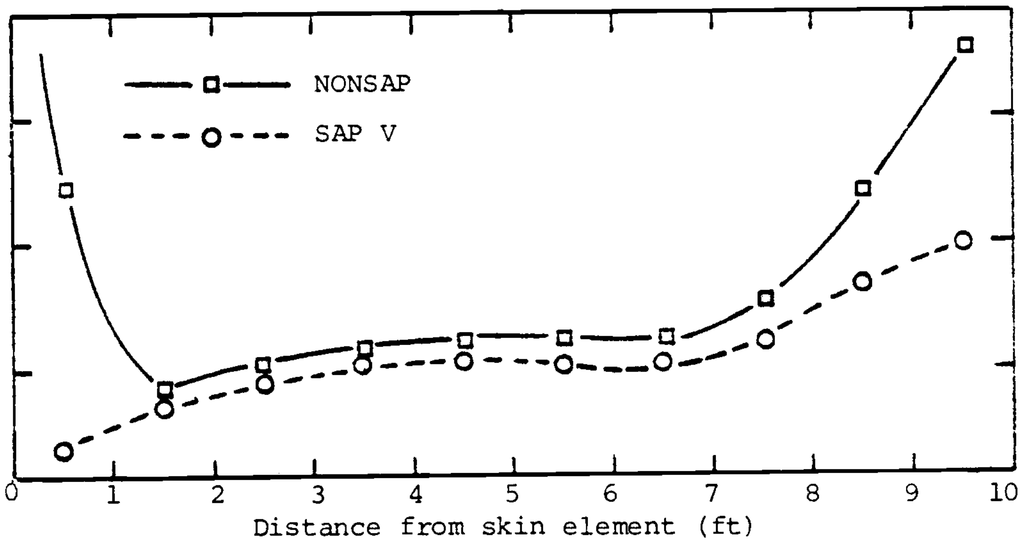


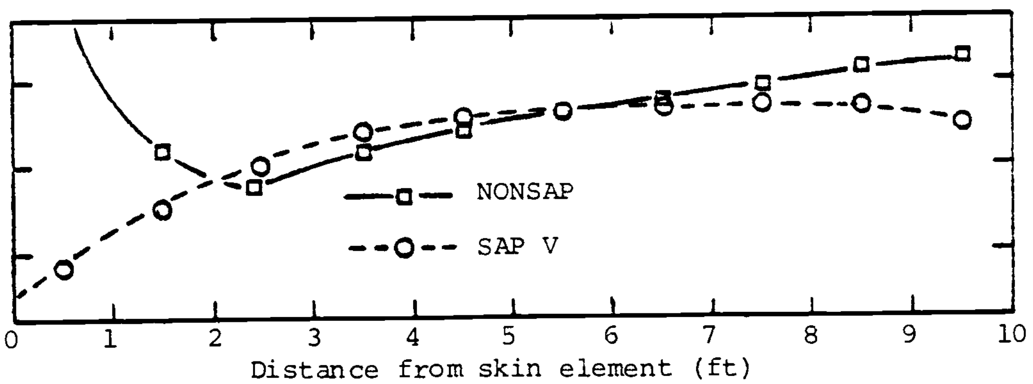
FIGURE 12. Drucker-Prager and von Mises Yield Surfaces
(after Zienkiewicz, 1977)



a. Slab Foundation, 1500 lb/ft^2 (71.8 kPa) Surcharge



b. Soft Foundation, 500 lb/ft^2 (23.9 kPa) Surcharge



c. Firm Foundation, 500 lb/ft^2 (23.9 kPa) Surcharge

FIGURE 13. Reinforcement Stress Distribution (for reinforcement 7 ft. below top of wall)

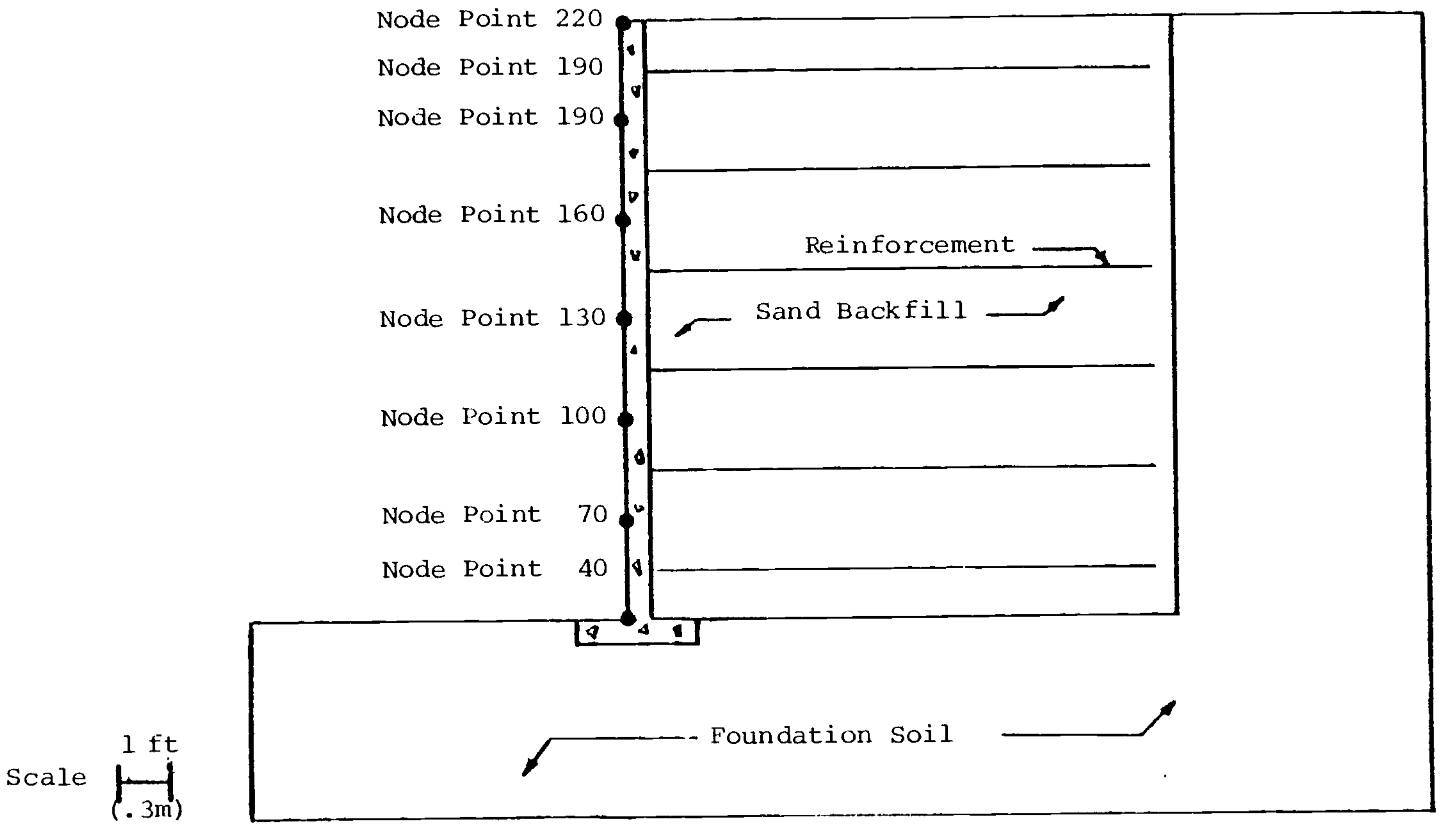


FIGURE 14. Retaining Wall

TABLE 8. Computed Retaining Wall Deflections.

	DEFLECTIONS (ft)											
Program: Surcharge: Foundation:	NONSAP 500 (lb/ft^2) Soft		SAP V 500 (lb/ft^2) Soft		NONSAP 500 (lb/ft^2) Firm		SAP V 500 (lb/ft^2) Firm		ANSYS 1500 (lb/ft^2) Soft		SAP V 1500 (lb/ft^2) Soft	
Direction:	Y	Z	Y	Z	Y	Z	Y	Z	Y	Z	Y	Z
Nodal Points 220	.0060	.0327	.0018	.0366	.0006	.0075	.0033	.0076	.0097	.0524	.0038	.0111
190	.0004	.0327	.0055	.0366	.0010	.0075	.0007	.0076	.0006	.0525	.0002	.0112
160	.0066	.0327	.0004	.0366	.0024	.0075	.0014	.0077	.0074	.0526	.0031	.0114
130	.0126	.0327	.0056	.0368	.0034	.0076	.0028	.0078	.0144	.0527	.0049	.0114
100	.0183	.0327	.0101	.0368	.0040	.0076	.0036	.0078	.0205	.0528	.0056	.0115
70	.0238	.0327	.0140	.0368	.0043	.0076	.0038	.0078	.0258	.0528	.0055	.0115
40	.0290	.0327	.0176	.0368	.0042	.0075	.0039	.0077	.0310	.0526	.0052	.0114

representation of nonlinear materials; however, it is questionable whether or not further studies are warranted based on the effects of slippage of reinforcement on analyses, as previously discussed, and the inability of linear programs to model slippage.

Soft Versus Firm Foundations

Naylor (1978) studied the effects the stiffness of the foundation had on finite element analyses of an idealized reinforced soil wall, 32.8 feet (10 m) high and 49.2 feet (15 m) deep with metal strip reinforcement extending 26.2 feet (8 m) back into the wall. A slipping strip analytical model (Naylor and Richards, 1978) was utilized in the analyses. As compared to rigid base analyses, Naylor noted the following effects of the foundation being soft:

1. A slight increase in the peak tension in the reinforcement of about 10 percent occurs.
2. Soil stresses are generally relaxed in the region above the foundation.

The analyses of the retaining walls which were used to study the effects of linear representation of nonlinear materials may also be used to study the effects of a soft foundation versus a firm foundation. Slippage is not possible and every component is represented discretely in these analyses, as opposed to the composite analyses by Naylor in which slippage was modeled. Other differences between models were that the soft foundation soil was modeled as being 49.2 feet (15 m), or 1.5 times the height of the wall, deep and extended 23 feet (7 m) behind the wall in the Naylor model and the soft foundation soil of the model in this study extended 4 feet (1.2 m), or 0.33 times the height of the wall, in depth and 5 feet (1.5 m) beyond the rear of the wall. Naylor considered only the weights of the components in his analyses.

The stresses developed in a reinforcing component, for a 500 lb/ft^2 (23.9 kPa) surcharge in combination with the weights of the components, are shown in Figure 13. Contrary to the results by Naylor the stresses in the reinforcement were found to decrease in the models with soft foundations. The soil stresses in the region above the foundation varied, as shown in Figure 15; however, a general trend is not perceivable.

The results of the two studies indicate the need of further finite element analyses of reinforced soil structures. Further research should study soft versus firm foundations with varying parameters such as depth of foundation, loading conditions, and composite or discrete analyses.

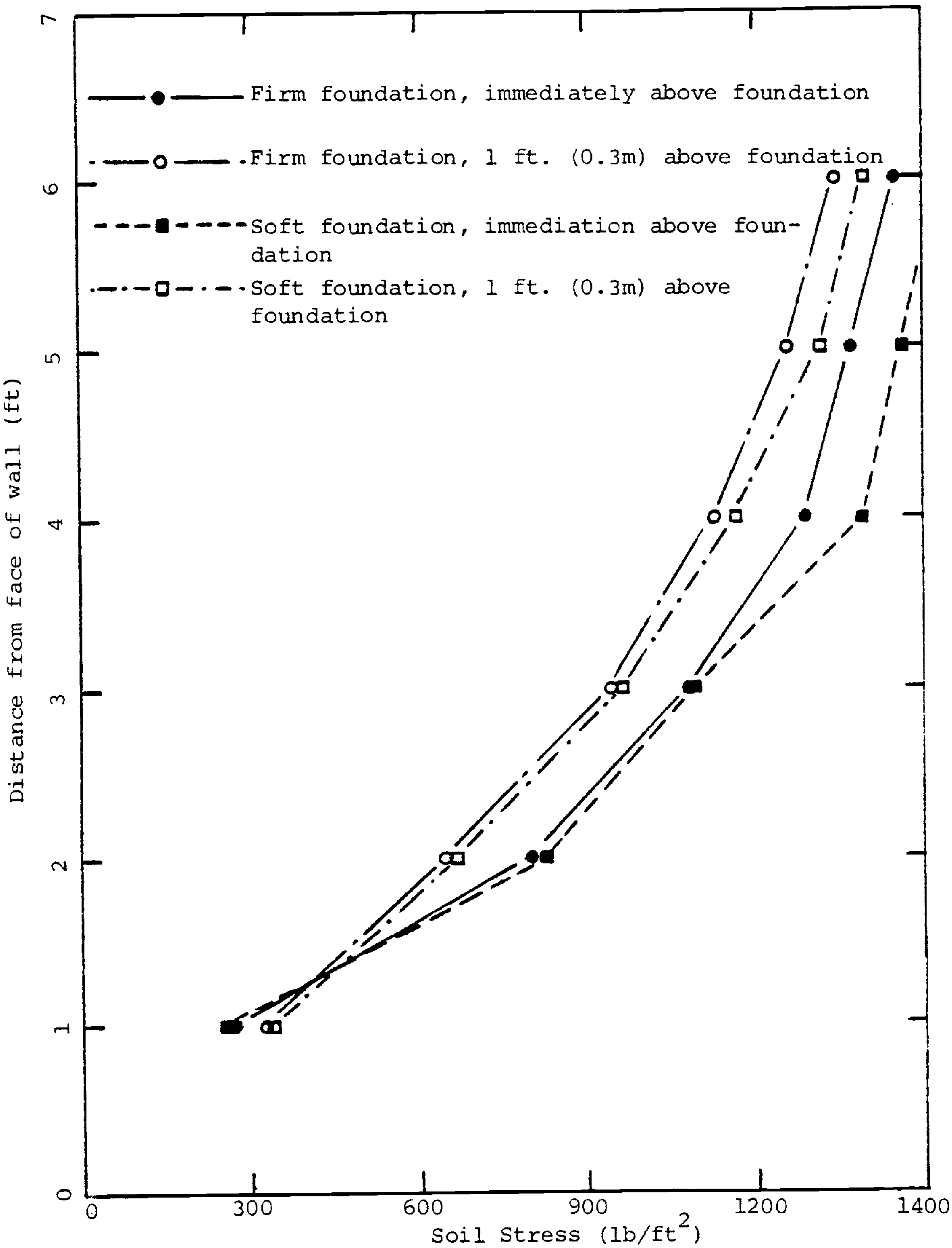


FIGURE 15. Soil Stresses Above Retaining Wall Foundation

CHAPTER VI. CONCLUSIONS AND RECOMMENDATIONS

The utility of a computer program for analysis of a particular reinforced soil structure may be assessed by consideration of the following requirements: two- or three-dimensional analysis, computer elements to represent the components of the structure, material models, loading conditions, and types of deformations. Typically, programs do not meet all requirements for the analysis of a reinforced soil structure; therefore, the consequences of not meeting various requirements, along with modeling techniques which may be utilized in analyses, must be considered.

Based on the literature reviewed and/or computer analyses, the following conclusions relative to various parameters and modeling techniques which affect finite element analyses of reinforced soil structures appear justified and important.

1. Not modeling slip of the reinforcement will decrease deformations and may shift the location of maximum stress in the reinforcement (Naylor and Richards, 1978).

2. Incorporating a slippage model into an existing finite element program is only valid if the program has provision for nonlinear material modeling.

3. Slippage models which employ normal and tangential springs require that the displacements of the two nodes of each normal spring be set equal, as stipulated by Peterson (1977) and reconfirmed with computer analyses of this study.

4. A two-dimensional analysis of a structure with strip reinforcement will result in an inaccurate transfer of shear perpendicular to the reinforcement and can also result in large errors in reinforcement slippage (Naylor and Richards, 1978).

5. Gap elements, orthotropic linear elastic material modeling (Bell, Greenway, and Vischer, 1977), or nonlinear

specification of material stress-strain characteristics may be used to provide tensile cut-off of materials represented by two-dimensional planar or three-dimensional solid elements.

6. Specification of zero or low compressive strength of the reinforcing is required in finite element analyses of reinforced soil slabs.

7. The evidence relative to the effects of soft versus firm foundations for reinforced soil walls is contradictory. This indicates the need for further research.

8. The effects of various parameters and modeling techniques on finite element analyses of reinforced soil structures requires further research. Specifically, additional studies are needed in the following areas:

   a. Modeling of reinforcement slippage with

      (1) master-slave node pairing.

      (2) stiff beam elements modeling the normal springs.

   b. Identifying the effects of two-dimensional representation of strip reinforcement on deflected shapes, reinforcement stresses, and soil stresses for various types of structures at working loads.

   c. Identifying the effects of sequential construction on deflected shapes, reinforcement stresses, and soil stresses for various structures.

   d. Identifying the effects of employing different material models on finite element analyses of reinforced soil structures.

Many of the commercially available finite element programs cannot meet the unique requirements which exist for finite element analyses of reinforced soil structures. Specifically, the requirements to model reinforcement slippage and sequential construction limit the utility of available programs. The consequences of not accounting for slippage or sequential construction could not be adequately evaluated in this study, due to the inability to model such. The study of other parameters, such as two-dimensional representation of strip reinforcement and linear representation of nonlinear materials, was limited, also due to the inability to model slippage and sequential construction. It is concluded that a comprehensive program, which meets all of the requirements for analysis of reinforced soil structures, and more data on actual structures are required to thoroughly evaluate finite element techniques for soil reinforcement application.

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APPENDICES

APPENDIX A

GEOMETRY OF COMPUTER MODELS

Three basic structures were utilized in the parametric studies: an embankment, a strip footing, and a retaining wall. The embankment, as shown in Figure A-1, was previously studied by Greenway (1976). The strip footing, as shown in Figure A-2, was presented as a design example by Binquet and Lee (1975). The retaining wall utilized in this study, as shown in Figure A-3, was similar to the wall studied by Al-Hussaini and Johnson (1977). Concrete facing elements were assumed in this study as opposed to aluminum skin elements used by Al-Hussaini and Johnson (1977).

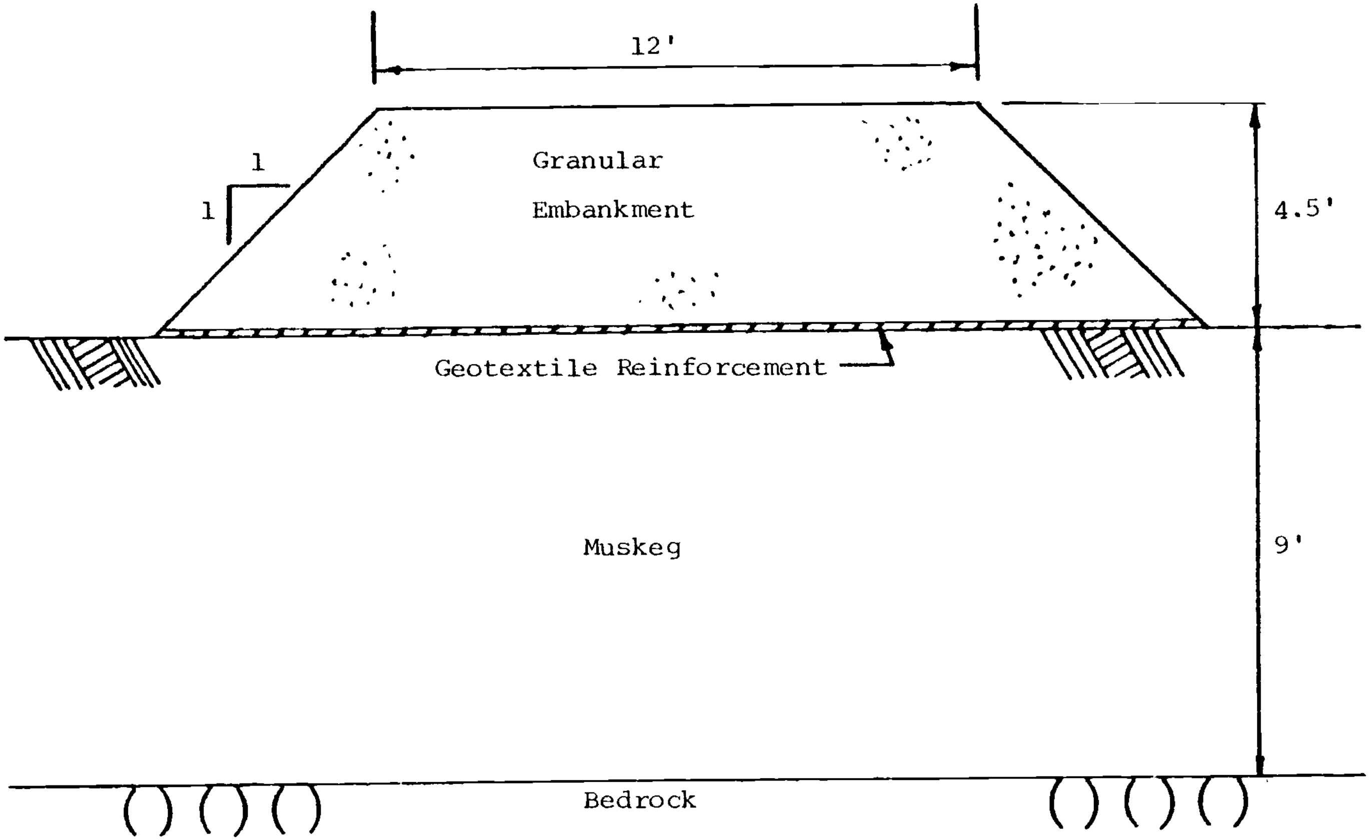


FIGURE A-1. Embankment

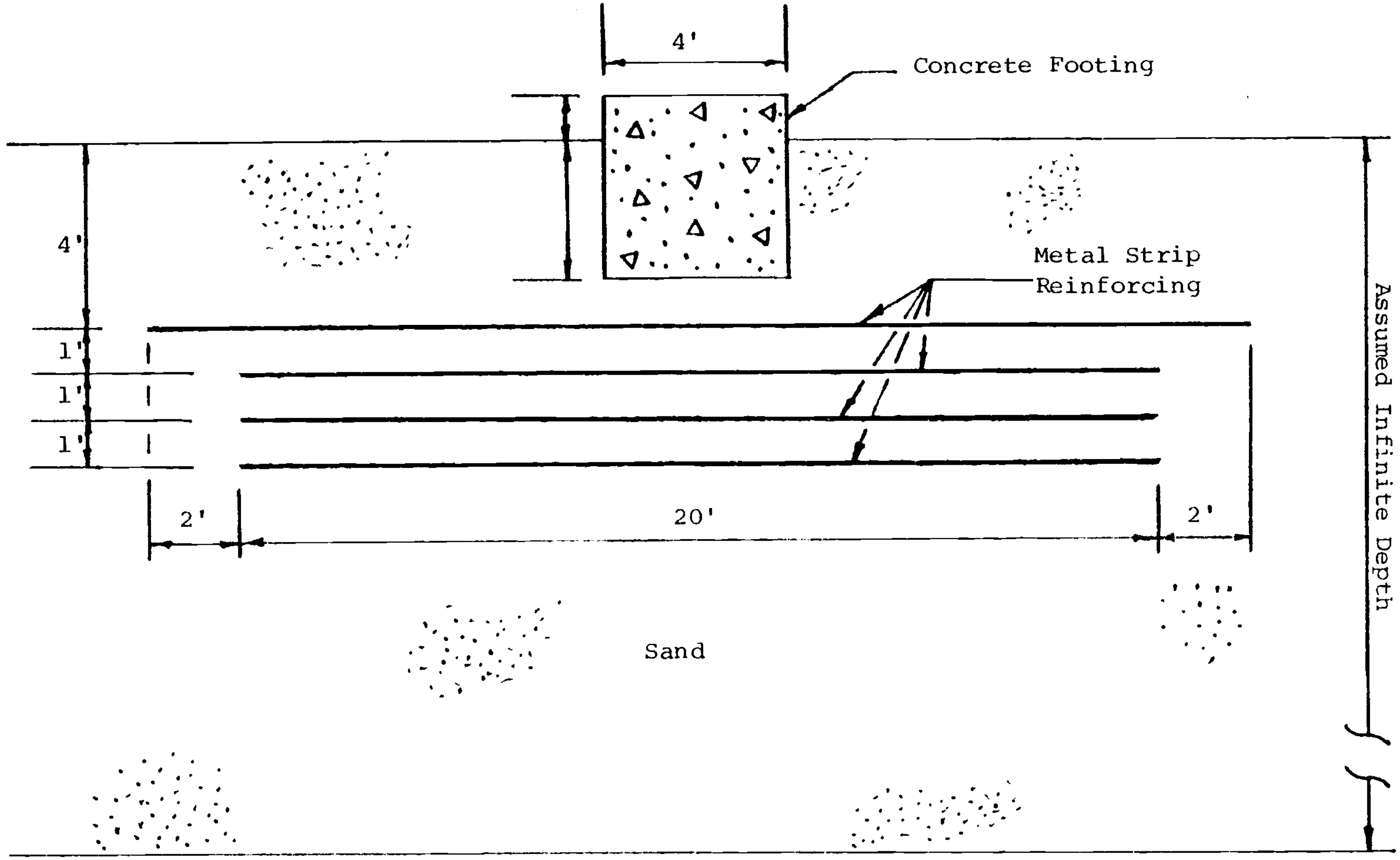


FIGURE A-2. Strip Footing.

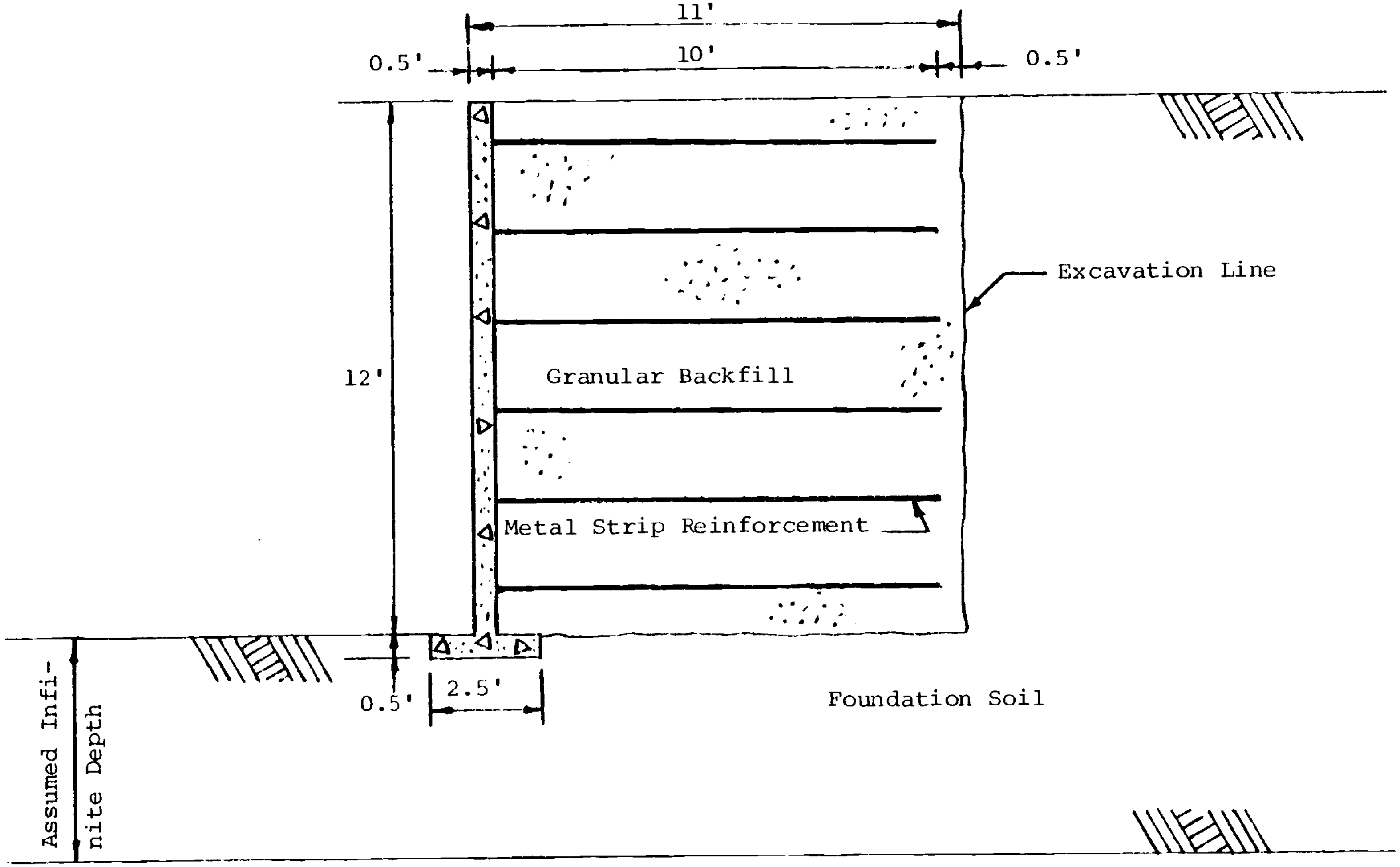


FIGURE A-3. Retaining Wall

APPENDIX B

DISCRETIZED COMPUTER MODELS

The embankment, strip footing, and retaining wall were subdivided, as shown in Figures B-1, B-2, and B-3, respectively, for the finite element analyses. A discretized model of the embankment, with possible slippage of the reinforcement, is shown in Figure 7.

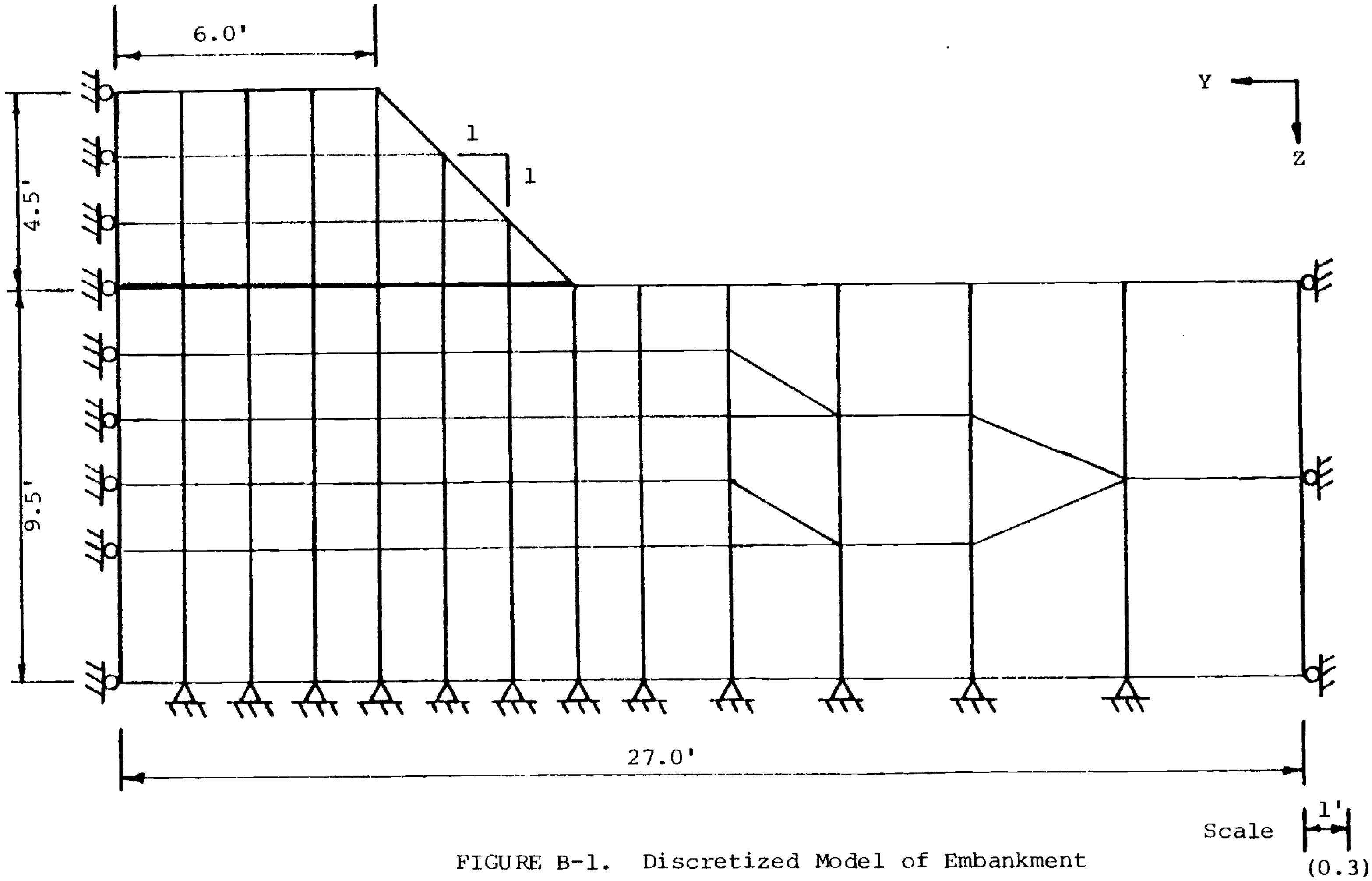


FIGURE B-1. Discretized Model of Embankment

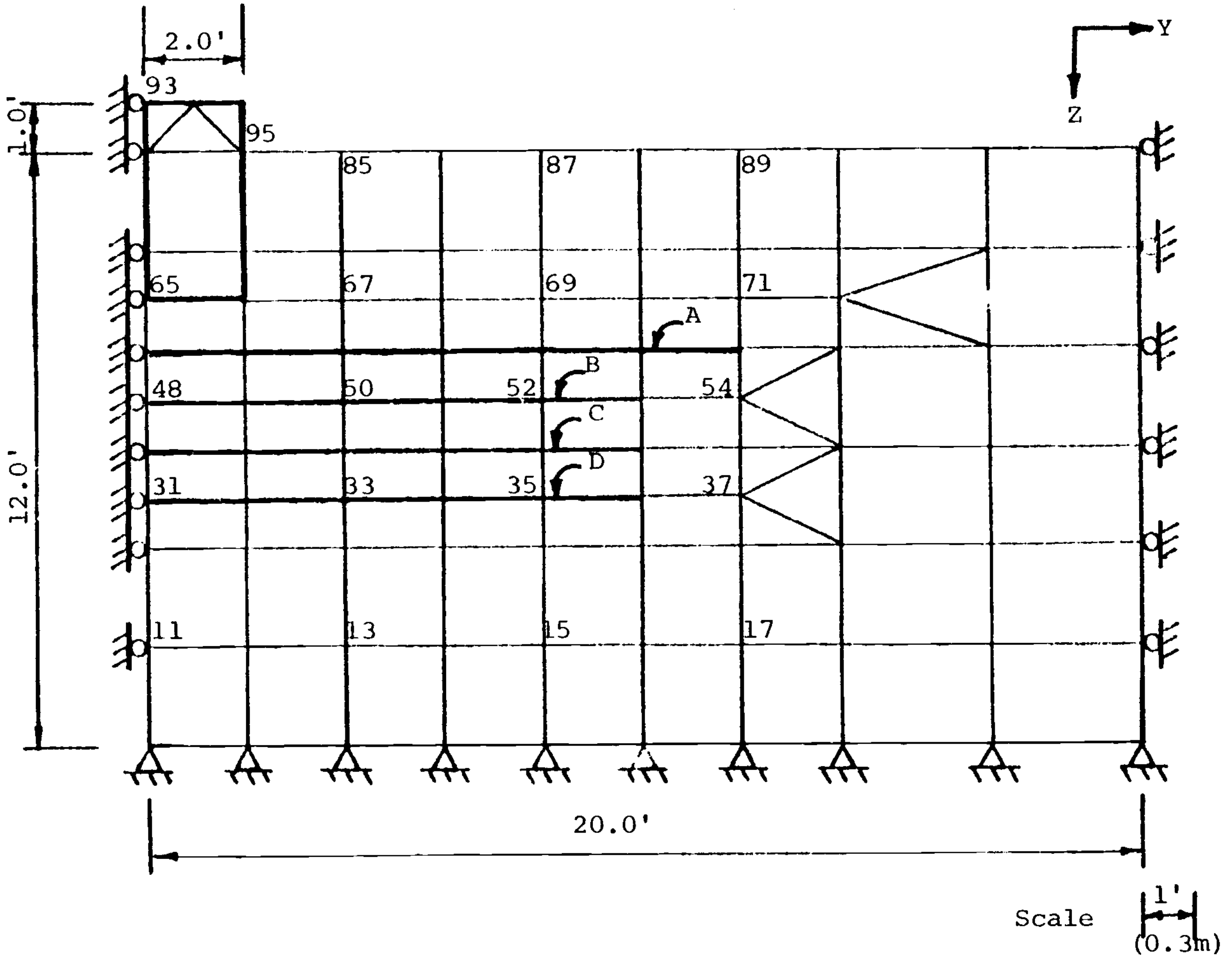


FIGURE B-2. Discretized Model of Strip Footing

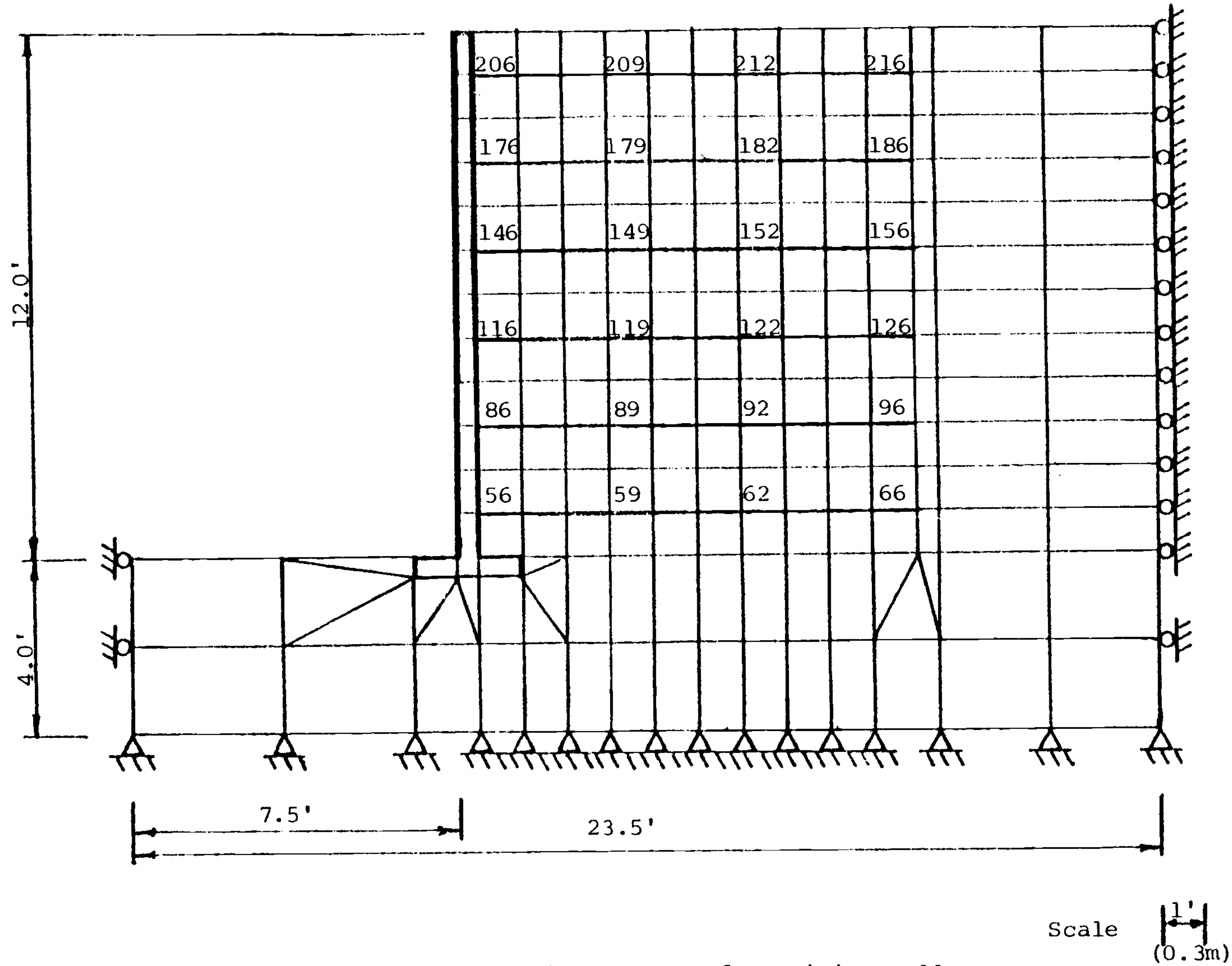


FIGURE B-3. Discretized Model of Retaining Wall.

APPENDIX C

MATERIAL MODELS

The following are descriptions of the materials which were used to define the components of the structures which were analyzed. A summary of the various computer models and materials used is given in Appendix D.

Clay L:

Linear elastic material, von Mises yield criteria.

E=72000 lb/ft^2 (3450kPa) $\nu = 0.4$

(Bowles, 1977; NAVFAC DM-7, 1977)

Clay NL:

Elastic-plastic material, Drucker-Prager yield criteria.

E=72000 lb/ft^2 (3450kPa) $\nu = 0.4$

$\emptyset = 1°$ c = 350 lb/ft^2 (16.8kPa)

(Lambe and Whitman, 1969; Bowles, 1977;
NAVFAC DM-7, 1977; Drucker and Prager, 1952)

Concrete:

Linear elastic material, von Mises yield criteria.

E=449600000 lb/ft^2 (2153MPa) $\nu = 0.20$

(Wang and Salmon, 1979)

Loess L:

Linear elastic material, von Mises yield criteria.

E=532800 lb/ft^2 (2550kPa) $\nu = 0.33$

(Al-Hussaini and Johnson, 1977)

Loess NL:

Elastic-plastic material, Drucker-Prager yield criteria.

$E=532800$ lb/ft^2 (2550kPa) $\nu = 0.33$

$\phi = 32°$ $c=576$ lb/ft^2 (27.6kPa)

(Al-Hussaini and Johnson, 1977; Drucker and Prager, 1952)

MS-L:

Metal strip reinforcement, linear elastic material.

$E=4480000000$ lb/ft^2 (935GPa)

(Al-Hussaini and Johnson, 1977)

MS-NL:

Metal strip reinforcement, elastic-plastic material.

$E=4480000000$ lb/ft^2 (935GPa)

Yield-7300000 lb/ft^2 (152GPa)

(Al-Hussaini and Johnson, 1977)

Muskeg:

Elastic-plastic material, Drucker-Prager yield criteria.

$E=3000$ lb/ft^2 (143.6kPa) $\nu = 0.25$

$\phi = 37°$ $c=100$ lb/ft^2 (4.8kPa)

(McFarlane, 1969; Drucker and Prager, 1952)

NWG-NL:

Nonwoven geotextile reinforcement, nonlinear material.

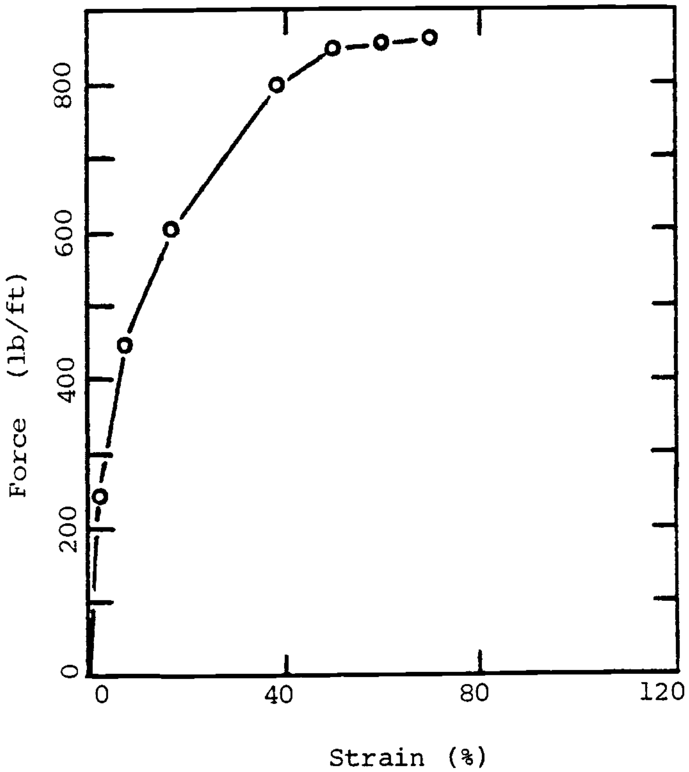


FIGURE C-1. Assumed Geotextile Property (Greenway, 1976; Rankilor, 1981)

Sand 1:

Linear elastic material, von Mises yield criteria.

E=600000 lb/ft^2 (28700kPa) ν = 0.1

(Greenway, 1976)

Sand 2:

Linear elastic material, von Mises yield criteria.

E=720000 lb/ft^2 (34400kPa) ν = 0.2

(Binquet and Lee, 1975)

<u>Sand 3L</u>:

Linear elastic material, von Mises yield criteria.

$E = 1280000\ lb/ft^2$ (61200kPa) $\nu = 0.3$

(Lambe and Whitman, 1969)

<u>Sand 3NL</u>:

Elastic-plastic material, Drucker-Prager yield criteria.

$E = 1280000\ lb/ft^2$ (61200kPa) $\nu = 0.3$

$\phi = 39°$ $c = 50$

(Lambe and Whitman, 1967; Drucker and Prager, 1952)

APPENDIX D

SUMMARY OF STRUCTURES ANALYZED

A summary of the computer models used in this study is presented in Table D-1. Three computer programs were utilized for the finite element analyses: ANSYS, NONSAP, and SAP V. The computers used were the CYBER 170 Model 720 computer at the Oregon State University campus and the VAX 11/780 computer at the University of Portland campus. Material descriptions of the components are given in Appendix C. Results of the analyses are given in the text and in Appendix E.

TABLE D-1. Description of Computer Models

Run Number	Program	Computer	Reinforcing Material	Foundation or Base Soil	Embankment or Backfill Soil	Facing Element or Footing	General Description
EMB-1	NONSAP	CYBER	NWG-NL	MUSKEG	SAND 1	N/A	Embankment, no slippage
EMB-2	NONSAP	CYBER	NWG-NL	MUSKEG	SAND 1	N/A	Embankment with slippage modeled
STR-1	NONSAP	CYBER	MS-NL	SAND 2	N/A	CONCRETE	Strip footings, one loading
STR-2	NONSAP	CYBER	MS-NL	SAND 2	N/A	CONCRETE	Strip footings, incremental loading
RW-1A	SAP V	CYBER	MS-L	CLAY L	SAND 3L	CONCRETE	Retaining wall, 500 lb/ft^2 surcharge
RW-1B	NONSAP	CYBER	MS-NL	CLAY NL	SAND 3NL	CONCRETE	Retaining wall, 500 lb/ft^2 surcharge
RW-2A	SAP V	CYBER	MS-L	CLAY L	SAND 3L	CONCRETE	Retaining wall, 1500 lb/ft^2 surcharge
RW-2B	ANSYS	VAX	MS-NL	CLAY NL	SAND 3L	CONCRETE	Retaining wall, 1500 lb/ft^2 surcharge
RW-3A	SAP V	CYBER	MS-L	LOESS L	SAND 3L	CONCRETE	Retaining wall, 500 lb/ft^2 surcharge
RW-3B	NONSAP	CYBER	MS-NL	LOESS NL	SAND 3NL	CONCRETE	Retaining wall, 100 lb/ft^2 surcharge

APPENDIX E

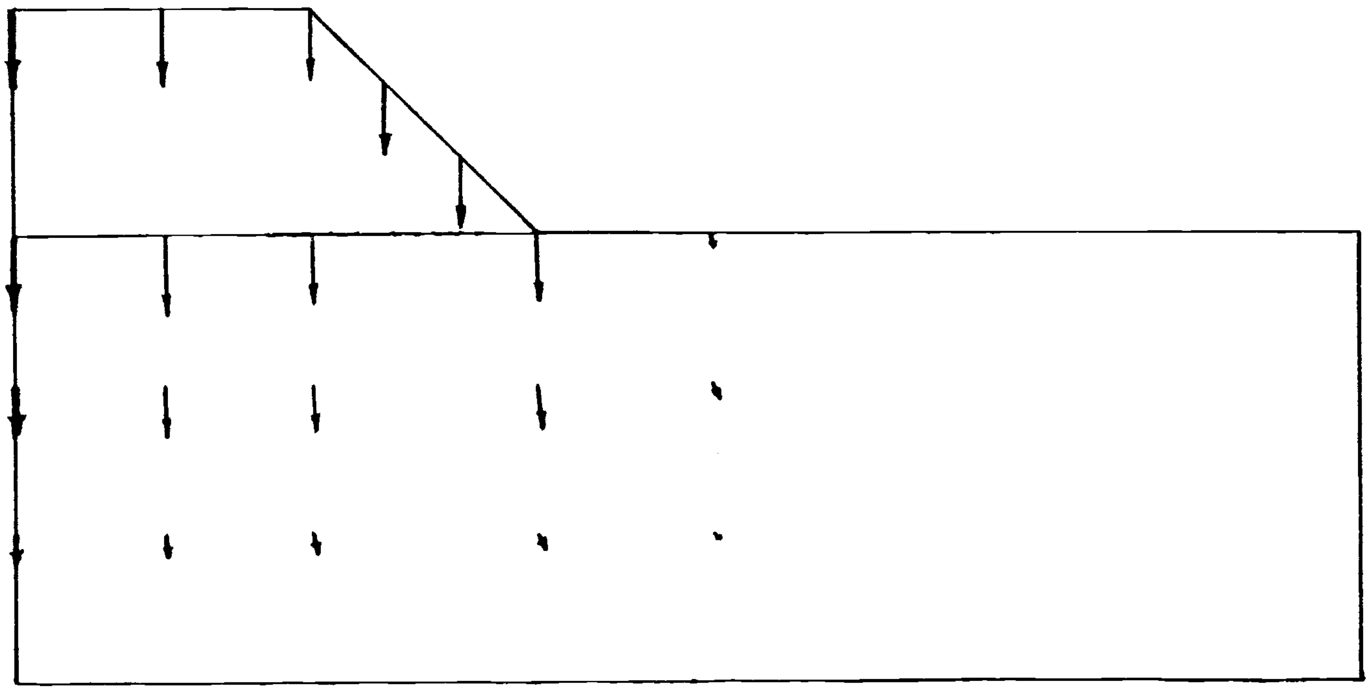
RESULTS OF COMPUTER ANALYSES

Some of the results from the finite element analyses of the reinforced soil structures were presented in the main body of this thesis, additional results are presented in this appendix. Results of the analyses are expressed as soil stresses, reinforcement stresses, and deformations.

Predicted deformations for embankment, for the case without reinforcement slippage and the case with reinforcement slippage, are shown in Figures E-1 and E-2. The geotextile stresses, as predicted in this study, are compared to the stresses predicted by Greenway (1976) in Figure E-3. An explanation for the large discrepancy between computed geotextile stresses has not been determined. A comparison of the computed ground surface deflections to the deflections predicted by Greenway (1976) shows a good correlation between values.

Stresses in the reinforcement of the strip footing are illustrated in Figure E-5. The individual reinforcements are labeled on the discretized model of the strip footing in Figure B-2. As stated in the text, only minute differences in predicted values of streses and deflections were computed in the incremental loading and one load analyses of the footing. Deflections at various nodal points of the strip footing are shown in Table E-1 for both load cases. The nodal points are also labeled in Figure B-2.

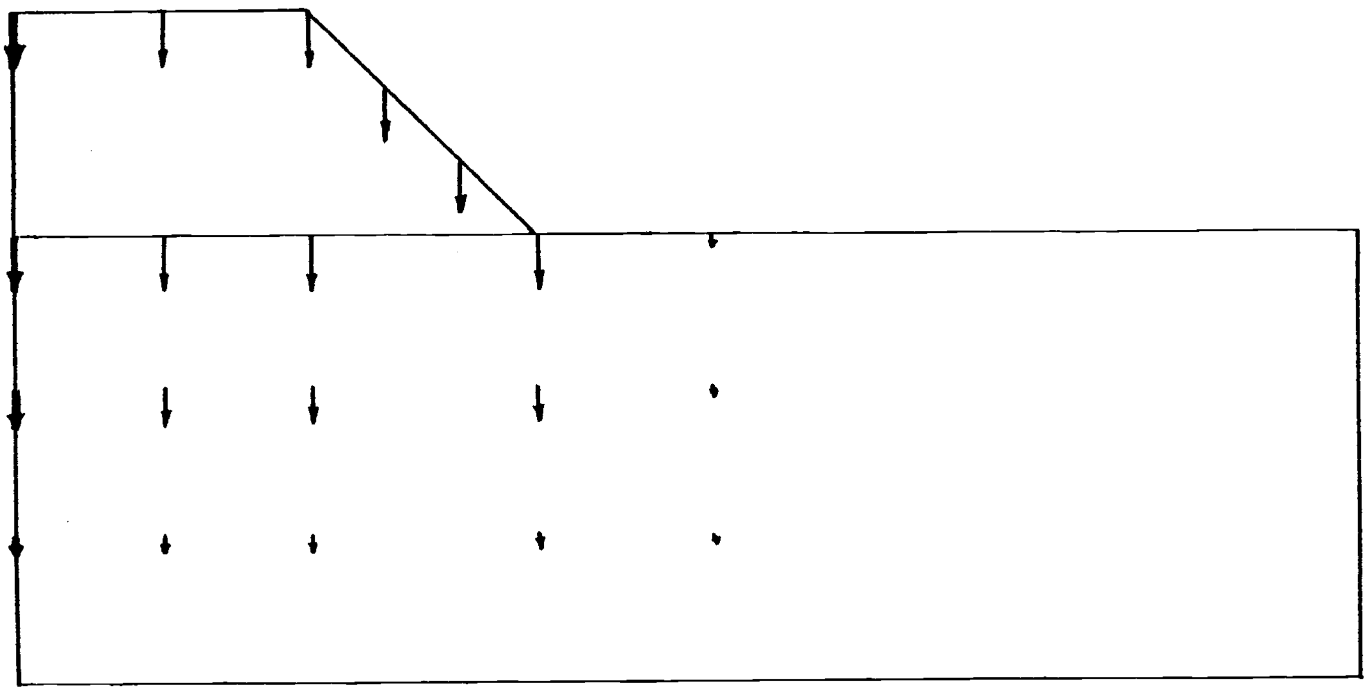
Stresses in the reinforcement of the retaining wall, for six cases, are given in Table E-2. The six cases are described in Table D-1. The nodal points along the reinforcement, which are used in Table E-2, are labeled in Figure B-3.



(Note: Deformations to scale)

Scale 1' (0.3m)

FIGURE E-1. Predicted Deformations--No Slip Case



(Note: Deformations to scale)

Scale 1' (0.3m)

FIGURE E-2. Predicted Deformations--Slip Case

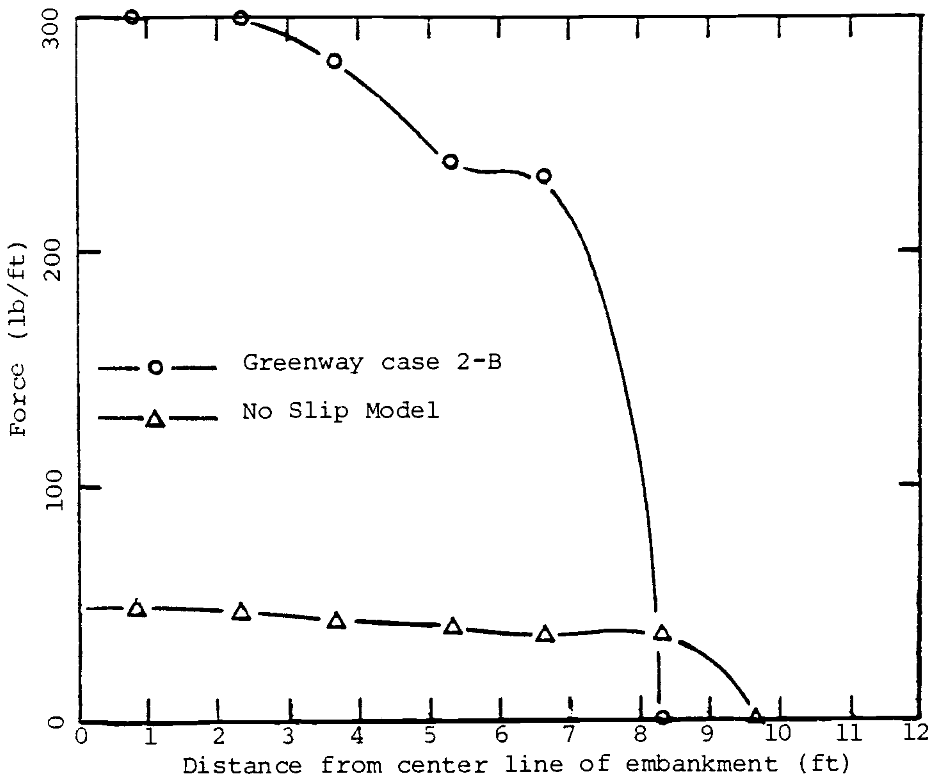


FIGURE E-3. Comparison of Geotextile Stresses

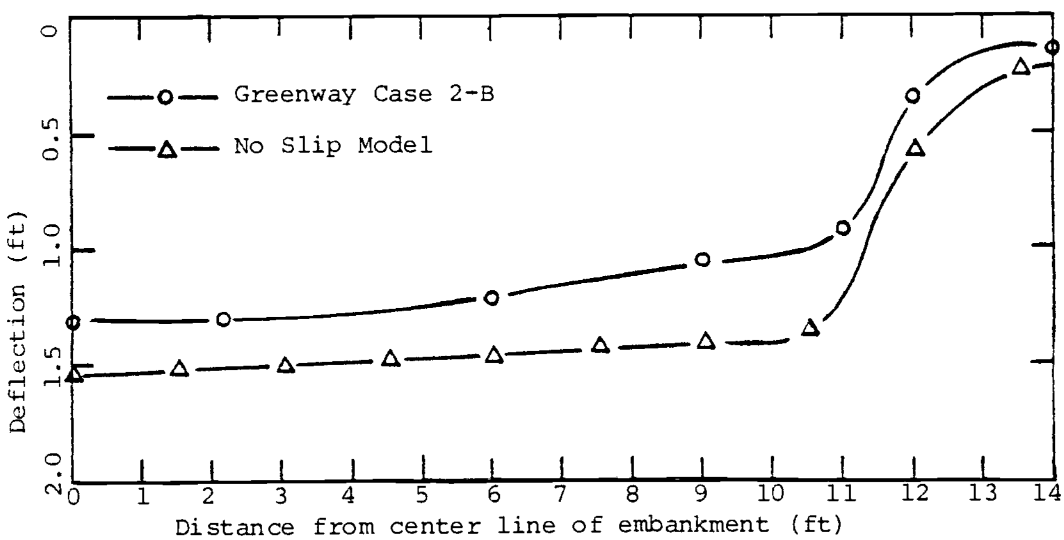


FIGURE E-4. Comparison of Ground Surface Deflections

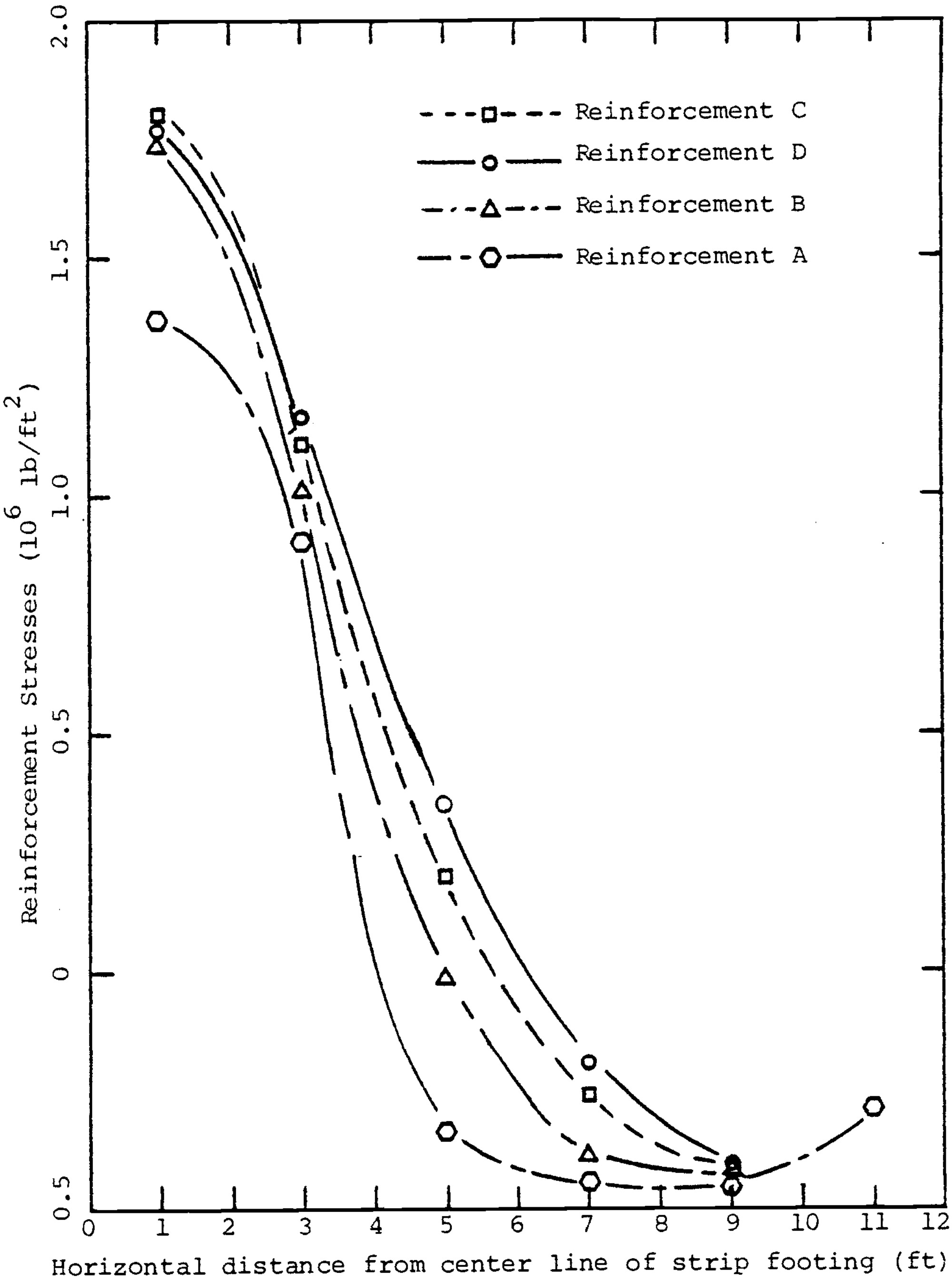


FIGURE E-5. Strip Footing Reinforcement Stresses

TABLE E-1. Deflections of the Strip Footing

Nodal Points	Deflections (ft)			
	One Load		Incremental Loading	
	Y	Z	Y	Z
11	0	.0329	0	.0329
13	.0097	.0253	.0097	.0253
15	.0103	.0131	.0103	.0131
17	.0065	.0062	.0065	.0062
31	0	.0900	0	.0900
33	.0136	.0647	.0136	.0647
35	.0142	.0294	.0142	.0294
37	.0088	.0121	.0088	.0121
48	0	.1343	0	.1343
50	.0125	.0898	.0125	.0898
52	.0102	.0375	.0102	.0375
54	.0054	.0140	.0054	.0140
65	0	.1786	0	.1786
67	.0046	.1069	.0046	.1069
69	.0004	.0418	.0004	.0418
71	.0017	.0153	.0017	.0153
85	.0340	.1203	.0340	.1203
87	.0292	.0446	.0292	.0446
89	.0148	.0171	.0148	.0171
93	0	.0788	0	.1788
95	.0001	.1788	.0001	.1788

TABLE E-2. Retaining Wall Reinforcement Stresses

	Stress (10^6 lb/ft^2)					
	Case RW-1A	Case RW-1B	Case RW-2A	Case RW-2B	Case RW-3A	Case RW-3B
56	-0.422*	-0.695	-0.222	N**	-0.605	-0.680
59	1.338	1.747	2.652		0.776	1.053
62	1.776	3.746	3.346		1.006	1.223
66	0.756	4.338	1.429		0.786	1.170
86	-0.082	0.118	0.229		-0.035	0.537
89	0.654	0.900	1.577		0.702	0.800
92	0.563	1.671	1.527		0.849	0.984
96	0.940	1.869	1.828		0.812	1.178
116	0.117	1.112	0.371		0.221	1.811
114	0.434	0.444	1.248		0.713	0.650
122	0.437	0.545	1.386		0.817	0.851
126	0.887	1.742	1.794		0.748	1.068
146	0.162	2.099	0.444		0.248	2.565
149	0.421	0.266	1.255		0.641	0.552
152	0.466	0.448	1.440		0.699	0.691
156	0.741	1.996	1.599		0.575	0.844
176	0.109	2.853	0.288		0.074	3.096
179	0.408	0.373	1.152		0.389	0.430
182	0.430	0.993	1.245		0.390	0.427
186	0.519	1.129	1.268		0.299	0.531
206	-0.107	2.853	-0.229		-0.242	4.455
209	0.067	3.616	0.201		-0.285	0.588
212	0.372	2.260	0.630		-0.259	0.215
216	0.461	1.209	0.997		-0.010	0.124

*Negative sign (-) indicates compression.
**No results obtained.