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FABRIC REINFORCED EMBANKMENTS
CONSTRUCTED ON WEAK FOUNDATIONS

Eva Boutrup

Robert D. Holtz

PURDUE UNIVERSITY

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Eva Boutrup

Robert D. Holtz

Final Report

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TO: H. L. Michael, Director DATE: July 6, 1983
Joint Highway Research Project
PROJECT: C-36-36M
FROM: R. D. Holtz, Research Engineer
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Attached is the Final Report on the HPR Part II research study entitled "Fabric Reinforced Embankments Constructed on Weak Foundations". This report essentially completes all of the tasks of the approved work plan of the revised proposal for this project.

Because of the unfortunate illness of Ms. Eva Boutrup, Graduate Instructor in Research, we were not able to do all we originally planned in 1977-78. Prior to her illness, Ms. Boutrup had written drafts of Chapters 2, 3 and 4, as well as the Appendices of this report. Those chapters required editing and minor revisions. I wrote Chapter 1 and 5 of the report. My colleague, Prof. C. W. Lovell, read Chapter 3 and 4 while they were in draft form and made many valuable comments.

The results of the study indicate that the primary influence of the fabric reinforcement is to reduce both shear stresses in soft foundation soils and the vertical differential settlements at the top of the embankment. As we expected, total settlements were found to be only slightly affected by the reinforcement. The degrees of improvement due to the presence of the geotextile was much more pronounced with higher modulus geotextiles and for the undrained loading case. This research has important practical implications for increasing the stability of embankments constructed on soft foundations, and recommendations for their design and construction are given in the report.

Copies of the report will be submitted to the IDOH and FHWA for their review. I look forward to receiving their comments on our research.

Sincerely yours,

R. D. Holtz, Ph.D., P.E.
Research Engineer

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16. Abstract A two-dimensional nonlinear, large displacement finite element program called GEONON (a modification of NONSAP) for the static analysis of embankments on weak foundations has been developed. Both linear and elasto-plastic stress-strain soil models are included, and incremental and stage construction and the process of compaction can be simulated. The program GEONON, however, is not developed herein as an adaptable design tool for the practicing highway engineer. The research investigated the response in terms of stresses and displacements of a low embankment constructed on soft muskeg soils in Alaska (Bell, Greenway, and Vischer, 1977). Both unreinforced and reinforced cases were considered for drained and undrained foundation conditions of varying stiffness. Single and multilift construction was investigated and the effect of compaction was simulated. The influence of embankment dead load and a simulated live load were studied, and both single and multiple layers of geotextiles of varying moduli were considered. It was found that the geotextile reinforcement significantly reduced the shear stresses in the foundation and decreased the vertical differential settlements at the top of the embankment, especially for the undrained case. This influence was more pronounced as the modulus of the geotextile was increased. Total settlements were only slightly affected by the presence of the reinforcement. Multiple layers of fabric in the embankment also reduced foundation shear stresses and differential settlements. The report also discusses the practical aspects of the design and construction of geotextile-reinforced embankments on weak foundations.					
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FOREWORD

In 1978, when we began the research described in this report, we knew from limited empirical observations that geotextiles could be used to increase the stability of embankments on weak foundations. However, we lacked a viable analytical model for reinforced embankment behavior. Such a model we felt would ultimately be required to explain this behavior as well as to allow our experience to be extended to different embankment and reinforcement configurations and to new sites. We knew that the limiting equilibrium approach, although quite satisfactory for design, could not be used to explain what we suspected was rather complex soil-reinforcement-foundation interaction. Further, we believed that the finite element method with small strains and simple linear elastic soil models also would prove to be inadequate. Consequently, we chose to approach the problem using a more complex analytical tool, a large finite element program with a large displacement formulation and nonlinear stress-strain soil response.

The computational difficulties have at times been enormous, especially when both nonlinear soil behavior and large strains are considered. Not all our original personal objectives have been realized, but we did complete essentially all the research tasks of the revised research proposal of March 17, 1981.

Prior to her unfortunate illness, Ms. Eva Boutrup had completed all of the computer runs and had written most of Chapters 2, 3, and 4 as well as the Appendices. R. D. Holtz completed those sections and wrote the literature review (Chapter 1) and Chapter 5 on the practical implications of the conclusions of the research and some recommendations for the design and construction of geotextile-reinforced embankments.

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Financial support for this research was provided by the State of Indiana, Department of Highways and the U.S. Federal Highway Administration through the Joint Highway Research Project at Purdue University. This support is gratefully acknowledged. We also want to thank the members of the JHRP Advisory Committee for this project. They gave us continual encouragement and provided many helpful suggestions during the conduct of this research. Members of the present Advisory Committee are Profs. C. W. Lovell and E. C. Ting of Purdue University, Mr. R. K. Smutzer, Division of Materials and Tests, IDOH, and Mr. P. A. Hoffmann, Field Technical Coordinator, Region 5, FHWA, in Indianapolis. Our FHWA Washington Office Contact was Mr. A. F. DiMillio, Materials Division, Office of Research. Mr. W. J. Sisiliano of IDOH was initially a member of the Committee, and occasionally FHWA engineers Mr. B. Johnson, Mr. G. Moss, and Mr. G. Andreko joined our meetings. We appreciate the contributions of all these individuals. Finally, we want to mention that Prof. Lovell critically read Chapters 3 and 4 and provided many helpful suggestions.

Mr. N. Sivakugan prepared many of the drawings, and the manuscript was typed by Mrs. Catherine Ralston and Ms. Bonnie Simmons.

Final Report
FABRIC REINFORCED EMBANKMENTS CONSTRUCTED ON WEAK FOUNDATIONS

HIGHLIGHT SUMMARY

A two-dimensional nonlinear, large displacement finite element program called GEONON (a modification of NONSAP) for the static analysis of embankments on weak foundations has been developed. Both linear and elastoplastic stress-strain soil models such as the Von Mises and the Drucker-Prager are included. The program is suitable for the estimation of stresses and displacements of both ordinary embankments and embankments reinforced with geotextiles. Incremental and stage construction can be simulated as can the process of compaction. The program GEONON, however, is not developed herein as an adaptable design tool for the practicing highway engineer.

After a rather extensive review of the literature on geotextiles and their use as reinforcement and related applications, the analytical soil models used in GEONON are described. Next a discussion of the large-displacement and incremental construction formulations is presented, and modifications to NONSAP in order to develop GEONON are also described.

The research investigated the response in terms of stresses and displacements in a low embankment constructed on soft muskeg soils in Alaska (Bell, Greenway, and Vischer, 1977). Both unreinforced and reinforced cases were considered for drained and undrained conditions in the foundations. Varying foundation stiffness was also considered for the unreinforced embankment, both single and multilift construction was investigated, and it was found that the multilift simulation should be used unless the foundation soils are relatively stiff and loading is undrained. In that case, the difference was insignificant. When compaction was simulated, large horizontal stresses were observed, especially in the soft foundation.

The same embankment studied above was reinforced with both single and multiple layers of geotextiles of varying moduli. The values of the moduli were chosen to be representative of typical woven and nonwoven geotextiles in use today for reinforcing embankments. Both the influence of the embankment dead load and a simulated live load were studied for relatively soft and stiff foundation soils and for undrained and drained conditions in the foundation. It was found that the geotextile significantly reduced the shear stresses in the foundation and decreased the vertical differential settlements at the top of the embankment. This influence was more pronounced as the modulus of the geotextile was increased. Total settlements were only slightly affected by the presence of the reinforcement. Also, the influence was much greater for the undrained case than for the drained; again the effect of increasing geotextile modulus was significant in reducing foundation shear stresses and decreasing the differential settlement. Multiple layers of fabric in the embankment also reduced foundation shear stresses and differential settlements.

The program GEONON is rather sophisticated and is probably more useful for indicating of trends of performance rather than providing exact numerical values for design. Therefore, the report also discusses the practical aspects of the design and construction of geotextile-reinforced embankments on weak foundations. Considered in some detail are the functions of the geotextile in the embankment, fabric and soil creep, design of reinforced embankments, factors of safety, selection of geotextile properties, subgrade stabilization and specifications for design and construction. Recommendations for additional research are also provided.

The report also contains extensive appendices. In addition to a complete listing of the program GEONON, a detailed description of GEONON is given as are details of input data preparation. Finally a study of the effect of different orders of Gaussian integration on the results are given.

Chapter 1

INTRODUCTION AND TECHNICAL BACKGROUND

1.1 EARTH REINFORCING AND GEOTEXTILES

It is well known that embankments constructed on soft foundations frequently require some type of reinforcing to maintain stability under dead load as well as anticipated live loads. Examples include roads constructed on logs or timbers (corduroy), as was done in colonial North America and Scandinavia, and bamboo fascines which have been commonly used for centuries under low embankments in southeast Asia. A modern counterpart to these systems is the Columbus vehicle and fascine mat developed in Sweden about 15 years ago (Holtz, 1976). Levee and road embankments have often been constructed directly on the brush and small trees which commonly grow on marshy ground. In recent years, an embankment reinforcing system consisting of two rows of short sheet piles or steel channel sections connected by steel tie rods has been developed at the Swedish Geotechnical Institute to increase the stability of embankments constructed on soft foundations (Wager and Holtz, 1976). Reinforcing has also been carried out using woven and nonwoven fabrics, plastic and steel nets and grids, used automobile tire casings, steel landing mats, "Columbus" fascine mats, and reinforced plastic and rubber membranes. Holtz (1975 and 1978) has summarized many of these developments for reinforcing both embankments and retaining walls.

The primary functional requirements of any reinforcement material are: (1) it must have a sufficiently high deformation modulus in tension, and (2) it must be able to develop sufficient frictional resistance with the subsoil and/or embankment materials. Furthermore, it is important that these properties be reasonably constant throughout the design life of the structure. Unfortunately, many plastics and nonwoven fabric materials have creep properties such that their effectiveness as reinforcement may decrease with time. However, in reinforced embankments the strength of the subsoil may increase faster than the corresponding creep in the reinforcement so that the effect of creep is effectively neutralized. In the case of reinforced sands or reinforced backfills for retaining walls, situations in which the soils do not consolidate and gain strength, creep deformations under high loads could be significant (Holtz, Tobin, and Burke, 1982).

1.1.1 Geotextiles

The use of woven and nonwoven fabric materials (ASTM: "geotextiles") for reinforcement and other applications is a relatively recent development in the U.S. With a few notable exceptions, most of the early research and development work with these materials was done in Europe, and applications were primarily directed toward stabilizing temporary roads on soft foundations. During the past 10 years, many European nonwoven fabrics have become available in the U.S., and their use for certain specific civil engineering situations is increasing

dramatically.

It is interesting to note that woven geotextile technology began in the U.S. and then moved to Europe (again, with only a very few exceptions). Initial geotextile applications in the U.S. used woven monofilament fabrics as "filter fabrics", that is, as an alternate to granular filters under riprap and in other erosion control applications. Recent developments in woven technology have included slit film fabrics, which are stronger and have a higher modulus than typical nonwovens, but cost about the same as nonwovens per unit area.

For the reader unfamiliar with geotextiles and their many applications in civil engineering, the Supplement on the subject in the British technical journal Civil Engineering (March, 1981) is highly recommended for a general overview of the subject. In somewhat more detail, two recent books, by Koerner and Welsh (1981) and Rankilior (1981), provide excellent descriptions of common geotextiles, and these books also give many examples of applications to civil engineering practice. Unfortunately, both books are somewhat deficient in providing detailed design procedures for especially reinforced embankments. Other useful sources of information include the Proceedings of the two International Conferences on Geotextiles (Paris, 1977 and Las Vegas, 1982) as well as several regional specialty conferences. Recommended are the First Canadian Symposium on Geotextiles (1980) in Calgary, Alberta, the preprint of the session on the Use of Geotextiles in Soil Improvement at the 1980 ASCE meeting in

Portland, Oregon, and the Seminar on the Use of Synthetic Fabrics in Civil Engineering (1981) in Toronto. Results of research reported in these publications that is pertinent to our own work will be summarized later in this Chapter.

It seems appropriate at this point to specifically mention the research and other publications on geotextiles that have been sponsored by the Federal Highway Administration. First is the recently completed project at Oregon State University on "Evaluation of Test Methods and Use Criteria for Geotechnical Fabrics in Highway Applications". An interim report by Bell, Hicks, et al. (1980) has been issued, and a final project report (1982) is currently being reviewed by FHWA. Also very useful is a report by Haliburton, Lawmaster, and McGuffey (1981), prepared for the Office of Development of the FHWA, but not yet approved for release (as of February, 1983). This report is an excellent summary of the current state of the art of the use of geotextiles in highway practice. Parts of the report pertinent to our work will be referenced in Chapter 5. Two other FHWA publications are highly recommended. Steward, Williamson, and Mohny (1977) is a summary of U.S. Forest Service experience with fabrics in connection with construction and maintenance of low volume roads. FHWA (1978) is a useful compendium of sample specifications from several organizations for a variety of fabric uses.

In our opinion, the recommendations in these FHWA reports as to design, testing, and specifications of geotextiles are worthy of serious consideration by the designer.

1.1.2 Geogrids

An even newer material called "geogrids" has recently been developed which has some features similar to geotextiles. Geogrids look like nets of plastic; however the strength of these materials is significantly greater than typical plastic nets. An English company, Netlon, produces the new material under the trade name of "Tensar", and they will soon be manufactured in Canada also. On a weight basis, Tensar nets are as strong as steel but their cost is on the order of the heavier woven geotextile materials. They also have an added advantage over ordinary geotextiles of providing "interlock", in addition to frictional resistance, if materials coarser than sands are used in construction.

1.2 RESEARCH ON EARTH REINFORCING

1.2.1 Theoretical and Analytical Research

At the start of our research in 1978, most of the theoretical research on earth reinforcing had been applied to "classical" reinforced earth retaining walls (Vidal, 1966; Schlosser and Vidal, 1969; Hausmann and Lee, 1976; and Juran and Schlosser, 1978). On the other hand, relatively little analytical work had been done on the problem of reinforced embankments. The little research that had been done was secondary to other work on embankments.

For example, Chirapuntu and Duncan (1976) examined the Wager method of reinforcing embankments with sheet piles and tie rods, which was mentioned earlier (Wager and Holtz, 1976). Their work examined the effect of the strength of the fill on the stability of embankments on soft clay foundations. They found that considerable reduction in safety factors occurred due to cracking of the embankment. Based on both conventional stability analyses and the finite element method (FEM), their study indicated that Wager-type reinforcing could be very effective in preventing cracking, and they recommended that additional studies of embankment reinforcing be carried out.

Morgat (1976) used a finite element analysis to investigate the performance of embankments on soft foundations. His study included the effect of a dry crust at the top of the foundation clay as well as what happens when tensile reinforcement is placed at the base of the embankment. Morgat found that the tensile reinforcement eliminated internal embankment tensile stresses and increased the overall factor of safety against instability by as much as 20%.

At the Paris conference in 1977, three analytical papers discussed the problem of reinforced embankments. Maagdenberg (1977) presented an analytical study based on membrane theory and limiting equilibrium concepts. He concluded that the membrane would increase the stability significantly only if it was able to develop high strength at small deformations, i.e., a high modulus fabric would perform better. Broms (1977) discussed theoretical

design concepts for embankments as well as retaining walls reinforced with fabrics, and Bell, Greenway, and Vischer (1977) presented an analysis of a fabric reinforced embankment constructed on muskeg soils in southern Alaska. This latter case history and the analytical techniques utilized provided an excellent example for our own research.

In a very interesting analytical study, Bassett and Last (1978) showed that horizontally-lying reinforcement layers may not be the optimum orientation for reinforcement under embankments. In any case, however, practical construction requirements would probably control the actual design.

Ohta, Mochinaga, and Kurihara (1980) used an elasto-plastic FEM analysis on an idealized model of a soft foundation to investigate the improvement of the bearing capacity of an embankment by the use of reinforcement. Their results were compared with measurements of a trial embankment constructed in northern Japan. They concluded that transverse surface reinforcement was definitely capable of reducing the amount of deformation in the foundation and improving the bearing capacity. McGown, et al (1981) described the deformation behavior of embankments reinforced with fabrics determined from both laboratory scale models and the FEM. They were primarily interested in the effect of reinforcing the embankment itself with multiple layers of reinforcement. They found that the presence of the reinforcement changed the stress distribution and therefore the settlement pattern of the embankment. Toe settlements were somewhat greater whereas settlements

near the central portion of the embankment were reduced. Much smaller horizontal deformations at the interface between the embankment and the foundation were caused by the reinforcement as compared to embankments without reinforcement.

Several papers at the Second International Conference on Geotextiles in Las Vegas, August 1982, reported on analytical investigations of fabric-reinforced embankments. Papers involving analysis and design using a rather conventional limiting equilibrium approach included those by Ingold (1982), Jewell (1982), Fowler (1982), and Christie (1982). Papers utilizing the finite element technique (FEM) were those by Rowe (1982), Andrawes, et al. (1982) and Petrick, Baslik and Leitner (1982). Remarks at this point will be limited to a discussion of the papers using the FEM since that is the analytical technique we chose to use for our research.

Petrick, Baslik and Leitner (1982) found that vertical deformations were only slightly influenced by the presence or absence of the reinforcement while horizontal deformations were substantially influenced by reinforcement. They found that bearing capacity and stability of the embankment were also increased considerably, while stiff reinforcements were able to reach a much higher strength mobilization in the embankment. Rowe (1982) agreed with previous investigators that the fabric had very little effect on vertical settlements but may significantly reduce lateral spreading. He was concerned about the fabric at the edge of the embankment being unstressed, and he suggested that

sufficient fabric anchorage could be mobilized without the expense and inconvenience of overlapping the fabric at the edge of the embankment. The fabric was found to definitely increase the stability of the embankment; however, for fabric with a low to moderate modulus, excessively large deformations may occur prior to the fabric reaching its tensile capacity. In these cases, failures may be deemed to have occurred prior to rupture of the fabric. Rowe recommended that both fabric stiffness and tensile capacity be determined under conditions of plane strain, and he recommended that for the range of cases he considered, precise determination of the stiffness of the embankment was unnecessary.

Andrawes, et al. (1982) showed the general usefulness of the FEM applied to soil-geotextile systems. They noted that if the soil properties could be correctly represented in the soil elements, a good correlation between predicted and measured data was obtained up to about 85% of peak load. They felt beyond this stress level, the FEM is inappropriate because local failures of the soil occur which cannot be accommodated in the FEM procedures.

1.2.2 Experimental Work ~~→~~ Laboratory

Laboratory experimental research on geotextiles has been of primarily two types: (1) to investigate fundamental fabric properties and soil-fabric interaction, and 2) laboratory scale model tests of geotextile-reinforced walls and embankments. Most

of this research has been reported in the conference proceedings mentioned earlier (Sect. 1.1.1). Work with which we are quite familiar and pertinent to our research includes the research on primarily woven fabrics which was conducted at the Swedish Geotechnical Institute and at Purdue University (Holtz, 1973 and 1977; Holtz and Broms, 1977; Salomone, 1978). Other relevant laboratory research includes that by Haliburton, Anglin, and Lawmaster (1978b) on the mechanical properties of geotextiles and by Haliburton and Lawmaster (1981) on soil-fabric interaction. Belfrage and Eriksson (1980) and Belfrage (1981) described some laboratory scale model tests of embankments reinforced with rubber membranes as well as woven geotextiles. They concluded that the vertical as well as the horizontal deformations under the embankment were changed as the strength of the fabric increased. Differential settlements were also altered significantly by the fabric. They found that the effect of the fabric was greater for low embankments and larger widths at the same embankment height. They concluded that the polyester fabric placed under an embankment on soft soil would have a positive influence on the bearing capacity. However, for every situation there is an upper limit of the fabric modulus in tension, and fabric strengths are not fully utilized beyond this limit. McGown, et al. (1981) described laboratory model tests of embankments in which the embankment itself was reinforced with several layers of fabric. The results of this research were mentioned above in connection with the description of FEM analyses.

1.2.3 Experimental Work→Field Scale Tests and Case Histories

Among the earliest uses of geotextiles for reinforcing highway embankment were three field installations in Sweden. The sites were successfully stabilized with a woven polyester fabric, and the results of measurements at the sites were discussed by Holtz (1975) and Holtz and Massarsch (1976).

In the U.S., Lukanen and Teig (1976) described the use of geotextiles and other reinforcing techniques including corduroy for roadway widening through swamps in northern Minnesota. Six different methods for increasing stability were used at three different sites in the same general area and each site had one control section using the conventional method of "floating" the widening. Several interesting conclusions were developed from this study. The corduroy section showed no distress and apparently worked very well. The other sections which also performed well included one in which the fabric was placed only in the ditch where the widening took place. It is interesting that sections utilizing fabric for the full width of the section showed numerous longitudinal cracks. Another method which worked very well was the use of wood chips in the embankment to reduce fill weight. The recommendation that the existing vegetation or meadow mat over the peat should be left intact in the vicinity of the widening is good practical advice. This layer is in effect a reinforcing layer which contributes significantly to the overall stability of the section.

At the first Paris Geotextiles Conference, three field tests were reported. Belloni and Sembenelli (1977) described a full-scale loading test on peat deposits in Africa which was instrumented with piezometers and settlement plates. The performance of sections placed on fascines or directly on the peat were compared with sections reinforced with geotextiles. The authors concluded that there was some improvement with the use of the geotextile over that of the fascines. Volman, Krekt and Risseeuw (1977) described several large-scale trials which were carried out in Holland wherein embankments were reinforced with woven fabrics. One reinforced embankment performed very well while a companion embankment without reinforcing experienced failure when the height was more than a meter less than the reinforced embankment height. The case history of a low embankment on muskeg described by Bell, Greenway and Vischer (1977) was already mentioned. The reinforced embankment performed well but they were unable to prove it by their FEM analysis. As mentioned, this case history provided the model for our research.

Research using fabrics to reinforce embankments constructed on extremely soft foundations has been conducted by Haliburton, Anglin, and Lawmaster, 1978a; Haliburton and Fowler, 1980; and Fowler, 1981. Of particular interest are the methods of construction they developed.

Several case histories of geotextile reinforced embankments were presented at the Geotextiles Conference in Las Vegas. Particularly interesting for our research were the papers by Brakel,

et al. (1982); Hannon (1982); Barsvary, MacLean, and Cragg (1982); and Olivera (1982). In all these cases, the embankments were well instrumented, and the authors concluded that the presence of the geotextile definitely improved the stability of the embankment. In some cases, calculation methods were presented and verified by the results of the experiments.

1.3 FABRIC FUNCTIONS: REINFORCEMENT VS. SEPARATION

Although as noted above, reinforcing embankments with fabrics has been found empirically to increase their stability, the exact mechanism for this increase is not clearly understood, especially for low embankments with high live loads. In this case, a distinction between the fabric functions of separation and reinforcement is difficult to make. In the case of high embankments, reinforcement seems to be clearly the function of the fabric, since in this case the live load would be small in comparison to the dead load of the embankment itself. In fact, Haliburton, Lawmaster and King (1980) and Haliburton and Lawmaster (1981) have found, among other things, that the potential improvement of the performance of embankments and airfield runways results from three different phenomena: (1) The geotextile appears to act as a separation medium which prevents the intrusion and deterioration of the aggregate materials in the embankment. This phenomena is especially pertinent when the subgrade is soft and cohesive. (2) There appears to be a degree of lateral restraint provided by the fabric to the embankment and

foundation materials. As the load of the embankment is applied to the soft foundation, it tends to spread laterally. The friction at the fabric and the embankment-foundation interface tends to prevent this spreading. A similar mechanism results when live loads are applied to the embankment. Again, the tendency for lateral spreading is limited by the lateral restraint of the fabric. (3) The third mechanism or benefit provided by the geotextile is that of membrane-type support. For this mechanism to occur, relatively large vertical deformations must take place in the subgrade to mobilize the full membrane resistance. In our research we investigated the effect of lateral restraint as well as that of membrane support.

1.4 REINFORCED HAUL ROADS, PAVEMENTS, AND MESL

1.4.1 Unpaved Roads

Considerable research has been conducted on the use of fabrics and other membranes under small embankments such as haul roads constructed on very soft foundations. Much of this research was sponsored by the manufacturers of nonwoven geotextiles. Consequently, in our opinion, some of their proposed design methods are somewhat self-serving. In several cases, the design assumptions are not clearly stated and rarely are the experimental data or theoretical analyses supporting the methods made available. Summaries of the various design methods have been given by Koerner and Welsh (1980, 1981) and Lai and Robnett (1981). The notable exception in the list of manufacturer-

sponsored methods is the procedure developed by Barenberg and his students. This procedure has a reasonably sound theoretical and empirical basis. For a description of this method, see Bender and Barenberg (1978) and Kinney and Barenberg (1982). Kinney (1979) has developed a "fabric tension model" by which the modulus of the geotextile as well as subgrade strength, traffic loads, and rut geometry can be appropriately considered.

Giroud and Noiray (1981) developed a method that has a very sound theoretical basis and takes into account data from full scale tests carried out at the U.S.A.E. Waterways Experiment Station. The method offers design charts that allow the determination of aggregate thicknesses for unpaved roads when geotextiles are used as reinforcement and when traffic is taken into account. The rut depth considered in the design charts is approximately 1 ft (0.30 m). Recently, Sivakugan (1982) prepared design charts for lesser rut depths. Since Giroud and Noiray's (1981) method is for temporary haul road applications, tire inflation pressures and axle loads given in the charts are typical of construction vehicles. The standard axle load is about 18,000 lbs (80 kN) and maximum tire inflation pressure is about 90 psi (620 kPa). For the purpose for which it was developed, the Giroud and Noiray (1981) method is both elegant and simple to use, and according to Giroud (1982, personal communication) the method has been used with considerable success in practice.

Other recent research on unpaved roads was reported at the Second International Conference on Geotextiles in Las Vegas.

Robnett, Lai, and Murch (1982), in an analysis of laboratory test results and other information, concluded that the most important fabric property in reducing the amount of rutting and therefore the amount of aggregate in an unpaved road was the modulus of the fabric. Sellmeijer, Kenter, and Van Den Berg (1982) presented a new calculation method for fabric reinforced haul roads. They concluded that the modulus of the geotextile was the most important factor in performance and aggregate savings. Raumann (1982) discussed design considerations for the use of geotextiles in unpaved roads. Her analysis was based on both field and laboratory tests. She concluded that the most suitable geotextiles combine high strength and intermediate modulus (tested in plane strain), but they also require adequate deformability as determined in an unrestrained (grab or strip tensile test) mode to allow for installation or construction requirements.

Kinney and Barenberg (1982) presented the results of tests on a two-dimensional experimental model of the soil-geotextile-aggregate system of an unpaved road. They described a technique for calculating the tension in the geotextile at any point within the deformed profile of the fabric. Barksdale, et al (1982) also discussed the experimental and theoretical behavior of geotextile reinforced aggregate soil systems. Tests conducted in the laboratory modeled haul road conditions, and a finite element program was used for the analytical studies. They concluded that both the model tests and the FEM analysis indicated that the presence of fabric results in a definite beneficial alteration of

the stress and plastic strain distribution in fabric-reinforced haul roads. (Our research indicated the same general conclusions for fabric-reinforced embankments.) Sowers, Collins and Miller (1982) described some experiments carried out in the field to investigate the mechanism of geotextile-aggregate support in unpaved roads. Their studies primarily considered the separation effect of the geotextile. Ruddock, Potter and McAvoy (1982) reported on full-scale field experiments of both aggregate and bituminous pavements on fabrics. They concluded that with aggregate embankments, the presence of the fabric reduced the rate of surface deformation as long as the fabric was intact. Permanent vertical strains were also reduced although transient vertical stresses and strains were apparently not changed. Transient and permanent horizontal strains were also reduced by the presence of the fabric. It is interesting to note that for bituminous pavements, the structural behavior of the pavements was not improved by the presence of the fabric. This is in contrast to the findings of Hamilton and Pearce (1981), described below.

1.4.2 Paved Roads

Hamilton and Pearce (1981) developed guidelines for the design of flexible pavements using slit film woven fabrics. The method is specifically applicable to the Texas Gulf Coast region where very poor subsoils predominate and suitable construction aggregates are either nonexistent or of poor quality. Significant haulage distances can result in extremely high construction costs. Hamilton and Pearce (1981) found that high modulus

geotextiles have the potential of solving many of the pavement problems in that region. They present a design method and suggest that the use of woven geotextiles offers (1) a reasonable and cost effective alternative to mechanical or chemical subgrade stabilization, (2) a reduction in required base thicknesses, and (3) an extended pavement life.

1.4.3 MESL

For about 15 years, the U.S.A.E. Waterways Experiment Station (WES) has been conducting research on membrane encapsulated soil layers (MESL). Both nonwoven geotextiles as well as military membranes such as the T-16, T-17 and WX-18 membrane mats have been used (Burns and Barber, 1969 and 1971). WES has also conducted research on "sand bag" type structures for the expedient construction of bridge piers and abutments in a theater of operations (Webster, 1975). Recent research by the same group has involved very unconventional reinforcing materials such as small plastic cylinders and paper and aluminum grids, both hexagonal and rectangular in shape. Roadways were constructed of reinforced sand on both very soft clay and sand subgrades. These tests are described by Webster and Alford (1978) and Webster (1979 and 1981).

1.5 STATEMENT OF THE PROBLEM

Although it appears that considerable research has been carried out on the reinforcing of embankments on soft foundations,

at the time we began our research, there were important deficiencies in the design methods available. Probably the most serious was the lack of a viable analytical model for the behavior of fabric-reinforced embankments. In addition, meaningful design parameters for the interaction of soil and fabric were not available (and, with few exceptions, they still are not). Considerable progress has been made in the development of analytical models as outlined in this Chapter. Our objectives at the time we began this research were to define the fundamental behavior of the soil reinforcing system, including the mechanism of shear stress transfer from the soil to the reinforcement. In addition, we wished to develop a theoretical model for reinforced embankments on soft soil foundations, including, if possible, the special cases of localized zones of little or no support and embankment widening. We were to consider the geotechnical and environmental factors applicable to the design of fabric-reinforced embankments, and we wanted to suggest simple and conservative procedures for the design and construction of such embankments.

With the exception of the case of embankment widening, all of these objectives have been met as will be described in the following report.

1.6 OUTLINE OF THE REPORT

In Chapter 2, a brief description is given of some of the analytical models used to describe reinforced soil systems. Modifications to the models and the FEM analysis used for this

research are also mentioned. Chapter 3 presents the finite element analyses of unreinforced embankments, and in Chapter 4, the influence of geotextile reinforcement on the stress-deformation behavior of embankments is presented. Conclusions and practical implications, recommendations for design and construction, and recommendations for further research are given in Chapter 5.

The program we used is listed in Appendix A and described in detail in Appendix B. How the input data are prepared is described in Appendix C. Appendix D presents the results of a study of different Gaussian integration orders.

Chapter 2

ANALYTICAL MODELS

The analytical method we utilized to investigate the influence of fabric reinforcement on the performance of embankments constructed on weak foundations is the method of finite elements (FEM). This procedure is well suited for plane deformation problems.

2.1 MODELING REQUIREMENTS AND SELECTION OF PROGRAM

The program selected should be capable of handling large deformations, have elements appropriate for modeling the soil as well as the fabric reinforcement, and be able to consider realistic models for the stress-strain behavior of the foundation and embankment soils.

Several alternatives were considered. Among them was the concept of utilizing a basic small displacement finite element program, and then make alterations as required to account for large displacements. Another approach was to find a program that already included a large displacement formulation. The program NONSAP (Bathe, Wilson, and Iding, 1974) has this special feature, and it was selected for the study.

NONSAP provides two-dimensional isoparametric elements suitable for modeling the soil, and bar or truss elements which can model the fabric. It also has a good selection of linear and nonlinear material behavior models.

2.2 SELECTION OF SOIL MODELS

Part of the process of developing a computer code for this study involved the choice of soil models. With FEM analyses, more sophisticated material models are possible, some of which are particularly applicable to soils. Which model to use depends to a great extent on the problem to be solved.

2.2.1 Linear Elastic Models

The simplest material model is the linear elastic model. The NONSAP program has both isotropic and orthotropic linear elastic models available.

In a first trial, the linear elastic model was used for both the embankment and foundation. This approach, however, resulted in large horizontal tensile stresses at the base of the embankment, and these occurred even for relatively small foundation settlements. Since the embankment soil is assumed to be granular, no tensile stresses can be taken by the soil; hence this model would not be appropriate for the embankment.

Similar results, i.e. the presence of undesirable tensile stresses in the lower part of the embankment, were reported by Greenway and Bell (1976) using linear theory. To minimize the tendency for these stresses to develop, they considered the lower two-thirds of the embankment to be an orthotropic linearly elastic material with a very small elastic modulus in the horizontal direction and a Poisson's ratio of zero.

Since a Poisson's ratio of zero corresponds to the case of no lateral strain due to the vertical stresses, this assumption helps to prevent the development of lateral tension. However, we do not consider it a realistic assumption. Further, it reduces the calculated lateral movement of the embankment, which is an important factor in the evaluation of the influence of the inclusion of fabric reinforcement in the embankment. As we shall later see, the smaller the lateral deformation, the smaller is the influence of horizontally-placed fabric reinforcement. We believe the problem of unwanted tension in the base of the embankment is better handled by the use of a more realistic soil model, e.g. the Drucker-Prager (1952) model.

For a cohesive foundation soil, on the other hand, a linearly elastic soil model may be quite reasonable for an evaluation of the initial (undrained) settlements of the foundation; at least, such an approach is common in foundation engineering practice.

2.2.2 Curve Description Models

A hyperbolic stress-strain relationship introduced by Kondner (1963) and further developed by Duncan and Chang (1970) is very popular in finite element analyses of soil structures. Laboratory data from ordinary triaxial tests can usually be easily fitted to this model.

Three curve description models are available in the NONSAP program, all of which may be fitted to the hyperbolic

relationship. For all of them, both loading and unloading moduli are defined. However, since none of them allows iteration for equilibrium, they are not suitable for large displacement problems, where iteration for equilibrium is essential.

2.2.3 Elasto-Plastic Models

Elasto-plastic models are suitable for large displacement problems. In the program NONSAP, two elasto-plastic models are available. One is the Von Mises criterion which is well known from the theory of plasticity (Chen and Saleeb, 1982). The other elasto-plastic model in NONSAP is the Drucker-Prager (1952) model. This model is an approximation in three dimensions of the well-known Mohr Coulomb failure criterion, which is the most common failure criterion in geotechnical engineering. Excellent descriptions of the Drucker-Prager failure criterion are given by Mizuno and Chen (1980a, 1980b) and Chen and Saleeb (1982). As the Drucker-Prager model appeared to be the most reasonable model for soils, it was chosen for most of the analyses reported herein. For undrained ($\phi = 0$) analyses of foundation soils, the Von Mises model, a special case of Drucker-Prager, was used. The Drucker-Prager relationships and material constants are relatively easily determined and they have certain mathematical advantages which are useful in FEM analyses. As pointed out by Mizuno and Chen (1980a), the Drucker-Prager model cannot predict plastic volumetric strain during hydrostatic loading, and to take care of this difficulty, the so-called Cap Models have been developed (Sandler, DiMaggio, and Baladi, 1976).

We also experienced numerical convergence difficulties during iteration with Drucker-Prager at large strains. We had to develop and use different convergence rules for these cases.

2.3 LARGE DISPLACEMENT AND INCREMENTAL CONSTRUCTION FORMULATION

The two basic ways for describing the deformations of a continuum are the Lagrangian and the Eulerian (Chen and Saleeb, 1982). The Lagrangian method is our familiar engineering strain in which the deformations are referenced to the initial position of the element. With Eulerian strain, the referenced state is the coordinates of the material element at the particular time of interest or at their present position. For small strains, it can be shown that both approaches yield the same thing, but this is not the case for large displacements. Thus, for the incremental loading analysis with large displacements in NONSAP, an alternate definition of strain was employed. In the analysis, the geometry of the embankment is "updated" after each construction lift is applied. This procedure requires that the strain calculation be based on updated rather than initial embankment geometry. To be consistent, the strain is redefined in terms of an approximate logarithmic strain instead of our familiar linear strain.

Conventional or engineering strain, ϵ , is

$$\epsilon = \frac{\Delta L}{L_0} = \frac{L - L_0}{L_0} = \frac{L}{L_0} - 1$$

Logarithmic strain, $\bar{\epsilon}$, is

$$\bar{\epsilon} = \ln \frac{L}{L_0}$$

Thus ϵ and $\bar{\epsilon}$ are related by

$$\bar{\epsilon} = \ln (1 + \epsilon)$$

In an incremental or differential formulation, conventional or Lagrangian strain is

$$d\epsilon = \frac{dL}{L_0}$$

while the differential logarithmic strain is

$$d\bar{\epsilon} = \frac{dL}{L_0} \frac{L_0}{L} = \frac{dL}{L}$$

The two are related by

$$d\bar{\epsilon} = \frac{d\epsilon}{1 + \epsilon}$$

Thus, an infinitesimal increment of the conventional strain expresses the change in length with respect to the original length L_0 of the element, whereas increments of logarithmic strain are calculated in terms of the instantaneous length L . As mentioned, for small strains, both definitions are identical (Krizek, 1966).

A detailed investigation of stress and strain definitions resulted in the conclusion that a Cauchy stress, which is based on current geometry, combined with a logarithmic definition of strain were the most logical formulations for the large strain deformation analysis. When these formulations are used, a work-

able computer code results which encompasses a much larger range of deformations than was the case with the original large displacement formulations that were found in NONSAP.

The program NONSAP was modified to consider incremental loading in order to better simulate the actual construction sequence of an embankment. Results from this modification were compared with those from a well-known FEM program CANDE (Katona, et al., 1976), which was developed for the analysis of buried culverts and which has provision for incremental construction. Agreement in terms of stresses and vertical displacements between our modified program and CANDE was considered to be good, which gave us confidence that our incremental loading modifications were properly carried out.

2.4 MODIFICATIONS TO NONSAP

Several modifications were necessary to make the program suitable for our problem. Even with these modifications, it is not an easy task to analyze large deformation problems, particularly when the foundation is assumed to be undrained (Poisson's ratio close to 0.5). It is a general deficiency of the conventional finite element formulations that problems having a Poisson's ratio of 0.5 are impossible to solve without some modifications or adjustments. The reason for this is that the finite element formulation solves the equation

$$[k]\{D\} = \{R\}$$

where $[k]$ is the structure stiffness matrix, $\{D\}$ the displacement vector, and $\{R\}$ the force vector.

If Poisson's ratio ν is 0.5, the bulk modulus

$$B = \frac{E}{3(1 - 2\nu)}$$

will be infinite and the above equation will "blow up" since B appears in the $[k]$ matrix.

When trying to solve a problem having a Poisson's ratio close to 0.5, convergence becomes a difficulty, which is further exacerbated by material and geometric nonlinearities.

The term "geometric nonlinearity" expresses the fact that the structure deforms so much that the formulation of the stiffness matrix must include an account for the change in geometry that occurs during the loading and/or unloading.

Material nonlinearity refers to any material model in which the behavior differs from that of a linear elastic material.

The NONSAP program is rather bulky. Since only a two-dimensional static analysis was of interest in this study, it was desirable to eliminate the three-dimensional and dynamic analysis parts of the program, parts that were not likely to be used. Hence, the three-dimensional element routines and the routines for dynamic analysis were eliminated.

The modified program is called GEONON. A listing of it is given in Appendix A, and it is described in some detail in Appendix B. For preparation of the data, see Appendix C. Appendix D is a study of the effect of higher order Gaussian integration on the results.

Chapter 3

ANALYSIS OF UNREINFORCED EMBANKMENTS

Prior to any analyses of fabric-reinforced embankments, an investigation of stresses and displacements in an unreinforced embankment-foundation system under various conditions was carried out. This was done in order to understand how various factors besides reinforcement influence the stresses and displacements.

Before presenting the results of these investigations, it should be emphasized that the finite element method is quite sensitive to any change in the "interior" factors such as the number of load increments used, the convergence tolerance, and the iteration scheme applied. So unless it is possible to keep these parameters constant while changing "exterior" factors such as the addition of fabric reinforcement, the results can only be compared with great caution.

3.1 SINGLE VERSUS MULTILIFT CONSTRUCTION

The embankment-foundation system analyzed in this and the following sections is adapted from an embankment studied by Greenway and Bell (1976), (also described by Bell, Greenway and Vischer, 1977) but with a slightly modified finite element mesh which is shown in Fig. 3.1. The embankment is assumed to be composed of granular fill with a slight amount of cohesion. A Drucker-Prager (1952) soil model (See Chapter 2) is used for the

embankment with a Young's modulus E of 600,000 psf (28,700 kPa), a Poisson's ratio ν of 0.25, compacted density of 120 pcf (1.92 Mg/m³), and strength parameters $\phi'_E = 35^\circ$ and $c'_E = 25$ psf (1.2 kPa). Foundation properties varied depending on the particular case analyzed.

The embankment was analyzed for one lift versus multilift construction on both stiff and soft foundations and for both undrained and drained responses of the foundation. Clough and Woodward (1967) considered the influence of multilift versus single lift construction by a finite element analysis, and the influence was significant, particularly on the deformations within the embankment proper.

For this study, the stresses in the embankment and foundation and the displacements of the embankment-foundation interface are of major interest because of the potential influence of the reinforcement layer (Chapter 4).

3.1.1 Stiff Foundation, Undrained Response

Figures 3.2 and 3.3 show the orientation and relative magnitude of the principal stresses for a stiff foundation with undrained response, for one lift and three lift construction simulation, respectively. The results are almost identical, and plots of the vertical and horizontal stress contours show very little difference (see Figs. 3.4 and 3.5). Also the settlements under the embankment are essentially identical (Figs. 3.6 to 3.8).

FIGURE 3.2 Orientation and relative magnitude of the principal stresses after embankment load is activated. Embankment is on a stiff foundation, with one lift construction and undrained response ($E_F = E_E = 600,000 \text{ psf}$, $\nu_F = 0.48$).

FIGURE 3.3 Orientation and relative magnitude of the principal stresses after embankment load is activated. Embankment is on a stiff foundation, with three lift construction and undrained response ($E_F = E_E = 600,000 \text{ psf}$, $\nu_F = 0.48$).

(a) One Lift

(b) Three Lifts

FIGURE 3.4 Contours of vertical stress (psf) for one lift and multilift construction, undrained response ($v_F = 0.48$), stiff foundation ($E_F = E_E = 600,000$ psf).

(a) One Lift

(b) Three Lifts

FIGURE 3.5 Contours of horizontal stress (psf) for one lift and multilift construction, undrained response ($v_F = 0.48$), stiff foundation ($E_F = E_E = 600,000$ psf).

FIGURE 3.6 Vertical displacement of ground surface; stiff foundation, undrained response.

FIGURE 3.7 Horizontal displacement of ground surface; stiff foundation, undrained response.

FIGURE 3.8 Horizontal displacement under toe of embankment slope; stiff foundation, undrained response.

3.1.2 Soft Foundation, Undrained Response

Figures 3.9 and 3.10 show the principal stresses for the soft foundation with undrained response for one and three lift construction simulation. Compared to the stiff foundation, large horizontal stresses developed in the upper part of the embankment for both the one lift and the three lift case, although they are most pronounced for the single lift case.

Figures 3.11 to 3.13 show the stress contour lines for vertical and horizontal normal stresses and for maximum shear stresses for these two cases. The vertical stresses in the foundation are significantly increased underneath the central portion of the embankment for the three lift case compared to the one lift case, and they are slightly reduced to the sides. This is caused by the fact that the embankment settles more in the center than on the sides. Therefore, each new horizontally placed lift will be thicker at the center line of the embankment and thinner towards the embankment slopes. The vertical stresses in the embankment are slightly increased at the bottom and slightly reduced at the top for the three lift case as compared to the one lift case. Notice also that the vertical stress contour lines for the stiff foundation response (Fig. 3.4) meet at the embankment-foundation interface. This is not the case for the soft foundation response where a discontinuity exists between the vertical stress contour lines for the embankment and those for the foundation (Fig. 3.11). A similar discontinuity appears for all the large displacement analyses and seems to be an inherent

FIGURE 3.9 Orientation and relative magnitude of the principal stresses after embankment load is activated. Embankment is on a soft foundation, with one lift construction and undrained response ($E_F = 3000 \text{ psf}$, $\nu_F = 0.48$).

FIGURE 3.10 Orientation and relative magnitude of the principal stresses after embankment load is activated. Embankment is on a soft foundation, with three lift construction and undrained response ($E_F = 3000 \text{ psf}$, $\nu_F = 0.48$).

(a) One Lift

(b) Three Lifts

FIGURE 3.11 Contours of vertical stress (psf) for one lift and multilift construction, undrained response ($v_F = 0.48$), soft foundation ($E_F = 3000$ psf)

(a) One Lift

(b) Three Lifts

1 ft = 0.305 m

1 psf = 47.88 N/m²

FIGURE 3.12 Contours of horizontal stress (psf) for one lift and multilift construction, undrained response ($\nu_F = 0.48$), soft foundation ($E_F = 3000$ psf).

FIGURE 3.13 Contours of maximum shear stress (psf) for one lift and multilift construction, undrained response ($v_F = 0.48$), soft foundation ($E_F = 3000$ psf).

deficiency of this type of analysis.

As can be seen in the principal stress plots (Figs. 3.2, 3.3, 3.9 and 3.10), as well as in Figs. 3.5 and 3.12, the horizontal stresses are tremendously increased in the upper portion of the embankment (as compared to the stiff foundation). This is most pronounced for the one lift case (compare Figs. 3.9 and 3.2). However, the horizontal stresses in the foundation are larger for the three lift case.

With respect to maximum shear stresses (Fig. 3.13), for the one lift case these are increased in the upper part of the embankment but are smaller in the foundation, than for the three lift case.

Plots of settlements for the soft foundation, undrained response are shown in Figures 3.14 to 3.16. The multilift case settles more in the center portion under the embankment and less under the embankment slope than does the one lift case. This is probably mostly due to the larger load in the center for the multilift case, as discussed earlier, but it may also be partly due to the more pronounced arching in the one lift case. Arching tends to redistribute the vertical load out towards the sides. The phenomenon of arching will be discussed later.

The horizontal displacements at the ground surface are somewhat smaller for the multilift case (Fig. 3.15). The horizontal displacements along a vertical line directly under the toe of the embankment slope are almost identical for the two cases.

FIGURE 3.14 Vertical displacement of the ground surface; soft foundation, undrained response.

FIGURE 3.15 Horizontal displacement of ground surface; soft foundation, undrained response.

FIGURE 3.16 Horizontal displacement under the toe of the embankment slope; soft foundation, undrained response.

3.1.3 Stiff Foundation, Drained Response

The principal stresses for the stiff foundation with a drained response are shown in Figs. 3.17 and 3.18. Contours of the various stresses are shown in Figs. 3.19 to 3.21. Vertical stresses are slightly increased below the central part of the embankment for the multilift case; otherwise the vertical stresses are almost identical for the one lift and multilift construction simulation. On the other hand, horizontal stresses are larger in the upper part of the one lift embankment, again due to a tendency for the development of arching. The horizontal foundation stresses are slightly larger for the multilift case.

For the undrained response of the foundation, maximum shear stresses were plotted, since they show which parts of the foundation are closest to yield. For the drained response of the foundation, proximity to failure depends on shear as well as normal stresses. To evaluate how close a particular state of stress is to failure in drained conditions, the ratio $\sqrt{J'_2}/\sqrt{J'_{2f}}$ was determined, where J'_2 is the second invariant of the deviatoric stresses and J'_{2f} is the value of J'_2 at failure (Chen and Saleeb, 1982). In the evaluation of J'_{2f} , J'_1 (the first invariant of the deviatoric stress: $J'_1 = \sigma_1 + \sigma_2 + \sigma_3$), is assumed constant; that is, $J'_{1f} = J'_1$. Figure 3.22 illustrates how the values of $\sqrt{J'_2}$ and $\sqrt{J'_{2f}}$ are determined for a particular state of stress. Since the octahedral shear stress $\tau_{oct} = \sqrt{\frac{2}{3}} \sqrt{J'_2}$, the ratio $\sqrt{J'_2}/\sqrt{J'_{2f}} = \tau_{oct}/\tau_{oct_f}$ is hereafter referred to as the

FIGURE 3.17 Orientation and relative magnitude of the principal stresses after embankment load is activated. Embankment is on a stiff foundation, with one lift construction and drained response ($E_F = E_E = 600,000 \text{ psf}$, $\nu_F = 0.25$).

FIGURE 3.18 Orientation and relative magnitude of the principal stresses after embankment load is activated. Embankment is on a stiff foundation, with three lift construction and drained response ($E_F = E_E = 600,000 \text{ psf}$, $\nu_F = 0.25$).

FIGURE 3.19 Contours of vertical stress (psf) for one lift and multilift construction; drained response ($\nu_F = 0.25$), stiff foundation ($E_F = E_E = 600,000$ psf).

FIGURE 3.20 Contours of horizontal stress (psf) for one lift and multilift construction; drained response ($\nu_F = 0.25$), stiff foundation ($E_F = E_E = 600,000$ psf).

FIGURE 3.21 Contours of octahedral shear stress ratio (expressed as a percent) for one lift and multilift construction; drained response ($\nu_F = 0.25$), stiff foundation ($E_F = E_E = 600,000$ psf).

J_1 = First invariant of the stress tensor

J_2' = Second invariant of the deviatoric stresses = $\frac{3}{2} \tau_{oct}^2$

Octahedral shear stress ratio:

$$\frac{\tau_{oct}}{\tau_{oct f}} = \frac{\sqrt{J_2'}}{\sqrt{J_{2f}'}} = \frac{\sqrt{J_2'}}{\alpha J_1 + k}$$

FIGURE 3.22 Illustration of the octahedral shear stress ratio.

octahedral shear stress ratio.

Figure 3.21 shows the contours of the octahedral shear stress ratio for the drained, stiff response of the foundation. The one lift case may be closer to yielding than is the three lift case, but the difference is small.

Figures 3.23 to 3.26 show the displacements for the drained, stiff response of the foundation. Whereas the vertical displacements of the ground surface are almost identical for the one lift and three lift construction simulation, the horizontal displacements of the ground surface underneath the embankment are significantly larger for the one lift case. The uneven deflection profile that occurs under the embankment (Figs. 3.24 and 3.25) seems to be caused by numerical oscillation due to the discontinuity of vertical loading. It is not considered to represent the actual displacement. The dashed line in Fig. 3.25 represents the probable actual horizontal displacement of the ground surface. If the horizontal displacements along a horizontal line are examined at some depth below the ground surface (Fig. 3.25), the deflection profile is much smoother and without any peaks. Figure 3.26 shows the horizontal displacement under the toe of the embankment slope. Again the one lift construction simulation deflects more than the multilift one.

3.1.4 Soft Foundation, Drained Response

Finally, Fig. 3.27 and 3.28 show the principal stresses for an embankment on soft foundation, drained response, one and

FIGURE 3.23 Vertical displacement of ground surface; stiff foundation, drained response.

FIGURE 3.24 Horizontal displacement of ground surface; stiff foundation, drained response.

FIGURE 3.25 Horizontal displacements of stiff foundation; drained response, multilift construction.

FIGURE 3.26 Horizontal displacement under toe of embankment slope, stiff foundation, drained response.

FIGURE 3.27 Orientation and relative magnitude of the principal stresses after embankment load is activated. Embankment is on a soft foundation, with one lift construction and drained response ($E_F = 3000 \text{ psf}$, $\nu_F = 0.25$).

FIGURE 3.28 Orientation and relative magnitude of the principal stresses after embankment load is activated. Embankment is on a soft foundation, with three lift construction and drained response ($E_F = 3000 \text{ psf}$, $\nu_F = 0.25$).

multilift construction simulation. Again large horizontal stresses can be observed in the upper part of the embankment, as was the case for the soft, undrained foundation response. This behavior is more pronounced for the one lift case. Figures 3.29 to 3.31 show the various stress contours for the one and multilift situations. Again the multilift construction simulation has higher vertical stresses below the central part of the embankment and slightly lower ones under the slope. The arching phenomenon is very pronounced for the one lift case for both vertical as well as horizontal embankment stresses. Since embankments are never constructed in one lift, the results do not directly apply to actual embankments. Even if the embankment does settle as one unit, e.g., as would occur for consolidation settlements, it is doubtful that the stresses would ever reach the high levels indicated by the finite element analysis.

The horizontal stresses in the foundation are quite similar for the two cases. The octahedral shear stress ratio, Fig. 3.31 and defined in Fig. 3.22, is somewhat larger underneath the central part of the embankment for the multilift construction simulation, and somewhat smaller under the toe of the embankment slope.

The displacements for these two cases are shown in Figs. 3.32 to 3.35. The vertical settlement of the ground surface (Fig. 3.32) again demonstrates that the multilift simulation settles more under the center of the embankment and less under the embankment slope than the one lift simulation. With respect to

FIGURE 3.29 Contours of vertical stress (psf) for one lift and multilift construction; drained response ($\nu = 0.25$), soft foundation ($E_F = 3000$ psf).

(a) One Lift

(b) Three Lifts

FIGURE 3.30 Contours of horizontal stress (psf) for one lift and multilift construction; drained response ($\nu = 0.25$), soft foundation ($E_F = 3000$ psf).

FIGURE 3.31 Contours of octahedral shear stress ratio (expressed as a percentage) for one lift and multilift construction; drained response ($v_F = 0.25$) soft foundation ($E_F = 3000$ psf).

FIGURE 3.32 Vertical displacement of the ground surface; soft foundation, drained response.

FIGURE 3.33 Horizontal displacement of the ground surface; soft foundation, drained response.

FIGURE 3.34 Horizontal displacements of soft foundation; drained response, multilift construction.

FIGURE 3.35 Horizontal displacement under toe of embankment slope; soft foundation, drained response.

horizontal displacements the multilift case also moves more than the one lift case. This is opposite to the stiff foundation, drained response case. Again a uneven deflection profile is observed, which is believed to represent a numerical error. Figure 3.34 shows the horizontal displacements at some depth in the foundation, and the deflection profile is smooth. The dashed line indicates the probable displacement of the ground surface.

The numerical error of the calculated horizontal displacement might have been reduced by using a finer finite element mesh. However, for the purpose of investigating the influence of fabric, only the settlements under the embankment proper are important, and these settlements are likely to be not significantly influenced by this numerical error.

Figure 3.35 shows the horizontal displacement with depth under the toe of the embankment slope. Again the multilift case shows the most displacement.

3.1.5 Conclusion

A general conclusion is that except for the stiff foundation and undrained response, it does make a difference whether the embankment construction is simulated in one or multiple lifts. Hence it was decided to base the following analyses on multilift construction simulation.

3.2 STIFF VERSUS SOFT FOUNDATION

Since stresses in a foundation often are calculated by linear elastic theory, which always presumes small strains and displacements, it is of interest to observe how the stress field changes when large displacements occur due to loading a soft foundation. Figures 3.36 to 3.38 show the stress contours for the undrained response of stiff and soft foundations. The vertical stresses increase underneath the embankment on the soft subsoil, and decrease to the sides as compared to the stiff subsoil. This difference, however, is totally due to the multilift construction simulation, which causes a thicker lift to be placed in the center of the embankment where the settlements are the greatest. A one lift simulation (comparing Figs. 3.4(a) and 3.11(a)) shows just the opposite result. The vertical stresses are reduced in the center and increased to the sides of the embankment for the soft foundation, probably due to the arching effect created by the large settlements under the center of the embankment. (See Section 3.3 for further discussion of the arching phenomenon.)

Also the horizontal stresses are significantly larger in the soft foundation (Fig. 3.37), which results in a reduction of the maximum shear stresses in the soft foundation as compared to the stiff (Fig. 3.38). Similar results can be observed for the one lift construction simulation (compare Figs. 3.5(a) and 3.12(a) for the horizontal stresses, and Figs. 3.38(a) and 3.13(a) for the maximum shear stresses). All stresses were identical for one

FIGURE 3.36 Contours of vertical stress (psf) for stiff and soft foundations, undrained response ($\nu_F = 0.48$), and multilift construction.

FIGURE 3.37 Contours of horizontal stress (psf) for stiff and soft foundations, undrained response ($\nu_F = 0.48$), and multilift construction.

FIGURE 3.38 Contours of maximum shear stress (psf) for stiff and soft foundations, undrained response ($v_F = 0.48$), and multilift construction.

lift and multilift construction simulation on stiff, undrained foundations.

Figures 3.39 to 3.41 shows the stress contours for the drained response of stiff and soft foundations. It is now observed that despite the heavier loading in the central part of the embankment for the soft foundation, the arching effect results in smaller vertical stresses in the central part and larger stresses towards the sides than for the stiff foundation (Fig. 3.39). The horizontal stresses are mostly increased somewhat for the soft foundation (Fig. 3.40). As shown in Fig. 3.41, the octahedral shear stress ratio is reduced under the central part of the embankment and increased towards the sides of the soft foundation (as compared to the stiff).

3.3 THE ARCHING PHENOMENON

The phenomenon of arching in an embankment was illustrated by Casagrande in 1936 (Fig. 3.42). When the foundation is relatively incompressible the stresses exerted on the ground are distributed as shown in Fig. 3.42(a), with the largest vertical stress under the center of the embankment. When the ground is soft and compressible, large settlements take place under the center of the embankment and smaller settlements occur under the slopes. The non-uniform settlement results in the development of a stress arc in the embankment, which reduces the vertical stresses under the center of the embankment and increases the stresses under the slopes, as shown in Fig. 3.42(b).

FIGURE 3.39 Contours of vertical stress (psf) for stiff and soft foundations, drained response ($v_F = 0.25$) and multilift construction.

FIGURE 3.40 Contours of horizontal stress (psf) for stiff and soft foundations, drained response ($\nu_F = 0.25$), and multilift construction.

FIGURE 3.41 Contours of the octahedral shear stress ratio for stiff and soft foundations, drained response ($\nu = 0.25$), and multilift construction.

(a)

(b)

FIGURE 3.42 Arching in an embankment on a soft foundation (after Casagrande, 1936).

Trollope (1957) presented a systematic arching theory applied to the stability analysis of embankments. He showed that arching within an embankment might create an adverse orientation of stresses, so that a failure within the embankment is possible. This concept is illustrated in Fig. 3.43. Figure 3.43(a) shows the no arching case where the orientation of principal stresses are close to the vertical and the horizontal. The orientation of potential failure lines makes an angle of $45^\circ + \phi/2$ with respect to the direction of the minor principal stress (or major principal plane). It is apparent that the orientation of these failure lines in the no arching case does not increase the danger of failure. On the other hand, in the full arching case [Fig. 3.43(b)], the orientation of the principal stresses is approximately at a 45° angle with respect to vertical and horizontal, and the possible failure planes intersect the slope in a manner that makes failure possible.

Figures 3.44 and 3.45 show the direction and relative magnitude of principal stresses for an embankment on both stiff and soft foundations for drained response. It is apparent that whereas the embankment stresses are mostly vertical-horizontal on the stiff foundation, significant reorientation of these stresses occurs for the embankment on the soft foundation. Since the state of stress is on the yield envelope, local failure of the embankment may occur, unless measures are taken to prevent it.

Figure 3.46 shows the distribution of vertical stresses 0.75 ft below the ground surface for both stiff and soft foundations

(a) No Arching Case

(b) Full Arching Case

- Major principal planes
- Minor principal planes
- Possible failure planes

FIGURE 3.44 Orientation and relative magnitude of principal stresses and of potential failure lines in an embankment on a stiff foundation for drained response.

FIGURE 3.45 Orientation and relative magnitude of principal stresses and of potential failure lines in an embankment on a soft foundation for drained response.

FIGURE 3.46 Vertical stresses at the 0.75 ft level in the foundation for (a) drained response, and (b) undrained response for both stiff and soft foundations.

and for both drained and undrained responses. The difference in stress distribution between a stiff and a soft foundation is more pronounced for drained response. All cases represent the one lift construction simulation, since the multilift construction simulation on soft ground introduces larger loads in the center of the embankment. This point makes a true comparison between stiff and soft foundation behavior difficult.

3.4 EFFECTS OF SIZE OF FINITE ELEMENT MESH

In order to verify that the results obtained are not significantly influenced by the coarseness of the mesh and the boundary conditions, two modifications of the original finite element mesh were analyzed. In one mesh, the embankment construction was simulated in six rather than three lifts. The other has the mesh extended almost twice as far to the left of the embankment. Comparisons are made for stress contours and ground surface settlements. Figure 3.47 shows the finite element mesh for the six lift construction case. Figure 3.48 shows the vertical stress contours for the three and six lift construction simulation. Stresses in the embankment are slightly reduced for the six lift case as compared to the three lift case. In the foundation, stresses have increased slightly under the center and reduced at the sides of the embankment.

The horizontal stresses (Fig. 3.49) are reduced in the embankment in the six lift case compared to the one for three lifts. For the foundation, the horizontal stresses are quite

FIGURE 3.47 Finite element mesh with six lift construction simulation on a soft foundation for undrained response. Also shown are the orientations and the relative magnitudes of the principal stresses.

FIGURE 3.48 Contours of vertical stress (psf) for three lift and six lift construction; undrained response ($v_F = 0.48$), soft foundation ($E_F = 3000$ psf).

FIGURE 3.49 Contours of horizontal stress (psf) for three lift and six lift construction; undrained response ($v_F = 0.48$), soft foundation ($E_F = 3000$ psf).

similar for the two cases.

The extended foundation mesh is shown in Fig. 3.50 and the stresses for this mesh and the regular finite element mesh are compared in Fig. 3.51 and 3.52. The two cases are quite similar, particularly underneath the embankment, which is the main area of interest in this study.

Figures 3.53 and 3.54 show the vertical and horizontal settlements for the regular mesh, the six lift mesh, and the extended foundation mesh. For the extended foundation mesh the settlements are significantly different to the left of the embankment edge, but all three models give essentially the same displacements at the embankment-foundation interface. Since the displacement in this location is crucial with respect to fabric interaction, it may be concluded that the regular mesh is sufficiently accurate for purposes of this research.

The extended foundation mesh was also examined for drained response, particularly because of the problem of an uneven horizontal deflection profile (see Figs. 3.33 and 3.34). The result is similar to that for the undrained case: settlements are quite different to the left of the toe of the embankment but very similar at the embankment-foundation interface (Fig. 3.55). The uneven deflection profile does not disappear. As before, we may conclude that the uneven deflection profile is due to numerical oscillation caused by the discontinuity in vertical loading. Further, the regular mesh is sufficiently accurate for investiga-

FIGURE 3.50 Extended finite element mesh for the soft foundation case with undrained response. Also shown are the orientations and relative magnitudes of the principal stresses.

FIGURE 3.51 Influence of left boundary on the vertical stress contours (psf) for three lift construction and undrained response ($v_F = 0.48$) of a soft foundation ($E_F = 3000$ psf).

FIGURE 3.52 Influence of left boundary on the horizontal stress contours (psf) for three lieft construction and undrained response ($v_F = 0.48$) of a soft foundation ($E_F = 3000$ psf).

FIGURE 3.53 Comparison of vertical displacements of ground surface for a soft foundation and undrained response.

FIGURE 3.54 Comparison of horizontal displacements of ground surface for a soft foundation and undrained response.

FIGURE 3.55 Horizontal displacement of ground surface for the extended mesh; soft foundation, drained response.

tion of fabric reinforcement effects.

3.5 INFLUENCE OF COMPACTION

Compaction is the process of densification of a soil mass through the application of transient forces. Holtz and Kovacs (1981) list several important engineering advantages of compaction, but two effects are important for this study: (1) compaction reduces future settlements by rearranging the soil aggregates to form a more compact mass; and (2) compaction increases the strength of the soil mass.

3.5.1 Compaction Simulation

To truly model the effects of compaction in a computer analysis, a soil model would be needed which is capable of modeling (1) densification, (2) increase in stiffness, (3) increase of strength, and (4) a build up of residual stresses. Several sophisticated soil models are currently in use which make it possible to approximately model each of these effects. They are mostly based on elasto-plastic material models, of which the Drucker-Prager (1952) model is an example (see Chapter 2).

(1) Densification would be accomplished if volumetric plastic (irrecoverable) strain takes place due to (a) increased hydrostatic (mean normal) stress, and (b) stress difference or shear.

If the so called cap model is used (Fig. 3.56), plastic

FIGURE 3.56 Elasto-plastic material model with yield and cap surfaces (adapted from Mizuno and Chen, 1980a).

volumetric strain occurs when the state of stress reaches the cap surface, at which point the soil starts to flow (Sandler, DiMaggio, and Baladi, 1976; Mizuno and Chen, 1980a and b). The cap subsequently expands to the current state of stress, so that unloading takes place elastically inside the cap.

(2) Increase in stiffness could also be accomplished with a cap model, where the moduli are functions of the volumetric plastic strain ϵ_v^P .

(3) Increase in strength, e.g. an increased ϕ angle with higher density, would require a shift in the yield surface as a function of the void ratio or density of the soil, which is again related to the plastic volumetric strain ϵ_v^P .

(4) The build up of residual horizontal stresses due to compaction can be illustrated by Fig. 3.57. Approximating the compaction impact by a moving line load P , the Boussinesq (linear elastic) solution gives increases in vertical stress,

$\Delta\sigma_z = 2P/\pi z$, directly beneath the load, where z is the depth. The increase in horizontal stress $\Delta\sigma_z$ is zero. For a granular soil, however, the horizontal stress σ_z can not take a value less than $\sigma_z = K_A \sigma_v$, where K_A is the active earth pressure coefficient and σ_z is the effective vertical stress. The shaded area in Fig. 3.57 shows the residual horizontal stresses that are created by the moving line load if in situ stresses are negligible. During loading, the horizontal stresses will take the value of $K_A \cdot \sigma_z$ or $\sigma_x^{\text{elastic}}$, whichever is the largest. During unload-

Figure 3.57 Residual horizontal stresses due to a moving line load.

ing, the horizontal stresses will decrease following the elastic curve. Hence the residual horizontal stress after the load has passed will be $\sigma_x^{res} = K_A \cdot \sigma_z - \sigma_x^{elastic}$ for $K_A \cdot \sigma_z > \sigma_x^{elastic}$, and $\sigma_x^{res} = 0$ otherwise. This compaction effect can be simulated directly by the Drucker-Prager elasto-plastic model, since it closely approximates the Mohr-Coulomb failure criteria. The minor principal stress has a lower bound of K_A times the major principal stress.

Extending the material response models in the program GEONON to include a cap model is possible, but it was considered to be beyond the scope of the present research. Also there is some doubt as to how well such models would perform, since it is recognized that the Drucker-Prager model produces numerical problems in certain large displacement problems (Mizuno and Chen, 1980a).

Consequently, it was not possible to simulate densification or increase in modulus or strength due to compaction, and initial values for these parameters were chosen to be compacted values rather than the values for the initially loosely placed fill. The compaction load has to be simulated by a line load in a two dimensional computer model. Therefore, a translation from an area load to a line load was necessary. The influence of compaction is sought at depths of 1 to 4 ft. Assuming compaction by a track type tractor with a track width of 1.67 ft, a unit contact pressure of 8 psi (1152 psf) and a track length 1 (contact length between tire and ground) of 6 in., an equivalent line load of $P =$

500 lb/ft was adopted. This value is based on a comparison of the elastic stress at equivalent depths under a rectangularly loaded area with that caused by a line load. For a rectangular area with dimensions $2 \times 2b$ of $20 \times 6 \text{ in}^2$, an influence factor at a depth of 1 ft was found to be $\sigma_z/(P/z) = 0.276$, whereas the influence factor for a line load is $\sigma_z/(P/z) = 2/\pi = 0.637$. Hence the equivalent line load $P_{\text{line}} \text{ (lb/ft)} = (0.276/0.637) \times P_{\text{area}} \text{ (psf)} = 0.433 P_{\text{area}}$. Hence, $P_{\text{line}} = 0.433 \times 1152 \text{ lb/ft} = 500 \text{ lb/ft}$. At a depth of 4 ft, the influence factor is $\sigma_z/(P/z) = 0.10$ and the conversion factor is $0.10/0.637 = 0.157$. When the pressure is matched for a depth of 1 ft it will be too large at greater depths. To truly match the actual pressures, one would have to place additional forces in the opposite direction at different depths. However, since the actual effect of the compaction equipment is not easy to assess, further complications in the simulation of compaction seems unjustified at this time.

3.5.2 Results of Compaction Simulation

Two basically different approaches were considered in the application of the compaction loads. First the compaction equivalent line loads were placed at each node at the surface of the embankment simultaneously, and then removed. This procedure produced only a marginal increase in the lateral stresses. The second approach was to load and unload the line loads sequentially, one node at a time. This method created a significant amount of residual horizontal stresses, and it is also the method that most realistically simulates the actual compaction sequence.

The results of these two procedures are illustrated in Fig. 3.58(b) and (c). In the upper right corner of the same figure is shown the finite element mesh used for analyzing compaction simulation of an embankment constructed on relatively incompressible ground (stiff foundation).

Figs. 3.59 through 3.62 show the influence of compaction of each 1 ft construction lift on the stresses in the bottom lift, for a total of four construction lifts. The stresses in the bottom layer are of particular interest with respect to the inclusion of fabric between embankment and foundation. For compaction of the first lift the horizontal stresses increase significantly and actually exceed the vertical stress in the central part of the embankment. As the construction proceeds, however, the influence of compaction of new lifts on the first lift diminishes as shown in Figs. 3.60 to 3.62. Before compaction of the first lift, it is in an elasto-plastic state. After compaction, and during all following loading, this lift remains in an elastic state.

For compaction simulation of an embankment on soft foundation the finite element mesh shown in Fig. 3.1 was used. Figures 3.63 to 3.65 show the influence of compaction of each 1.5 ft construction lift on the stresses in the bottom lift, for a total of three construction lifts. A striking change in behavior occurs, compared to the compaction simulation of the embankment on a stiff foundation. Not only does the horizontal stress change significantly, but so do the vertical normal and shear stresses.

Finite Element Mesh, Incremental Construction
With Compaction Simulation

Stress in first layer

(a) Due to own weight, $\gamma = 120$ pcf

(b)

(a) + uniform compaction simulation *)

(c)

(a) + sequential compaction simulation **)

*) Nodal boundary loads applied and removed simultaneously

**) Load applied at node 57 and removed; then applied at node 58 and removed, etc. (Node 56 not loaded since compaction is difficult at edge of embankment).

FIGURE 3.58 Results of uniform and sequential compaction simulation.

(a) Stresses Before Compaction (Mostly plastic state)

(b) Stresses After Compaction (Elastic)

$$1 \text{ ft} = 0.305 \text{ m}$$

$$1 \text{ psf} = 47.88 \text{ N/m}^2$$

FIGURE 3.59 Incremental construction with compaction simulation of the first layer. Stresses in first layer before and after compaction simulation. Embankment slope 1.5/1 on a stiff foundation, $\nu = 0.48$.

(a) Stress Before Compaction (Elastic)

(b) Stress After Compaction (Elastic)

1 ft = 0.305 m

1 psf = 47.88 N/m²

FIGURE 3.60 Incremental construction with compaction simulation of the second layer. Stresses in first (bottom) layer before and after compaction simulation. Embankment slope 1.5/1 on a stiff foundation, $\nu = 0.48$.

(a) Stresses Before Compaction (Elastic)

(b) Stresses After Compaction (Elastic)

$$1 \text{ ft} = 0.305 \text{ m}$$

$$1 \text{ psf} = 47.88 \text{ N/m}^2$$

FIGURE 3.61 Incremental construction with compaction simulation of the third layer. Stresses in first layer before and after compaction simulation. Embankment slope 1.5/1 on a stiff foundation, $\nu = 0.48$.

(a) Stresses Before Compaction (Elastic)

(b) Stresses After Compaction (Elastic)

FIGURE 3.62 Incremental construction with compaction simulation of the fourth layer. Stresses in first layer before and after compaction simulation. Embankment slope 1.5/1 on a stiff foundation, $\nu = 0.48$.

(a) Stresses Before Compaction (Plastic)

(b) Stresses After Compaction (Mostly elastic)

1 ft = 0.305 m

1 psf = 47.88 N/m²

FIGURE 3.63 Incremental construction with compaction simulation of the first layer. Stresses before and after compaction simulation. Soft foundation.

(a) Stresses Before Compaction (Mostly plastic)

(b) Stresses After Compaction (Elastic)

1 ft = 0.305 m

1 psf = 47.88 N/m²

FIGURE 3.64 Incremental construction with compaction simulation of the second layer. Stresses in first layer before and after compaction simulation. Soft foundation.

(a) Stresses Before Compaction (Plastic)

(b) Stresses After Compaction (Elastic)

$$1 \text{ ft} = 0.305 \text{ m}$$

$$1 \text{ psf} = 47.88 \text{ N/m}^2$$

FIGURE 3.65 Incremental construction with compaction simulation of the third layer. Stresses in first (bottom) layer before and after compaction simulation. Soft foundation.

For the embankment on the stiff foundation, all stresses increased upon further loading (placement of consecutive lifts). For the embankment on the soft foundation, the vertical stress keeps increasing with additional lifts and their compaction. However, the horizontal stresses fluctuate up and down, up due to the impact of compaction and down when a new lift is added. Furthermore, the increase in horizontal stresses after compaction exceeds by far the amount that can be explained due to the compaction load. Even directly underneath the load where $\Delta\sigma_h^{res}/(P/z)$ is a maximum (0.173 from Fig. 3.57), the maximum residual horizontal stress due to compaction would be only $\Delta\sigma_h^{res} = 0.5(0.173 \times 500/0.37 + 0.173 \times 500/1.12) = 156$ psf, and the total horizontal stress after compaction should not exceed $\sigma_h = \sigma_{h_0} + \Delta\sigma_h^{res} = 276 + 156 = 432$ psf. The actual value is 600 psf. Also the initial horizontal stress at 276 psf is very large. This is due to the fact that the embankment in the finite element analysis acts like a beam. If a linear elastic soil model was used for the embankment, we would have observed large tensile stresses at the lower integration points, and large compressive stresses at the upper integration points. The Drucker-Prager model prevents the large tensile stresses from developing, but not the large compressive stresses, which contribute to the stresses observed. The additional large horizontal large horizontal stresses which develop during compaction simulation are caused by elastic rebound from the large deformations that take place while the compactive load is being applied. This also explains why these stresses decrease again when additional

load is applied by the next construction lift [Figs. 3.64(a) and 3.65(a)]. Whether this behavior reflects actual material behavior or is a deficiency of the present model is subject to question.

Observe again the discrepancy between the vertical stresses in the embankment and those in the foundation. At the top of the foundation, the vertical stress is 181 psf, reflecting the 1.5 ft x 120 pcf load of the first construction lift, whereas the vertical stress in this lift is 248 psf. Since all stresses in the embankment are plastic, the large vertical stresses must somehow be related to the large horizontal and shear stresses (note that the vertical and horizontal stresses are not principal stresses at this construction stage). Again it is doubtful that these represent actual stresses in the embankment.

Figures 3.66 and 3.67 compare the ground surface settlements for a soft foundation with and without compaction simulation. The larger vertical and horizontal displacements that take place for the compacted embankment are primarily due to plastic yielding of the foundation, intensified by a thicker second and third construction lift due to the larger vertical deflection.

3.5.3 Concluding Remarks

Due to the elastic rebound from large deformations, compaction of an embankment on a soft foundation results in much larger horizontal stresses than compaction of an embankment on a stiff foundation. Because this behavior and the fact that the

1 ft = 0.305 m

1 psf = 47.88 N/m²

FIGURE 3.66 Influence of compaction simulation on vertical settlement of ground surface for an embankment on a soft foundation.

FIGURE 3.67 Influence of compaction simulation on horizontal settlement of ground surface for an embankment on a soft foundation.

sequential compaction simulation (loading and unloading one node at a time) produces a rather costly computational procedure when large settlements take place, it was decided that the influence of compaction, so far as reinforcement is concerned, could be judged by placing and removing of the live load of the final incrementally constructed embankment.

CHAPTER 4

INFLUENCE OF GEOTEXTILE REINFORCEMENT

The influence of geotextile reinforcement on the behavior of a highway embankment was studied using the embankment analyzed previously in Chapter 3. It will be recalled that soft as well as stiffer foundations were considered, as well as undrained and drained foundation response. The effect of multi stage construction and compaction was also investigated. The undrained case is usually appropriate for the immediate or end-of-construction response of an embankment on a soft cohesive foundation, while the drained response represents the long term stability condition. However, the drained strength may also represent end-of-construction response of a slightly overconsolidated clay, as pointed out by Tavenas (1979). Initially most natural clay deposits are more or less overconsolidated and have a relatively high coefficient of consolidation c_v . Hence a significant amount of dissipation of excess pore pressure (consolidation) takes place during construction loading until the preconsolidation pressure is reached. Thereafter, the clay is normally consolidated and responds to the remaining load in an undrained manner.

The finite element mesh for the embankment studied is shown in Fig. 4.1. Geometry and soil properties were similar to those used in Chapter 3. The embankment is assumed to be composed of granular fill with a slight amount of cohesion. The elastoplastic Drucker-Prager (1952) material model (see Chapter 2) is used for the embankment soil with a Young's modulus E of 600,000

FIGURE 4.1 Embankment geometry and finite element mesh for analysis of fabric-reinforced embankments. Both elements and nodes are numbered as shown.

psf (28,700 kPa) and a Poisson's ratio ν of 0.25. The strength parameters are $\phi'_E = 35^\circ$ and $c'_E = 25$ psf (1.2 kPa). For the undrained response of the foundation, a von Mises type elastoplastic material model (Chen and Saleeb, 1982) is used with a Poisson's ratio ν of 0.48, modulus E_F of 3000 psf, and an undrained shear strength $c_u = 150$ psf (72 kPa). For the drained foundation response, the Drucker-Prager (1952) model is used with a Poisson's ratio ν and a strength intercept of 150 psf (7.2 kPa). The foundation soil is assumed weightless (buoyant weight = 0). The friction angle ϕ'_F of 41.8° is chosen to match the Poisson's ratio of $\nu = 0.25$, based on the relationship $K_0 = \nu/(1-\nu) = 1 - \sin \phi'$ (Jaky, 1948), where K_0 is the coefficient of earth pressure at rest for zero lateral strain.

As discussed in Chapter 1, the fabric has at least two functions: (1) It acts as a separator between the soft foundation soil and the granular fill, preventing the two from intermixing; and (2) it provides a certain amount of tensile or membrane strength to the fabric-soil system. This is a reinforcing function similar to that of the steel bars in reinforced concrete. The separation function cannot be evaluated with the finite element method, because the very nature of the finite element mesh implies a complete separation between embankment and foundation. Anything less cannot be analyzed at present, even if in reality without the fabric separator, the soils would intermix because of small localized bearing capacity failures. These failures would occur either when the fill is placed or under traffic loads.

What can be modeled with a finite element analysis is the reinforcing effect of the fabric.

4.1 UNDRAINED RESPONSE OF FOUNDATION

Figure 4.2 shows the results of the analysis of the unreinforced embankment for undrained foundation response. Results are presented in terms of the orientation and relative magnitude of the principal stresses, the maximum shear stresses, and the maximum vertical settlements of the foundation. Also shown is the vertical differential settlement at the crown of the embankment δ .

The maximum vertical settlement of 0.53 ft (0.16 m) occurs at the centerline of the embankment, or at node 78 (see Fig. 4.1). The maximum horizontal displacement was 0.37 ft (0.11 m) and it occurred 3 ft (0.91 m) below the original ground surface, and close to the toe of the embankment slope (at node 47, Fig. 4.1, and marked with a small dot on Fig. 4.2). The ratio of maximum horizontal to maximum vertical displacement at the crown amounts to 0.70 while the differential settlement δ at the top of the embankment is 0.060 ft (0.018 m). Embankment settlement response is summarized in Table 4.1.

The direction and relative magnitude of the principal stresses are shown in Fig. 4.2 at the center of each element, with the vertical stress of element 61 at the centerline taken as a reference stress. To show the overall accuracy of the solu-

FIGURE 4.2 Response of the unreinforced embankment on an undrained foundation after the embankment dead load is activated. Orientation and relative magnitude of the principal stresses are shown. Numerical values within each element indicate the maximum shear stress in psf.

Table 4.1 Summary of Settlement and Deflection Response for Various Loading and Reinforcement Configurations; Undrained Foundation

Case	Figure	Maximum Vertical Settlement (ft)	Node	Maximum Horizontal Deflection (ft)	Node	Ratio of Maximum H to V	Maximum Differential Settlement (ft)
1. Unreinforced DL only	4.2	0.53	78	0.37	47	0.70	0.060
2. Unreinforced DL + LL	4.3	0.90	78	0.65	46	0.72	0.163
3. Reinforced with nonwoven geotextile; DL only	4.5	0.51	78	0.36	47	0.71	0.053
4. Same as Case 3, DL + LL	4.7	0.87	78	0.64	46	0.74	0.152
5. Same as Case 3, woven geotextile, DL only	4.9	0.50	78	0.35	46	0.70	0.050
6. Same as Case 4 woven geotextile, DL + LL	4.11	0.86	78	0.64	46	0.74	0.147
7. Same as Case 3, high modulus woven geotextile, DL only	4.14	0.49	78	0.35	46	0.71	0.044
8. Same as Case 4, high modulus woven geotextile, DL + LL	4.16	0.83	78	0.62	46	0.75	0.134

tion, the applied vertical stress at the centerline was $\gamma h = 540$ psf while the calculated stress in the foundation directly under the embankment was 550 psf, an error of less than 2%.

The numerical values in psf of the maximum shear stress, $\tau_{\max} = (\sigma_1 - \sigma_3)/2$, are also shown inside each element. The stresses for an element are calculated as the average of the stresses at the four integration points. When the maximum shear stress in any one element (or parts of an element) reaches the undrained shear strength c_u , the element becomes plastic. However, this did not occur, and the foundation remained completely elastic. Contour lines of vertical, horizontal and maximum shear stresses for this case were shown previously in Fig. 3.36 to 3.38, case (b).

4.1.1 Results for Live Load

Figure 4.3 shows the effect of adding a live load such as a dump truck. Two live loads of 2000 lb/ft were added as shown. This same loading scheme was used by Greenway and Bell (1976). However, the values they choose seem somewhat high compared to the compaction live load calculated in Chapter 3. Settlement response is summarized in Table 4.1. Maximum vertical settlement as well as the horizontal deflection increased significantly, and their ratio was also slightly increased. The differential settlement at the crown of the embankment increased about three times to 0.16 ft (0.05 m), and the foundation has started to yield (shaded elements in Fig. 4.3).

FIGURE 4.3 Response of the unreinforced embankment on an undrained foundation due to embankment dead load plus live load (dump truck). Orientation and relative magnitude of principal stresses are shown. Numerical values within each element indicate the maximum shear stress in psf.

If an additional 2 x 250 lb/ft live load is added, the zones of yielding expand as shown in Fig. 4.4. A somewhat well defined "active" failure zone has developed, whereas the "passive" failure zone has not yet formed. These results agree well with observations of the actual progress of failure in model studies performed at the Swedish Geotechnical Institute. Belfrage and Eriksson (1980) and Belfrage (1981) reported that the "active" portion of the failure surface developed first, and with additional loading the "passive" portion also developed. The total load carried by the embankment in Figs. 4.3 and 4.4 was compared with the ultimate load in bearing capacity, $q_{ult} = (\pi + 2) c_u = 5.14 c_u$ of the foundation. It was found that the load slightly exceeds the ultimate bearing capacity. For example, in Fig. 4.3, the average applied vertical stress was 805 psf. So $q_{avg}/c_u = 5.30$, which is slightly greater than the theoretical maximum of 5.14. This result is reasonable since the bearing capacity equation assumes vertical load only, with no boundary shear forces and no resistance mobilized in the embankment. Both assumptions are on the conservative side with respect to estimating the ultimate embankment load.

4.1.2 Results with Geotextile Inclusion

Figure 4.5 shows the result when one layer of fabric reinforcement is placed between the embankment and the foundation. The geotextile is assumed to be linearly elastic with a stiffness EA of 6000 lb/ft (8 kN/m), where E is the Young's modulus of the fabric and A is the cross sectional area for a unit (1 ft) width

FIGURE 4.4 Expanded zones of yielding upon further loading. Same embankment as in Fig. 4.3.

FIGURE 4.5 Response of the embankment on an undrained foundation reinforced with a medium weight nonwoven geotextile, dead load only. Orientation and relative magnitude of principal stresses are shown. Numerical values within each element indicate the maximum shear stress in psf.

of the fabric. This value of EA represents a medium weight nonwoven geotextile. Maximum settlements (Table 4.1) are only slightly reduced by the reinforcement, about 3%. A larger reduction, about 12%, occurred in the differential settlements at the crown of the embankment.

The percentage reduction in the maximum shear stress for each element in the foundation due to the fabric inclusion is shown in Fig. 4.6. These values were obtained by comparing $\tau_{\max}$ values in Figs. 4.2 and 4.5, and calculating the percent change. The influence of the reinforcement is seen to be largest close to the fabric layer and close to the centerline of the embankment. It diminishes with depth and towards the side slopes of the embankment. Some elements immediately under the toe of the embankment actually experience an increase in maximum shear stress. This is indicated by a minus sign in the figure.

Figures 4.7 and 4.8 show the effect of adding live load. Comparing Figs. 4.3 and 4.7, it will be seen that the zone of yielding has been decreased due to the fabric reinforcement.

4.1.3 Influence of Increasing Fabric Stiffness

A stiffness of $EA = 6000 \text{ lb/ft}$ (88 kN/m) is representative of typical medium weight nonwoven geotextiles (Koerner and Welsh, 1980). Figures 4.9 to 4.12 give the response of the embankment reinforced with geotextile of stiffness $EA = 10000 \text{ lb/ft}$ (146 kN/m), which corresponds to a medium weight woven fabric (Koerner and Welsh, 1980). These moduli were obtained from testing the

FIGURE 4.6 Percent reduction in maximum shear stresses due to the geotextile reinforcement. Minus sign indicates an increase. Compare Figs. 4.2 and 4.5.

FIGURE 4.7 Same embankment and reinforcement as Fig. 4.5, but with live load added.

FIGURE 4.8 Percent reduction in maximum shear stresses due to geotextile reinforcement. Minus sign indicates an increase. Compare Figs. 4.3 and 4.7.

FIGURE 4.9 Same embankment and foundation as Figs. 4.2 and 4.5, but reinforced with a woven geotextile; dead load only.

FIGURE 4.10 Percent reduction in maximum shear stresses due to the geotextile reinforcement. Minus sign indicates an increase. Compare Figs. 4.2 and 4.9.

FIGURE 4.11 Same embankment and reinforcement as Fig. 4.9, but with live load added.

FIGURE 4.12 Percent reduction in maximum shear stresses due to the geotextile reinforcement. Minus sign indicates an increase. Compare Figs. 4.3 and 4.11.

fabric in isolation in an ordinary strip tensile test (ASTM method, D-1682). However, it has been observed by McGown, et al (1981) and McGown, Andrawes, and Kabir (1982) that the fabric confined in the soil has a much stiffer response than the same fabric tested in isolation. Pullout tests on a woven geotextile performed at Purdue (Salomone, 1978), showed an increase in stiffness of two to three times the values measured in isolation. Figure 4.13 shows results of a strip tensile test of a medium weight nonwoven confined in sand under various confining pressures (McGown, et al. 1981). For a confining pressure of 540 psf (26 kPa) corresponding to the height of the embankment, the modulus is almost doubled. Figures 7.14 to 7.17 show the results of the analyses similar to the previous ones with the geotextile modulus doubled to 20000 lb/ft (300 kN/m) to account for confinement. Settlement response for the various cases is summarized in Table 4.1.

4.1.4 Results after Unloading of Live Load

As stated in Chapter 3, the influence of compaction might be approximated by the loading and unloading of a live load on a soft foundation. Figure 4.18 shows the results for just such an analysis for the embankment without reinforcement, and Fig. 4.19 shows them with one layer of fabric of stiffness $EA = 20000 \text{ lb/ft}$ (300 kN/m) per unit width. Settlement and deflection response is summarized in Table 4.2. The percent reduction in maximum shear stresses due to the fabric inclusion is illustrated in Fig. 4.20. For most of the elements there is a slightly greater reduction of

FIGURE 4.13 Stress-strain relationships for a nonwoven fabric in isolation and confined in sand (McGown, et al., 1981).

FIGURE 4.14 Same embankment and foundation as Figs. 4.2 and 4.9, but reinforced with a high modulus woven geotextile; dead load only.

FIGURE 4.15 Percent reduction in maximum shear stresses due to the geotextile reinforcement. Minus sign indicates an increase. Compare Figs. 4.2 and 4.14.

FIGURE 4.17 Percent reduction in maximum shear stresses due to the geotextile reinforcement. Minus sign indicates an increase. Compare Figs. 4.3 and 4.16.

$$\sigma_V = 579 \text{ PSF}$$

FIGURE 4.18 Response of the unreinforced embankment after the live load was removed; undrained foundation. Compare with Fig. 4.3.

FIGURE 4.19 Response of the embankment reinforced with a high modulus woven geotextile after removal of the live load; undrained foundation. Compare with Fig. 4.16.

Table 4.2 Summary of Settlement and Deflection
Response for Unloading Live Load;
Undrained Foundation

Case	Figure	Maximum Vertical Settlement (ft)	Node	Maximum Horizontal Deflection (ft)	Node	Ratio of Maximum H to V	Maximum Differential Settlement (ft)
1. Unreinforced, DL only	4.18	0.64	78	0.45	47	0.70	0.134
2. Reinforced with high modulus woven geotextile	4.19	0.58	78	0.41	47	0.71	0.105

FIGURE 4.20 Percent reduction in maximum shear stress due to the geotextile reinforcement. Minus sign indicates an increase. Compare with Fig. 4.15.

maximum shear stresses as compared to Fig. 4.15, which represents the case before live load is added. For the elements under the toe of the embankment a significant reduction in maximum shear stress occurs for the case with geotextile reinforcement. This is in contrast to the cases prior to the removal of live load, where the inclusion of the geotextile produced an increase in maximum shear stresses for these elements.

The fabric appears to act in two ways: (1) as a membrane, reducing vertical stresses under the center of the embankment and increasing them towards the sides; hence, a more uniform load transfer results; and (2) as a restrictor of lateral movements, with the result that the horizontal stresses are increased under the embankment but decreased to the sides of the embankment. The combined result of these two actions is, in most cases, a reduction in maximum shear stresses in the foundation. However, directly under the toe, vertical stresses are increased and horizontal stresses are reduced; hence an increase in maximum shear stresses results. Why this pattern should change after loading and unloading of a live load is not understood, and it may reflect an inadequacy of the analytical model (the "Updated Geometry" model). To investigate this point further, the vertical and horizontal displacements were plotted in Figs. 4.21 and 4.22 for all three cases: (i) embankment load, (ii) live load, and (iii) removal of live load for the unreinforced embankment. As would be expected, the settlements increase due to live load, but a rebound upon unloading is observed around the embankment

1 ft = 0.305 m

1 lb/ft = 14.59 N/m

1 psf = 47.88 N/m²

FIG. 4.21 Vertical displacements of ground surface on a soft foundation, undrained response, for the three cases shown; unreinforced embankment.

FIG. 4.22 Horizontal displacements of ground surface, soft foundation, undrained response, for the three cases shown; unreinforced embankment.

toe that exceeds the deflection which occurred due to the live load. This behavior is not logical and for this reason the values of maximum shear stresses for the elements directly under the toe for the unload of live load case are doubtful.

The vertical and horizontal displacements for the embankment dead load with and without fabric reinforcement are shown in Figs. 4.23 and 4.24. As happens with the stresses, the vertical displacement is reduced under the center of the embankment and increased under the slope. The horizontal displacement is reduced at all points due to the inclusion of the geotextile reinforcement.

The tension in the geotextile for the three fabric stiffnesses analyzed is shown in Fig. 4.25(a) for embankment dead load only and in Fig. 4.25(b) for embankment dead load plus live load. The tension in the fabric increases with increasing stiffness, as would be expected. The stiffer fabric also gives a larger reduction in maximum shear stresses, as shown in Fig. 4.26 for two elements in the foundation: Element No. 7, which has the highest maximum shear stress, and Element No. 61 which shows the greatest percent reduction in maximum shear stress due to the presence of the geotextile reinforcement. For an explanation of the differences in integration order, see Appendix D.

4.1.5 The No-Slippage Criteria

All of the above results assume there is no slippage between the geotextile and the soil. The validity of this assumption can

1 ft = 0.305 m

1 psf = 47.88 N/m²

FIG. 4.23 Vertical displacement of ground surface for embankment dead load with and without geotextile reinforcement, soft foundation, undrained response.

FIG. 4.24 Horizontal displacement of ground surface for embankment dead load with and without geotextile reinforcement, soft foundation, undrained response.

FIG. 4.25 Tension in the geotextile for the undrained response of the foundation. (a) Embankment dead load, and (b) embankment dead load plus live load. Fabric stiffnesses: Curve (1) $EA = 6,000$ lb/ft; Curve (2) $EA = 10,000$ lb/ft; and Curve (3) $EA = 20,000$ lb/ft.

- Element No. 7— Embankment has 2nd order integration
- ⊗ Element No. 7— Embankment has 4th order integration
- △ Element No. 61— Embankment has 2nd order integration
- ⊗△ Element No. 61— Embankment has 4th order integration

FIG. 4.26 Reduction in maximum shear stress as a function of geotextile stiffness for elements No. 7 and No. 61; undrained response of the foundation.

be checked by comparing the maximum force transferred between soil and geotextile with the maximum force that can be transferred by friction between geotextile and soil.

Laboratory pullout tests have indicated that the friction in pull-out between a woven fabric and sand is very close to the triaxial friction angle of the sand alone (Holtz, 1973 and 1977). Direct shear type tests conducted by Haliburton, Anglin, and Lawmaster (1978a and b), Collios, et al. (1980), Andrawes, McGown, and Wilson-Fahmy (1980), and Myles (1982) showed that the geotextile-sand friction was about the same as for the sand alone for loose sands but slightly less for dense sands. How much less depended on the surface characteristics of the fabric and the size of the soil grains relative to the size of the "holes" in the geotextile. Typical values were in the range of 90-95% of ϕ_{sand} for dense sands in contact with common geotextiles. Very few tests have been carried out with cohesive soils and geotextiles, but those that have (Collios, et al., 1980; Ingold, 1981; and Ingold and Miller, 1982) indicate relatively high adhesion factors between common geotextiles and clay in undrained shear. When the geotextile is placed at the interface between a granular embankment and a cohesive foundation, as is the case for our embankment, the strain field is very complex and difficult to analyze. For design in the absence of laboratory tests, Bell (1980) conservatively recommends a coefficient of friction with cohesive backfills of $1/3 \phi$ (of the granular material) with clay on one side of the geotextile. With all-clay backfills, he

recommends an adhesion factor of two-thirds.

For a reinforced embankment, the largest force transmitted from the foundation soil and embankment to the geotextile occurs where the rate of change in fabric tension is the largest, i.e., where the slope of the tension-distance curve is the greatest. As can be seen in Fig. 4.25(b), the greatest slope is for Curve 3 ("stiffest fabric and live load") near the toe of the embankment, and it has a value of approximately 200 lb/ft over the first 0.5 ft from the toe. Because of arching in the embankment on soft ground, the vertical stresses are somewhat higher in the region of the toe than they are in an embankment on a stiffer foundation. Thus, substantial frictional forces can be mobilized in this region. The frictional force is equal to $580 \text{ psf} \times \tan 35^\circ \times 0.5 \text{ ft} \times 2 \text{ sides} = 406 \text{ lb/ft}$. (The vertical stress in the region of the toe is 580 psf, as determined by the finite element analysis for this case, stiffest fabric and live load). The factor of safety against slippage equals the available friction force divided by the maximum local fabric force, or $406/200$. Hence the minimum factor of safety against slippage of the fabric is about 2. Thus our assumption of no slippage between soil and geotextile is quite satisfactory.

4.1.6 Higher Gaussian Integration Scheme

Due to the nonrectangular shape of the embankment elements, it was possible that a higher Gaussian integration order for these elements might improve the results. A general study of

this question is reported in Appendix D. It was found that a higher Gaussian integration order for the embankment elements significantly changed the stresses in the embankment itself, but it did not change any conclusions about the influence of fabric. The largest change in terms of fabric influence was found for the undrained case.

Figure 4.27 shows the results of the undrained response with 4th order integration for the embankment elements (compare with Fig. 4.2). Figures 4.28 and 4.29 show the results when one layer of fabric with $EA = 20000 \text{ lb/ft}$ (300 kN/m) is placed between embankment and foundation. Figures 4.28 and 4.29 can be compared to Figs. 4.14 and 4.15, in which 2nd order integration was used, to assess the influence of 4th order versus 2nd order integration. For two foundation elements, Nos. 7 and 61, 2nd versus 4th order results are also shown in Fig. 4.26.

Settlement response is summarized in Table 4.3. These values can be compared with those in Table 4.1.

4.1.7 Influence of Multiple Layers of Reinforcement

The influence of multiple layers of geotextiles placed higher up in the embankment was studied for the undrained response of the foundation. Figures 4.30 and 4.31 and Table 4.3 show the results of placing three reinforcement layers in the embankment, with 4th order integration for the embankment elements. With just one layer of fabric placed between embankment and foundation, the maximum reduction in shear stress in the

FIGURE 4.27 Same embankment and foundation as Fig. 4.2; dead load only, undrained foundation, but with 4th order Gaussian integration.

FIGURE 4.28 Same embankment and foundation as Fig. 4.27 but reinforced with a high modulus woven geotextile; dead load only; 4th order Gaussian integration.

FIGURE 4.29 Percent reduction in maximum shear stresses due to the geotextile reinforcement. Minus sign indicates an increase. Compare Figs. 4.27 and 4.28.

Table 4.3 Summary of Settlement and Deflection Response for Embankment Loading; Unreinforced and Reinforced; Undrained Soft($E_F = 3000$ psf) Foundation, and 4th Order Integration

Case	Figure	Maximum Vertical Settlement (ft)	Node	Maximum Horizontal Deflection (ft)	Node	Ratio of Maximum Settlement H to V	Maximum Differential Settlement (ft)
1. Unreinforced DL only	4.27	0.52	78	0.37	47	0.71	0.060
2. Reinforced with one layer of high modulus woven geotextile	4.28	0.49	78	0.35	46	0.71	0.048
3. Reinforced with three layers of high modulus woven geotextile	4.30	0.48	78	0.34	46	0.71	0.044
4. Reinforced with two layers of high modulus woven or one layer of very high modulus geotextile	4.32	0.47	78	0.34	46	0.72	0.040
5. Reinforced with inter- face nodes fixed horizontally	4.36	0.31	78	0.27	45	0.87	0.004

FIGURE 4.30 Same embankment and foundation as Fig. 4.28 except with three layers of high modulus woven geotextile; 4th order integration.

FIGURE 4.31 Percent reduction in maximum shear stresses due to three layers of high modulus woven geotextile reinforcement in the embankment. Minus sign indicates an increase. Compare Figs. 4.27 and 4.30.

foundation was 11% (Fig. 4.29) and this occurred under the centerline. With three layers placed between the consecutive construction lifts, the reduction at the same spot amounted to 13% (Fig. 4.31).

If, on the other hand, two layers of fabric are placed between embankment and foundation (Fig. 4.32), an almost 18% reduction in maximum shear stresses results (Fig. 4.33). The same effect can be obtained with and in fact was modeled by one layer of reinforcement having twice the modulus. Hence, to produce reduction of shear stresses in the foundation, it is more beneficial to place more fabric layers at the bottom of the embankment, or use higher modulus fabrics, than it is to place additional reinforcement layers higher up in the embankment. Figures 4.34 and 4.35 illustrate the reduction in maximum shear stresses as a function of depth and distance from the centerline of the embankment for both one and three layers of geotextile reinforcement with stiffnesses as shown.

However, if adverse orientation of stresses develop in the embankment due to arching, as discussed in Chapter 3 (Trollope, 1957), layers of fabric placed up in the embankment could improve the stability of the embankment itself. Horizontal reinforcement layers may not produce the optimum influence on embankment stability (McGown, et al, 1981), although inclined inclusions are probably impractical to construct in the field.

FIGURE 4.33 Percent reduction in maximum shear stresses due to two reinforcement layers or one layer of a very high modulus geotextile between embankment and foundation. Compare Figs. 4.27 and 4.32.

FIG. 4.34 Reduction in maximum shear stresses due to geotextile reinforcement as a function of depth for foundation elements located under the centerline of the embankment; undrained response of a soft foundation.

FIG. 4.35 Reduction in maximum shear stresses due to the geotextile reinforcement as a function of the distance from the centerline of the embankment for elements at the bottom of the foundation (where highest maximum shear stresses develop); undrained response of a soft foundation.

The inclusion of a layer of fabric between embankment and foundation does not change the stress directions significantly. The same is true for more layers of fabric placed up in the embankment (Table 4.4). The stresses in the embankment remain on the yield envelope, although some change in them does take place. Horizontal stresses are in general increased, and vertical stresses are reduced. Mean and deviatoric stresses are mostly reduced.

The effect of the fabric on local embankment stability is difficult to assess from the finite element analysis. However, layers of fabric placed near the slope edges, with every lift of the fill, make it possible to improve compaction near the shoulders of the embankment. This will result in a more uniform compaction throughout the embankment. Such a method has been successfully employed in the construction of Japanese railroad embankments, where plastic nets were used as reinforcement (Iwasaki and Watanabe, 1978). It would also be reasonable to assume that the fabric could improve the local stability of the arching embankment when placed to intersect the probable failure plane (see Figure 3.45). McGown, et al. (1981) also discuss this point in some detail.

4.1.8 Infinitely Stiff Reinforcement

Chirapuntu and Duncan (1976) analyzed the Wager method of embankment reinforcement by short sheet piles and horizontal tie rods described in Chapter 1 (Wager and Holtz, 1976). They

Table 4.4 Comparison of Major Principal Stress Directions in the Embankment, in Degrees Counterclockwise with Respect to Vertical.

Layer	Element	Stiff Foundation	Soft Foundation		
			No reinforcement	1 layer	3 layers
1	1	-28.2	-1.3	-6.5	-4.3
	2	-14.2	-2.6	-4.5	-2.8
	3	-7.9	-0.6	-1.4	-0.2
	4	-4.0	-0.4	-0.7	0.1
	5	-1.6	0.1	-0.4	0.1
	6	-0.4	0.3	0	0.4
	7	-0.1	0	0.2	0.3
2	1	-22.1	-37.5	-41	-42.2
	2	-8.9	-14.5	-22.1	-27.2
	3	-4.1	-7.8	-11.5	-14.3
	4	-1.6	-5.2	-7.3	-8.6
	5	-0.2	-3.5	-4.7	-6.9
	6	0.2	-2.7	-2.6	-2.7
	7	0.1	-2.0	-0.8	-0.8
3	1	-29.5	-55.1	-54.9	-55.0
	2	-7.7	-67.9	-68.1	-68.6
	3	-2.8	-76.1	-76.3	-76.8
	4	1.5	-81.1	-81.0	-81.6
	5	1.8	-85	-84.3	-90.0
	6	1.3	88	-87.0	-82.0
	7	0.4	86.3	-89.0	-89.3

assumed the rods to be infinitely stiff by fixing all the points at the base of the embankment. To compare the results of this assumption with the results of geotextile inclusions, a calculation was made whereby the nodes at the interface between the embankment and the foundation were fixed horizontally. The results of this analysis are shown in Figs. 4.36 and 4.37. The trends are the same as for the geotextile inclusions, namely, a reduction in shear stresses in most of the elements but an extremely large increase in the shear stresses under the toe of the embankment. In addition, there was a significant decrease in vertical settlement, and the differential settlement was almost non-existent (Table 4.3). However, the changes appear to be tremendously exaggerated, and they are probably not realistic for geotextiles. The forces which would develop at the fabric-soil interface would exceed those which could be transferred by frictional resistance between the soil and the fabric. However, it is possible that a Wager-type reinforcement system with steel tie rods and anchor plates might produce results close to those obtained by the fixed-node analysis. It is unfortunate that Chirapuntu and Duncan (1976) did not present their results in a manner similar to Figs. 4.36 and 4.37 so that direct comparison would be possible.

Tables 4.5 and 4.6 summarize the results obtained thus far for the elements that experience the largest maximum shear stresses. See Fig. 4.1 for the location of those elements.

FIGURE 4.36 Same embankment and foundation as Fig. 4.2 and 4.27, but with nodes at foundation-embankment interface fixed horizontally; 4th order integration.

FIGURE 4.37 Percent reduction in maximum shear stresses due to horizontal fixity of interface nodes. Minus sign indicates an increase. Compare Figs. 4.36 and 4.27.

Table 4.5 Comparison of maximum shear stresses (in psf) for certain elements experiencing the highest maximum shear stress values; undrained foundation conditions, $E_f = 3000$ psf, $\nu_f = 0.48$; dead load of embankment only. Embankment elements have 2nd order integration. Percent reduction is with respect to the unreinforced embankment case.

Element number	Embankment without reinforcement	Embankment with one layer of geotextile reinforcement for different geotextile moduli					
		EA = 6000 lb/ft	Percent reduction	EA = 10000 lb/ft	Percent reduction	EA = 20000 lb/ft	Percent reduction
Fig. 4.1	Fig. 4.2	Fig. 4.5	Fig. 4.6	Fig. 4.9	Fig. 4.10	Fig. 4.14	Fig. 4.15
5	83.5	83.0	0.6	82.7	1.0	82.1	1.7
6	94.3	93.5	0.8	93.0	1.4	92.1	2.3
7	96.2	94.9	1.4	94.1	2.2	92.6	3.7
8	92.5	90.9	1.7	89.9	2.8	88.0	4.9
9	84.6	83.0	1.9	81.9	3.2	80.0	5.4
10	74.3	72.7	2.2	71.7	3.5	70.0	5.8
20	70.8	69.4	2.0	68.5	3.2	66.9	5.5
21	76.4	74.8	2.1	73.8	3.4	72.0	5.8
22	81.6	79.8	2.2	78.6	3.7	76.6	6.1
23	85.4	83.4	2.3	82.1	3.9	79.9	6.4
24	87.4	85.2	2.5	83.9	4.0	81.6	6.6
34	77.2	75.4	2.3	74.2	3.9	72.1	6.6
35	87.9	85.6	2.6	84.2	4.2	81.6	7.2
36	95.4	92.7	2.8	91.0	4.6	88.1	7.7
37	99.1	96.2	2.9	94.5	4.6	91.4	7.8
46	81.0	78.5	3.1	76.9	5.1	74.1	8.5
47	88.6	85.6	3.4	83.7	5.5	80.5	9.1
48	92.4	89.0	3.7	87.1	5.7	83.5	9.6
61	69.5	65.9	5.2	63.9	8.1	60.5	12.9

Table 4.6 Comparison of maximum shear stresses (in psf) for certain elements experiencing the highest maximum shear stress values; undrained foundation conditions, $E_F = 3000$ psf, $v_F = 0.48$; dead load of embankment only. Embankment elements have 4th order integration. Percent reduction is with respect to the unreinforced embankment case.

Element number	Embankment without reinforcement	Embankment with one layer of geotextile reinforcement at base of embankment for two geotextile moduli			
		EA = 20000 lb/ft	Percent reduction	EA = 40000 lb/ft	Percent reduction
Fig. 4.1	Fig. 4.27	Fig. 4.28	Fig. 4.29	Fig. 4.32	Fig. 4.33
5	83.3	82.1	1.4	81.4	2.3
6	94.0	92.1	2.0	90.9	3.3
7	95.7	92.6	3.2	90.6	5.3
8	91.8	88.0	4.1	85.6	6.8
9	83.9	80.0	4.6	77.5	7.6
10	73.6	70.0	4.9	67.6	8.2
20	70.3	67.0	4.7	65.0	7.5
21	75.8	72.0	5.0	69.7	8.0
22	80.9	76.6	5.3	73.9	8.7
23	84.6	80.0	5.4	77.0	9.0
24	86.5	81.7	5.5	78.5	9.2
34	76.6	72.2	5.7	69.6	9.1
35	87.1	81.7	6.2	78.4	10.0
36	94.4	88.3	6.5	84.4	10.6
37	98.1	91.6	6.6	87.4	10.9
46	80.3	74.3	7.5	70.7	12.0
47	87.6	80.7	8.9	76.4	12.8
48	91.2	84.0	7.9	79.3	13.0
61	68.4	60.9	11.0	56.3	17.7

Table 4.6 (Continued)

Element number	Three layers of geotextile reinforce- ment		Nodes at embankment- foundation interface fixed horizontally	
	EA = 20000 lb/ft	Percent reduction	EA $\rightarrow \infty$	Percent reduction*
Fig. 4.1	Fig. 4.30	Fig. 4.31	Fig. 4.36	Fig. 4.37
5	81.9	1.7	76.8	7.8
6	91.8	2.3	83.3	11.4
7	92.1	3.8	77.4	19.1
8	87.4	4.8	68.9	24.9
9	79.4	5.4	59.7	28.8
10	69.4	5.7	50.7	31.1
20	66.4	5.5	54.4	22.6
21	71.4	5.8	54.5	21.3
22	75.9	6.2	54.7	32.4
23	79.2	6.4	54.8	35.2
24	80.9	6.5	54.9	36.5
34	71.5	6.7	54.8	28.5
35	80.8	7.2	56.0	35.7
36	87.3	7.5	56.8	39.8
37	90.5	7.7	57.3	41.6
46	73.3	8.7	48.4	39.7
47	79.5	9.2	47.6	45.7
48	82.6	9.4	47.1	48.4
61	59.5	13.0	26.9	60.7

*Note that maximum shear stresses under the toe of the embankment are tremendously increased; see Fig. 4.37 and compare Fig. 4.36 with 4.27.

4.1.9 Stiffer Foundation

Figures 4.38 to 4.40 show the results for undrained embankment loading on a stiffer foundation. The modulus of the foundation was increased a factor of ten to 30000 psf. In this case the fabric reinforcement reduced the maximum shear stresses throughout the foundation including below the toe. As a matter of fact, the largest percentage reduction in maximum shear stress occurred below the toe, rather than under the centerline of the embankment, as was the case for the softer foundation. Figures 4.41 and 4.42 show the reduction in maximum shear stresses due to the reinforcement for the elements at the bottom and the top of the stiff foundation, and these should be compared with Figs. 4.43 and 4.35, which show the same thing for the soft foundation. The reduction in maximum shear stresses at the top of the stiffer foundation (Fig. 4.42) is about 4% under the central part of the embankment, and about 10% under the toe of the embankment. The corresponding numbers for the soft foundation (Fig. 4.43) are a 12% reduction under the central part, and an increase under the toe. For the bottom of the stiffer foundation (Fig. 4.41) the reduction in maximum shear stress is on the order of 0.5%, compared to about 5% for the soft foundation (Fig. 4.35).

Settlement response, summarized in Table 4.7, was the same for both cases.

FIGURE 4.38 Same embankment as Fig. 4.2, but on a stiff foundation ($E_F = 30,000 \text{ psf}$; $\nu_F = 0.48$); undrained loading, dead load only.

FIGURE 4.40 Percent reduction in maximum shear stresses due to the geotextile reinforcement. Minus sign indicates an increase. Compare Figs. 4.38 and 4.39.

1 ft = 0.305 m

1 psf = 47.88 N/m²

FIG. 4.41 Reduction in maximum shear stresses due to the geotextile reinforcement as a function of the distance from the centerline of the embankment for elements at the bottom of the foundation; undrained response of a stiff foundation; high modulus ($EA = 20000$ lb/ft) woven geotextile.

1 ft = 0.305 m

1 psf = 47.88 N/m²

FIG. 4.42 Reduction in maximum shear stresses due to the geotextile reinforcement as a function of the distance from the centerline of the embankment for elements at the top of the foundation; undrained response of a stiff foundation; high modulus ($EA = 20000$ lb/ft) woven geotextile.

1 ft = 0.305 m

1 psf = 47.88 N/m²

FIG. 4.43 Reduction in maximum shear stresses due to the geotextile reinforcement as a function of the distance from the centerline of the embankment for elements at the top of the foundation; undrained response of a soft foundation; high modulus ($EA = 20000$ lb/ft) woven geotextile.

Table 4.7 Summary of Settlement and Deflection
Response for Embankment Loading,
Unreinforced and Reinforced, and
Undrained Stiff ($E_F = 30000$ psf)
Foundation

Case	Figure	Maximum Vertical Settlement (ft)	Node	Maximum Horizontal Deflection (ft)	Node	Ratio of Maximum H to V	Maximum Differential Settlement (ft)
1. Unreinforced, DL only	4.38	0.06	78	0.04	47	0.67	0.010
2. Reinforced with one layer of high modulus woven geotextile	4.39	0.06	78	0.04	47	0.67	0.010

4.1.10 Results for Other Mesh Configurations

Figure 4.44 shows the maximum shear stresses in the soft foundation for the embankment constructed in six lifts instead of three, as was shown in Fig. 4.2. Fig. 4.45 shows the response for the three-lift embankment on a soft foundation, but with an extended foundation mesh (not shown in the figure beyond 27 ft). See Fig. 3.50 for an example of the extended mesh. This result should also be compared to Fig. 4.2. The response for six-lift construction does not differ much from three-lift construction, as was already demonstrated by the stress contour lines of Fig. 3.48 and 3.49. The results with the extended foundation (Figs. 4.45 to 4.47) mesh do vary significantly to the left of the embankment and to some extent under the toe; but under the center of the embankment, the maximum shear stresses are very similar. The results to the left of the embankment do not significantly affect the influence of the geotextile inclusions. Figs. 4.48 to 4.50 show the reduction in maximum shear stresses at various places in the foundation for the regular mesh, the extended foundation mesh, and the extended foundation mesh with six-lift construction. None of these cases deviate much from each other. The largest difference occurs at the top of the foundation for the extended mesh with six-lift construction. The results for this case are shown in Figs. 4.51 to 4.53.

FIGURE 4.44 Response of the unreinforced embankment on a soft foundation ($E_F = 3000$ psf; $\nu_F = 0.48$), undrained loading, six lift construction.

FIGURE 4.45 Same unreinforced embankment and soft foundation as Fig. 4.2, but with the foundation mesh extended to the left (this part not shown).

FIGURE 4.46 Same embankment with an extended foundation mesh as Fig. 4.45 but reinforced with one layer of a high modulus woven geotextile.

FIGURE 4.47 Percent reduction in maximum shear stresses due to the reinforcement. Minus sign indicates an increase. Compare Figs. 4.45 and 4.46.

FIG. 4.48 Reduction in maximum shear stresses due to the geotextile reinforcement as a function of the depth for foundation elements located under the centerline of the embankment; undrained response of a soft foundation; regular and extended foundation meshes; high modulus ($EA = 20000 \text{ lb/ft}$) woven geotextile.

FIG. 4.49 Reduction in maximum shear stresses due to the geotextile reinforcement as a function of distance from the centerline of the embankment for elements at the top of the foundation; undrained response of a soft foundation; regular and extended foundation meshes.

FIG. 4.50 Reduction in maximum shear stresses due to the geotextile reinforcement as a function of distance from the center-line of the embankment for elements at the bottom of the foundation; undrained response of a soft foundation; regular and extended foundation meshes; high modulus ($EA = 20000$ lb/ft) woven geotextile.

FIGURE 4.51 Same embankment (six-lift construction) on an extended foundation mesh as in Figs. 4.44 and 4.45; dead load only.

FIGURE 4.52 Same embankment and foundation as Fig. 4.51 but reinforced with one layer of a high modulus woven geotextile.

FIGURE 4.53 Percent reduction in maximum shear stress due to the reinforcement. Minus sign indicates an increase. Compare Figs. 4.51 and 4.52.

4.2 DRAINED RESPONSE OF FOUNDATION

Figure 4.54 shows the results of the analysis of the embankment on a soft foundation ($E_F = 3000$ psf) but with drained responses ($\nu_F = 0.25$). Results are shown in terms of principal stresses and an octahedral shear stress ratio for each element. The octahedral shear stress ratio is defined (Chapter 3) as τ_{oct}/τ_{oct_f} where τ_{oct} is the octahedral shear stress of the element (average over the integration points) and τ_{oct_f} is the value of τ_{oct} at failure. The octahedral normal stress (mean normal stress) is assumed to remain constant. The maximum vertical and horizontal displacements of the foundation, their ratio, and the vertical differential settlement at the crown of the embankment are summarized in Table 4.8. The reader is reminded that the drained response of a soft foundation is analyzed rather than the process of consolidation, since consolidation cannot be analyzed by the program GEONON at the present time. However, observations of settlements of actual embankments seem to verify that the consolidation settlements quite closely approximate the settlements obtained from a drained analysis. Tavenas (1979) gives the ratios of maximum horizontal to maximum vertical settlements of the foundation as 0.18 ± 0.09 , 0.91 ± 0.2 and 0.16 ± 0.02 for initially drained, initially undrained, and long term consolidation settlements, respectively. These values are quite similar to those obtained from undrained and drained analyses; that is, 0.70 to 0.75 for undrained (Table 4.1) and 0.17 to 0.20 for drained response (Table 4.8).

FIGURE 4.54 Same unreinforced embankment as Fig. 4.2 on a soft ($E_f = 3000$ psf) but drained foundation ($v_f = 0.25$); dead load only. Orientations and relative magnitude of the principal stresses are shown. Numerical values indicate the octahedral shear stress ratios, $\tau_{oct}/\tau_{oct f}$, expressed as a percent.

Table 4.8 Summary of Settlement and Deformation Response for Various Loading and Reinforcement Configurations; Drained Soft Foundation.

Case	Figure	Maximum Vertical Settlement (ft)	Node	Maximum Horizontal Deflection (ft)	Node	Ratio of Maximum H to V	Maximum Differential Settlement (ft)
1. Unreinforced, DL only	4.54	1.10	78	0.19	46	0.17	0.033
2. Unreinforced DL + LL	4.55	1.95	78	0.37	46	0.19	0.150
3. Unreinforced, DL + unload of LL	4.56	1.22	78	0.25	46	0.20	0.119
4. Reinforced with one layer of high modulus woven geotextile, DL only	4.57	1.08	78	0.18	46	0.17	0.029
5. Reinforced with one layer of high modulus woven geotextile, DL + LL	4.59	1.90	78	0.35	46	0.18	0.124
6. Reinforced with one layer of high modulus woven geotextile, DL + unload of LL	4.61	1.18	78	0.23	46	0.19	0.095

Figure 3.3(b) shows the contours of the octahedral shear stress ratio corresponding to Fig. 4.54. The maximum values appear under the centerline and right under the toe of the embankment which indicates where failure would start if the ratio reached 100%. A toe failure is most likely for the drained situation, whereas a base failure would apparently occur in the undrained situation (see Fig. 4.3).

Figures 4.55 and 4.56 give results for the live load and the unload of live load cases. The next six figures, 4.57 through 4.62, give the results when one layer of a high modulus woven geotextile with stiffness $EA = 20000 \text{ lb/ft}$ (300 kN/m) is placed between the embankment and the foundation. The octahedral shear stress ratio increased due to live load, but it does not quite return to its original value after the live load is removed. However, the unloading results are somewhat questionable due to the approximation of the large displacement formulation. The foundation remains elastic throughout the loading. Hence, any difference in foundation stresses after application and removal of live load, as compared to initial load of the embankment alone, must be either due to plastic deformations in the embankment or an accumulation of errors in the numerical procedure for the large displacement analysis.

Figures 4.63 and 4.64 show the percent reduction in octahedral shear stress ratio due to the geotextile reinforcement as a function of depth and distance from the centerline of the embankment, respectively.

FIGURE 4.56 Same embankment and foundation as Fig. 4.55 but with live load removed.

FIGURE 4.57 Same embankment and foundation as Fig. 4.54 but reinforced with one layer of a high modulus woven geotextile of modulus $EA = 20,000$ lb/ft.

FIGURE 4.58 Percent reduction in octahedral shear stress ratio due to reinforcement. Minus sign indicates an increase. Compare Figs. 4.54 and 4.57.

FIGURE 4.59 Same embankment foundation and live load as Fig. 4.55, but reinforced as in Fig. 4.57.

FIGURE 4.60 Percent reduction in octahedral shear stress ratio due to reinforcement. Minus sign indicates an increase. Compare Figs. 4.55 and 4.59.

FIGURE 4.61 Same embankment and foundation as Fig. 4.59 but with live load removed.

FIGURE 4.62 Percent reduction in octahedral shear stress ratio due to reinforcement and unloading. Minus sign indicates an increase. Compare Figs. 4.56 and 4.61.

FIG. 4.63 Reduction in octahedral shear stress ratios due to the geotextile reinforcement as a function of depth for foundation elements located under the centerline of the embankment; drained response of a soft foundation; high modulus ($EA = 20000$ lb/ft) woven geotextile.

FIG. 4.64 Reduction in octahedral shear stress ratios due to the geotextile reinforcement as a function of distance from the centerline of the embankment for elements at the top of the foundation; drained response of a soft foundation.

If the trends for the unload of live load are correct, compaction will definitely improve the effectiveness of the fabric for the drained response of the foundation. Figure 4.65 shows the percent reduction in maximum shear stresses due to geotextile reinforcement, as a function of depth and for undrained foundation response. Comparing Figs. 4.63 and 4.65, the added benefits of compaction on the geotextile inclusion are comparatively small for the undrained foundation response, whereas the overall effect of the geotextile is definitely larger.

Figures 4.66 and 4.67 show the vertical and horizontal displacements of the ground surface for the weight of embankment alone, with live load, and after removal of the live load. This is, of course, with a drained foundation. As for the undrained case, the vertical rebound of the foundation after removal of the live load exceeds the initial deflection under the toe of the embankment. Such behavior is questionable.

It is also seen that whereas the vertical displacements upon unloading rebound to almost the same amount as before loading of the live load, this is not the case with the horizontal displacements, which show very little rebound. In fact, the horizontal displacement actually increases significantly beyond the toe of the embankment after unloading of the live load.

The lack of rebound of the horizontal displacements was also observed for the undrained response (see Figs. 4.21 and 4.22). However, this behavior may be caused by the computer model rather

FIG. 4.65 Reduction in maximum shear stresses due to the geotextile reinforcement as a function of depth for the foundation elements located under the centerline of the embankment; undrained response of a soft foundation.

1 ft = 0.305 m

1 lb/ft = 14.59 N/m

1 psf = 47.88 N/m²

FIG. 4.66 Vertical displacements of the embankment on a soft foundation; drained response, $E_F = 3000$ psf, $\nu_F = 0.25$.

FIG. 4.67 Horizontal displacements of the embankment on a soft foundation; drained response.

than be representative of the actual behavior of embankments on soft foundations.

Figures 4.68 and 4.69 show the vertical and horizontal displacements of the ground surface for the soft foundation and drained response, with and without reinforcement. The influence of the geotextile on the vertical settlements is negligible; for the horizontal displacements, slightly more influence is observed. The greatest influence in terms of settlements is found for the vertical differential settlements at the embankment crown.

4.3 COMPARISON BETWEEN DRAINED AND UNDRAINED RESPONSE

The maximum vertical, horizontal, and differential settlements are given in Table 4.9 and 4.10 for undrained and drained foundation response and a fabric stiffness EA of 20000 lb/ft (high modulus woven geotextile). Comparisons are made with the case without fabric for embankment dead load only, with live load, and live load removed. The percent reduction in settlements and displacements due to the presence of the reinforcement are also given. For the undrained response of the soft foundation (Table 4.9) there is a reduction in differential settlement of up to 27% for the embankment load alone. The reduction in maximum total settlement amounts to less than 10%. When live load is added, the reduction in differential settlements is about 18% for the same fabric stiffness. Also the percentage reductions in total settlement have decreased. After live load is

FIG. 4.68 Vertical displacement of ground surface for the embankment on a soft foundation; drained response, with and without fabric.

1 ft = 0.305 m

FIG. 4.69 Horizontal displacement of the ground surface for the embankment on a soft foundation; drained response, with and without fabric.

Table 4.9 Influence of the Reinforcement on Total and Differential Settlements of the Embankment on a Soft Foundation ($E_F = 3000$ psf), Undrained Loading ($\nu_F = 0.48$), and a High Modulus ($EA = 20000$ lb/ft) Geotextile Reinforcement.

	Maximum Vertical Settlement (ft)	Maximum Horizontal Deflection (ft)	Maximum Differential Settlement (ft)
Unreinforced embankment	0.530	0.370	0.060
Reinforced embankment	0.487	0.346	0.044
Difference	0.043	0.024	0.016
Percent difference	8.1	6.5	26.7
Unreinforced embankment + live load	0.895	0.654	0.163
Reinforced embankment + live load	0.833	0.623	0.134
Difference	0.062	0.031	0.029
Percent difference	6.9	4.7	17.8
Unreinforced embankment + unload live load	0.636	0.448	0.134
Reinforced embankment + unload live load	0.575	0.408	0.105
Difference	0.061	0.040	0.029
Percent difference	9.6	8.9	21.6

Table 4.10 Influence of the Reinforcement on Total and Differential Settlements of the Embankment on a Soft Foundation ($E_f = 3000$ psf), Drained Loading ($\nu_f = 0.25$), and a High Modulus ($E_A = 20000$ lb/ft) Geotextile Reinforcement.

	Maximum Vertical Settlement (ft)	Maximum Horizontal Deflection (ft)	Maximum Differential Settlement (ft)
Unreinforced embankment	1.10	0.19	0.033
Reinforced embankment	1.08	0.18	0.029
Difference	0.02	0.01	0.004
Percent Difference	1.8	5.3	12.1
Unreinforced embankment + live load	1.95	0.37	0.150
Reinforced embankment + live load	1.90	0.35	0.124
Difference	0.05	0.02	0.026
Percent Difference	2.6	5.4	17.3
Unreinforced embankment + unload, live load	1.22	0.25	0.119
Reinforced embankment + unload, live load	1.18	0.23	0.095
Difference	0.04	0.02	0.024
Percent Difference	3.3	8.0	20.2

removed, the reduction in differential settlements is increased somewhat to about 22%, but it is less than the value prior to the application of the live load. In contrast, the maximum settlements are reduced more by fabric reinforcement after removal of live load than before the load was applied.

Also for the drained response of the foundation (Table 4.10), the differential settlements are reduced more than the maximum total settlements. There is a notable difference between the drained and undrained response with respect to the decreases in settlements. Decreases in maximum vertical, maximum horizontal and differential settlements of the crown due to the geotextile reinforcement are larger when live load is applied, and they remain larger even after the live load has been removed, compared to the reductions for embankment load alone. However, results after removal of live load are somewhat questionable, as explained previously.

The reduction in the maximum total settlements can be compared to the reduction in maximum shear stresses for the undrained case (Fig. 4.65) and reduction in the octahedral shear stress ratios for the drained case (Fig. 4.63). For the undrained case (Fig. 4.65), the addition of live load decreases the reduction of maximum shear stresses. However, after the load is removed, there is an increase in the reduction of the maximum shear stresses (as compared to the case with the embankment alone). For the drained foundation response, on the other hand, the reduction in octahedral shear stress ratios increases for

both live load and removal of live load. Again the results for unloading must be regarded as questionable.

Overall, the influence of a geotextile reinforcement placed between a soft foundation and a relatively stiff embankment is much larger for undrained response than for a drained response of the foundation. The difference seems to be related to the ratio of maximum horizontal to maximum vertical displacements, or perhaps to the horizontal displacement only. The ratio of maximum horizontal to maximum vertical displacement is about 0.2 for the drained case (Table 4.8), and 0.7 for the undrained case. Even though the drained case settles about twice as much as the undrained case, the benefit of the geotextile is significantly less for the drained case. In many cases the reduction in maximum shear stresses for the undrained case is more than twice the amount of the reduction in octahedral shear stress ratios for the drained case (see Fig. 4.65 and 4.63). Similar results are obtained for displacements (Tables 4.9 and 4.10). The relative benefit of the fabric can also be seen from the tensile forces that develop in the fabric as shown in Fig. 4.70. The tension in the fabric is much larger for undrained foundation response.

The fact that the benefit from the fabric inclusion is greater for larger horizontal displacements seems reasonable, since the fabric is placed horizontally. The important horizontal displacement is the displacement of the nodes along the embankment foundation interface. The node at the toe of the embankment, node 71 (Fig. 4.1), has a 0.1 ft (0.03 m) lateral

FIG. 4.70 Tension in the high modulus ($EA = 20000$ lb/ft) geotextile for both drained and undrained response of a soft ($E_F = 3000$ psf) foundation. (a) Embankment dead load only; and (b) embankment dead load plus live load.

Fig. 4.70 Continued. (c) Unload of live load.

displacement due to the weight of the embankment alone on the soft drained foundation. This movement is reduced to 0.086 ft (0.026 m) when one layer of a high modulus geotextile is placed between the foundation and the embankment. The same numbers for the undrained case are 0.192 ft (0.059 m) and 0.154 ft (0.047 m), respectively. This means that the geotextile produces a 0.014 ft (0.004 m) reduction in lateral displacement for the drained case, and a 0.038 ft (0.012 m) reduction for the undrained case.

Chapter 5

CONCLUSIONS AND PRACTICAL IMPLICATIONS;
RECOMMENDATIONS FOR DESIGN AND CONSTRUCTION;
RECOMMENDATIONS FOR FURTHER RESEARCH

5.1 CONCLUSIONS AND PRACTICAL IMPLICATIONS

A two-dimensional finite element program for the static analysis of embankments on weak foundations reinforced with geotextiles has been developed. The program called GEONON is a modification of the structural analysis program NONSAP, and it includes provisions for both small and large displacements and both linear and nonlinear stress-strain material models for the soils. The elasto-plastic soil models utilized are the Drucker-Prager, which closely approximates the well-known Mohr Coulomb failure, and the Von Mises failure criterion. The program is quite sophisticated and sufficiently general that it can be used for the estimation of stresses and displacements of ordinary embankments and embankment-foundation systems, as well as for reinforced embankments. The program also has incremental construction capability to more closely model the way embankments are actually built, and it is also possible to consider compaction of the soil layers.

The present research has attempted to assess the influence of fabric reinforcement on stresses and displacements in an embankment and its foundation. The program GEONON was used to analyze several typical cases of both reinforced and unreinforced

embankments on drained and undrained foundations. For the unreinforced embankment, single and multilift construction on both soft and stiff foundations were investigated. The influence of compaction of the embankment was also studied. For reinforced embankments, the primary variables were modulus of the geotextile reinforcement, the number of layers of reinforcement, stiffness of the foundation, and the presence of live load as well as embankment dead load.

Based on the analyses using GEONON reported herein, it is possible to draw the following conclusions. The practical implications of these conclusions will also be given where appropriate.

1. In general, the simulation of embankment construction by multiple lifts appears to be a more realistic model of embankment-foundation performance than the application of full embankment load in a single lift. Exceptions to this observation occur when the foundation is relatively stiff and loading is undrained, in which case the difference is negligible.

2. Compaction of an unreinforced embankment on a soft foundation results in much larger horizontal stresses than compaction of that same embankment on a relatively stiff foundation, probably due to the elastic rebound from the large deformations imposed by compaction.

3. When the geotextile reinforcement is placed horizontally between the embankment and the foundation, its primary influence

fabric can interfere with likely failure surfaces. Another advantage of placing the fabric up in the embankment is improved lateral support for compaction equipment. This should produce better compaction, particularly near the edge of the embankment.

7. The reinforcing effect of the geotextile decreases with increasing stiffness of the foundation, again probably due to a reduced lateral deformation. The presence of the geotextile reinforcement reduces the differential settlement at the top of the embankment. As expected, total settlements are only slightly affected by the reinforcement.

8. The finite element analysis is a powerful tool for investigating the performance of geotextile-reinforced embankments. However, it has its limitations, particularly for the large displacement analysis and for nonlinear material models such as elasto-plastic models. Therefore, the use of finite element analyses should be limited to providing trends of performance rather than exact numerical results. Also the results are greatly dependent on a good estimate of the soil properties in the prototype. These are usually difficult to predict with a great degree of certainty.

In summary, there is a greater improvement in terms of increased stability and less differential settlement at the top of the embankment due to the presence of the geotextile reinforcement if the following factors exist:

1. undrained foundation,

is to reduce both the shear stresses in the foundation and the vertical differential settlements of the top of the embankment.

4. The benefit of the reinforcement is much greater for the undrained response of the foundation. This is believed to be due to the larger horizontal displacements in the undrained foundation. The fact that the undrained response benefits more than drained from the geotextile is fortunate, since normally reinforcement is needed more when the shear strength is the smallest, or at the end of construction. After consolidation occurs, the soil gains strength and the need for reinforcement decreases.

5. The benefit of the geotextile increases with higher fabric moduli, so long as the forces developed in the geotextile can be transferred by friction to the foundation.

6. Placing geotextile layers up in the embankment gives an additional reduction in the maximum shear stresses in the foundation for undrained foundation response. (The drained case was not analyzed for multiple layers.) However, the greatest reduction of shear stresses in the foundation is obtained when the geotextile layers are positioned at or close to the embankment-foundation interface. Even better results might be obtained if the reinforcement was placed approximately 3 ft or 1 m below the ground surface where the maximum horizontal displacements occur. However, this is not a practical location in terms of construction costs. On the other hand, fabric placed up in the embankment itself can increase local embankment stability, since the

2. weaker foundation,
3. higher fabric modulus,
4. multiple layers of reinforcement.

5.2 RECOMMENDATIONS FOR DESIGN AND CONSTRUCTION

5.2.1 Function of Geotextile Reinforcement

As described in Chapter 1, the possibility of reinforcing embankments on soft foundations must be considered in terms of two different situations: (1) low embankments, wherein the live load is relatively large and the dead load is relatively small, and (2) high embankments where the dead load is large and the live load is relatively small. For the case of low embankments, the function of the fabric is one of separation as well as reinforcement, while in the case of high embankments the fabric functions as reinforcement only. Reinforcement in high embankments may be required (1) to prevent a potential slope failure in the embankment itself, and (2) where the foundation is soft and the embankment must be reinforced to prevent foundation instability. In the latter case, the tendency of the embankment to spread imposes horizontal shear stresses which must somehow be resisted by the foundation. If the foundation soil is too soft and weak, failure by rupture can occur. It is useful to look at the possible ways in which an embankment can fail (Fig. 5.1). In all three cases if the fabric reinforcement is properly designed and installed, it serves to prevent instability and failure.

REQ'D - FABRIC TENSILE
MODULUS TO CONTROL
LATERAL SPREADING
(NOT ILLUSTRATED)

REQ'D - FABRIC TENSILE
STRENGTH TO RESIST
SPLITTING

A. POTENTIAL EMBANKMENT FAILURE FROM LATERAL EARTH PRESSURE

REQ'D - FABRIC TENSILE
STRENGTH TO RESIST
ROTATIONAL FAILURE
AND FABRIC TENSILE
MODULUS TO RESIST
EXCESSIVE DEFORMATION

B. POTENTIAL EMBANKMENT ROTATIONAL SLOPE/FOUNDATION FAILURE

REQ'D - FABRIC TENSILE
MODULUS TO CONTROL
FOUNDATION DISPLACEMENT

C. POTENTIAL EMBANKMENT FAILURE FROM EXCESSIVE DISPLACEMENT

Figure 5-1. Potential Fabric-Reinforced Embankment Failure Modes
(after Haliburton, Lawmaster, and McGuffey, 1981).

A second purpose of reinforcement is to prevent excessive vertical and horizontal deformations. In this case, the concept is closer to that of subgrade reinforcement, as might be required for roadways, railroads, etc.

5.2.2 Creep

There has been some concern about the use of relatively lightweight geotextiles to reinforce embankments on soft foundations, primarily because such fabrics are subject to creep even when the sustained loads are relatively modest. The primary concern is the relative rates of creep of the geotextile and the soil. If the creep rate of the soil is faster than the creep rate and subsequent weakening of the fabric, then there is no problem, because the soil will gain strength faster than the reinforcement loses strength. Since many unstable soils such as peats and silts do consolidate relatively rapidly, the potential detrimental effect of creep of geotextile reinforcement for embankments on these soils is not a serious problem. What to do about creep for design will be discussed below.

5.2.3 Design of Reinforced Embankments

The finite element program developed in this research is not likely to be readily an adaptable design tool by practicing highway engineers. Limitations are apparent especially for large displacement analyses and for nonlinear material behavior. As was mentioned in the conclusions, the approach should be at this time limited to indicating trends of performance rather than

exact results suitable for design calculations. Consequently, it is recommended that design engineers follow a relatively simple approach to the design of fabric reinforced embankments. This can involve analyses similar to (1) a conventional bearing capacity analysis, or (2) a conventional slope stability analysis. A bearing capacity analysis would of course assume that the embankment is a very wide footing. A slope stability analysis would involve calculations for stability based on a series of assumed sliding surfaces. In the latter case, the reinforcement acts as a horizontal force to provide an increase in the resisting moment in a "Swedish circle" type of analysis. Several examples of such calculations are available in the references mentioned in Chapter 1. Recommended are Netlon Ltd. (1982), Steward, Williamson, and Mohny (1977), and Haliburton, Lawmaster and McGuffey (1981). Also, procedures given by Ingold (1982), Jewel (1982), and Fowler (1982) should be considered. All of these authors utilize simple limiting equilibrium concepts which are familiar to most geotechnical engineers.

5.2.4 Factor of Safety

Because of the uncertainties with the use of geotextile reinforcement, there is some question as to the choice of factor of safety for construction with geotextiles, especially when the calculated factor of safety is significantly less than unity. When the calculated factor of safety is greater than one but less than the minimum allowable factor of safety for design, the fabric reinforcement acts as an additional safety factor or

"second line of defense" against failure. On the other hand, there is considerable experience, particularly by Haliburton and his co-workers, with the construction of embankments on extremely soft foundations, foundations so soft that they were difficult to walk on. For these foundations, the factor of safety for an unreinforced embankment was very much less than 1. Thus, exceptionally strong geotextiles were utilized as a reinforcement, and special techniques for construction must be followed. For these cases, we recommend the design procedures described by Haliburton, Anglin, and Lawmaster (1978a), Fowler (1981), and summarized by Haliburton, Lawmaster, and McGuffey (1981) particularly for highway construction.

5.2.5 Selection of Geotextile Properties

For a conservative choice of properties for use in the suggested design calculations, we recommend that the procedures suggested by Bell (1980) be utilized. These are summarized as follows.

For the tensile strength of the fabric, a wide strip tensile test should be used, or if possible a confined tensile test (McGown, et al., 1982). The wide strip tensile test is recommended for especially nonwoven geotextiles in order to provide as close to plane strain conditions as possible during the test. A standard for this test is currently under development by ASTM Committee D13.61/D18.19 and it should be approved within the near future. In the meantime, use the procedures outlined by Bell and

Hicks (1982). Traditional grab or strip tensile tests as described in ASTM method D1682 are not appropriate for obtaining properties for design for reinforcing fabrics.

If the dead load is large, then the following design values for the tensile strength of the fabric are recommended: For the ordinary case the tensile stress at 20% fabric strain is probably satisfactory. If significant creep of the fabric is expected, then the tensile stress at 10% strain should be utilized. If large strains in the fabric are likely to occur during construction, then it may be best to use a design tensile stress determined at 50% fabric elongation. However, for exceptionally soft foundations, where the fabric reinforcement will be subject to very large tensile stresses during construction, an allowable fabric elongation of 50% may be excessive. In this case, the recommendations by Haliburton, Lawmaster and McGuffey (1981) should be followed. For the case of relatively small dead load and large live load, the fabric functions more as a separator layer and a different design approach should be followed. This approach will be discussed later.

If significant creep of the fabric is expected and if large creep strains would be detrimental to the performance of the embankment, then the relative amount of live load versus dead load should be considered. It is possible that if the "overstress" is due primarily to live load, no additional fabric strength is required. The choice of polymer should be polyester because its creep resistance is greater than polypropylene, the

other common geotextile polymer. As was mentioned before relative creep rates of the fabric versus the creep rate in the soil and the consequent strength increase should be taken into consideration in any creep analysis.

5.2.6 Subgrade Stabilization

If the embankment has a relatively large live load and a relatively small dead load, then alternate design approaches are recommended. In this case the geotextile has at least two functions, that of a separator or "filter", to prevent the migration of soil fines up into the embankment aggregate, and that of membrane reinforcement. A third fabric function, that of lateral confinement and interference with the development of the bearing capacity failure surfaces, has been proposed by Haliburton, Lawmaster, and King (1981). An additional complication with low embankments is that the live loads are usually dynamic and durability of the fabric to abrasion is a serious concern.

For design of a simple separator layer, Bell (1980) recommends the use of a nonwoven or a slit film fabric with a weight greater than 4 oz. per sq. yard (approximately 140 g/m^2). The permeability of the fabric should be much greater than the permeability of the soil.

When some reinforcing effect of the fabric is considered, the empirical design procedures for unpaved roads recommended are those by Steward, Williamson, and Mohny (1977), and Giroud and Noiray (1980, 1981). The Steward procedure is based on lab scale

tests (Barenberg and Kinney, 1980) with subgrade CBRs less than 2.5. The limiting rut depth is 2 in. (50 mm). Design curves for required aggregate thicknesses for different vehicles and fabric moduli have been published by Mirafi, Inc., the manufacturer who sponsored Barenberg's research. Steward et al. (1977) presents practical examples of the Barenberg procedure.

Perhaps the most elegant and best documented of the design procedures for unpaved roads is that by Giroud and Noiray (1980, 1981). Appropriate consideration is given to the geotextile modulus, the effect of traffic (repetitive load), the undrained shear strength of the subgrade, and the rut depth. Design curves are presented in which a comparison of designs with and without geotextile for different vehicles, tire pressures, etc. is easily considered. In this procedure, the maximum number of load repetitions is 10,000, rut depth is 1 ft (30 cm). Recently, Sivakugan (1981) developed design charts for lesser rut depths. The procedure probably should not be used for permanent construction.

Recommended geotextile properties for unpaved roads include a minimum tensile strength of 50 to 60 lb/in. at a strain of 20%, and this should be determined, as before, with the wide strip tensile test or a confined tensile test. The modulus which is required for Giroud and Noiray's (1980, 1981) procedure should be determined in a similar manner. The properties of creep, durability and constructibility (Bell and Hicks, 1980; Bell, 1980) discussed previously applies here also.

So much of the effect by especially fabric manufacturers has been directed towards savings in aggregate thickness by the use of geotextiles. There are other advantages also (B. R. Christopher, personal communication, 1981): For example, in temporary roadways, the fabric allows the construction to be carried out ("constructibility"). The geotextile may also extend service life, although this effect is unknown. The fabric promotes drainage and prevents contamination of the aggregate, which would cause a reduction in permeability and drainage of the aggregate. It is possible that the geotextile may make settlements more uniform, as the results reported in Chapters 3 and 4 suggest. Finally, geotextiles are very useful for that occasional very heavy traffic. In this situation, the design for normal traffic operations is acceptable, but the facility is very occasionally subjected to extremely heavy traffic loads, and it would be uneconomical to design just for that extremely heavy load. In this case the fabric can provide additional factor of safety to allow for the adequate support of the heavy traffic.

There are also possible improvements to permanent roadways with geotextiles. These include encapsulation to prevent or reduce frost action, salt intrusion, shrinkage, and swelling. It may also be possible to use poorer, and therefore cheaper, aggregates in base courses in conjunction with geotextiles, and it may also be possible to use thinner base and subbase sections. Many of the same improvements were discussed by Hamilton and Pearce (1981).

5.2.7 Specifications for Geotextiles in Construction

For assistance in writing specifications for geotextiles and construction with geotextiles, the recommendations by Haliburton, Lawmaster, and McGuffey (1981) and FHWA (1978) should be followed.

5.3 RECOMMENDATIONS FOR FURTHER RESEARCH

The case of embankment widening and grade raising should be investigated utilizing the methods outlined in this research. This is a case wherein existing roadways are stable but must be widened or raised, and it is possible that geotextile reinforcement would assist in maintaining stability and reducing differential settlement between the new and old construction.

Different embankment and foundation geometries and soil properties should be investigated. Particularly of interest is the case of a very deep, soft foundation. Also it is of considerable interest to use our procedure as a check of some of the reinforced embankment FEM solutions recently published.

Extension of the available soil models in GEONON to include the Cap models is recommended. Research must also be done on extending the large displacement formulation and the updated geometry procedures. Numerical convergence difficulties for those cases also require additional study. Research also needs to be done on a procedure for establishing a more realistic initial state of stress in the foundation.

As pointed out by Holtz (1982) there is a serious lack of well-documented case histories of fabric-reinforced embankments on soft foundations. Some of the case histories described in Chapter 1 have provided valuable insight into the performance of such embankments, but additional well-documented, and well-instrumented, case histories are required in order to increase our confidence in current design procedures and construction methods.

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NOTICE

The Appendices to this report are not included in this publication. These Appendices are titled as follows:

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