SCHOOL OF
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FHWA/IN/JHRP/92/19

Final Report

PHASE II - EMBANKMENT WIDENING AND
GRADE RAISING ON SOFT FOUNDATION SOILS

Scott J. Ludlow

Wai-Fah Chen

Philippe L. Bourdeau

C. William Lovell

PURDUE UNIVERSITY

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Final Report

**Phase II - Embankment Widening and Grade
Raising on Soft Foundation Soils**

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Joint Highway Research Project
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and the
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The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration or of the Indiana Department of Transportation. This report does not constitute a standard, specification, or regulation.

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DRAFT FINAL REPORT

TO: Vincent P. Drnevich, Director
Joint Highway Research Project

July 23, 1992

FROM: W. F. Chen, Research Engineer
Joint Highway Research Project

Project No: C-36-36U
File: 6-14-21

Attached is the draft final Report on the HPR Part II study entitled, "Embankment Widening and Grade Raising on Soft Foundation Soils". The authors of the report are Scott J. Ludlow, Wai-Fah Chen, Philippe L. Bourdeau and C. William Lovell.

Copies of this report will be submitted to the IDOH and FHWA for their review. My co-authors and I look forward to receiving their comments on the report.

Respectfully submitted,

A handwritten signature in cursive script, appearing to read "W.F. Chen".

W.F. Chen
Research Engineer

WFC/cak

Attachment

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Appendix D: Interim Report: "Embankment Widening and Grade Raising on Soft Foundation Soils: Example 2 - Indiana State Route 1 over Ramsey Creek in Franklin County, Indiana," <u>Report No. FHWA/IN/</u> <u>JHRP - 92-9</u> , 173 pp.	

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ABSTRACT

The finite element technique using a cap elastic-plastic work-hardening soil behavior model was applied to the analysis of embankments constructed on soft foundation soils. A procedure was provided to estimate the cap model parameters from conventional field and laboratory test results. A sensitivity analysis of the cap model parameters comparing the observed and calculated responses was also provided. Results indicate that the undrained shear strength and over-consolidation ratios were observed to have the most-significant influence on the predicted model behavior.

The technique was then applied to the analysis of two examples. The examples were based on actual highway projects in Indiana where information on these projects was provided by INDOT personnel. Results of the analysis were used to determine the influence of several factors on reinforced and unreinforced embankment behavior. The results indicated that the crust strength and foundation compressibility had the most-significant influence on embankment fill and foundation soil behavior and the potential benefit possible with reinforcement. Reinforcement type/modulus also influenced the behavior of the embankment fill and foundation soil but to a lesser extent when compared to crust strength. The use of reinforcement for widening and grade raising of existing embankments appeared to be beneficial in reducing lateral movement.

IMPLEMENTATION REPORT

PHASE II - EMBANKMENT WIDENING AND GRADE RAISING OVER SOFT FOUNDATION SOILS

The Need

The draft final report for this project includes the study of embankment widening and grade raising over soft foundation soils. With the urgent need to upgrade our nation's highway system, engineers face challenging problems of widening existing highway embankments and raising new embankments, particularly over soft foundation soils. For the analysis/design of these problems, engineers often rely on a combination of limit equilibrium type methods; simplified load/deformation responses; and experience. However, as a result of the complex interaction of the embankment and foundation soils, the above methods provide only approximate information concerning the embankment and foundation deformations and the serviceability of the embankment following construction.

To assess the performance of the embankment/foundation system, advanced modeling techniques are needed that account for nonlinear and elastic-plastic material behavior. A technique that has gained acceptance for the analysis of such problems is the finite element method (FEM).

The FEM

Use of finite element techniques have the ability to allow us to:

1. model complex geometry and deformation patterns;
2. simulate nonlinear material behavior, including yielding, using advanced constitutive models;
3. analyze the interaction of different materials; and

4. combine geometrical nonlinearity with material nonlinearity.

Thus, the FEM can help engineers to:

1. improve their understanding of observed behavior in the field;
2. model the response of an embankment/foundation under increasing load level;
3. analyze the effects of changes in the components of the system (i.e. the properties of the embankment and foundation soil); and
4. assess the impact of construction procedures and the response of the system.

The finite element technique is well recognized as being a powerful tool for design and analysis of embankment/foundation systems, and numerous examples can be found in the literature. However, the use of finite element analysis for performing studies which could answer the questions raised above is subject to some important limitations.

If one is to model the response of the embankment in the range of large deformations, then it is essential to employ a finite element formulation and constitutive model which:

1. models the stress-dependent properties of the embankment fill and foundation soil(s);
2. correctly models plastic yielding and flow in the fill and foundation soil; and
3. allows for potential slip at the interface of soil and reinforcement in cases where geosynthetics are used to improve the stability of the embankment.

The Problem

It is unfortunate that such methodologies that have been proposed in the geotechnical literature are still of very sparse

use in the practicing community. A major hurdle for the application of the FEM to the analysis of such previously-mentioned problems is the lack of practical guidelines and step-by-step procedures to determine the material parameters required by the model and interpret the results in a design context. The ultimate goal of the present study is to bridge this gap. This study outlines a set of procedures to enable engineers to make use of available technology, and provides guidelines for the design and analysis of reinforced and unreinforced embankments over soft foundation soils.

The Objective

The major objective of this study (Phase II) has been to outline a set of procedures to enable engineers to make use of the available finite element methodology, and provide guidelines for the design and analysis of reinforced and unreinforced embankments, particularly over soft foundation soils. In order to achieve this objective, it is necessary to provide:

1. a simple and reliable procedure to derive the cap plasticity model parameters from data obtained by conventional field and laboratory soil tests;
2. background on the application/modeling techniques of the FEM to NFAP program;
3. complete case example studies in order to illustrate the capabilities of the method for the analysis and design of embankment widening and grade raising over soft foundation soils; and
4. practical guidelines for the analysis of embankment widening and grade raising over soft foundation soils using the FEM methodology.

Results and Guidelines for Practical Implementation

• Practical Procedure for Cap Parameter Determination

A straight forward procedure to determine the cap parameters for normally consolidated soils from conventional laboratory and field tests and a sensitivity study which examines the effect of the input soil parameters was provided. The main input soil properties are the compressibilities (C_c, C_r), the effective Mohr-Coulomb shear strength parameters ($\bar{\phi}, \bar{c}$), the undrained shear strength ratio ($USR; \frac{S_u}{\sigma_{vo}}$) and the over-consolidation ratio (OCR). Solutions are given in graphical form and equations suitable for hand calculation.

The procedure was used to determine the cap parameters for an impact-compacted lacustrine clay using results from isotropically-consolidated undrained-compression (CIUC) tests. These parameters were then used in a computer program called CAP to calculate stress-strain curves, pore pressure response and effective stress paths. Comparisons were made to observed test results. In general, there was good agreement except for a discrepancy for pore pressures and effective stress paths.

• Sensitivity Study of Cap Model Parameters

In addition, a sensitivity study was made of the effect of the input soil properties on calculated CIUC triaxial sample behavior. The results show which parameters have the greatest effect on computed behavior and provide guidance in the selection and adjustment of input soil properties to obtain a better fit between the calculated and observed response. The USR and OCR were

observed to have the most-significant influence on the predicted behavior.

• Practical Case Study

A comparative finite element study of two practical examples involving a total of 30 cases was made using an incremental procedure which simulated embankment construction. Embankment fill and foundation soil behavior was represented with an isotropic, strain-hardening cap-plasticity model.

The results indicated that the properties of a stiff or of a tensile resistant reinforcement had the most significant influence on embankment fill and foundation soil behavior. Reinforcement type/modulus also influenced the behavior of the embankment fill and foundation soil but to a lesser extent when compared to crust strength.

For compressible foundation soils and a relatively stiff embankment, a high-modulus woven geotextile was very effective in reducing displacements within the embankment fill and foundation soils and increasing the stability near the embankment toe. For a given embankment stiffness, the foundation compressibility directly influences the effectiveness of the crust strength. The foundation compressibility had only a limited influence on embankment behavior and the benefit from using reinforcement was modest except for that noted previously. The effect of increasing the embankment width by the use of a stabilizing berm only slightly reduced the displacements within the embankment fill and foundation soil and modestly increased the stability at the toe.

The widening and grade raising of an embankment locally altered the magnitude of the state of stress and stability but did not alter the overall pattern. Displacements developed primarily near the former shoulder and the widened toe (horizontally) and along the sideslope of the former embankment (vertically).

In several cases, the local factors of safety within the foundation soil near the centerline of the embankment and continuing outwardly to the toe and slightly beyond the toe were yielding (strain hardening). The other portions of the embankment fill and foundation soils were either experiencing stress states on the cap (elastic-plastic), or within the cap (elastic-state).

Limitations

• On Strain-Softening Soil

The cap model has some limitations which impact the calculated response. One limitation which was observed during this study was the inability of the model to predict the reduction in undrained strength or increase in pore pressure that occurs after the peak strength is reached in strain softening soil. Consequently, this would overestimate the shear strength and underestimate the pore pressures and deformations.

• On Over-consolidated Soils

A second limitation is the ability to model the behavior of over-consolidated soils. The cap model correctly predicts the undrained strength at large strains. However, pore pressures are not predicted correctly and plastic strains which many over-

consolidated soils experience for stress changes in the elastic region bounded by the cap and ultimate failure surfaces are not accounted for. Also, strain softening after failure can not be modeled. It is speculated that this would lead to underestimation of deformations.

• On Trial-and-error Procedure

Another limitation which can be attributed to the model's formulation is the need to perform a trial-and-error calculation to estimate the best-fit of the observed response. Although time-consuming, a reasonable fit of the observed response can be obtained rather quickly by making a critical review of the OCR and USR parameters and then modifying one of the other five (from the previous sensitivity study) parameters, if necessary.

• On Long-term Behavior of Soils

Another limitation which requires considerable attention and can also be attributed to the model's formulation is the model's inability to account for long-term behavior (e.g. creep and consolidation). Presently, the model is best-suited for depicting short-term behavior only. To account for creep and consolidation, the model would require extensive modifications.

Further Research Needs

The work presented in the study aims to facilitate the application of the cap plasticity model to practical problems and in design of embankments constructed on soft foundation soils. However, there are some aspects of unreinforced and reinforced embankment analysis that require further development of the

software program for this study. The ability of the cap model to accurately predict soil behavior could be extended to a wider range of soils, stress paths and drainage conditions. The following are additional research needs:

1. A parametric study of the effect of reinforcement should be made using NFAP for a wider range of embankment geometries and soil properties. It should include narrower embankments, different crust thicknesses and different reinforcement moduli. The tensile forces in the reinforcement should be analyzed in this study.
2. The cap plasticity model should be extended to improve its ability to model soil behavior during rotation of principal stresses and the behavior of over-consolidated soils.
3. Compacted cohesive soils are more often used as fill material for embankments. The behavior of embankments with cohesive fill (which include the effects of compactive prestress) should be evaluated with the cap plasticity model.
4. Presently, the cap plasticity model is better at accommodating either drained or undrained conditions. The model should be extended to partially drained conditions and then evaluated.
5. Slippage often occurs at or near the interface of the embankment fill/geosynthetic/foundation soil interface. NFAP should be modified to include slippage considerations.
6. To provide beneficial use to practicing engineers, NFAP should be modified to include pre- and post-processing graphics capabilities.
7. Post construction effects need to be considered, particularly since the original embankment and widening/raising have different chronologies. Foundation creep and embankment settling upon wetting in service are examples.

CHAPTER 1

INTRODUCTION

1.1 Motivation

With the urgent need to upgrade our nation's highway system, engineers face challenging problems of widening existing highway embankments and raising new embankments, particularly over soft foundation soils. For the analysis/design of these problems, engineers often rely on a combination of limiting equilibrium type methods; simplified load/deformation responses; and experience. However, as a result of the complex interaction of the embankment and foundation soils, the above methods provide only approximate information concerning the embankment and foundation deformations and the serviceability of the embankment following construction.

To assess the performance of the embankment/foundation system, advanced modeling techniques are needed that account for nonlinear and elastic-plastic material behavior. A technique that has gained acceptance for the analysis of such problems is the finite element method (FEM).

Use of finite element techniques have the ability to allow us to:

1. model complex geometry and deformation patterns;
2. simulate nonlinear material behavior, including yielding, using advanced constitutive models; and
3. analyze the interaction of different materials.

Thus, the FEM can help engineers to:

1. improve their understanding of observed behavior in the field;

2. model the response of an embankment/foundation under increasing load level;
3. analyze the effects of changes in the components of the system (i.e. the properties of the embankment and foundation soil); and
4. assess the impact of construction procedures and the response of the system.

The finite element technique is well recognized as being a powerful tool for design and analysis of embankment/foundation systems, and numerous examples can be found in the literature (e.g. Boutrup and Holtz, 1983; Hird and Jewell, 1990; Kwok, 1987; Leroueil et al., 1978; Rowe, 1982). However, the use of finite element analysis for performing studies which could answer the questions raised above is subject to some important limitations.

If one is to model the response of the embankment in the range of large deformations, then it is essential to employ a finite element formulation and constitutive model which:

1. models the stress-dependent properties of the embankment fill and foundation soil(s);
2. correctly models plastic yielding and flow in the fill and foundation soil; and
3. allows for potential slip at the interface of soil and reinforcement in cases where geosynthetics are used to improve the stability of the embankment.

It is unfortunate that such methodologies that have been proposed in the geotechnical literature are still of very sparse use in the practicing community. A major hurdle for the application of the FEM to the analysis of such previously-mentioned problems is the lack of practical guidelines and step-by-step procedures to determine the material parameters required by the

model and interpret the results in a design context. The ultimate goal of the present study is to bridge this gap. This study outlines a set of procedures to enable engineers to make use of available technology, and provides guidelines for the design and analysis of reinforced and unreinforced embankments over soft foundation soils.

1.2 General Background

Prior to widening or raising an embankment, it is common practice to partially or totally undercut and replace soft or otherwise unstable foundation soils with compacted "special" fill, which is expensive (not only the fill but also the construction techniques). Alternatively, the foundation soils can be treated in such a way as to accommodate the proposed loads. Numerous techniques which are within the category of ground modification are available, e.g. dewatering, grouting, preloading, compacting, etc. However, these techniques are site specific, often costly and generally time consuming. In this study, the focus with respect to soil improvement is on the use of geosynthetic reinforcement (specifically geotextile or geogrid) placed directly at the interface of the soft ground and the new or added embankment. The result of the embankment loading is to induce excess pore water pressure in the underlying foundation soils which is subsequently dissipated when consolidation occurs. During the critical stage for the stability (short term), the function of the geosynthetic is of reinforcing and providing support (horizontally) to the embankment. However, the situation may change somewhat with time

as the foundation soil gains strength through consolidation and begins to support part, or all, of the embankment loading.

The software used in this study is based on a general purpose finite element program named NFAP (nonlinear finite analysis program) developed by Chang (1980). NFAP was originally developed for the use of nonlinear large deformation analysis of structures. However, the program was later expanded and refined by Mizuno (1981) and McCarron and Chen (1985) to include a cap plasticity model to simulate the behavior of soils. Then, Humphrey (1986) condensed McCarron's version of the program for use on personal computers.

During Phase I of this research study, Huang (1991) modified and extended NFAP to include: double precision/floating point variables; foundations with sloping boundaries and a variable groundwater level; and a procedure for determining the initial/existing stress state within an embankment. A flow chart for the NFAP computer program is shown in Figure 1.1.

As previously stated, NFAP employs a cap plasticity model to simulate the behavior of the soil. From a theoretical point of view, the cap model employed herein is particularly appropriate in modeling soil behavior, because it is capable of treating the conditions of stress history, stress path dependency, dilatancy, and the effect of the intermediate principal stress. At present, however, many practicing engineers are not familiar with the process of determining the material parameters of the cap model. Often, existing procedures from well-known texts or more recent

Figure 1.1 Flow Chart of NFAP

literature, which are mostly developed by mechanics from the engineering-mechanics viewpoint, are employed rather than using data obtained from commonly performed soil tests.

1.3 Objective and Scope of Work

The major objective of this study (Phase II) has been to outline a set of procedures to enable engineers to make use of the available finite element methodology of NFAP, and provide guidelines for the design and analysis of reinforced and unreinforced embankments, particularly over soft foundation soils. In order to achieve this objective, it is necessary to provide:

1. a simple and reliable procedure to derive the cap plasticity model parameters from data obtained by conventional field and laboratory soil tests;
2. background on the application/modeling techniques of the FEM to the NFAP program;
3. complete case example studies in order to illustrate the capabilities of the method for the analysis and design of embankment widening and grade raising over soft foundation soils; and
4. practical guidelines for the analysis of embankment widening and grade raising over soft foundation soils using the FEM methodology.

1.4 Report Organization

No theoretical or technical details are included beyond what is absolutely necessary for following the report and making use of the procedures and recommendations contained herein. Those interested in acquiring a more thorough knowledge of the applications of the cap plasticity model and finite element formulation and related work on soil reinforcement of soft embankment foundations are referred to the references listed in Appendix A.

Chapter 2 discusses a proposed procedure for estimating cap plasticity model parameters from conventional field and laboratory test results. Often during a design process, many geotechnical engineers are unable to perform a sufficient site investigation to accurately estimate the soil parameters because of the project's budgetary restraints. Consequently, the soil parameters are obtained from empirical correlations using conventional field and laboratory test results and/or engineering judgement. This chapter employs these correlations and judgement to estimating cap plasticity model parameters. In addition, a sensitivity study of the cap parameters is performed.

Chapter 3 concentrates on the application and modeling techniques of the FEM to the NFAP program.

Chapter 4 contains case studies of application of the procedures outlined in Chapters 2 and 3. The cases in Chapter 4 are based on actual highway projects in Indiana where information on these projects was provided by Indiana Department of Transportation (INDOT) personnel.

Chapter 5 presents the developed guidelines for the analysis of embankment widening and grade raising over soft foundation soils by using the NFAP methodology, based on the results obtained from Chapter 4.

Chapter 6 contains conclusions and recommendations, including further work involving soil reinforcement, and expanding the capabilities of the analytical model to better accommodate the deformations of embankment/foundations in service.

CHAPTER 2

A PROCEDURE FOR ESTIMATING CAP PLASTICITY MODEL PARAMETERS FROM CONVENTIONAL FIELD AND LABORATORY TEST RESULTS

2.1 Introduction

The deformational behavior of soils is primarily influenced by stress-strain history, soil type and direction, rate, and magnitude of loading. To successfully apply a soil model, it is essential that the model expresses the significant characteristics of the soil response for a particular state. One model that has received acceptance in the area of finite element analysis of geotechnical engineering problems is the cap plasticity (cap) model (e.g. Nelson and Baladi 1977; Baladi and Rohani 1979; Chen and McCarron 1983; Mizuno and Chen 1984; Daddazio et.al. 1987; McCarron and Chen 1987).

In general, cap models describe the failure and yielding behavior of soil with an ultimate yield surface that is fitted with a movable end cap (see Figure 2.1). Both the ultimate or failure and cap yield surfaces are symmetrical about the hydrostatic axis. A "hardening rule" specifies the movement of the cap as a result of the hardening or softening behavior of the soil when loaded. Other cap models also permit the movement of the failure surface in addition to the cap. Movement in these types of models are also expressed by a hardening rule. It should also be noted that strains are elastic for stress changes that lie within the region of the failure and cap surfaces but are both elastic and plastic for stress changes on the surfaces. The cap model utilized in this study is an isotropic strain-hardening plasticity model with an

(a) $\bar{I}_1 - J_2^{1/2}$ Plane

(b) Deviatoric Plane

Figure 2.1 Cap Plasticity Model

elliptical cap. This model is briefly described in the following sections. Readers interested in acquiring more knowledge about cap models are referred to a recent book by Chen and Mizuno (1990).

In this chapter, the reader is first given a brief description of the cap model including a discussion of the yield functions (ultimate and cap yield surfaces) and the hardening rule. Then, a procedure for estimating the cap model parameters from conventional field and laboratory test results is given. Finally, the results of a sensitivity study are given which examine the effect of the input soil properties on the calculated response.

2.2 Description of the Cap Model

The primary features of the cap model include the cone-shaped ultimate/failure surface and an elliptical-shaped cap. Figure 2.1 illustrates the model in an $\bar{I}_1$ - $J_2^{1/2}$ space where $\bar{I}_1$ is the first invariant of the effective stress tensor and J_2 is the second invariant of the deviatoric stress tensor. The invariants are defined in terms of the principal stresses as:

$$\bar{I}_1 = \bar{\sigma}_1 + \bar{\sigma}_2 + \bar{\sigma}_3 \quad (2.1)$$

$$J_2 = \frac{1}{6} [(\bar{\sigma}_1 - \bar{\sigma}_2)^2 + (\bar{\sigma}_2 - \bar{\sigma}_3)^2 + (\bar{\sigma}_3 - \bar{\sigma}_1)^2] \quad (2.2)$$

where $\bar{\sigma}_1, \bar{\sigma}_2, \bar{\sigma}_3$ are the major, intermediate and minor effective principal stresses, respectively.

The equation for the ultimate failure surface, f_f , is based on a generalization of the Mohr-Coulomb hypothesis (Drucker and Prager, 1952) which is a smooth surface of the simple form:

$$\bar{f}_f = \alpha \bar{I}_1 - J_2^{1/2} + \kappa = 0 \quad (2.3)$$

where α and κ are material parameters that are related to the angle of internal friction and cohesion of the soil, respectively. It should also be noted that the sign convention used herein is positive for compressive stresses and strains, and therefore consistent with soil mechanics sign convention.

The equation used to describe the shape of the cap is based on a quarter of an ellipse:

$$f_c = (\bar{I}_1 - L(l))^2 + R^2 J_2 - (x - L(l))^2 = 0 \quad (2.4)$$

where R is defined as the (aspect) ratio of the major and minor radii in Figure 2.1. In addition, x and $L(l)$ are defined as the hardening rule and failure function, respectively. It should also be recognized that x and $L(l)$ define the $\bar{I}_1$ -value at the intersections of the elliptical cap with the $\bar{I}_1$ -axis and the failure function, respectively. The location, x , of the cap is related to the plastic volumetric strain, e_v^p , and this relation is assumed as (DiMaggio and Sandler, 1971):

$$x = -\frac{1}{D} \ln(1 - \frac{e_v^p}{W}) \quad (2.5)$$

where D is a curve fitting parameter with units of (stress) $^{-1}$ and W is the limiting value of e_v^p at high stress.

Elastic volumetric and shear (distortion) strains are governed by the bulk, K , and shear, G , moduli, respectively. Elastic volumetric strain, e_v^p , is produced by changes in $\bar{I}_1$.

$$\epsilon_v^e = \frac{\bar{I}_1}{(3K)} \quad (2.6)$$

The elastic shear strain is given by:

$$\epsilon_{ij} = \frac{s_{ij}}{(2G)} \quad (2.7)$$

where ϵ_{ij} is the deviatoric strain tensor and s_{ij} is the deviatoric stress tensor. To reiterate, stress changes in the region bounded by the ultimate and cap yield surfaces produce only elastic strains. However, loading on the ultimate yield surface or beyond the initial location of the cap results in both elastic and plastic strains.

In order to compute plastic strains for loading on the ultimate and cap yield surfaces, an associated flow rule is assumed. The flow rule defines the relationship between the next increment of the plastic strain increment, $d\epsilon_{ij}^p$, and the present state of stress, σ_{ij} , for an element of soil that is either on the ultimate or cap yield surfaces and when subjected to further loading. As indicated by Figure 2.2a, an increment of loading on the cap causes it to expand and results in positive plastic volumetric (contraction) strain and plastic shear strain. However, loading on the ultimate yield surface produces negative plastic volumetric (dilation) strain and plastic shear strain. As a result of the dilation, the cap contracts (as shown in Figure 2.2b) by an amount that is proportional to the hardening rule (Equation 2.5). For a stress state at the intersection between the cap and the ultimate yield surface, the increment of plastic strain is taken to

Figure 2.2 Response due to an increment of loading
 (a) loading on cap yield surface
 (b) Loading on ultimate yield surface
 (c) loading at corner

be normal to the cap. Therefore, plastic shear strain occurs at a constant volumetric strain (Figure 2.2c). In addition, this is consistent with the observed behavior of soils at large strain.

2.3 A Procedure for Estimating Cap Model Parameters for NFAP

There are 12 cap model parameters and they are generally grouped into five categories. These categories include: 1) an ultimate yield surface (3 parameters); 2) elastic and plastic behavior (4); 3) cap surface (2); 4) initial stress state (2); and 5) pore pressure response factor (1). Table 2.1 provides a summary of the parameters, while the required constants for NFAP for determining the model parameters are indicated on Table 2.2. The difference between the required constants for NFAP and the conventional parameters is in the descriptions of the elastic behavior and hardening rule.

Parameters that are required to perform the analysis can be obtained from soil properties of commonly performed laboratory and field tests [e.g. consolidation (oedometer) test; unconsolidated and consolidated undrained triaxial tests; standard penetration test; field vane shear; and empirical correlations from index property tests]. In some cases, it may be necessary to estimate a few of these properties. The following procedures are given for the use of estimating cap model parameters for NFAP. Chapter 4 will apply the procedures contained herein to estimate cap model parameters from conventional field and laboratory test results.

2.3.1 Ultimate Failure Surface (α, κ, T_c)

As stated previously, the equation for the ultimate yield

Table 2.1 Summary of Cap Model Parameters

Ultimate yield surface

α	slope of ultimate yield surface in $\bar{I}_1 - J_2^{1/2}$ space (degrees)
κ	slope intercept along $J_2^{1/2}$ axis in $\bar{I}_1 - J_2^{1/2}$ space (force per unit area)
T_c	tension cut-off/crack potential (force per unit area - also related to the minimum principal tensile strength of the soil)

Elastic and Plastic Behavior

$K_{\min}$	minimum value of the elastic bulk modulus (force per unit area)
$A_t = \frac{C_c}{2.303(1+e_o)}$	total bulk modulus parameter
$A_e = \frac{C_r}{2.303(1+e_o)}$	elastic bulk modulus parameter
	where e_o is the initial void ratio and C_c and C_r are the compression and recompression indices, respectively
ν	Poisson's ratio

Cap Surface

R	cap aspect ratio (ratio of the major and minor radii in Figure 2.1)
OCR	overconsolidation ratio

Initial Stress State

γ	saturated or moist unit weight of the soil (force per unit volume)
K_o	initial coefficient of lateral earth pressure

Pore Pressure Response Factor

β	a ratio relating the apparent bulk modulus of the fluids to the total stiffness (which includes both the soil and fluid)
---------	--

Table 2.2 Constants for Determining the Cap Model Parameters

$K_{\min}$	minimum value of the elastic bulk modulus (force per unit area)
ν	Poisson's ratio
e_o	initial void ratio
ϕ	angle of internal friction (degrees)
c	cohesion (force per unit area)
C_c	compression index
C_r	recompression index
$\frac{S_u}{\bar{\sigma}_o}$	undrained shear strength to effective over- burden pressure
OCR	overconsolidation ratio
T_c	tension cut-off (force per unit area)
γ	saturated or moist unit weight of the soil (force per unit volume)
K_o	coefficient of initial lateral earth pressure
β	pore water pressure response factor

surface , f_f , is based on a generalization of the Mohr-Coulomb hypothesis (Drucker and Prager, 1952) which is:

$$f_f = \alpha \bar{I}_1 - J_2^{1/2} + \kappa = 0 \quad (2.8)$$

where α and κ are material parameters that are related to the angle of internal friction and cohesion of the soil, respectively. However, before estimating the parameters, it is important to recognize that there are several ways to approximate the Mohr-Coulomb hexagonal failure surface by the Drucker-Prager circular section in three-dimensional stress space as indicated in Figure 2.3. For geotechnical engineering applications, soil will either experience some compression or extension depending primarily upon the stress-strain history, soil type and direction of loading.

To simulate triaxial compression behavior of the soil (where $\sigma_2 = \sigma_3$), the two sets of material constants (α , κ and c , ϕ) are related by (Chen and Saleeb, 1982):

$$\alpha = \frac{2 \sin \phi}{\sqrt{3} (3 - \sin \phi)} \quad (2.9)$$

$$\kappa = \frac{6 c \cos \phi}{\sqrt{3} (3 - \sin \phi)} \quad (2.10)$$

To simulate triaxial extension behavior of the soil, the relations are given by:

$$\alpha = \frac{2 \sin \phi}{\sqrt{3} (3 + \sin \phi)} \quad (2.11)$$

$$\kappa = \frac{6 c \cos \phi}{\sqrt{3} (3 + \sin \phi)} \quad (2.12)$$

For plane strain conditions (e.g. a relatively long embankment),

Figure 2.3 Drucker-Prager and Mohr-Coulomb failure criterion matched on compression and extension meridians (after Chen and Saleeb, 1982)

the material constants are related by (Drucker and Prager, 1952):

$$\alpha = \frac{\tan\phi}{(9 + 12\tan^2\phi)^{\frac{1}{2}}} \quad (2.13)$$

$$\kappa = \frac{3c}{(9 + 12\tan^2\phi)^{\frac{1}{2}}} \quad (2.14)$$

However, for stress states within the soil mass other than triaxial compression or extension, the match between the Mohr-Coulomb and the Drucker-Prager criteria depends upon the Lode angle, θ , or the intermediate principal stress at failure. Therefore, the material constants are given by:

$$\alpha = \frac{\frac{1}{3}\sin\phi}{[\sin(\theta + \frac{\pi}{3}) - \frac{1}{\sqrt{3}}\cos(\theta + \frac{\pi}{3})\sin\phi]} \quad (2.15)$$

$$\kappa = \frac{c\cos\phi}{[\sin(\theta + \frac{\pi}{3}) - \frac{1}{\sqrt{3}}(\cos(\theta + \frac{\pi}{3})\sin\phi)]} \quad (2.16)$$

where

$$\theta = \frac{1}{3}\cos^{-1}\left(\frac{3\sqrt{3}J_3}{2J_2^{\frac{3}{2}}}\right) \quad (2.17)$$

J_2 and J_3 in Equation 2.17 are the second and third invariants of the deviatoric stress tensor, respectively.

When the stress state within the soil mass reaches the intersection of the ultimate and cap yield surfaces, only shear (distortion) strains (γ) occur without plastic volume change

(i.e. $d\epsilon_v^p=0$). As a result, the direction of the incremental plastic strain must be normal to the elliptical cap rather than the ultimate surface in order to satisfy normality conditions. For the plane strain case with shear distortion allowed only at failure, the material constants are derived in a similar manner by Drucker and Prager (1952) and expressed as:

$$\alpha = \frac{1}{3} \sin \phi \quad (2.18)$$

$$\kappa = c \cos \phi \quad (2.19)$$

Often, soils will exhibit a greater angle of internal friction under plane strain conditions than those obtained from triaxial test conditions (Lee, 1970). This indicates that the failure surface, which connects the triaxial compression and extension zones on the deviatoric plane for the Mohr-Coulomb failure criterion (see Figure 2.4), underestimates the actual behavior of the soil. In order to represent the actual behavior of the soil mass, Dafalias and Herrmann (1986) modified the material constants with a non-circular cross section of the failure surface on the deviatoric plane such that the parameters α and κ depend on the Lode angle, θ , where:

$$\alpha = \alpha_c g(\theta, m) \quad (2.20)$$

$$\kappa = \kappa_c g(\theta, m) \quad (2.21)$$

and

Figure 2.4 Non-circular and Mohr-Coulomb failure surfaces on deviatoric plane

$$m = \frac{\alpha_t}{\alpha_c} = \frac{\kappa_t}{\kappa_c} = \frac{3 - \sin\phi}{3 + \sin\phi} \quad (2.22)$$

in which

$$g(\theta, m) = \frac{2m}{1 + m - (1 - m)\sin(\frac{\pi}{2} + 3\theta)} \quad (2.23)$$

where the subscripts t and c denote the values with respect to triaxial extension ($\theta=60^\circ$) and compression ($\theta=0^\circ$). For a condition of pure shear ($\theta=30^\circ$), Equation 2.23 reduces to:

$$g(\theta, m) = \frac{2m}{1 + m} \quad (2.24)$$

In addition to the compressive stresses within the soil mass, tensile stresses may occur. In NFAP, the tension potential is evaluated using the minimum principal stress in the "in-plane" stress state for the two-dimensional plane strain case. Since most soils are not capable of supporting significant tensile stresses, a small value of tension cut-off is generally assumed.

2.3.2 Elastic and Plastic Behavior ($K_{\min}, A_t, A_e, v$)

Prior to discussing the elastic and plastic behavior of soils within the cap model framework, it is important to realize that two assumptions are incorporated herein. These include: (1) the one-dimensional compression and rebound curves are parallel and linear to the respective curves for hydrostatic loading; and (2) the ratio of C_c and C_r to the specific volume, V , ($1+e_o$) is constant for all values of $\bar{p}$, the (mean) effective hydrostatic stress.

Based on the above assumptions, an idealized response of the

one-dimensional consolidation behavior (for simulating isotropic consolidation conditions) is shown in Figure 2.5. The equation for the virgin isotropic consolidation line is:

$$C_c = -\frac{de}{d(\log \bar{p})} \quad (2.25)$$

where e is the void ratio and C_c is the compression index. Recall that $\bar{p}$ is the effective hydrostatic stress where:

$$\bar{p} = \frac{\bar{I}_1}{3} \quad (2.26)$$

Based on the assumption that one-dimensional consolidation behavior represents isotropic consolidation conditions, the total volumetric strain, de_v^t , is then a function of the change in the void ratio where:

$$de_v^t = -\frac{de}{1 + e_o} \quad (2.27)$$

Substituting Equation 2.27 into Equation 2.25 and rearranging results:

$$\frac{C_c}{1 + e_o} = \frac{de_v^t}{d(\log \bar{p})} \quad (2.28)$$

or

$$K_t = \frac{d\bar{p}}{de_v^t} = \frac{\bar{p}}{A_t} \quad (2.29)$$

where K_t is the total bulk modulus and is a function of the effective hydrostatic stress, $\bar{p}$, and A_t is a constant which has the form:

Figure 2.5 Idealized response of soil to hydrostatic stress

Figure 2.6 Cap model response for undrained shear

$$A_t = \frac{C_c}{2.303(1 + e_o)} \quad (2.30)$$

The elastic bulk modulus, K_e , can be derived in the same manner and expressed as:

$$K_e = \frac{d\bar{p}}{de_v^e} = \frac{\bar{p}}{A_e} \quad (2.31)$$

with

$$A_e = \frac{C_r}{2.303(1 + e_o)} \quad (2.32)$$

where C_r is the recompression index and e_v^e is the elastic or recoverable volumetric strain. The total volumetric strain consists of elastic/recoverable strains and plastic/irrecoverable strains.

$$e_v^t = e_v^e + e_v^p \quad (2.33)$$

Based on Equations 2.29 and 2.31, it is apparent that both bulk moduli, K_t and K_e , are proportional to the effective hydrostatic stress. Therefore, $K_{\min}$, which is the minimum value of the elastic bulk modulus for a specific effective hydrostatic stress or possibly a free stress state is required and can be estimated from elastic moduli relations.

As mentioned earlier, the other elastic constant that is required is the shear modulus, G , which can be evaluated from Poisson's ratio, ν , which must also be determined or approximated:

$$G = \frac{3K_o(1 - 2\nu)}{2(1 + \nu)} \quad (2.34)$$

2.3.3 Cap Parameters (R , OCR)

The aspect ratio, R , is generally evaluated from shear tests on normally consolidated soil where the loading path causes the cap to expand. It is assumed that R remains constant as the state of stress moves from the initial to the failure condition and that R is independent of the initial hydrostatic stress state.

The initial state of stress ($\bar{I}_{1o}$, $J_{2o}^{1/2}$) for normally consolidated soil is on the cap and the failure state of stress ($\bar{I}_{1f}$, $J_{2f}^{1/2}$) is at the intersection of the cap and ultimate failure surface as shown in Figure 2.6 (page 24). The initial state of stress is calculated from the effective vertical consolidation stress $\bar{\sigma}_{vo}$ and the at-rest normally-consolidated coefficient of lateral earth pressure K_o which is given by:

$$K_o = \frac{\bar{\sigma}_{ho}}{\bar{\sigma}_{vo}} \quad (2.35)$$

where $\bar{\sigma}_{ho}$ is the horizontal effective stress. Equations 2.1 and 2.2 are used to calculate $\bar{I}_{1o}$ and $J_{2o}^{1/2}$ for $\bar{\sigma}_1 = \bar{\sigma}_{vo}$ and $\bar{\sigma}_2 = \bar{\sigma}_3 = \bar{\sigma}_{ho}$.

$$\bar{I}_{1o} = \bar{\sigma}_{vo}(1 + 2K_o) \quad (2.36)$$

$$J_{2o}^{1/2} = \frac{1}{\sqrt{3}}\bar{\sigma}_{vo}(1 - K_o) \quad (2.37)$$

However, at failure, Equation 2.3 is used to relate the final states of stress ($\bar{I}_{1f}$, $J_{2f}^{1/2}$)

$$\bar{I}_{1f} = \frac{(J_{2f}^{1/2} - \kappa)}{\alpha} \quad (2.38)$$

The undrained shear strength ratio (USR) is introduced as a normalized measure of the shear stress at failure. At final state, the USR can be expressed as:

$$USR = \frac{S_u}{\bar{\sigma}_{vo}} = \frac{(\sigma_1 - \sigma_3)_f}{2\bar{\sigma}_{vo}} \quad (2.39)$$

where $(\sigma_1 - \sigma_3)_f$ is the principal stress difference at failure. At failure, the second invariant of the deviatoric stress tensor J_{2f} is:

$$J_{2f} = \frac{1}{3} (\sigma_1 - \sigma_3)_f^2 \quad (2.40)$$

or from Equation 2.39:

$$J_{2f}^{\frac{1}{2}} = \frac{1}{\sqrt{3}} (2 USR \bar{\sigma}_{vo}) \quad (2.41)$$

Substitution of Equation 2.41 into Equation 2.8 gives:

$$\bar{I}_{1f} = \frac{1}{\alpha} (-\kappa + 2 \frac{\bar{\sigma}_{vo}}{\sqrt{3}} USR) \quad (2.42)$$

Using these relations and Equation 2.29, the elastic volumetric strain change can be evaluated as:

$$de_v^e = A_e \frac{\bar{I}_{1f} - \bar{I}_{1a}}{(\bar{I}_{1f} + \bar{I}_{1a})/2} \quad (2.43)$$

For undrained behavior, $de_v^p = -de_v^e$. From Equations 2.36, 2.42 and 2.43, de_v^p can also be written as:

$$de_v^p = -2A_e \frac{\left(-\frac{\kappa}{\sigma_{vo}} + 2\frac{USR}{\sqrt{3}}\right) - (1 + 2K_o)\alpha}{\left(-\frac{\kappa}{\sigma_{vo}} + 2\frac{USR}{\sqrt{3}}\right) + (1 + 2K_o)\alpha} \quad (2.44)$$

The hardening rule as described earlier by Equation 2.5 will not be used herein. However, a modification as proposed by Huang and Chen (1991) describing the evolution of the cap will be used. Since the hardening rule describes the evolution of subsequent loading or yielding surfaces, it will be necessary to determine the initial yielding surface, and then any subsequent yielding surface due to additional loading. From Equations 2.29 and 2.31, the plastic volumetric strain change, de_v^p , can then be related to the effective hydrostatic stress change, $d\bar{p}$, as:

$$de_v^p = de_v^t - de_v^e = (A_t - A_e) \frac{d\bar{p}}{\bar{p}} \quad (2.45)$$

and assuming K_o -consolidation in which:

$$X = (1 + 2K_o) \bar{p} \quad (2.46)$$

and

$$dX = (1 + 2K_o) d\bar{p} \quad (2.47)$$

rearranging yields:

$$de_v^p = -(A_t - A_e) \frac{dX}{X} \quad (2.48)$$

Then, the evolution of the cap, dX , can be approximated by Equation 2.48 as:

$$dX = \frac{X_a}{A_t - A_\theta} d\epsilon_v^p \quad (2.49)$$

where X_a is the initial cap position (see Figure 2.6) for a stress state at point A which can also be evaluated for subsequent loadings as follows.

The equations for the ultimate failure (Equation 2.3) and cap (Equation 2.4) surfaces are used to determine L_a .

$$(1 - \alpha^2 R^2) L_a^2 - 2(\bar{I}_{1a} + \alpha \kappa R^2) L_a + (\bar{I}_{1a}^2 + R^2 J_{2a} - R^2 \kappa^2) = 0 \quad (2.50)$$

Then, the cap position X_a at A is:

$$X_a = L_a + (\kappa + \alpha L_a) R \quad (2.51)$$

and the position of the cap at state B, X_b is:

$$X_b = X_a + dX \quad (2.52)$$

where dX and X_a can be obtained respectively from Equations 2.49 and 2.51. The value of the cap aspect ratio is:

$$R = \frac{X_b - \bar{I}_{1f}}{J_{2f}^{\frac{1}{2}}} \quad (2.53)$$

From the above equations, the cap aspect ratio can then be evaluated through a trial-and-error procedure. However, if K_0 equals one, the stress state experiences a hydrostatic condition and the cap aspect ratio can be evaluated after substitution:

$$= \frac{\sqrt{3}m}{2\alpha USR} - \frac{1}{\alpha} + \frac{3\sqrt{3}}{2USR} \left[1 - \frac{2A_e \left(\frac{m}{\alpha} + 2 \frac{USR}{\sqrt{3}\alpha} - 3 \right)}{(A_t - A_e) \left(\frac{m}{\alpha} + 2 \frac{USR}{\sqrt{3}\alpha} + 3 \right)} \right] \quad (2.54)$$

where $m = \frac{\kappa}{\bar{\sigma}_{vo}}$. When $\kappa = 0$, Equation 2.54 simplifies to:

$$R = -\frac{1}{\alpha} + 3 \frac{\sqrt{3}}{2USR} \left[1 - \frac{2A_e \left(2 \frac{USR}{\sqrt{3}\alpha} - 3 \right)}{(A_t - A_e) \left(2 \frac{USR}{\sqrt{3}\alpha} + 3 \right)} \right] \quad (2.55)$$

It should be noted that the foregoing discussion is limited to a triaxial condition only. For the plane strain case, the initial stress state at A can be expressed as:

$$\sigma_{1a} = \bar{\sigma}_{vo} \quad (2.56)$$

$$\sigma_{3a} = K_o \bar{\sigma}_{vo} \quad (2.57)$$

$$\sigma_{2a} = \nu (\sigma_1 + \sigma_3) = \nu (1 + K_o) \bar{\sigma}_{vo} \quad (2.58)$$

Assuming undrained conditions ($\nu = 0.5$), σ_{2a} becomes:

$$\sigma_{2a} = 0.5 (1 + K_o) \bar{\sigma}_{vo} \quad (2.59)$$

From Equations 2.56, 2.57 and 2.58, the values of $\bar{I}_1$ and $J_2^{\frac{1}{2}}$ at point A are obtained as:

$$\bar{I}_{1a} = 1.5 (1 + 2K_o) \bar{\sigma}_{vo} \quad (2.60)$$

$$J_{2a}^{\frac{1}{2}} = 0.5 (1 - K_o) \bar{\sigma}_{vo} \quad (2.61)$$

The same procedure to evaluate the cap aspect ratio under plane strain conditions can be obtained in a similar manner as the triaxial condition.

As for the overconsolidation ratio, OCR , it is used to specify the initial location of the cap along the hydrostatic axis. If only elastic deformation occurs during unloading, the initial cap location for the over-consolidated soil will be n times less than the currently over-consolidated state. The n -value is determined as:

$$n = \frac{(1 + 2K_{on})}{(1 + 2K_{ov})} OCR \quad (2.62)$$

where K_{on} and K_{ov} are respectively the initial coefficients of lateral earth pressure under normally consolidated and over-consolidated conditions.

For determining the initial location and shape of the cap, K_o , S_u and OCR are required. A simple first-order approximation of K_o for normally consolidated (cohesive) soils was proposed by Jaky (1944):

$$K_o = 1 - \sin\phi \quad (2.63)$$

The undrained shear strength, S_u , can be evaluated from undrained triaxial compression tests. The over-consolidated behavior can be evaluated through consolidation test results and correlated to K_o . Additional references for the evaluation of soil properties for the determination of the initial location and shape of the cap can be found in the books mentioned previously or in papers and reports by: Ladd et al. (1977); Mayne (1980); Mayne and Kulhawy (1982); and Kulhawy and Mayne (1990).

2.3.4 Initial Stress State (γ, K_o)

From the boundary geometry, location of the water level and unit weight and coefficient of at-rest lateral earth pressure of the embankment and foundation soils, the initial stress field within the embankment and foundation soils can be constructed. The stress field within the embankment and foundation soils is represented in NFAP as:

$$\sigma_x = A_1 + B_1Y + C_1Z \quad (2.64)$$

$$\sigma_y = A_2 + B_2Y + C_2Z \quad (2.65)$$

$$\sigma_z = A_3 + B_3Y + C_3Z \quad (2.66)$$

$$\sigma_{yz} = A_4 + B_4Y + C_4Z \quad (2.67)$$

where σ_y and σ_z are respectively the normal stresses in the horizontal and vertical directions, σ_{yz} is the shear stress, and σ_x is the normal stress in the longitudinal direction. Constants, A_i, B_i, C_i are dependent on the boundary geometry, location of the ground water level and K_o .

2.3.5 Pore Pressure Response Factor (β)

For soils experiencing either drained or undrained conditions, it is convenient to consider a common formulation to include the effect of pore pressure (e.g. Naylor, et al., 1981; Herrmann, et al., 1982). This can be achieved by superimposing a large bulk modulus over the soil stiffness if slight compressibility of soil is recognized for an undrained condition. For a drained condition (i.e. no pore pressure development), the bulk modulus can be

assumed to be zero while a partially-saturated soil can be managed as a value which is a function of the Henkel pore pressure parameter (Naylor, et al., 1981).

The total stiffness, $\bar{D}_t$, is given by:

$$\bar{D}_t = \bar{D}_s + \bar{D}_f \quad (2.68)$$

where $\bar{D}_t$ is the total stiffness, $\bar{D}_s$ is the soil stiffness and $\bar{D}_f$ is the stiffness contributed by the pore fluid/solid system. $\bar{D}_f$ has the form $[K_{ij}]$ where K_{ij} are the components of the stiffness matrix. Since fluids cannot resist shear distortions, $\bar{D}_f$ has the form:

$$\bar{D}_f = \begin{bmatrix} K_f & K_f & K_f & 0 & 0 & 0 \\ K_f & K_f & K_f & 0 & 0 & 0 \\ K_f & K_f & K_f & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \end{bmatrix} \quad (2.69)$$

where K_f is the apparent bulk modulus of pore fluid/solid system. For a given displacement field, the resistance provided by the soil is

$$[\bar{\sigma}] = \bar{D}_s [\epsilon] \quad (2.70)$$

where the transpose of $\bar{\sigma}$ is

$$[\bar{\sigma}]^T = [\bar{\sigma}_x \ \bar{\sigma}_y \ \bar{\sigma}_z \ \tau_{xy} \ \tau_{yz} \ \tau_{zx}] \quad (2.71)$$

and the transpose of ϵ is

$$[\epsilon]^T = [\epsilon_x \ \epsilon_y \ \epsilon_z \ \gamma_{xy} \ \gamma_{yz} \ \gamma_{zx}] \quad (2.72)$$

The pore pressure developed is

$$\mu = K_f e_v \quad (2.73)$$

where e_v is the total volumetric strain given by

$$e_v = e_x + e_y + e_z \quad (2.74)$$

The total stress $[\sigma]$ is then

$$[\sigma] = [\bar{\sigma}] + \mu [I] \quad (2.75)$$

where

$$[I]_T = [1 \ 1 \ 1 \ 0 \ 0 \ 0] \quad (2.76)$$

The apparent bulk modulus K of the pore fluid/solid particle system for saturated undrained conditions is (Naylor, et al., 1981)

$$\frac{1}{K} = \frac{n}{K_w} + \frac{1-n}{K_s} \quad (2.77)$$

where K_w , K_s are respectively the bulk modulus of the water and solid particles and n is the porosity. Since $K_s \approx 30K_w$ (Lambe and Whitman, 1969), the second term in Equation 2.77 can be neglected with only a few percent error, therefore,

$$K = \frac{K_w}{n} \quad (2.78)$$

The bulk modulus of water is 2.1×10^6 kPa (43×10^6 psf; 300,000 psi). It is convenient to express K_f in terms of the bulk modulus of the soil skeleton K using a pore pressure response factor β which is defined as:

$$\beta = \frac{K_f}{K} \quad (2.79)$$

For partially saturated conditions K_f is related to the Henkel pore pressure parameter (Naylor, et al., 1981)

$$K_f = \frac{KB_h}{1 - B_h} \quad (2.80)$$

where

$$\Delta\mu = B_h \left(\frac{\Delta I_1}{3} \right) + A_h (3J_2)^{\frac{1}{2}} \quad (2.81)$$

A_h and B_h are the Henkel pore pressure parameters (Henkel, 1960; Henkel and Wade, 1966). B_h is zero for dry soil and approaches 1 for saturated soils. Comparison of Equations 2.79 and 2.80 shows that

$$\beta = \frac{B_h}{1 - B_h} \quad (2.82)$$

It may therefore be possible to use β to simulate partially drained conditions.

2.4 Discussion of Sensitivity of Cap Model Parameters

A sensitivity study was performed to show the effect of varying the input soil properties on calculated stress-strain curves, pore pressure response and effective stress paths. The results show which parameters have the greatest effect on computed behavior and provide guidance in the selection and adjustment of input soil properties to obtain a better fit between the calculated and observed response. Sample response calculated using the parameters shown in Table 2.3 and for test SW23-276 (Nwabuokei, 1984) was used as the basis for comparison. Test SW23-276

TABLE 2.3. SUMMARY OF SENSITIVITY ANALYSIS OF THE CAP PLASTICITY MODEL PARAMETERS

PARAMETER CASE	ν	C_c	C_r	β	ϕ	USR	OCR
1A 1B 1C	0.15 0.30 0.45	0.162	0.044	10	24	0.22	1.49
2A 2B 2C	0.45	0.107 0.162 0.240	0.044	10	24	0.22	1.49
3A 3B 3C	0.45	0.162	0.029 0.044 0.066	10	24	0.22	1.49
4A 4B 4C	0.45	0.162	0.044	1 5 10	24	0.22	1.49
5A 5B 5C	0.45	0.162	0.044	10	19 24 30	0.22	1.49
6A 6B 6C	0.45	0.162	0.044	10	24	0.22 0.31 0.43	1.49
7A 7B 7C	0.45	0.162	0.044	10	24	0.22	1.0 1.49 2.25

- Notes: 1. Refer to the figures of the results of the sensitivity study in Appendix B entitled: "A Sensitivity Analysis of the Parameters for a Cap Plasticity Model".
2. For each case, the value of one parameter was varied while the others remained constant.
3. The constant values of each parameter except for the Poisson's Ratio refer to the reference values (observed response) obtained from the laboratory test results. A value of 0.45 rather than 0.49 was used for the Poisson's Ratio to avoid numerical instability.

consisted of an impact-compacted lacustrine clay soil. Compaction was performed in accordance with Standard AASHTO procedures and wet-of-optimum. Following compaction, the clay soil was subjected to a second level of consolidation pressure (isotropically-consolidated undrained-compression conditions; CIUC) of 276 kPa (40 psi) after saturation (via back-pressure) and, then sheared in an undrained condition. The soil properties v , C_c , C_r , ϕ , USR , and OCR were varied individually and the effect on the calculated response was observed. The reference values of the required soil properties for the lacustrine clay are summarized as:

$$\begin{aligned} v &= 0.49 \\ C_c &= 0.162 \\ C_r &= 0.044 \\ \phi &= 24 \\ USR &= 0.22 \\ OCR &= 1.49 \end{aligned}$$

The influence of β was also investigated. The figures of the results of the sensitivity study are presented in Appendix B as part of an earlier report (Ludlow, et al., 1992).

2.4.1 Poisson's Ratio (v)

Three values of Poisson's Ratio, v : 0.15, 0.30 and 0.45 were investigated. From Equation 2.34, Poisson's ratio will affect the elastic shear modulus. A lower v -value results in an increase of elastic shear modulus, which in turn yields a steeper initial stress-strain and excess pore pressure response, as shown in Figure A.1.1 of Appendix B. However, the values have little to no effect at higher axial strain. The effective stress path is independent of the variation of v (Figure A.1.3). This is because the

determination of the cap shape is not related to v .

2.4.2 Compression Index (C_c)

The effect of the compression index with a $\pm 50\%$ variation was examined. Figure A.2.1 illustrates that a lower compression index results in a slightly-higher calculated stress-strain response and a moderately-lower excess pore pressure. Within 2% axial strain, the effects are insignificant. A comparison of the calculated and observed response indicates relatively good agreement for the stress-strain relation with the "best-fit" occurring within the range of 0.107 and 0.162. However, the calculated excess pore pressure in all cases is considerably less than the observed response. The influence on the effective stress path was relatively small.

2.4.3 Recompression Index (C_r)

The influence of the recompression index with a $\pm 50\%$ variation was investigated. From Equations 2.31 and 2.32, C_r will affect the elastic bulk modulus, which results in a considerable change in the stress-strain relation and pore pressure response (Figure A.3.1). A lower C_r results in a steeper initial stress-strain response and a higher pore pressure. However, at larger strains, lower C_r -values result in a slightly lower stress-strain response. A comparison of the calculated and observed response indicates relatively good agreement for the stress-strain relation with the "best-fit" occurring within the range of 0.044 and 0.066. However, the calculated excess pore pressure in all cases is considerably less than the observed

response. The influence on the effective stress path or also cap shape was relatively small (Figure A.3.3).

2.4.4 Pore Pressure Response Factor (β)

The influence of the pore pressure response factor (β : 1, 5 and 10) on the calculated response was examined. It should be noted that a β -value of 10 is recommended by Naylor, et al. (1983) for simulating an undrained condition in practice.

Lower values of β result in a higher stress-strain response and a lower pore pressure (Figure A.4.1). A comparison of the calculated and observed response indicates relatively good agreement for the stress-strain relation with the "best-fit" occurring near 5. However, the calculated excess pore pressure in all cases is considerably less than the observed response. For smaller β -values in which partial drainage is permitted, the results indicate a lower pore pressure response which also shifts the effective stress path to the right (Figure A.4.3).

2.4.5 Angle of Internal Friction (ϕ)

The influence of varying the effective stress angle of internal friction was studied. The parameters corresponding to a $\pm 25\%$ variation of ϕ are shown in Figures A.5.1 and A.5.3.

Observations indicate that increasing ϕ results in a slightly flatter stress-strain curve, causes higher pore pressures at failure and shifts the effective stress path to the left which results in a greater aspect ratio of the cap. A comparison of the calculated and observed response indicates relatively good agreement for the stress-strain relation with the "best-fit"

occurring within the range of 24 and 30 degrees. However, the calculated excess pore pressure in all cases is considerably less than the observed response.

2.4.6 Undrained Shear Strength Ratio (*USR*)

The effect of the *USR* with values of 0.22, 0.31 and 0.43 was investigated; other parameters are given in Table 2.3. Figures A.6.1 and A.6.3 indicate a significant change in the calculated response of the *USR*. A higher *USR* results in a lower aspect ratio of the cap which shifts the effective stress path to the right (Figure A.6.3). For a higher value of *USR*, a shorter path is required and a smaller pore pressure is induced at failure.

2.4.7 Over-consolidation Ratio (*OCR*)

The variation of the *OCR* with values of 1.0, 1.49 and 2.25 was examined. The calculated response for describing the over-consolidated (describes the initial location of the cap along the hydrostatic axis) condition is shown on Figures A.7.1 and A.7.3.

Observations of the figures indicate that an increasing *OCR* causes a significantly steeper and higher stress-strain response. A comparison of the calculated and observed response indicates relatively good agreement for the stress-strain relation with the "best-fit" occurring near an *OCR* value of 1.49. However, the calculated excess pore pressure is considerably less than the observed response except in the case where the *OCR* has a value of 2.25 and limited in strain.

2.5 Limitations of the Cap Model

The cap model has some limitations which impact the calculated

response. One limitation which was observed during this study was the inability of the model to predict the reduction in undrained strength or increase in pore pressure that occurs after the peak strength is reached in strain softening soil. Consequently, this would overestimate the shear strength and underestimate the pore pressures and deformations.

A second limitation is the ability to model the behavior of over-consolidated soils. The cap model correctly predicts the undrained strength at large strains. However, pore pressures are not predicted correctly and plastic strains which many over-consolidated soils experience for stress changes in the elastic region bounded by the cap and ultimate failure surfaces are not accounted for. Also, strain softening after failure can not be modeled. It is speculated that this would lead to underestimation of deformations.

Another limitation which can be attributed to the model's formulation is the need to perform a trial-and-error calculation to estimate the best-fit of the observed response. Although time-consuming, a reasonable fit of the observed response can be obtained rather quickly by making a critical review of the OCR and USR parameters and then modifying one of the other five (from the previous sensitivity study) parameters, if necessary.

Another limitation which requires considerable attention and can also be attributed to the model's formulation is the model's inability to account for long-term behavior (e.g. creep and consolidation). Presently, the model is best-suited for depicting

short-term behavior only. To account for creep and consolidation, the model would require extensive modifications.

2.6 Summary

A straight forward procedure to determine the cap parameters for normally consolidated soils from conventional laboratory and field tests and a sensitivity study which examines the effect of the input soil parameters was provided. The main input soil properties are the compressibilities (C_c, C_r), the effective Mohr-Coulomb shear strength parameters ($\bar{\phi}, \bar{c}$), the undrained shear strength ratio ($USR; \frac{S_u}{\sigma_{vo}}$) and the over-consolidation ratio (OCR). Solutions are given in graphical form and equations suitable for hand calculation.

The procedure was used to determine the cap parameters for an impact-compacted lacustrine clay using results from isotropically-consolidated undrained-compression (CIUC) tests. These parameters were then used in a computer program called CAP to calculate stress-strain curves, pore pressure response and effective stress paths. Comparisons were made to observed test results. In general, there was good agreement except for a discrepancy for pore pressures and effective stress paths. The discrepancy is at least partially because this formulation of the cap model does not allow plastic volumetric strain for stress changes within the region bounded by the cap and ultimate failure surfaces.

In addition, a sensitivity study was made of the effect of the input soil properties on calculated CIUC triaxial sample behavior. The results show which parameters have the greatest effect on

computed behavior and provide guidance in the selection and adjustment of input soil properties to obtain a better fit between the calculated and observed response. The *USR* and *OCR* were observed to have the most-significant influence on the predicted behavior.

CHAPTER 3

APPLICATION OF THE FINITE ELEMENT METHOD TO NFAP

3.1 Introduction

The software used in this study is based on a general purpose finite element program named NFAP (nonlinear finite analysis program) developed by Chang (1980). NFAP was originally developed for the use of nonlinear large deformation analysis of structures. However, the program was later expanded and refined by Mizuno (1981) and McCarron and Chen (1985) to include a cap plasticity model to simulate the behavior of soils. Then, Humphrey (1986) condensed McCarron's version of the program for use on personal computers.

During Phase I of this research study, Huang (1991) modified and extended NFAP to include: double precision/floating point variables; foundations with sloping boundaries and a variable groundwater level; and a procedure for determining the initial/existing stress state within an embankment.

NFAP employs a cap plasticity model, as discussed in the previous chapter, to simulate the behavior of the soil. From a theoretical point of view, the cap model employed herein is particularly appropriate in modeling soil behavior, because it is capable of treating the conditions of stress history, stress path dependency, dilatancy (positive and negative), and the effect of the intermediate principal stress.

The program performs an incremental load-displacement analysis. After each increment of load is applied, the

displacement field is modified using an iterative procedure (The Newton-Raphson or the modified Newton-Raphson can be selected.) until an equilibrium configuration is reached. Convergence is based on the difference between two successive displacement norms, and 1% is used in the case studies of Chapter 4.

NFAP also incorporates a large strain deformation analysis which is based on the assumption of small strain and large rotation, for which the structural stiffness of the global equilibrium equations is symmetric. For solution of the equilibrium equations, gaussian quadrature using second or third order integration is recommended. In addition, a node-renumbering system in NFAP is introduced to minimize the band-width of the global stiffness matrix and improve solution efficiency. It should also be noted that, presently, NFAP is only capable of accepting a finite element mesh with 500 or fewer nodes. However, there are no restrictions to the number of elements. If necessary, NFAP can be easily modified to accommodate a mesh that may have more than 500 nodes. This can be achieved by changing the FORTRAN code "common" statement within the subroutine "GIBB" to the number of nodes that are required. Presently, the size of the one-dimensional arrays are established at 500.

3.2 Modeling Foundation Soils

In NFAP, the foundation soils are modeled with two-dimensional 4-, 6- or 8-node isoparametric (implying that the shape functions for the displacements are the same as those for coordinate transformation) plane strain continuum elements. Figure 3.1

illustrates a typical finite element mesh. Soil behavior is represented by the strain-hardening cap model (which is time and temperature independent) described in the previous chapter or a linear elastic response by specifying Young's modulus and Poisson's ratio. In addition, cap parameters for the foundation soils were chosen using the procedure given in Chapter 2. The linear elastic response was not discussed herein since the behavior of soils is often non-linear and inelastic and because one of the major thrusts of this study was to analyze practical engineering problems using the cap model. However, it should be noted that the solutions of linear elastic models are often used as a benchmark to evaluate other results from more sophisticated models (e.g. cap plasticity model).

3.3 Modeling Embankment Fill

The embankment fill is also modeled with two-dimensional 4-, 6- or 8-node isoparametric plane strain continuum elements. Again, soil behavior is represented by the strain-hardening cap model described in the previous chapter or a linear elastic response by specifying Young's modulus and Poisson's ratio. Just as in the case of the foundation soil, use of the cap model would also require knowledge of the compaction-induced preconsolidation pressure or compactive prestress (stiffness related) and at-rest lateral earth pressure coefficient in the compacted fill. For compacted clay fills, these are generally unknown. Therefore, plastic strains within the soil mass may not be accurately predicted by the model. In addition, finite element studies (e.g.

Figure 3.1 Typical finite element representation

Rowe, 1982; Kwok, 1987; Hird and Jewell, 1989) have shown that an increase in the embankment stiffness can modestly reduce the deformations in the soft foundation soils.

3.4 Modeling Reinforcement

The reinforcement is modeled with 2 or 3 node one-dimensional truss elements which can only sustain axial tensile load. One- or multiple-layer reinforcement can be modeled during embankment construction. The reinforcement behavior is modeled as a linear elastic or non-linear elastic response where the stress-strain relation of the reinforcement is specified. For this study, a non-linear elastic response of the reinforcement was used in the case studies of Chapter 4.

Slippage at the embankment fill-reinforcement or reinforcement-foundation soil interfaces cannot be explicitly modeled by NFAP (i.e. assumes perfect adherence between the soil/fill and reinforcement). This assumption is adequate provided: (1) the shear stresses developed at the interface are less than the interface strength; or (2) the soil-reinforcement interface shear strength equals or exceeds the soil-soil shear strength. The latter case was assumed for this study. It implies that slip will occur in the adjacent soil when the shear strength is exceeded rather than at the interface. This may underestimate the interface movement. Experimental studies reported in the literature indicate soil-reinforcement interface resistance is often greater than 80% of the soil shear strength.

However, if the interaction between the soil mass (embankment

fill and foundation soil) and the reinforcement became a concern for a particular application, three approaches could be considered of which two would require extensive modification to the existing program. The approaches are: (1) the use of joint elements (Al-Hussaini and Johnson, 1978); (2) nodal-compatibility slip elements; or (3) substructuring which involves refinement of the mesh and soil element properties at and near the interface of the reinforcement (applicable to NFAP).

Common approaches to modeling the soil reinforcement interface by the joint element involve three nodes at each point along the reinforcement; one attached to the soil above the reinforcement, one on the reinforcement, and one to the soil below the reinforcement. The nodal-compatibility slip element which may be formulated initially in terms of normal and tangential (shear) springs with very high stiffnesses:

1. ensures compatible displacement between a pair of dual nodes until some failure criterion is reached; and
2. replaces the compatibility conditions by a failure condition and dilatancy equation once the interface strength is exceeded.

Joint elements allow relative deformation of the soil and reinforcement, prior to failure of the interface. This is based on some assumed constitutive relationship of what is in effect on the interface layer between the reinforcement and the soil continuum. The joint element may be modelled as a pair of normal and tangential springs. As the stiffness of a joint element increases, it tends to a nodal-compatibility slip element. The distinction between the two is related to the question of whether a discrete

interface layer exists or whether the deformations at the interface (prior to failure) are simply due to the interaction between the reinforcement and the soil on the either side of the reinforcement.

According to Rowe (1988), any modeling of the interface behavior must consider three possible mechanisms of failure as noted below.

1. If there is insufficient anchorage capacity, failure will occur at the soil reinforcement interface above and below the reinforcement as the reinforcement is pulled out of the soil. This "pullout" mode involves displacement of the reinforcement relative to the soil on both sides of the reinforcement.
2. If the shear strength of the soil reinforcement is less than the shear strength of the soil alone, then failure may occur by sliding of the soil along the upper surface of the reinforcement and the upper soil mass moves relative to both the reinforcement and the underlying soil.
3. The soil below the reinforcement (for cases of soft foundation soils) may be "squeezed out" laterally from beneath the lowest reinforcement layer. In this case, the lower soil may move relative to the reinforcement and the overlying soil.

In the case of geotextiles, since the reinforcement is in the form of a sheet, which completely separates the soil above and below the reinforcement, the interface resistance can be readily determined by direct shear tests (Rowe et.al., 1985). In this case, provision for slip at the interface is the same irrespective of the mechanism of failure (that is, direct shear or pullout). This difference, in addition to boundary effects and stress concentrations, makes the pull-out test results difficult to analyze. It is noted that during direct shear tests the geotextile reinforcement experiences simple shear deformation instead of

tensile deformation developed in the pull-out mode. However, if the reinforcement consists of a geogrid, with openings that are large compared to the grain size of the soil, or if the reinforcement consists of separate reinforcing strips, then special care is required to correctly model the failure mechanism.

For planar reinforcement, independent movement of the soil may occur above and below the reinforcement following either a direct shear or pullout failure. On the hand, for strip reinforcement, independent movement of the soil above and below the plane of reinforcement can only occur during a direct shear mode of failure (Rowe et.al., 1985). Pullout of strips is really a three dimensional phenomenon in which the strips move relative to the soil around them but the soil between strips remains continuous. As noted by Naylor and Richards (1978), the common approach of using a conventional joint element (or nodal compatibility element) implicitly treats the strips as an equivalent two dimensional sheet and will cause error since it interrupts the transfer of shear stress through the soil.

3.5 Incremental Construction

Construction of the embankment is simulated using an incremental loading technique (McCarron, 1985) which does not directly account for compactive prestress during embankment construction. The embankment is usually divided into horizontal layers each represented by a row of elements with zero initial stress. During each increment, the gravity stresses in the next layer are increased from zero to their full value in one or more

subincrements (which are specified by the user). Additional layers are then applied until construction of the embankment is complete and/or failure of the side slope or foundation occurs.

There is a trade-off between layer thickness and the number of subincrements. Thicker layers require more subincrements to achieve a stable solution (Naylor, et al., 1981), so there is little savings in solution time by using excessively thick layers. Thicker layers also result in a stiffer embankment response. It is not necessary to represent each construction lift (typically 6 to 8 inches thick) by a layer of elements. Provided the thickness of the layers is chosen so that 4 to 5 layers are applied prior to embankment failure, computed behavior is approximately the same as obtained with thinner layers. This recommendation should be re-examined for layers that are many times thicker than the construction lift.

As stated previously, the incremental loading technique used in NFAP does not directly account for stresses produced by the compaction equipment during construction of the embankment. Compaction prestress which is analogous to preconsolidation stress represents the fraction of the compaction energy which is effectively transmitted to the soil matrix due to plastic straining/deformations. To account for compactive prestresses, knowledge of the compressibility and shear strength responses would be required within defined layers of the embankment. Often, this is not known. One approach, again which would require knowledge of the compressibility and shear strength responses of the fill (which

is limited to cohesive type soils only), is SHANSEP (Soil History and Normalized Soil Engineering Properties).

3.6 Concluding Remarks

The following comments are also related to the application of the FEM and should be regarded as general guidelines for the setup and analysis of practical engineering problems. These points include:

1. Create a general subsurface (soil/groundwater) profile from the test boring information and appropriate laboratory test results;
2. Determine the cap parameters from the procedure outlined in the previous chapter;
3. Input the parameters from step 2 into the CAP program and perform the analysis;
4. Compare the computed (from the CAP program) and observed (from the laboratory test results) results and iterate to obtain the "best-fit";
5. Draw a finite element mesh showing the geometric boundaries and the different soil layers (regions);
 - Draw the elements as square as possible keeping an aspect ratio (length to height) of the element less than 2L:1H;
 - Avoid inverted elements (interior angle of any element must be less than 180 degrees);
 - Avoid triangle elements in a 4-node element scheme (if needed, use them in a region away from the zone of interest);
 - Keep the mesh fine near the region of interest or where changes in shear stress and strain are significant;
 - Avoid sudden jumps in element size;
 - Ideally model the subsurface profile to a relatively incompressible layer (e.g. rock or dense sand and gravel) or model the bottom foundation layer to a depth of two to three times the

embankment height and/or width;

- Ideally model the foundation layer to a distance away from the embankment toe of two to four times the embankment width;
 - Take advantage of embankment symmetry, if possible;
 - Minimize bandwidth of global element stiffness matrix by decreasing the maximum difference of the node numbering in each element;
 - When performing an incremental analysis, simulate the construction sequencing as much as possible; and
 - Keep it simple (i.e. do not try to model every detail of the soil-embankment system);
6. Setup the data file and check the data input; and
 7. Perform the finite element analysis and interpret and check the results. Verification of the solution should include:
 - A. A check against a well-documented case history or physical model;
 - B. Simple hand calculation; and
 - C. Reasonable checks of output including:
 - effective vertical stress;
 - effective horizontal stress (for rough case only);
 - stress level;
 - boundary conditions and deformed mesh;
 - shear strain; and
 - orientation and magnitude of principal stresses (best check).

A few additional notes on the application of the FEM include:

1. The FEM is relatively good at predicting stresses but somewhat poor at estimating strains and displacements;
2. The FEM formulation over-estimates the global element

stiffness matrix because of the assumption that displacements across an element are defined by some interpolation function of the displacements at the nodes of the element; and

3. Four primary errors associated with the FEM include:
 - error due to numerical integration (slightly under-estimates the stiffness);
 - error due to above-mentioned item 2 (over-estimates the stiffness);
 - FEM used to analyze boundary value problem (i.e. error from not accurately modeling system geometry and boundary conditions); and
 - error due to constitutive law from not modeling the soil behavior (dominant).

CHAPTER 4

CASE STUDIES

4.1 Introduction

This chapter summarizes the case studies contained in this report and describes the implementation procedures, which are outlined in Chapters 2 and 3, that were used to setup each example. The examples in this chapter are based on actual highway projects in Indiana where information on these projects was provided by INDOT personnel. These examples include: (1) State Route 55 over Turkey Creek in Lake County; and (2) State Route 1 over Ramsey Creek in Franklin County.

The influence of several factors on the short term behavior of reinforced and unreinforced embankments was studied. The influence of reinforcement was assessed by comparing behavior of reinforced and unreinforced embankments. A total of 30 cases (12 cases for State Route 55 and 18 cases for State Route 1) including various embankment geometries and soil properties was considered. The effect of the crust strength, foundation thickness and compressibility and embankment width and widening and grade raising were examined.

4.2 Example 1 - State Route 55 over Turkey Creek in Lake County

This example involved a 13-ft high and 60-ft wide (at the top) approach embankment for State Route 55 over Turkey Creek in Lake County, Indiana. A representative section was assumed near Station 52+00 along the centerline of the proposed roadway. Again, information regarding this example was provided by INDOT personnel

[Project No. ST-4145(B), Structure No. 55-45-7366] and summarized in a geotechnical report prepared by Engineering Testing Service (1989). For this case, engineering properties of the foundation soils and embankment fill were limited and as a result, many of the cap model parameters were estimated from empirical correlations. The rationale for estimating these parameters is contained in Appendix C (entitled Interim Report: "Embankment Widening and Grade Raising on Soft Foundation Soils: Example 1 - Indiana State Route 55 over Turkey Creek in Lake County, Indiana") and a general soil profile is summarized in Table 4.1.

As stated previously, 12 cases were analyzed for this example. In all cases, the groundwater level was assumed to be at the ground surface. Typically, the drainage conditions during simulation of the embankment construction were assumed to be undrained except for case 2 where drainage of the embankment fill was permitted.

In all of the cases except for three, the finite element representation of the embankment fill and foundation soil consisted of 349 nodes and 102 eight-node isoparametric plane strain continuum elements (Figure 4.1). The exceptions consisted either of: (1) 355 nodes and 104 eight-node isoparametric plane strain continuum elements [case 5 - which included the construction of a stabilizing berm (Figure 4.2)]; or (2) 397 nodes and 118 eight-node isoparametric plane strain continuum elements [cases 6 and 8 - which included embankment widening and grade raising (Figure 4.3)]. It should also be noted that the embankment construction was simulated with eight 1.625-ft (0.5 m) thick lifts (or sixteen 0.81-

Table 4.1. General Soil Profile for State Route 55

Cap Model Parameter	Foundation Layer/Value ¹			
	1	2	3	4
K_{min}	300 ksf	300 ksf	300 ksf	300 ksf
ν	0.45	0.45	0.45	0.45
e_o	0.700	0.700	0.700	0.700
$\bar{\phi}$	33°	33°	33°	33°
$\bar{c}$	0 psf	0 psf	0 psf	0 psf
C_c	0.246	0.246	0.246	0.246
C_r	0.0246	0.0246	0.0246	0.0246
$\frac{S_u}{\bar{\sigma}_o}$	0.36	0.36	0.36	0.36
OCR	3.0	2.5	1.5	1.0
γ	125 pcf	125 pcf	125 pcf	125 pcf
K_o	0.71	0.66	0.54	0.46
β	10	10	10	10

- Notes:**
1. Refer to Figure 4.1 for location of foundation layers.
 2. Refer to Geotechnical Investigation of State Route 55 (1989) for additional soils information.
 3. Refer to Chapter 2 for additional information regarding the cap plasticity model.
 4. Refer to Appendix C for rationale for estimating the cap model parameters.

Notes:

1. 355 Nodes;
104 eight-node isoparametric
plane strain elements
Cases 5 includes the construction of a
stabilizing berm
2. Refer to Table 4.2 for additional information
- 3.

Project No. ST-4145(B)
 Structure No. 55-45-7366
 SR 55 over Turkey Creek
 In Lake County, Indiana
 Station: 52+00
 Scale : 1" = 10'

Figure 4.2 Finite element representation for Case 5

Project No. ST-4145(B)
 Structure No. 55-45-7366
 SR 55 over Turkey Creek
 in Lake County, Indiana
 Station: 52+00
 Scale: 1" = 10'

Figure 4.3 Finite element representation for Cases 6 and 8

ft thick subincrements). Figures 4.1 through 4.3 illustrate the finite element representation of the embankment/foundation soil geometries.

Cases 3, 4, 8, 10, 11 and 12 involved the use of either a low-modulus nonwoven geotextile (LMNG) or a high-modulus woven geotextile (HMGW) placed either: at the mid-height of the embankment and embankment/foundation interface; or at the embankment/foundation interface only. The geotextile was modeled with a 3-node one-dimensional truss element (non-linear elastic stress-strain relation) as discussed in Chapter 3. A "non-linear elastic stress-strain response" implies that the strains are recoverable and proportional to the non-linear response of the material. Stress-strain relations of the geotextile were obtained from Koerner (1990). Refer to Table 4.2 for a summary of the cases.

4.3 Example 2 - State Route 1 over Ramsey Creek in Franklin County

This example involved a 90-ft high and 50-ft wide (at the top) embankment for State Route 1 in Franklin County, Indiana. A representative section was assumed near Station 1226+90 along the centerline of the proposed roadway. Information regarding this example was provided by INDOT personnel [Project No. RS-5124(2)] and summarized in a geotechnical report prepared by The H.C. Nutting Company (1986). For this case, engineering properties of the foundation soils were available from laboratory tests (e.g. isotropically-consolidated undrained compression and consolidation test results) and as a result, many of the cap model parameters were estimated from the test results. After obtaining the

TABLE 4.2. EXAMPLE 1 - SUMMARY OF CASES FOR INDIANA STATE ROUTE 55 OVER TURKEY CREEK/LAKE COUNTY, INDIANA				
CONDITION(S) CASE	DRAINAGE CONDITIONS DURING EMBANKMENT CONSTRUCTION	EMBANKMENT CONSTRUCTION AND SIMULATION	FINITE ELEMENT REPRESENTATION	COMMENTS
CASE 1	Undrained conditions with the water level at the ground surface.	Gravity build-up with eight 1.625-ft (.5 m) thick lifts.	349 nodes and 102 eight-noded isoparametric plane strain solid elements.	Refer to Appendix A.1 for rational soil parameters. Slope angle 1.5H to 1V.
CASE 2	Undrained and drained conditions for the foundation soil and embankment fill, respectively. Water level at the ground surface.	Gravity build-up with eight 1.625-ft (.5 m) thick lifts.	349 nodes and 102 eight-noded isoparametric plane strain solid elements.	
CASE 3	Undrained conditions with the water level at the ground surface.	Gravity build-up with placement of a low-modulus nonwoven geotextile (LMNG) at the interface of the embankment fill and foundation soil.	349 nodes and 102 eight-noded isoparametric plane strain and 10 three-noded truss elements.	
CASE 4	Undrained conditions with the water level at the ground surface.	Gravity build-up with placement of two LMNG layers: one each at the interface of the embankment fill and foundation soil and at the mid-height of the embankment.	349 nodes and 102 eight-noded isoparametric plane strain and 18 three-noded truss elements.	

TABLE 4.2. cont'd EXAMPLE 1 - SUMMARY OF CASES FOR INDIANA STATE ROUTE 55 OVER TURKEY CREEK/LAKE COUNTY, INDIANA				
CONDITION(S) CASE	DRAINAGE CONDITIONS DURING EMBANKMENT CONSTRUCTION	EMBANKMENT CONSTRUCTION AND SIMULATION	FINITE ELEMENT REPRESENTATION	COMMENTS
CASE 5	Undrained conditions with the water level at the ground surface.	Gravity build-up with eight 1.625-ft (.5 m) thick lifts. Construction of a 3.25-ft (1 m) high and 10-ft (3.1 m) wide stabilizing berm.	355 nodes and 104 eight-noded isoparametric plane strain solid elements.	Stabilizing berm constructed simultaneously with the lower two lifts of the embankment.
CASE 6	Undrained conditions with the water level at the ground surface.	Gravity build-up of the original embankment prior to widening and grade raising. Construction simulated with sixteen 1.625-ft (.5 m) thick lifts.	397 nodes and 118 eight-noded isoparametric plane strain solid elements. Perform analysis storing the initial stress state of the original embankment prior to widening and grade raising.	Widening (10 ft; 3.1 m) and grade raising (3.25 ft; .5 m) of the existing embankment. New embankment height and width of 16.25 ft (5 m) and 30 ft (9.1 m), respectively.
CASE 7	Undrained conditions with the water level at the ground surface.	Gravity build-up with eight 1.625-ft (.5 m) thick lifts.	349 nodes and 102 eight-noded isoparametric plane strain solid elements.	Similar to Case 1 except the over-consolidation ratio was modified.
CASE 8	Undrained conditions with the water level at the ground surface.	Gravity build-up of the original embankment prior to widening and grade raising. Construction simulated with sixteen 1.625-ft (.5 m) thick lifts.	397 nodes and 118 eight-noded isoparametric plane strain solid elements. Perform analysis storing the initial stress state of the original embankment prior to widening and grade raising.	Placement of a LMNG layer at the interface of the embankment fill and foundation soil. Similar to Case 6 except for geotextile.

TABLE 4.2. cont'd EXAMPLE 1 - SUMMARY OF CASES FOR INDIANA STATE ROUTE 55 OVER TURKEY CREEK/LAKE COUNTY, INDIANA				
CONDITION(S) CASE	DRAINAGE CONDITIONS DURING EMBANKMENT CONSTRUCTION	EMBANKMENT CONSTRUCTION AND SIMULATION	FINITE ELEMENT REPRESENTATION	COMMENTS
CASE 9	Undrained conditions with the water level at the ground surface.	Gravity build-up with eight 1.625-ft (.5 m) thick lifts.	349 nodes and 102 eight-noded isoparametric plane strain solid elements.	Similar to Case 7 except the undrained shear strength ratio and effective strength parameters were modified.
CASE 10	Undrained conditions with the water level at the ground surface.	Gravity build-up with placement of a high-modulus woven geotextile (HMG) at the interface of the embankment fill and foundation soil.	349 nodes and 102 eight-noded isoparametric plane strain and 10 three-noded truss elements.	Similar to Case 3 except for geotextile type and modulus.
CASE 11	Undrained conditions with the water level at the ground surface.	Gravity build-up with placement of two HMG layers: one each at the interface of the embankment fill and foundation soil and at the mid-height of the embankment.	349 nodes and 102 eight-noded isoparametric plane strain and 18 three-noded truss elements.	Similar to Case 4 except for geotextile type and modulus.
CASE 12	Undrained conditions with the water level at the ground surface.	Gravity build-up with placement of a HMG at the interface of the embankment fill and foundation soil.	349 nodes and 102 eight-noded isoparametric plane strain and 10 three-noded truss elements.	Similar to Case 7 except for incorporation of a geotextile.

appropriate information from the laboratory test results, a comparison of the calculated and observed (from laboratory test results) responses was performed using the CAP program. Several iterations were performed to obtain the "best-fit" with results shown in Appendix D (entitled Interim Report: "Embankment Widening and Grade Raising on Soft Foundation Soils: Example 2 - Indiana State Route 1 over Ramsey Creek in Franklin County, Indiana"). A general soil profile is also provided for review in Table 4.3.

As stated previously, 18 cases were analyzed for this example. In all cases, the groundwater level was assumed to be at 11.5 ft below the existing ground surface except for case 3 where the groundwater level was assumed to be at 7.5 ft. Typically, the drainage conditions during simulation of the embankment construction were assumed to be undrained and drained for the foundation soils and embankment fill, respectively. Based on the geotechnical report, rock fill from roadway excavations would be used to construct the embankment fill.

In all of the cases, the finite element representation of the embankment fill and foundation soil consisted of 460 nodes and 409 four-node subparametric plane strain continuum elements. It should also be noted that the embankment construction was simulated with eighteen 5.0-ft (1.52 m) thick lifts. Figure 4.4 illustrates the finite element representation of the embankment/foundation soil geometries. This example focuses more on the effect of the reinforcement and the foundation soil properties rather than the issue of embankment widening and grade raising. This is because

Table 4.3. General Soil Profile for State Route 1							
Cap Model Parameter	Foundation Layer/Value ¹						
	1	2	3	4	5	6	7
K_{min}	300 ²	300 ²	300 ²	300 ²	300 ²	300 ²	300 ²
ν	0.40	0.40	0.40	0.40	0.40	0.40	0.40
e_o	0.700	0.700	0.700	0.700	0.700	0.700	0.700
$\bar{\phi}$	21	21	21	21	21	21	21
$\bar{c}$	0.720 ²	0.720 ²	0.720 ²	0.720 ²	0.720 ²	0.720 ²	0.720 ²
C_c	0.230	0.250	0.250	0.250	0.250	0.250	0.124
C_r	0.039	0.050	0.050	0.050	0.050	0.020	0.020
$\frac{S_u}{\bar{\sigma}_o}$	1.58	1.46	1.07	0.96	0.87	0.80	0.63
OCR	8.0	7.2	4.9	4.3	3.8	3.4	2.5
γ	0.125 ²	0.120 ²	0.120 ²	0.120 ²	0.120 ²	0.120 ²	0.120 ²
K_o	1.50	1.44	1.23	1.17	1.11	1.06	0.94
β	10	10	10	10	10	10	10

Notes:

1. Refer to Figure 4.4 for location of foundation layers.
2. Units of ksf.
3. Refer to Geotechnical Investigation of State Route 1 (1986) for additional soils information.

Figure 4.4 Typical finite element representation for State Route 1

the number of nodes used in this example was near the maximum number (500) permitted by NFAP. Further modification of the mesh for embankment widening and grade raising would require additional nodes which would have exceeded the capacity of the program.

Cases 4, 5, 6, 7, 8, and 12 through 17 involved the use of either a LMNG or a HMWG placed either: at the 30-ft and 60-ft height of the embankment and at the embankment/foundation interface; or at the embankment/foundation interface only. The geotextile was modeled with a 2 node one-dimensional truss element (non-linear elastic stress-strain relation). Again, stress-strain relations of the geotextile were obtained from Koerner (1990). Refer to Table 4.4 for a summary of the cases.

TABLE 4.4. EXAMPLE 2 - SUMMARY OF CASES
FOR INDIANA STATE ROUTE 1 OVER RAMSEY CREEK/FRANKLIN COUNTY, INDIANA

CONDITION(S) CASE	DRAINAGE CONDITIONS DURING EMBANKMENT CONSTRUCTION	EMBANKMENT CONSTRUCTION AND SIMULATION	FINITE ELEMENT REPRESENTATION	COMMENTS
CASE 1	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52 m) thick lifts.	460 nodes and 409 four-noded subparametric plane strain solid elements.	Refer to Appendix D for estimated soil response using a cap plasticity model. Slope angle 3H to 1V. Seven foundation soil layers.
CASE 2	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up to Elevation 700 with thirteen 5-ft (1.52 m) thick lifts.	460 nodes and 409 four-noded subparametric plane strain solid elements. Turn gravity off after lift 13.	
CASE 3	Drained conditions with the water level at 7.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52m) thick lifts.	460 nodes and 409 four-noded subparametric plane strain elements.	
CASE 4	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with placement of one low-modulus nonwoven geotextile (LMNG) at the interface of the embankment fill and foundation soil.	460 nodes and 409 four-noded subparametric plane strain and 20 two-noded truss elements.	

TABLE 4.4. cont'd EXAMPLE 2 - SUMMARY OF CASES FOR INDIANA STATE ROUTE 1 OVER RAMSEY CREEK/FRANKLIN COUNTY, INDIANA				
CONDITION(S) CASE	DRAINAGE CONDITIONS DURING EMBANKMENT CONSTRUCTION	EMBANKMENT CONSTRUCTION AND SIMULATION	FINITE ELEMENT REPRESENTATION	COMMENTS
CASE 5	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52 m) thick lifts. Placement of 3 LMNG layers; one each at the interface of the embankment fill and foundation soil and at the 30-ft and 60-ft heights of the embankment.	460 nodes and 409 four-noded subparametric plane strain solid and 42 two-noded truss elements.	Modified the OCR and USR of the embankment fill.
CASE 6	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52 m) thick lifts. Placement of a LMNG layer at the interface of the embankment fill and foundation soil.	460 nodes and 409 four-noded subparametric plane strain solid and 22 two-noded truss elements.	Similar to Cases 3 and 4 (combined).
CASE 7	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52 m) thick lifts. Placement of a high-modulus woven geotextile (HMWG) at the interface of the embankment fill and foundation soil.	460 nodes and 409 four-noded subparametric plane strain solid and 22 two-noded truss elements.	Similar to Case 4 except for the geotextile type.

TABLE 4.4. cont'd EXAMPLE 2 - SUMMARY OF CASES
FOR INDIANA STATE ROUTE 1 OVER RAMSEY CREEK/FRANKLIN COUNTY, INDIANA

CONDITION(S) CASE	DRAINAGE CONDITIONS DURING EMBANKMENT CONSTRUCTION	EMBANKMENT CONSTRUCTION AND SIMULATION	FINITE ELEMENT REPRESENTATION	COMMENTS
CASE 8	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52 m) thick lifts. Placement of 3 HWG layers; one each at the interface of the embankment fill and foundation soil and at the 30-ft and 60-ft heights of the embankment.	460 nodes and 409 four-noded subparametric plane strain solid and 42 two-noded truss elements.	Similar to Case 5 except for the geotextile type.
CASE 9	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52m) thick lifts.	460 nodes and 409 four-noded subparametric plane strain elements.	Similar to Case 1 except OCR of embankment fill has been modified.
CASE 10	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52m) thick lifts.	460 nodes and 409 four-noded subparametric plane strain elements.	Similar to Case 9 except the OCR and USR of the crust (uppermost soil layer) has been modified.
CASE 11	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52 m) thick lifts.	460 nodes and 409 four-noded subparametric plane strain elements.	Similar to Case 10 except the OCR and USR of the underlying soil profile has been modified.

TABLE 4.4. cont'd EXAMPLE 2 - SUMMARY OF CASES FOR INDIANA STATE ROUTE 1 OVER RAMSEY CREEK/FRANKLIN COUNTY, INDIANA				
CONDITION(S) CASE	DRAINAGE CONDITIONS DURING EMBANKMENT CONSTRUCTION	EMBANKMENT CONSTRUCTION AND SIMULATION	FINITE ELEMENT REPRESENTATION	COMMENTS
CASE 12	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52 m) thick lifts. Placement of a high-modulus woven geotextile (HMG) at the interface of the embankment fill and foundation soil.	460 nodes and 409 four-noded subparametric plane strain solid and 22 two-noded truss elements.	Similar to Case 9 except for incorporation of geotextile.
CASE 13	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52 m) thick lifts. Placement of a LMNG layer at the interface of the embankment fill and foundation soil.	460 nodes and 409 four-noded subparametric plane strain solid and 22 two-noded truss elements.	Similar to Case 4 except for incorporation of geotextile.
CASE 14	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52 m) thick lifts. Placement of a high-modulus woven geotextile (HMG) at the interface of the embankment fill and foundation soil.	460 nodes and 409 four-noded subparametric plane strain solid and 22 two-noded truss elements.	Similar to Case 10 except for incorporation of geotextile.

TABLE 4.4. cont'd EXAMPLE 2 - SUMMARY OF CASES FOR INDIANA STATE ROUTE 1 OVER RAMSEY CREEK/FRANKLIN COUNTY, INDIANA				
CONDITION(S) CASE	DRAINAGE CONDITIONS DURING EMBANKMENT CONSTRUCTION	EMBANKMENT CONSTRUCTION AND SIMULATION	FINITE ELEMENT REPRESENTATION	COMMENTS
CASE 15	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52 m) thick lifts. Placement of a LMNG layer at the interface of the embankment fill and foundation soil.	460 nodes and 409 four-noded subparametric plane strain solid and 22 two-noded truss elements.	Similar to Case 10 except for incorporation of geotextile.
CASE 16	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52 m) thick lifts. Placement of a high-modulus woven geotextile (HMG) at the interface of the embankment fill and foundation soil.	460 nodes and 409 four-noded subparametric plane strain solid and 22 two-noded truss elements.	Similar to Case 11 except for incorporation of geotextile.
CASE 17	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52 m) thick lifts. Placement of a LMNG layer at the interface of the embankment fill and foundation soil.	460 nodes and 409 four-noded subparametric plane strain solid and 22 two-noded truss elements.	Similar to Case 11 except for incorporation of geotextile.
CASE 18	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52 m) thick lifts.	460 nodes and 409 four-noded subparametric plane strain solid and 22 two-noded truss elements.	Modified the OCR and USR of the embankment fill.

CHAPTER 5

DESIGN GUIDELINES

5.1 Introduction

This chapter investigates the effect of reinforcement, crust strength, stability, foundation compressibility, embankment width and embankment widening and grade raising on short term deformations and stresses to identify which aspects of embankment geometry and foundation properties have the greatest effect on the overall behavior. To evaluate these effects, 30 cases as described in the previous chapter were considered. Although a significant number of cases were analyzed with NFAP, the effect of the above-mentioned aspects, which are discussed herein, provides merely a qualitative assessment of the behavior. During the design process, the results of NFAP should be only used to assess/check the design and not be used as a "stand alone" design tool.

To discern these effects, computer-generated contour plots of the horizontal and vertical displacements, principal stress ratios and local factors of safety were created. These plots for examples 1 (State Route 55) and 2 (State Route 1) are provided in Appendices C and D, respectively.

After executing the program NFAP, an output file known as "TTT" is generated in ASCII format. The file contains information regarding coordinates of the nodal points (original mesh), nodal connectivity, coordinates of the deformed mesh and local factors of safety (defined as the ratio between the available shear strength and mobilized shear stress at the same average compressive stress,

$\bar{I}_1$) and principal stresses (and angle of rotation) of each element in sequential order. A post-processing program written in BASIC language interfaces with the TTT file and a commercial software package (Surfer, 1989) which was used to generate the contour plots.

5.2 Design Analysis

5.2.1 Deformation Behavior of Reinforced and Unreinforced Embankments

In this section a detailed examination is presented of the behavior of reinforced and unreinforced embankments in relationship with the examples discussed previously in Chapter 4. A comparison of similar cases in Appendices C and D involving reinforced and unreinforced embankments indicates that the reinforcement locally alters the magnitude of the short term displacements but does not alter the overall pattern of displacements. Horizontal displacements at the embankment toe and maximum settlement (not in the sense of consolidation but rather displacements/deformations during undrained shear) at the base of the embankment were slightly reduced with single and multiple layers of reinforcement and with the type/modulus of the reinforcement. However, in comparison of cases 7 and 12 of State Route 55, a considerable reduction of the horizontal and vertical displacements at the toe of the embankment (in the foundation soil) and beneath the shoulder (embankment fill) were observed. The reduction was about 33% at each location. It should be noted that the over-consolidation ratio of the foundation soil was reduced to yield a softer response for cases 7 and 12 and a high-modulus woven geotextile was placed at the interface of the

embankment fill and foundation soil for case 12 to permit the comparison.

In general, comparisons indicate that the largest horizontal displacements develop beneath the shoulder/slope and at the toe (more so at the toe than the shoulder). These findings are in general agreement with the results of previous studies (e.g., Ohta, et al., 1980; Andrawes, et al., 1980, 1982; McGown, et al., 1981; Rowe, 1982; Boutrup and Holtz, 1982, 1983; McCarron, 1985; and Humphrey, 1986). In cases where reinforcement was used, the deformations were decreased and, as will be discussed in Section 5.2.3, stability was increased. However, the reinforcement did not have much of an effect at other sections within the embankment and at greater depths in the foundation. The location of the maximum displacements and the embankment stability is believed to depend primarily on the reinforcement type/modulus, embankment width, foundation depth and compressibility and the relative stiffness of the embankment fill and foundation soils. The beneficial effects of reinforcement on the short term deformation patterns are summarized in Table 5.1.

Table 5.1. Relative Influence of Reinforcement		
	Stiff Embankment	Soft Embankment
Stiff Foundation	Little Effect	Little Effect
Soft Foundation	Significant Effect	Little to Moderate Effect

5.2.2 Effect of Crust Strength

Displacements for embankments on foundations with strong and

weak crust are compared on the basis of the examples as discussed previously in Chapter 4. It should be noted that for these cases that the crust was modeled as a moderately to highly over-consolidated soil as discussed in Section 3.2 of Chapter 3. A comparison of similar cases with crust reported in Appendices C and D indicates that the crust strength has a significant effect on the overall performance (short term only) of the embankment fill and foundation soils. In comparison of cases 7 and 9 of State Route 55, the displacements are reduced by almost 80% for a given crust/foundation thickness and an increase in crust strength, especially when the cohesion was increased. A strong crust strength alters the magnitude of the displacements and causes the maximum horizontal displacement to develop at a slightly greater depth, directly beneath the embankment slope and in the foundation soil rather than near the toe for the weaker case. Again, similar conclusions as discussed in the previous section regarding reinforcement can be made here. The beneficial effects of crust strength are summarized in Table 5.2.

Table 5.2. Relative Influence of Crust Strength		
	Stiff Embankment	Soft Embankment
Strong Crust	Moderate to Significant Effect	Moderate Effect
Weak Crust	Little Effect	Little Effect

Based on the information in this section and the previous, a strong crust strength appears to significantly enhance the overall performance of the embankment fill and foundation soil and is

somewhat analogous to reinforcement. However, the beneficial effects of crust strength appear to outweigh those of reinforcement for the range of crust/foundation thicknesses ratio (i.e. 20% to 25%) considered in the examples.

5.2.3 Effect of Reinforcement on Stability

The effect of reinforcement on stability was investigated. A comparison of similar cases in Appendices C and D involving reinforced embankments indicates that the reinforcement locally alters the magnitude of the safety factor but does not influence the overall pattern of stability. Safety factors in the zone of the embankment toe and sideslope were slightly increased by the presence of reinforcement and more significantly by the presence of multiple layers of reinforcement. The potential benefit was more noticeable when using a high-modulus woven geotextile. In a comparison of cases 7 and 12 of State Route 55, a pronounced increase of the stability by about 20% at the toe (foundation soil) and by nearly 8% beneath the shoulder (embankment fill) was observed. It should be noted that the over-consolidation ratio of the foundation soil was reduced to yield a softer response for cases 7 and 12 and a high-modulus woven geotextile was placed at the interface of the embankment fill and foundation soil for case 12 to allow comparison. The effects of reinforcement on stability are extremely beneficial in situations where a relatively stiff embankment is established on very weak and compressible foundation soils. An interesting point in the comparison of cases 10 and 14 of State Route 1 indicates an increase by about 20% in the

embankment stability near the toe upon incorporation of a high-modulus woven geotextile placed at the interface of the embankment fill and foundation soil.

5.2.4 Effect of Foundation Compressibility

The effect of foundation compressibility was investigated on the basis of the examples discussed in Tables 4.2 and 4.4 of Chapter 4. A comparison of similar cases in Appendices C and D indicates that the foundation compressibility locally alters the magnitude of the short term displacements but does not alter the overall pattern of displacements. In general, comparisons indicate that the largest horizontal displacements develop at the toe. However, the largest vertical displacements develop near the shoulder and to a lesser extent at the center of the embankment and migrate to the sideslope with increasing foundation compressibility. Recall that a strong crust strength alters the magnitude of the short term displacements and causes the maximum horizontal displacement to develop at a slightly greater depth, directly beneath the embankment slope and in the foundation soil rather than near the toe for the weaker case. For a given embankment stiffness, the foundation compressibility directly influences the effectiveness of the crust strength. The effects of the foundation compressibility on the short term performance of the crust strength are summarized in Table 5.3.

Table 5.3. Relative Influence of Foundation Compressibility
on Crust Strength

	Normal Compressibility	High Compressibility
Strong Crust	Little Effect	Significant Effect
Weak Crust	Moderate to Significant Effect	Significant Effect

5.2.5 Effect of Embankment Width

The effect of embankment width was investigated by comparing cases 1 and 5 of State Route 55. The embankment width of case 5 [originally 45 ft (13.7 m) in case 1] was increased by the placement of a 10-ft (3 m) wide and 3.25-ft (1 m) high stabilizing berm of similar soil properties as the embankment fill. The effect of increasing the embankment width slightly reduces the short term displacements within the embankment fill and foundation soil. However, at a distance of approximately 20 ft (6.1 m) to 30 ft (9.1 m) from the toe of the berm, foundation heave slightly increases. In addition, the stability increases modestly near the toe of the embankment. Although the height of the berm is relatively low, the beneficial effect is somewhat modest in comparison to the use of reinforcement and/or an increase of crust strength.

5.2.6 Effect of Embankment Widening and Grade Raising

The effect of embankment widening and grade raising was investigated by comparing cases 1, 6 and 8 of State Route 55. Case 6 involves the widening (10 ft; 3 m) and grade raising (3.25 ft; 3 m) of the existing embankment in case 1. Case 8 includes the placement of a low-modulus nonwoven geotextile at the interface of

the embankment fill and foundation soil. Case 8 is similar to case 6 except for the incorporation of a geotextile.

Comparisons of cases 1 and 6 indicate that the horizontal displacements increase by as much as 0.4 in. (1 cm) to 0.6 in. (1.5 cm) following widening and grade raising. Settlements increase by as much as 1 in. (2.5 cm). Displacements develop primarily near the former shoulder and the widened toe (horizontally) and along the sideslope of the former embankment (vertically). Because of the loading and undrained conditions, some displacements develop causing movement of the existing embankment fill and foundation soil toward and upward along the centerline of the embankment. However, these displacements are relatively small.

The widening and raising locally alters the magnitude of the state of stress and stability but does not alter the overall pattern. Recall that the local factor of safety is defined as the ratio between the available shear strength (which is defined on the ultimate yield surface) and mobilized shear stress [which is defined by the stress state either on the ultimate yield surface (failure), within the cap (for overly-consolidated soils/elastic response) or on the cap (for normally-consolidated soils/elastic-plastic response)] at the same hydrostatic stress state. In review of the local factors of safety of cases 1 and 6, a greater portion of the foundation soil near the centerline of the embankment is yielding in case 1. By definition, a factor of safety of 1 implies that the strength is fully mobilized [i.e. the stress state is located at the corner of the ultimate and cap yielding surfaces

(see Figure 2.2c)]. As the embankment is widened, foundation soils near the centerline of the embankment and outwardly from the toe experience slight increases and decreases of the stability, respectively. In other words for the foundation soil near the centerline, the mobilized shear stress is slightly reduced upon widening and hence the stability is somewhat increased. While outwards from the toe, the widening of the embankment causes a slight increase in the mobilized shear stress in the foundation soil and hence the stability is somewhat decreased.

As stated previously, case 8 involves the incorporation of a low-modulus nonwoven geotextile at the interface of the embankment fill and foundation soil prior to widening and grade raising. Comparisons of cases 6 and 8 indicate only localized beneficial effects by the reinforcement. Again, the location of the maximum displacements and the embankment stability is believed to depend primarily on the reinforcement type/modulus, embankment width, foundation depth and compressibility and the relative stiffness/strength of the embankment fill and foundation soils.

5.2.7 Summary

Finite element analyses of two practical examples involving a total of 30 cases were performed. An incremental procedure was used in these computations to simulate embankment construction. The responses of the modeled embankment fill and foundation soils were represented using an isotropic, strain-hardening cap-plasticity model.

The results indicated that the presence of a stiff superficial

crust or of high tensile modulus geosynthetic reinforcement had the most significant influence and potential benefit on the system's short term performance. Reinforcement type/modulus also influenced the behavior of the embankment fill and foundation soil but to a lesser extent when compared to crust strength.

For compressible foundation soils and a relatively stiff embankment, a high-modulus woven geotextile was very effective in reducing displacements within the embankment fill and foundation soils and increasing the stability near the embankment toe. For a given embankment stiffness, the foundation compressibility directly influences the effectiveness of the crust strength. The foundation compressibility had only a limited influence on embankment behavior and the benefit from using reinforcement was modest except for that noted previously. The effect of increasing the embankment width by the use of a stabilizing berm only slightly reduced the displacements within the embankment fill and foundation soil and modestly increased the stability at the toe.

The widening and grade raising of an embankment locally altered the magnitude of the state of stress and stability but did not alter the overall pattern. Displacements developed primarily near the former shoulder and the widened toe (horizontally) and along the sideslope of the former embankment (vertically).

In several cases, the local factors of safety within the foundation soil near the centerline of the embankment and continuing outwardly to the toe and slightly beyond the toe were yielding (strain-hardening). The other portions of the embankment fill and

foundation soils were either experiencing stress states on the cap (compressive hardening), or within the cap (elastic state) as discussed in Chapter 2.

5.3 Design Analysis Limitations of NFAP program

NFAP is an extremely useful tool in gaining insight on the short-term behavior of embankments constructed on soil foundation soils. However, to fully appreciate the program's unique capabilities as a practical analysis tool requires a basic knowledge of such items as: the cap plasticity model (Chapter 2); the application of the FEM to NFAP (Chapter 3); and interpretation of the results as discussed in this chapter. NFAP does not have the capability to answer such questions as: "Is the design acceptable?"; and "How will the embankment behave during long-term conditions (e.g. consolidation, creep and certain environmental conditions/changes)?" . Nevertheless, these are questions that the engineer must answer during the design process. These types of questions usually require engineering judgement or experience on the part of the engineer and there may not be one "right" solution but rather several.

As discussed in Chapter 2, NFAP employs a cap plasticity model to simulate the behavior of the soil. From a theoretical point of view, the cap model employed herein is particularly appropriate in modeling soil behavior, because it is capable of handling the conditions of stress history, stress path dependency, dilatancy, and the effect of the intermediate principal stress. However, the model is incapable of treating such time-dependant phenomena as

consolidation and creep. Rather than using the cap model to simulate quasi-time-dependant effects, it is recommended that the engineer: (1) modify the output subroutine of NFAP to yield information regarding excess pore pressures or utilize the CAP program to estimate excess pore pressures based on loading conditions similar to that at critical locations within the embankment. In either circumstance estimate the consolidation settlements using an approximate coefficient permeability of the soils. Another approach to simulate long term time-dependant effects (e.g. consolidation) could be the use of isotropically-consolidated drained compression (CIDC) tests. Although embankments are typically constructed in a relatively short time frame, this solution will provide a reasonable upper bound to the overall performance (deformation and stability) of the embankment/foundation system. In the case of creep, use a variable moduli model (e.g. Duncan and Chang, 1970) however, the Duncan and Chang model is not based on plasticity theory and nor will it adapt to NFAP.

At present, the FEM is the only technique available to the geotechnical engineering community which allows a detailed assessment of deformations within the embankment fill and foundation soils during and following construction. The FEM technique is based on continuum mechanics. In the continuum theory of soil mechanics that includes the mathematical theory of elasticity, plasticity and viscosity, the following three basic sets of equations must be applied:

1. Equations of equilibrium of motion for a static or dynamic analysis, respectively;

2. Conditions of geometry or the compatibility of strains and displacements; and
3. Material constitutive laws or stress-strain relations.

As a result of the formulation, NFAP and for that matter most other FEM-based approaches are incapable of producing a physical crack and the associated propagation of the crack (violates item 2 above). However, NFAP is capable of evaluating the crack potential. By reviewing the output, the user is able to locate zones of potential tensile stresses (e.g. at the embankment toe or shoulder). Presently, the crack potential is evaluated in NFAP using the minimum principal stress in the in-plane stress state for two-dimensional plane strain analysis. When tension is recognized, the program performs a stress re-distribution (Chen, 1982). Upon completion of the analysis, the output of the program provides the computed stress at the Gauss points as well as the stress situation for each element, including the potential for tension. Hence, the user can identify where such a potential crack exists. From a practical standpoint, this could lead the analyst to modify the geometry/soil characteristics and re-analyze the problem, or possibly to perform a limit equilibrium analysis that includes a tension crack.

Another point of particular interest and related to the previous discussions is the aspect of reversal of principal stresses. Reversal of principal stresses develops when the minor principal stress is increased to failure or the major principal stress is decreased to failure. This occurs for K_0 consolidated samples sheared on axial extension and lateral compression stress

paths (e.g. near the toe of an embankment). The state of stress begins on the cap, however, the shear stress is initially reduced and the stress state moves into the elastic region. The cap model predicts a stress path having the shape shown in Figure 5.1. For convenience, states of stress where $\bar{\sigma}_v < \bar{\sigma}_h$ are plotted below the $\bar{I}_1$ axis. The path moves vertically downward until it reaches the cap and only elastic strains occur. Then, the path follows the cap and the cap expands causing plastic strains. Failure occurs when the state of stress reaches the ultimate yield surface. This stress path differs significantly from the behavior of many soils (Figure 5.1) and often $J_{2f}^{\frac{1}{2}}$ is less than $J_{2o}^{\frac{1}{2}}$. It is not possible to calibrate the model for this situation. However, when $J_{2o}^{\frac{1}{2}}$ is less than $J_{2f}^{\frac{1}{2}}$ the model can be calibrated to yield the correct $J_{2f}^{\frac{1}{2}}$, although predicted pore pressures will likely be in error.

The inability of the model to represent this behavior is because stress changes within the region bounded by the cap and ultimate yield surfaces cause only elastic strains and the cap and ultimate yield surfaces are symmetric about the $\bar{I}_1$ axis. This is not the case for real soils.

It should also be recognized during the design process that NFAP does not account for three-dimensional effects. NFAP is based on a two-dimensional formulation where the tensorial and engineering shear strains are set equal to zero. In the practical sense, this may not be so where the edge of a slip/failure surface intersects the longitudinal axis of an embankment.

Figure 5.1 Effective stress path predicted by cap model for samples that experience a reversal of principal stresses

5.4 Design Recommendations

Examples 1 and 2 involved the use of empirical correlations and laboratory test data, respectively, to determine the cap plasticity model parameters. Once an initial estimate of the parameters were determined for Example 2 from laboratory test results, an analysis using the CAP program was performed. Results from the CAP program were then compared to the observed behavior to assess the "fit". Once a reasonable fit of the observed data was attained for Example 2, the data file was constructed. However in Example 1, no comparison of the calculated and observed behavior was performed since no laboratory tests were performed. Development of the data file also requires the construction of a finite element mesh which represents the geometry and soil conditions of the specific problem. Once the mesh and data file are prepared, an analysis of the design can be performed using NFAP. Following the analysis, it is recommended that the results be carefully reviewed/evaluated with respect to those points discussed in Chapter 3.

CHAPTER 6

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary and Conclusions

6.1.1 General

The finite element technique using the cap soil behavior model was applied to the analysis of embankments constructed on soft foundation soils. A procedure was provided to estimate the cap model parameters from conventional field and laboratory test results. A sensitivity analysis of the cap model parameters comparing the observed and calculated responses was also provided. The technique was then applied to the analysis of two examples. The examples were based on actual highway projects in Indiana where information on these projects was provided by INDOT personnel. Results of the analysis were used to determine the influence of several factors on reinforced and unreinforced embankment behavior.

6.1.2 Cap Plasticity Model

A straight forward procedure to determine the cap parameters for normally consolidated soils from conventional laboratory and field tests is proposed. Table 6.1 on the following page summarizes the procedure for determining the cap parameters from conventional laboratory and field tests. A sensitivity study which examines the effect of the input soil parameters was performed. The main input soil properties are the compressibilities (C_c, C_r), the effective Mohr-Coulomb shear strength parameters ($\bar{\phi}, \bar{c}$), the undrained shear strength ratio ($USR; \frac{S_u}{\sigma_{vo}}$) and the over-consolidation ratio (OCR). Solutions are given in

Table 6.1. Summary of Procedures for Determining Cap Parameters from Conventional Laboratory and Field Tests		
Cap Model Parameter	Procedure	
	Cohesive Soil	Granular Soil
K_{min}	CIUC/CIDC/Correlations	Correlations/ Standard Penetration Test (SPT)/Pressuremeter Testing (PMT)/Vacuum Triaxial
v	CIUC/CIDC/Correlations	Correlations/Vacuum Triaxial
e_o	CIUC/CIDC/Correlations/ Consolidation Testing	Correlations/ SPT/Index Property Testing
$\bar{\phi}$	CIUC/CIDC/Correlations	Correlations/Vacuum Tri- axial/SPT/Cone Penetrometer Testing (CPT)/ Dilatometer Testing (DMT)/PMT
$\bar{c}$	CIUC/CIDC/Correlations	-
C_c	Correlations/Consolidation Testing	Correlations/Consolidation Testing
C_r	Correlations/Consolidation Testing	Correlations/Consolidation Testing
$\frac{S_u}{\sigma_o}$	CIUC/CIDC/Correlations/Vane Shear/CPT	Assume a fictitious value and iterate using the CAP program comparing results
OCR	Correlations/Consolidation Testing	Correlations/Consolidation Testing/Plate Load Tests
γ	Correlations/Index Property Tests	Correlations/Index Property Testing
K_o	Correlations/CIUC/CIDC PMT/DMT/CPT	Correlations/PMT/DMT/ CPT/SPT

graphical form and equations suitable for hand calculation.

The procedure was used to determine the cap parameters for an impact-compacted lacustrine clay using results from isotropically-consolidated undrained-compression (CIUC) tests. These parameters were then used in a computer program called CAP to calculate stress-strain curves, pore pressure response and effective stress paths. Comparisons were made to observed test results. In general, there was good agreement except for a discrepancy for pore pressures and effective stress paths. The discrepancy is at least partially because this formulation of the cap model does not allow plastic volumetric strain for stress changes within the region bounded by the cap and ultimate failure surfaces.

In addition, a sensitivity study was made of the effect of the input soil properties on calculated CIUC triaxial sample behavior. The results show which parameters have the greatest effect on computed behavior and provide guidance in the selection and adjustment of input soil properties to obtain a better fit between the calculated and observed response. The *USR* and *QCR* were observed to have the most-significant influence on the predicted behavior.

6.1.3 Case Studies

A comparative finite element study of two practical examples involving a total of 30 cases was made using an incremental procedure which simulated embankment construction. Embankment fill and foundation soil behavior was represented with an isotropic, strain-hardening cap-plasticity model.

The results indicated that the properties of a stiff or of a tensile resistant reinforcement had the most significant influence on embankment fill and foundation soil behavior. Reinforcement type/modulus also influenced the behavior of the embankment fill and foundation soil but to a lesser extent when compared to crust strength.

For compressible foundation soils and a relatively stiff embankment, a high-modulus woven geotextile was very effective in reducing displacements within the embankment fill and foundation soils and increasing the stability near the embankment toe. For a given embankment stiffness, the foundation compressibility directly influences the effectiveness of the crust strength. The foundation compressibility had only a limited influence on embankment behavior and the benefit from using reinforcement was modest except for that noted previously. The effect of increasing the embankment width by the use of a stabilizing berm only slightly reduced the displacements within the embankment fill and foundation soil and modestly increased the stability at the toe.

The widening and grade raising of an embankment locally altered the magnitude of the state of stress and stability but did not alter the overall pattern. Displacements developed primarily near the former shoulder and the widened toe (horizontally) and along the sideslope of the former embankment (vertically).

In several cases, the local factors of safety within the foundation soil near the centerline of the embankment and continuing outwardly to the toe and slightly beyond the toe were yielding

(strain hardening). The other portions of the embankment fill and foundation soils were either experiencing stress states on the cap (elastic-plastic), or within the cap (elastic state).

6.2 General Recommendations

The work presented herein aims to facilitate the application of the cap plasticity model to practical problems and in design of embankments constructed on soft foundation soils. However, there are some aspects of unreinforced and reinforced embankment analysis that require further development of the NFAP program. The ability of the cap model to accurately predict soil behavior could be extended to a wider range of soils, stress paths and drainage conditions. The following are recommended as additional research topics:

1. A parametric study of the effect of reinforcement should be made using NFAP for a wider range of embankment geometries and soil properties. It should include narrower embankments, different crust thicknesses and different reinforcement moduli. The tensile forces in the reinforcement should be analyzed in this study.
2. The cap plasticity model should be extended to improve its ability to model soil behavior during rotation of principal stresses and the behavior of over-consolidated soils.
3. Compacted cohesive soils are more often used as fill material for embankments. The behavior of embankments with cohesive fill (which include the effects of compactive prestress) should be evaluated with the cap plasticity model.
4. Presently, the cap plasticity model is better at accommodating either drained or undrained conditions. The model should be extended to partially drained conditions and then evaluated.
5. Slippage often occurs at or near the interface of the embankment fill/geosynthetic/foundation soil interface. NFAP should be modified to include slippage consider-

ations.

6. To provide beneficial use to practicing engineers, NFAP should be modified to include pre- and post-processing graphics capabilities.
7. Post construction effects need to be considered, particularly since the original embankment and widening/raising have different chronologies. Foundation creep and embankment settling upon wetting in service are examples.

Appendix A
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Appendix B:

Interim Report: "A Sensitivity Analysis of the Parameters for a Cap Plasticity Model," Report No. FHWA/JHRP/IN - 92-1, 23 pp.

Joint Highway Research Project

Draft Interim Report
FHWA/JHRP/IN - 92/1

A SENSITIVITY ANALYSIS OF THE
PARAMETERS FOR A CAP
PLASTICITY MODEL

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**Interim Report: "A Sensitivity Analysis
of the Parameters for a Cap Plasticity Model"**

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Joint Highway Research Project
HPR 2031

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the Indiana Department of Highways

and the

U.S. Department of Transportation
Federal Highway Administration

The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein.

Purdue University
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Date: January 23, 1991

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APPENDIX A

A.1 - Case 1

TABLE 1. SUMMARY OF SENSITIVITY ANALYSIS OF THE CAP PLASTICITY MODEL PARAMETERS

PARAMETER CASE	v	C_c	C_r	β	ϕ	USR	OCR
1A 1B 1C	0.15 0.30 0.45	0.162	0.044	10	24	0.22	1.49
2A 2B 2C	0.45	0.107 0.162 0.240	0.044	10	24	0.22	1.49
3A 3B 3C	0.45	0.162	0.029 0.044 0.066	10	24	0.22	1.49
4A 4B 4C	0.45	0.162	0.044	1 5 10	24	0.22	1.49
5A 5B 5C	0.45	0.162	0.044	10	19 24 30	0.22	1.49
6A 6B 6C	0.45	0.162	0.044	10	24	0.22 0.31 0.43	1.49
7A 7B 7C	0.45	0.162	0.044	10	24	0.22	1.0 1.49 2.25

Figure A.1.1 - Effect of Poisson's Ratio on the Principal Stress Difference and Excess Pore Pressure vs. Axial Strain

Figure A.1.2 - Effect of Poisson's Ratio on the Principal Stress Ratio vs. Axial Strain

Figure A.1.3 - Effect of Poisson's Ratio on the location of the Cap in the $J_2^{1/2}$ - I_1 Space and on the q - p' Diagram

A.2 - Case 2

Figure A.2.1 - Effect of the Compression Index on the Principal Stress Difference and Excess Pore Pressure vs. Axial Strain

Figure A.2.2 - Effect of the Compression Index on the Principal Stress Ratio vs. Axial Strain

Figure A.2.3 - Effect of the Compression Index on location of the Cap in the $J_2^{1/2}$ - I_1 Space and on the q - p' Diagram

A.3 - Case 3

Figure A.3.1 - Effect of the Recompression Index on the Principal Stress Difference and Excess Pore Pressure vs. Axial Strain

Figure A.3.2 - Effect of the Recompression Index on the Principal Stress Ratio vs. Axial Strain

Figure A.3.3 - Effect of the Recompression Index on location of the Cap in the $J_2^{1/2}$ - I_1 Space and on the q - p' Diagram

A.4 - Case 4

Figure A.4.1 - Effect of the Pore Pressure Response Factor on the Principal Stress Difference and Excess Pore Pressure vs. Axial Strain

Figure A.4.2 - Effect of the Pore Pressure Response Factor on the Principal Stress Ratio vs. Axial Strain

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A.5 - Case 5

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Figure A.5.2 - Effect of the Angle of Internal Friction on the Principal Stress Ratio vs. Axial Strain

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Figure A.7.2 - Effect of the Over-consolidation Ratio on the Principal Stress Ratio vs. Axial Strain

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APPENDIX B

Computer disk with input data files

APPENDIX C

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Appendix C:

Interim Report: "Embankment Widening and Grade Raising on Soft Foundation Soils: Example 1 - Indiana State Route 55 over Turkey Creek in Lake County, Indiana," Report No. FHWA/IN/JHRP - 91-18, 56 pp.

JOINT HIGHWAY RESEARCH PROJECT

FHWA/IN/JHRP - 91-18

Interim Report

Embankment Widening and Grade
Raising on Soft Foundation Soils

Scott J. Ludlow
Wai-Fah Chen
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C. William Lovell

**Interim Report: "Embankment Widening and Grade
Raising on Soft Foundation Soils:
Example 1 - Indiana State Route 55 over
Turkey Creek in Lake County, Indiana"**

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Joint Highway Research Project
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in Cooperation with
the Indiana Department of Highways
and the
U.S. Department of Transportation
Federal Highway Administration

The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein.

Purdue University
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Date: December 23, 1991

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TABLE 1. EXAMPLE 1 - SUMMARY OF CASES
FOR INDIANA STATE ROUTE 55 OVER TURKEY CREEK/LAKE COUNTY, INDIANA

CONDITION(S) CASE	DRAINAGE CONDITIONS DURING EMBANKMENT CONSTRUCTION	EMBANKMENT CONSTRUCTION AND SIMULATION	FINITE ELEMENT REPRESENTATION	COMMENTS
CASE 1	Undrained conditions with the water level at the ground surface.	Gravity build-up with eight 1.625-ft (.5 m) thick lifts.	349 nodes and 102 eight-noded isoparametric plane strain solid elements.	Refer to Appendix A.1 for rational soil parameters. Slope angle 1.5H to 1V.
CASE 2	Undrained and drained conditions for the foundation soil and embankment fill, respectively. Water level at the ground surface.	Gravity build-up with eight 1.625-ft (.5 m) thick lifts.	349 nodes and 102 eight-noded isoparametric plane strain solid elements.	
CASE 3	Undrained conditions with the water level at the ground surface.	Gravity build-up with placement of a low-modulus nonwoven geotextile (LMNG) at the interface of the embankment fill and foundation soil.	349 nodes and 102 eight-noded isoparametric plane strain and 10 three-noded truss elements.	
CASE 4	Undrained conditions with the water level at the ground surface.	Gravity build-up with placement of two LMNG layers: one each at the interface of the embankment fill and foundation soil and at the mid-height of the embankment.	349 nodes and 102 eight-noded isoparametric plane strain and 18 three-noded truss elements.	

TABLE 1. cont'd EXAMPLE 1 - SUMMARY OF CASES
FOR INDIANA STATE ROUTE 55 OVER TURKEY CREEK/LAKE COUNTY, INDIANA

CONDITION(S) CASE	DRAINAGE CONDITIONS DURING EMBANKMENT CONSTRUCTION	EMBANKMENT CONSTRUCTION AND SIMULATION	FINITE ELEMENT REPRESENTATION	COMMENTS
CASE 5	Undrained conditions with the water level at the ground surface.	Gravity build-up with eight 1.625-ft (.5 m) thick lifts. Construction of a 3.25-ft (1 m) high and 10-ft (3.1 m) wide stabilizing berm.	355 nodes and 104 eight-noded isoparametric plane strain solid elements.	Stabilizing berm constructed simultaneously with the lower two lifts of the embankment.
CASE 6	Undrained conditions with the water level at the ground surface.	Gravity build-up of the original embankment prior to widening and grade raising. Construction simulated with sixteen 1.625-ft (.5 m) thick lifts.	397 nodes and 118 eight-noded isoparametric plane strain solid elements. Perform analysis storing the initial stress state of the original embankment prior to widening and grade raising.	Widening (10 ft; 3.1 m) and grade raising (3.25 ft; .5 m) of the existing embankment. New embankment height and width of 16.25 ft (5 m) and 30 ft (9.1 m), respectively.
CASE 7	Undrained conditions with the water level at the ground surface.	Gravity build-up with eight 1.625-ft (.5 m) thick lifts.	349 nodes and 102 eight-noded isoparametric plane strain solid elements.	Similar to Case 1 except the over-consolidation ratio was modified.
CASE 8	Undrained conditions with the water level at the ground surface.	Gravity build-up of the original embankment prior to widening and grade raising. Construction simulated with sixteen 1.625-ft (.5 m) thick lifts.	397 nodes and 118 eight-noded isoparametric plane strain solid elements. Perform analysis storing the initial stress state of the original embankment prior to widening and grade raising.	Placement of a LMNG layer at the interface of the embankment fill and foundation soil. Similar to Case 6 except for geotextile.

TABLE 1. cont'd EXAMPLE 1 - SUMMARY OF CASES
FOR INDIANA STATE ROUTE 55 OVER TURKEY CREEK/LAKE COUNTY, INDIANA

CONDITION(S) CASE	DRAINAGE CONDITIONS DURING EMBANKMENT CONSTRUCTION	EMBANKMENT CONSTRUCTION AND SIMULATION	FINITE ELEMENT REPRESENTATION	COMMENTS
CASE 9	Undrained conditions with the water level at the ground surface.	Gravity build-up with eight 1.625-ft (.5 m) thick lifts.	349 nodes and 102 eight-noded isoparametric plane strain solid elements.	Similar to Case 7 except the undrained shear strength ratio and effective strength parameters were modified.
CASE 10	Undrained conditions with the water level at the ground surface.	Gravity build-up with placement of a high-modulus woven geotextile (HMG) at the interface of the embankment fill and foundation soil.	349 nodes and 102 eight-noded isoparametric plane strain and 10 three-noded truss elements.	Similar to Case 3 except for geotextile type and modulus.
CASE 11	Undrained conditions with the water level at the ground surface.	Gravity build-up with placement of two HMG layers: one each at the interface of the embankment fill and foundation soil and at the mid-height of the embankment.	349 nodes and 102 eight-noded isoparametric plane strain and 18 three-noded truss elements.	Similar to Case 4 except for geotextile type and modulus.
CASE 12	Undrained conditions with the water level at the ground surface.	Gravity build-up with placement of a HMG at the interface of the embankment fill and foundation soil.	349 nodes and 102 eight-noded isoparametric plane strain and 10 three-noded truss elements.	Similar to Case 7 except for incorporation of a geotextile.

APPENDIX A

**Example 1 - Indiana State Route 55 over Turkey
Creek in Lake County, Indiana**

Rational for estimating cap/soil model parameters

Objective: Estimate cap/soil model parameters from index property tests (test results summarized in a report prepared by ETS of Indianapolis, Indiana on May 20, 1989). For an explanation of the cap model parameters, refer to Huang and Chen (1991).

I. Foundation Soil.

- A. Determine the effective stress parameters for the ultimate failure surface (α, κ, T_c).

Using a relationship developed by Mitchell from isotropically consolidated, undrained triaxial compression (CIUC) tests with pore water pressure measurements, (appropriate for normally-consolidated and remolded clays only)

$$\sin \bar{\phi}_{cv} = 0.8 - 0.094 \ln PI$$

(first-order approximation)

where, $\bar{\phi}_{cv}$, is the critical void ratio friction angle for insensitive, uncemented normally-consolidated clay soils and, PI , is the plasticity index. From the report, PI is 13. Therefore, $\bar{\phi}_{cv}$ is about 33° .

Based on the geology of northwestern Indiana, the type of clay encountered at the site is likely lacustrine in origin. A personal data base by the author for clay soils in this area indicates that the PI is likely near the lower bound for lacustrine clay soils in northwestern Indiana. The plasticity index typically ranges from 12 to 18 averaging about 15. In addition, the liquid limit typically ranges from 30% to 38% averaging about 34%.

The ultimate failure surface used herein is described by the Drucker-Prager criterion (which is a circular surface in section unlike the Mohr-Coulomb surface which is hexagonal). For triaxial compression, the material constants, α and κ are related to ϕ and c by:

$$\alpha = \frac{2 \sin \phi}{\sqrt{3}(3 - \sin \phi)} \quad , \quad \kappa = \frac{6 c \cos \phi}{\sqrt{3}(3 - \sin \phi)}$$

For a plane strain condition, soils often display a slightly greater angle of internal friction than those obtained during a triaxial compression test. To simulate this behavior, the material constants depend on the Lode angle, θ , (Dafalias and Herrmann, 1986), according to:

$$\alpha = \alpha_c g(\theta, m) \quad , \quad \kappa = \kappa_c g(\theta, m)$$

and

$$m = \frac{3 - \sin \phi}{3 + \sin \phi}$$

$$g(\theta, m) = \frac{2m}{1+m}$$

Assume $\kappa=0$ and $\alpha=0.209$ from the above-mentioned relationships.

B. Determine the elastic and plastic parameters ($K_{\min}, A_c, A_e, v$)

$K_{\min}$; Because of the problems with the laboratory determination of undrained elastic modulus, E_u , and because large-scale field loading tests are expensive, it is common to assume that E_u is somehow related to the undrained shear strength. For example, Bjerrum (1972) proposed that the ratio E_u/S_u ranges from 500 to 1500, with S_u determined by the vane shear test. However, there appears to be much scatter about the data. Another way to estimate $K_{\min}$ is from Q_u test results. Based on the laboratory test results, the undrained elastic modulus varies from 9.4 to 25 ksf. In addition, the undrained shear strength ratio, $S_u/\bar{\sigma}_o$, varies from 0.23 (in Boring TB-4) to 0.8 (in Boring TB-2). E_u appears to be considerably low and is likely a result of sample disturbance. In addition, the Q_u tests were performed on splitspoon samples. Based on the author's personal data base, Q_u tests performed on lacustrine clays from similar depths and moisture contents, in which a piston sampler (Osterberg type) was used, indicate E_u values as high as 150 ksf. Use an E_u of 90 ksf (E_u for soft clays typically ranges from 100 to 500 ksf). Therefore,

$$K_{\min} = \frac{E_u}{3(1-2v)}$$

$$K_{\min} = 300 \text{ ksf}$$

assuming v is 0.45.

A_c, A_e ; No laboratory consolidation tests were performed. Recall that:

$$A_c = \frac{C_c}{2.303(1+e_o)} \quad \text{and}$$

$$A_e = \frac{C_r}{2.303(1+e_o)}$$

Estimate e_o :
$$e_o = \frac{w_c G_s}{S}; \quad G_s \text{ typically ranges from } 2.71 \text{ to } 2.75. \text{ Say } w_c \text{ is about } 26\%.$$
 Therefore, e_o is approximately 0.7.

Estimate $C_c; C_c$:

For Chicago clays: $C_c = 0.01(w_c)$ therefore, 0.26.

For undisturbed clays with low to medium plasticity: $C_c = 0.009(LL-10)$; LL implies the liquid limit, approximately 34%. Therefore, C_c is 0.216.

Author's personal data base: C_c ranges from 0.2 to 0.3, averaging about 0.246.

Use 0.246. Therefore, A_c is 0.0625.

Say C_r is 10% of C_c . Therefore, 0.00625.

Values of C_r outside the range of 0.005 to 0.05 should be considered questionable.

C. Estimate the cap surface (R , OCR).

R ; implies the cap aspect ratio (ratio of the major and minor radii of an ellipse). The cap aspect ratio, R , can be evaluated provided that the Drucker-Prager constants, $\alpha, \kappa, K_o, S_u/\bar{\sigma}_o$ are known. Presently, NFAP (nonlinear finite element analysis program) makes provisions to evaluate R and this will not be discussed herein.

OCR ; overconsolidation ratio. To accurately estimate the OCR of the lacustrine clay soils, a consolidation test(s) should have been performed. Based on a comparison of the moisture content values and Atterberg limits, the clay soils do not appear to be overly-consolidated. Although not site specific, results from consolidation tests performed on lacustrine clay soils from northwestern Indiana indicate that the clay soils may be slightly over-consolidated. Refer to the data input for the OCR values for analysis.

D. Initial stress state; ($\gamma_{s,m}, K_o$)

$$\gamma_{s,m} = 125 \text{ pcf}$$

Estimate K_o ;

$$K_o = (1 - \sin \phi) \quad \text{therefore, } 0.44.$$

$$K_o = 0.44 + 0.42 (PI/100) \quad \text{therefore, } 0.49.$$

Say 0.5.

E. Pore pressure response factor; (β)

Say 10 for undrained conditions otherwise 0 for drained. Note NFAP does not account for dissipation of pore water pressure. The pore pressure response factor is actually a ratio of the apparent bulk modulus of the fluids to the total (soil and water) bulk modulus.

F. Undrained shear strength ratio; ($\frac{S_u}{\sigma_o}$)

For normally- to slightly over-consolidated clays:

$$\frac{S_u}{\sigma_o} = 0.11 + 0.0037 (PI)$$

$\frac{S_u}{\sigma_o}$ typically ranges from 0.16 to 0.6.

Based on Boring TB-1 (sample depth/interval; 33.5 to 35 ft), say 0.36 (first-order approximation). Another technique in estimating the undrained shear strength ratio is to use SHANSEP (Ladd et al).

II. Embankment Fill.

For a first approximation of the embankment behavior, assume the soil conditions of the fill are similar to those of the foundation soil near the surface.

Figure A.2.1 Case 1 - Contours of Horizontal Displacement
Figure A.2.2 Case 1 - Contours of Vertical Displacement
Figure A.2.3 Case 1 - Contours of Principal Stress Ratio
Figure A.2.4 Case 1 - Contours of Local Factors of Safety

Figure A.2.1 Case 1 - Contours of Horizontal Displacement

Figure A.2.2 Case 1 - Contours of Vertical Displacement

Figure A.2.3 Case 1 - Contours of Principal Stress Ratio

Figure A.2.4 Case 1 - Contours of Local Factors of Safety

Figure A.3.1 Case 2 - Contours of Horizontal Displacement
Figure A.3.2 Case 2 - Contours of Vertical Displacement
Figure A.3.3 Case 2 - Contours of Principal Stress Ratio
Figure A.3.4 Case 2 - Contours of Local Factors of Safety

Figure A.3.1 Case 2 - Contours of Horizontal Displacement

Figure A.3.3.2 Case 2 - Contours of Vertical Displacement

Figure A.3.3 Case 2 - Contours of Principal Stress Ratio

Figure A.3.4 Case 2 - Contours of Local Factors of Safety

Figure A.4.1 Case 3 - Contours of Horizontal Displacement
Figure A.4.2 Case 3 - Contours of Vertical Displacement
Figure A.4.3 Case 3 - Contours of Principal Stress Ratio
Figure A.4.4 Case 3 - Contours of Local Factors of Safety

Figure A.4.1 Case 3 - Contours of Horizontal Displacement

Figure A.4.2 Case 3 - Contours of Vertical Displacement

Figure A.4.3 Case 3 - Contours of Principal Stress Ratio

Figure A.4.4 Case 3 - Contours of Local Factors of Safety

Figure A.5.1 Case 4 - Contours of Horizontal Displacement
Figure A.5.2 Case 4 - Contours of Vertical Displacement
Figure A.5.3 Case 4 - Contours of Principal Stress Ratio
Figure A.5.4 Case 4 - Contours of Local Factors of Safety

Contour Interval; 0.2 in.

Depth (ft)

Figure A.5.1 Case 4 - Contours of Horizontal Displacement

Figure A.5.2 Case 4 - Contours of Vertical Displacement

Figure A.5.3 Case 4 - Contours of Principal Stress Ratio

Figure A.5.4 Case 4 - Contours of Local Factors of Safety

Figure A.6.1 Case 5 - Contours of Horizontal Displacement
Figure A.6.2 Case 5 - Contours of Vertical Displacement
Figure A.6.3 Case 5 - Contours of Principal Stress Ratio
Figure A.6.4 Case 5 - Contours of Local Factors of Safety

Figure A.6.1 Case 5 - Contours of Horizontal Displacement

Contour Interval; 0.2 in.

Depth (ft)

Figure A.6.2 Case 5 - Contours of Vertical Displacement

Figure A.6.3 Case 5 - Contours of Principal Stress Ratio

Figure A.6.4 Case 5 - Contours of Local Factors of Safety

Figure A.7.1 Case 6 - Contours of Horizontal Displacement
Figure A.7.2 Case 6 - Contours of Vertical Displacement
Figure A.7.3 Case 6 - Contours of Principal Stress Ratio
Figure A.7.4 Case 6 - Contours of Local Factors of Safety

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Depth (ft)

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APPENDIX B

Computer disk with input data files

APPENDIX C

List of References

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Appendix D:

Interim Report: "Embankment Widening and Grade Raising on Soft Foundation Soils: Example 2 - Indiana State Route 1 over Ramsey Creek in Franklin County, Indiana," Report No. FHWA/IN/JHRP - 92-9, 173 pp.

JOINT HIGHWAY RESEARCH PROJECT

FHWA/IN/JHRP 92/9

Interim Report: Embankment Widening
and Grade Raising on Soft Foundation
Soils

Example 2 - Indiana State Route 1 Over
Ramsey Creek in Franklin County

Philippe Bourdeau,
Wai-Fah Chen,
C. William Lovell, and
Scott J. Ludlow

**Interim Report: "Embankment Widening and Grade
Raising on Soft Foundation Soils:
Example 2 - Indiana State Route 1 over
Ramsey Creek in Franklin County, Indiana"**

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Joint Highway Research Project
HPR 2031

Project No.: C-36-36U
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This study is proposed to be conducted by
the Joint Highway Research Project
in Cooperation with
the Indiana Department of Highways
and the

U.S. Department of Transportation
Federal Highway Administration

The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein.

Purdue University
West Lafayette, Indiana 47907

Date: March 31, 1991

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TABLE 1. EXAMPLE 2 - SUMMARY OF CASES
FOR INDIANA STATE ROUTE 1 OVER RAMSEY CREEK/FRANKLIN COUNTY, INDIANA

CONDITION(S) CASE	DRAINAGE CONDITIONS DURING EMBANKMENT CONSTRUCTION	EMBANKMENT CONSTRUCTION AND SIMULATION	FINITE ELEMENT REPRESENTATION	COMMENTS
CASE 1	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52 m) thick lifts.	460 nodes and 409 four-noded subparametric plane strain solid elements.	Refer to Appendix A.1 for estimated soil response using a cap plasticity model. Slope angle 3H to 1V. Seven foundation soil layers.
CASE 2	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up to Elevation 700 with thirteen 5-ft (1.52 m) thick lifts.	460 nodes and 409 four-noded subparametric plane strain solid elements. Turn gravity off after lift 13.	
CASE 3	Drained conditions with the water level at 7.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52m) thick lifts.	460 nodes and 409 four-noded subparametric plane strain elements.	
CASE 4	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with placement of one low-modulus nonwoven geotextile (LMNG) at the interface of the embankment fill and foundation soil.	460 nodes and 409 four-noded subparametric plane strain and 20 two-noded truss elements.	

TABLE 1. cont'd EXAMPLE 2 - SUMMARY OF CASES FOR INDIANA STATE ROUTE 1 OVER RAMSEY CREEK/FRANKLIN COUNTY, INDIANA				
CONDITION(S) CASE	DRAINAGE CONDITIONS DURING EMBANKMENT CONSTRUCTION	EMBANKMENT CONSTRUCTION AND SIMULATION	FINITE ELEMENT REPRESENTATION	COMMENTS
CASE 5	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52 m) thick lifts. Placement of 3 LMWG layers; one each at the interface of the embankment fill and foundation soil and at the 30-ft and 60-ft heights of the embankment.	460 nodes and 409 four-noded subparametric plane strain solid and 42 two-noded truss elements.	Modified the OCR and USR of the embankment fill.
CASE 6	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52 m) thick lifts. Placement of a LMWG layer at the interface of the embankment fill and foundation soil.	460 nodes and 409 four-noded subparametric plane strain solid and 22 two-noded truss elements.	Similar to Cases 3 and 4 (combined).
CASE 7	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52 m) thick lifts. Placement of a high-modulus woven geotextile (HMWG) at the interface of the embankment fill and foundation soil.	460 nodes and 409 four-noded subparametric plane strain solid and 22 two-noded truss elements.	Similar to Case 4 except for the geotextile type.

TABLE 1. cont'd EXAMPLE 2 - SUMMARY OF CASES
FOR INDIANA STATE ROUTE 1 OVER RAMSEY CREEK/FRANKLIN COUNTY, INDIANA

CONDITION(S) CASE	DRAINAGE CONDITIONS DURING EMBANKMENT CONSTRUCTION	EMBANKMENT CONSTRUCTION AND SIMULATION	FINITE ELEMENT REPRESENTATION	COMMENTS
CASE 8	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52 m) thick lifts. Placement of 3 HWG layers; one each at the interface of the embankment fill and foundation soil and at the 30-ft and 60-ft heights of the embankment.	460 nodes and 409 four-noded subparametric plane strain solid and 42 two-noded truss elements.	Similar to Case 5 except for the geotextile type.
CASE 9	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52m) thick lifts.	460 nodes and 409 four-noded subparametric plane strain elements.	Similar to Case 1 except OCR of embankment fill has been modified.
CASE 10	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52m) thick lifts.	460 nodes and 409 four-noded subparametric plane strain elements.	Similar to Case 9 except the OCR and USR of the crust (uppermost soil layer) has been modified.
CASE 11	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52 m) thick lifts.	460 nodes and 409 four-noded subparametric plane strain elements.	Similar to Case 10 except the OCR and USR of the underlying soil profile has been modified.

TABLE 1. cont'd EXAMPLE 2 - SUMMARY OF CASES FOR INDIANA STATE ROUTE 1 OVER RAMSEY CREEK/FRANKLIN COUNTY, INDIANA				
CONDITION(S) CASE	DRAINAGE CONDITIONS DURING EMBANKMENT CONSTRUCTION	EMBANKMENT CONSTRUCTION AND SIMULATION	FINITE ELEMENT REPRESENTATION	COMMENTS
CASE 12	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52 m) thick lifts. Placement of a high-modulus woven geotextile (HMG) at the interface of the embankment fill and foundation soil.	460 nodes and 409 four-noded subparametric plane strain solid and 22 two-noded truss elements.	Similar to Case 9 except for incorporation of geotextile.
CASE 13	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52 m) thick lifts. Placement of a LMNG layer at the interface of the embankment fill and foundation soil.	460 nodes and 409 four-noded subparametric plane strain solid and 22 two-noded truss elements.	Similar to Case 4 except for incorporation of geotextile.
CASE 14	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52 m) thick lifts. Placement of a high-modulus woven geotextile (HMG) at the interface of the embankment fill and foundation soil.	460 nodes and 409 four-noded subparametric plane strain solid and 22 two-noded truss elements.	Similar to Case 10 except for incorporation of geotextile.

TABLE 1. cont'd EXAMPLE 2 - SUMMARY OF CASES
FOR INDIANA STATE ROUTE 1 OVER RAMSEY CREEK/FRANKLIN COUNTY, INDIANA

CONDITION(S) CASE	DRAINAGE CONDITIONS DURING EMBANKMENT CONSTRUCTION	EMBANKMENT CONSTRUCTION AND SIMULATION	FINITE ELEMENT REPRESENTATION	COMMENTS
CASE 15	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52 m) thick lifts. Placement of a LMNG layer at the interface of the embankment fill and foundation soil.	460 nodes and 409 four-noded subparametric plane strain solid and 22 two-noded truss elements.	Similar to Case 10 except for incorporation of geotextile.
CASE 16	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52 m) thick lifts. Placement of a high-modulus woven geotextile (HMWG) at the interface of the embankment fill and foundation soil.	460 nodes and 409 four-noded subparametric plane strain solid and 22 two-noded truss elements.	Similar to Case 11 except for incorporation of geotextile.
CASE 17	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52 m) thick lifts. Placement of a LMNG layer at the interface of the embankment fill and foundation soil.	460 nodes and 409 four-noded subparametric plane strain solid and 22 two-noded truss elements.	Similar to Case 11 except for incorporation of geotextile.
CASE 18	Drained conditions with the water level at 11.5 ft below the ground surface. Undrained foundation.	Gravity build-up with eighteen 5-ft (1.52 m) thick lifts.	460 nodes and 409 four-noded subparametric plane strain solid and 22 two-noded truss elements.	Modified the OCR and USR of the embankment fill.

APPENDIX A

Indiana State Route 1 over Ramsey Creek
in Franklin County, Indiana

A.1 Estimated Soil Response using a Cap
Plasticity Model

Indiana State Route 1 over Ramsey Creek Franklin County, Indiana (10 psi)

0

1. 0.40 0.75 21. 5.0 0.25 0.05 .63 2.5 0.01 10.0 .730 10

900

0. 0.020 0. 0.

0

0. 0. 0. 0.

Computer Program Input

Figure A.2.1 Case 1 - Contours of Horizontal Displacement

Figure A.2.2 Case 1 - Contours of Vertical Displacement

Figure A.2.3 Case 1 - Contours of Principal Stress Ratio

Figure A.2.4 Case 1 - Contours of Local Factors of Safety

State Route 1 over Ramsey Creek - Franklin County, Indiana Case 1

460	0	1	1	18	1
1	0	15	.01		
0.0	0.5	9.0			
1	1	0	0.000	0.000	
2	1	0	0.000	-7.500	1
7	1	0	0.000	-27.500	
8	1	1	0.000	-35.000	
9	0	0	12.500	0.000	
17	0	0	25.000	0.000	8
337	0	0	625.000	0.000	
345	1	0	640.000	0.000	
10	0	0	12.500	-7.500	1
15	0	0	12.500	-27.500	
16	0	1	12.500	-35.000	
18	0	0	25.000	-7.500	8
338	0	0	625.000	-7.500	
346	1	0	640.000	-7.500	1
351	1	0	640.000	-27.500	
352	1	1	640.000	-35.000	
19	0	0	25.000	-11.500	8
339	0	0	625.000	-11.500	
20	0	0	25.000	-15.500	8
340	0	0	625.000	-15.500	
21	0	0	25.000	-19.500	8
341	0	0	625.000	-19.500	
22	0	0	25.000	-23.500	8
342	0	0	625.000	-23.500	
23	0	0	25.000	-27.500	8
343	0	0	625.000	-27.500	
24	0	1	25.000	-35.000	8
344	0	1	625.000	-35.000	
353	1	0	0.000	10.000	1
361	1	0	0.000	90.000	
362	0	0	12.500	10.000	1
370	0	0	12.500	90.000	
371	0	0	25.000	10.000	1
387	0	0	265.000	10.000	
388	0	0	280.000	5.000	
389	0	0	250.000	15.000	
390	0	0	25.000	20.000	1
404	0	0	235.000	20.000	
405	0	0	220.000	25.000	
406	0	0	25.000	30.000	1
418	0	0	205.000	30.000	
419	0	0	190.000	35.000	
420	0	0	25.000	40.000	1
430	0	0	175.000	40.000	
431	0	0	160.000	45.000	
432	0	0	25.000	50.000	1
440	0	0	145.000	50.000	
441	0	0	130.000	55.000	
442	0	0	25.000	60.000	1
448	0	0	115.000	60.000	
449	0	0	100.000	65.000	
450	0	0	25.000	70.000	1
454	0	0	85.000	70.000	
455	0	0	70.000	75.000	
456	0	0	25.000	80.000	1
458	0	0	55.000	80.000	

459	0	0	40.000	85.000				
460	0	0	25.000	90.000				
8								
1								
0.	640.		-7.5	0.	0.	0.		
0.	0.		0.	-11.5	0.125	0.0626	21.	8.0
1								
0.	640.		-11.5	-7.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	7.20
2								
0.	640.		-15.5	-11.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	4.9
2								
0.	640.		-19.5	-15.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	4.30
2								
0.	640.		-23.5	-19.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	3.8
2								
0.	640.		-27.5	-23.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	3.40
2								
0.	640.		-35.0	-27.51	0.	0.		
0.	0.		0.	-11.5	0.130	0.0676	21.	2.5
3								
0.	295.		0.	90.	0.	0.		
0.1	0.50							
0	0	0						
2	409	4	10	8	13	4	2	16
1								
300.	0.4		0.7500	21.	0.720	0.230	0.039	1.58
8.0	0.01		0.000	10.	2.			
2								
300.	0.4		0.7500	21.	0.720	0.250	0.050	1.46
7.20	0.01		0.000	10.	2.			
3								
300.	0.4		0.7500	21.	0.720	0.250	0.050	1.07
4.9	0.01		0.000	10.	2.			
4								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.96
4.30	0.01		0.000	10.	2.			
5								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.87
3.8	0.01		0.000	10.	2.			
6								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.80
3.40	0.01		0.000	10.	2.			
7								
300.	0.4		0.6630	21.	0.720	0.124	0.020	0.63
2.5	0.01		0.000	10.	2.			
8								
300.	0.4		0.7500	60.	0.000	0.124	0.020	1.58
20.0	0.01		0.125	0.	2.			
1	4	1	1	8	0.			
9	1	2	10	0	0	0	0	
43	4	1	1		0.			
345	337	338	346	0	0	0	0	
44	4	1	2	8	0.			
10	2	3	11	0	0	0	0	
86	4	1	2		0.			

388	388	153	161	0	0	0	0
322	4	1	8			1.	
363	354	353	362	0	0	0	0
323	4	1	8			1.	
390	363	362	371	0	0	0	0
324	4	1	8	1		1.	
391	390	371	372	0	0	0	0
337	4	1	8			1.	
404	403	384	385	0	0	0	0
338	4	1	8			1.	
389	404	385	386	0	0	0	0
339	4	1	8			1.	
389	389	386	387	0	0	0	0
340	4	1	8			2.	
364	355	354	363	0	0	0	0
341	4	1	8			2.	
406	364	363	390	0	0	0	0
342	4	1	8	1		2.	
407	406	390	391	0	0	0	0
353	4	1	8			2.	
418	417	401	402	0	0	0	0
354	4	1	8			2.	
405	418	402	403	0	0	0	0
355	4	1	8			2.	
405	405	403	404	0	0	0	0
356	4	1	8			3.	
365	356	355	364	0	0	0	0
357	4	1	8			3.	
420	365	364	406	0	0	0	0
358	4	1	8	1		3.	
421	420	406	407	0	0	0	0
367	4	1	8			3.	
430	429	415	416	0	0	0	0
368	4	1	8			3.	
419	430	416	417	0	0	0	0
369	4	1	8			3.	
419	419	417	418	0	0	0	0
370	4	1	8			4.	
366	357	356	365	0	0	0	0
371	4	1	8			4.	
432	366	365	420	0	0	0	0
372	4	1	8	1		4.	
433	432	420	421	0	0	0	0
379	4	1	8			4.	
440	439	427	428	0	0	0	0
380	4	1	8			4.	
431	440	428	429	0	0	0	0
381	4	1	8			4.	
431	431	429	430	0	0	0	0
382	4	1	8			5.	
367	358	357	366	0	0	0	0
383	4	1	8			5.	
442	367	366	432	0	0	0	0
384	4	1	8	1		5.	
443	442	432	433	0	0	0	0
389	4	1	8			5.	
448	447	437	438	0	0	0	0
390	4	1	8			5.	
441	448	438	439	0	0	0	0
391	4	1	8			5.	

346	338	339	347	0	0	0	0
87	4	1	3	8		0.	
11	3	4	12	0	0	0	0
129	4	1	3			0.	
347	339	340	348	0	0	0	0
130	4	1	4	8		0.	
12	4	5	13	0	0	0	0
172	4	1	4			0.	
348	340	341	349	0	0	0	0
173	4	1	5	8		0.	
13	5	6	14	0	0	0	0
215	4	1	5			0.	
349	341	342	350	0	0	0	0
216	4	1	6	8		0.	
14	6	7	15	0	0	0	0
258	4	1	6			0.	
350	342	343	351	0	0	0	0
259	4	1	7	8		0.	
15	7	8	16	0	0	0	0
301	4	1	7			0.	
351	343	344	352	0	0	0	0
302	4	1	8			0.01	
362	353	1	9	0	0	0	0
303	4	1	8			0.01	
371	362	9	17	0	0	0	0
304	4	1	8			0.01	
372	371	17	25	0	0	0	0
305	4	1	8			0.01	
373	372	25	33	0	0	0	0
306	4	1	8			0.01	
374	373	33	41	0	0	0	0
307	4	1	8			0.01	
375	374	41	49	0	0	0	0
308	4	1	8			0.01	
376	375	49	57	0	0	0	0
309	4	1	8			0.01	
377	376	57	65	0	0	0	0
310	4	1	8			0.01	
378	377	65	73	0	0	0	0
311	4	1	8			0.01	
379	378	73	81	0	0	0	0
312	4	1	8			0.01	
380	379	81	89	0	0	0	0
313	4	1	8			0.01	
381	380	89	97	0	0	0	0
314	4	1	8			0.01	
382	381	97	105	0	0	0	0
315	4	1	8			0.01	
383	382	105	113	0	0	0	0
316	4	1	8			0.01	
384	383	113	121	0	0	0	0
317	4	1	8			0.01	
385	384	121	129	0	0	0	0
318	4	1	8			0.01	
386	385	129	137	0	0	0	0
319	4	1	8			0.01	
387	386	137	145	0	0	0	0
320	4	1	8			0.01	
388	387	145	153	0	0	0	0
321	4	1	8			0.01	

441	441	439	440	0	0	0	0
392	4	1	8			6.	
368	359	358	367	0	0	0	0
393	4	1	8			6.	
450	368	367	442	0	0	0	0
394	4	1	8	1		6.	
451	450	442	443	0	0	0	0
397	4	1	8			6.	
454	453	445	446	0	0	0	0
398	4	1	8			6.	
449	454	446	447	0	0	0	0
399	4	1	8			6.	
449	449	447	448	0	0	0	0
400	4	1	8			7.	
369	360	359	368	0	0	0	0
401	4	1	8			7.	
456	369	368	450	0	0	0	0
402	4	1	8			7.	
457	456	450	451	0	0	0	0
403	4	1	8			7.	
458	457	451	452	0	0	0	0
404	4	1	8			7.	
455	458	452	453	0	0	0	0
405	4	1	8			7.	
455	455	453	454	0	0	0	0
406	4	1	8			8.	
370	361	360	369	0	0	0	0
407	4	1	8			8.	
460	370	369	456	0	0	0	0
408	4	1	8			8.	
459	460	456	457	0	0	0	0
409	4	1	8			8.	
459	459	457	458	0	0	0	0

Figure A.2.1 Case 1 - Contours of Horizontal Displacement

Contour Interval; 0.4'

Distance from Centerline of Embankment (ft)

14

Depth (ft) Contour Interval; 8

Distance from Centerline of Embankment (ft)

Figure A.2.3 Case 1 - Contours of Principal Stress Ratio

Contour Interval; 2

16

Computer Program Input

Figure A.3.1 Case 2 - Contours of Horizontal Displacement

Figure A.3.2 Case 2 - Contours of Vertical Displacement

Figure A.3.3 Case 2 - Contours of Principal Stress Ratio

Figure A.3.4 Case 2 - Contours of Local Factors of Safety

State Route 1 over Ramsey Creek - Franklin County, Indiana Case 2

460	0	1	1	13	1
1	0	15	.01		
0.0	0.5	9.0			
1	1	0	0.000	0.000	
2	1	0	0.000	-7.500	1
7	1	0	0.000	-27.500	
8	1	1	0.000	-35.000	
9	0	0	12.500	0.000	
17	0	0	25.000	0.000	8
337	0	0	625.000	0.000	
345	1	0	640.000	0.000	
10	0	0	12.500	-7.500	1
15	0	0	12.500	-27.500	
16	0	1	12.500	-35.000	
18	0	0	25.000	-7.500	8
338	0	0	625.000	-7.500	
346	1	0	640.000	-7.500	1
351	1	0	640.000	-27.500	
352	1	1	640.000	-35.000	
19	0	0	25.000	-11.500	8
339	0	0	625.000	-11.500	
20	0	0	25.000	-15.500	8
340	0	0	625.000	-15.500	
21	0	0	25.000	-19.500	8
341	0	0	625.000	-19.500	
22	0	0	25.000	-23.500	8
342	0	0	625.000	-23.500	
23	0	0	25.000	-27.500	8
343	0	0	625.000	-27.500	
24	0	1	25.000	-35.000	8
344	0	1	625.000	-35.000	
353	1	0	0.000	10.000	1
361	1	0	0.000	90.000	
362	0	0	12.500	10.000	1
370	0	0	12.500	90.000	
371	0	0	25.000	10.000	1
387	0	0	265.000	10.000	
388	0	0	280.000	5.000	
389	0	0	250.000	15.000	
390	0	0	25.000	20.000	1
404	0	0	235.000	20.000	
405	0	0	220.000	25.000	
406	0	0	25.000	30.000	1
418	0	0	205.000	30.000	
419	0	0	190.000	35.000	
420	0	0	25.000	40.000	1
430	0	0	175.000	40.000	
431	0	0	160.000	45.000	
432	0	0	25.000	50.000	1
440	0	0	145.000	50.000	
441	0	0	130.000	55.000	
442	0	0	25.000	60.000	1
448	0	0	115.000	60.000	
449	0	0	100.000	65.000	
450	0	0	25.000	70.000	1
454	0	0	85.000	70.000	
455	0	0	70.000	75.000	
456	0	0	25.000	80.000	1
458	0	0	55.000	80.000	

459	0	0	40.000	85.000				
460	0	0	25.000	90.000				
8								
1								
0.	640.		-7.5	0.	0.	0.		
0.	0.		0.	-11.5	0.125	0.0626	21.	8.0
1								
0.	640.		-11.5	-7.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	7.20
2								
0.	640.		-15.5	-11.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	4.9
2								
0.	640.		-19.5	-15.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	4.30
2								
0.	640.		-23.5	-19.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	3.8
2								
0.	640.		-27.5	-23.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	3.40
2								
0.	640.		-35.0	-27.51	0.	0.		
0.	0.		0.	-11.5	0.130	0.0676	21.	2.5
3								
0.	295.		0.	90.	0.	0.		
0.1	0.50							
0	0	0						
2	409	4	10	8	13	4	2	16
1								
300.	0.4		0.7500	21.	0.720	0.230	0.039	1.58
8.0	0.01		0.000	10.	2.			
2								
300.	0.4		0.7500	21.	0.720	0.250	0.050	1.46
7.20	0.01		0.000	10.	2.			
3								
300.	0.4		0.7500	21.	0.720	0.250	0.050	1.07
4.9	0.01		0.000	10.	2.			
4								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.96
4.30	0.01		0.000	10.	2.			
5								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.87
3.8	0.01		0.000	10.	2.			
6								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.80
3.40	0.01		0.000	10.	2.			
7								
300.	0.4		0.6630	21.	0.720	0.124	0.020	0.63
2.5	0.01		0.000	10.	2.			
8								
300.	0.4		0.7500	60.	0.000	0.124	0.020	1.58
20.0	0.01		0.125	0.	2.			
1	4	1	1	8	0.			
9	1	2	10	0	0	0	0	
43	4	1	1		0.			
345	337	338	346	0	0	0	0	
44	4	1	2	8	0.			
10	2	3	11	0	0	0	0	
86	4	1	2		0.			

346	338	339	347	0	0	0	0
87	4	1	3	8		0.	
11	3	4	12	0	0	0	0
129	4	1	3			0.	
347	339	340	348	0	0	0	0
130	4	1	4	8		0.	
12	4	5	13	0	0	0	0
172	4	1	4			0.	
348	340	341	349	0	0	0	0
173	4	1	5	8		0.	
13	5	6	14	0	0	0	0
215	4	1	5			0.	
349	341	342	350	0	0	0	0
216	4	1	6	8		0.	
14	6	7	15	0	0	0	0
258	4	1	6			0.	
350	342	343	351	0	0	0	0
259	4	1	7	8		0.	
15	7	8	16	0	0	0	0
301	4	1	7			0.	
351	343	344	352	0	0	0	0
302	4	1	8	0.01			
362	353	1	9	0	0	0	0
303	4	1	8	0.01			
371	362	9	17	0	0	0	0
304	4	1	8	0.01			
372	371	17	25	0	0	0	0
305	4	1	8	0.01			
373	372	25	33	0	0	0	0
306	4	1	8	0.01			
374	373	33	41	0	0	0	0
307	4	1	8	0.01			
375	374	41	49	0	0	0	0
308	4	1	8	0.01			
376	375	49	57	0	0	0	0
309	4	1	8	0.01			
377	376	57	65	0	0	0	0
310	4	1	8	0.01			
378	377	65	73	0	0	0	0
311	4	1	8	0.01			
379	378	73	81	0	0	0	0
312	4	1	8	0.01			
380	379	81	89	0	0	0	0
313	4	1	8	0.01			
381	380	89	97	0	0	0	0
314	4	1	8	0.01			
382	381	97	105	0	0	0	0
315	4	1	8	0.01			
383	382	105	113	0	0	0	0
316	4	1	8	0.01			
384	383	113	121	0	0	0	0
317	4	1	8	0.01			
385	384	121	129	0	0	0	0
318	4	1	8	0.01			
386	385	129	137	0	0	0	0
319	4	1	8	0.01			
387	386	137	145	0	0	0	0
320	4	1	8	0.01			
388	387	145	153	0	0	0	0
321	4	1	8	0.01			

388	388	153	161	0	0	0	0
322	4	1	8			1.	
363	354	353	362	0	0	0	0
323	4	1	8			1.	
390	363	362	371	0	0	0	0
324	4	1	8	1		1.	
391	390	371	372	0	0	0	0
337	4	1	8			1.	
404	403	384	385	0	0	0	0
338	4	1	8			1.	
389	404	385	386	0	0	0	0
339	4	1	8			1.	
389	389	386	387	0	0	0	0
340	4	1	8			2.	
364	355	354	363	0	0	0	0
341	4	1	8			2.	
406	364	363	390	0	0	0	0
342	4	1	8	1		2.	
407	406	390	391	0	0	0	0
353	4	1	8			2.	
418	417	401	402	0	0	0	0
354	4	1	8			2.	
405	418	402	403	0	0	0	0
355	4	1	8			2.	
405	405	403	404	0	0	0	0
356	4	1	8			3.	
365	356	355	364	0	0	0	0
357	4	1	8			3.	
420	365	364	406	0	0	0	0
358	4	1	8	1		3.	
421	420	406	407	0	0	0	0
367	4	1	8			3.	
430	429	415	416	0	0	0	0
368	4	1	8			3.	
419	430	416	417	0	0	0	0
369	4	1	8			3.	
419	419	417	418	0	0	0	0
370	4	1	8			4.	
366	357	356	365	0	0	0	0
371	4	1	8			4.	
432	366	365	420	0	0	0	0
372	4	1	8	1		4.	
433	432	420	421	0	0	0	0
379	4	1	8			4.	
440	439	427	428	0	0	0	0
380	4	1	8			4.	
431	440	428	429	0	0	0	0
381	4	1	8			4.	
431	431	429	430	0	0	0	0
382	4	1	8			5.	
367	358	357	366	0	0	0	0
383	4	1	8			5.	
442	367	366	432	0	0	0	0
384	4	1	8	1		5.	
443	442	432	433	0	0	0	0
389	4	1	8			5.	
448	447	437	438	0	0	0	0
390	4	1	8			5.	
441	448	438	439	0	0	0	0
391	4	1	8			5.	

441	441	439	440	0	0	0	0
392	4	1	8			6.	
368	359	358	367	0	0	0	0
393	4	1	8			6.	
450	368	367	442	0	0	0	0
394	4	1	8	1		6.	
451	450	442	443	0	0	0	0
397	4	1	8			6.	
454	453	445	446	0	0	0	0
398	4	1	8			6.	
449	454	446	447	0	0	0	0
399	4	1	8			6.	
449	449	447	448	0	0	0	0
400	4	1	8			7.	
369	360	359	368	0	0	0	0
401	4	1	8			7.	
456	369	368	450	0

Figure A.3.2 Case 2 - Contours of Vertical Displacement

Depth (ft)

Contour Interval; 4

Distance from Centerline of Embankment (ft)

Figure A.3.3 Case 2 - Contours of Principal Stress Ratio

Computer Program Input

Figure A.4.1 Case 3 - Contours of Horizontal Displacement

Figure A.4.2 Case 3 - Contours of Vertical Displacement

Figure A.4.3 Case 3 - Contours of Principal Stress Ratio

Figure A.4.4 Case 3 - Contours of Local Factors of Safety

State Route 1 over Ramsey Creek - Franklin County, Indiana Case 3

460	0	1	1	18	1
1	0	15	.01		
0.0	0.5	9.0			
1	1	0	0.000	0.000	
2	1	0	0.000	-7.500	1
7	1	0	0.000	-27.500	
8	1	1	0.000	-35.000	
9	0	0	12.500	0.000	
17	0	0	25.000	0.000	8
337	0	0	625.000	0.000	
345	1	0	640.000	0.000	
10	0	0	12.500	-7.500	1
15	0	0	12.500	-27.500	
16	0	1	12.500	-35.000	
18	0	0	25.000	-7.500	8
338	0	0	625.000	-7.500	
346	1	0	640.000	-7.500	1
351	1	0	640.000	-27.500	
352	1	1	640.000	-35.000	
19	0	0	25.000	-11.500	8
339	0	0	625.000	-11.500	
20	0	0	25.000	-15.500	8
340	0	0	625.000	-15.500	
21	0	0	25.000	-19.500	8
341	0	0	625.000	-19.500	
22	0	0	25.000	-23.500	8
342	0	0	625.000	-23.500	
23	0	0	25.000	-27.500	8
343	0	0	625.000	-27.500	
24	0	1	25.000	-35.000	8
344	0	1	625.000	-35.000	
353	1	0	0.000	10.000	1
361	1	0	0.000	90.000	
362	0	0	12.500	10.000	1
370	0	0	12.500	90.000	
371	0	0	25.000	10.000	1
387	0	0	265.000	10.000	
388	0	0	280.000	5.000	
389	0	0	250.000	15.000	
390	0	0	25.000	20.000	1
404	0	0	235.000	20.000	
405	0	0	220.000	25.000	
406	0	0	25.000	30.000	1
418	0	0	205.000	30.000	
419	0	0	190.000	35.000	
420	0	0	25.000	40.000	1
430	0	0	175.000	40.000	
431	0	0	160.000	45.000	
432	0	0	25.000	50.000	1
440	0	0	145.000	50.000	
441	0	0	130.000	55.000	
442	0	0	25.000	60.000	1
448	0	0	115.000	60.000	
449	0	0	100.000	65.000	
450	0	0	25.000	70.000	1
454	0	0	85.000	70.000	
455	0	0	70.000	75.000	
456	0	0	25.000	80.000	1
458	0	0	55.000	80.000	

459 0 0 40.000 85.000
 460 0 0 25.000 90.000

8

1

0. 640. -7.5 0. 0. 0.
 0. 0. 0. -7.5 0.125 0.0626 21. 8.0

1

0. 640. -11.5 -7.51 0. 0.
 0. 0. 0. -7.5 0.120 0.0576 21. 7.20

2

0. 640. -15.5 -11.51 0. 0.
 0. 0. 0. -7.5 0.120 0.0576 21. 4.9

2

0. 640. -19.5 -15.51 0. 0.
 0. 0. 0. -7.5 0.120 0.0576 21. 4.30

2

0. 640. -23.5 -19.51 0. 0.
 0. 0. 0. -7.5 0.120 0.0576 21. 3.8

2

0. 640. -27.5 -23.51 0. 0.
 0. 0. 0. -7.5 0.120 0.0576 21. 3.40

2

0. 640. -35.0 -27.51 0. 0.
 0. 0. 0. -7.5 0.130 0.0676 21. 2.5

3

0. 295. 0. 90. 0. 0.
 0.1 0.50

0

0

0

2 409 4 10 8 13 4 2 16

1

300. 0.4 0.7500 21. 0.720 0.230 0.039 1.58
 8.0 0.01 0.000 10. 2.

2

300. 0.4 0.7500 21. 0.720 0.250 0.050 1.46
 7.20 0.01 0.000 10. 2.

3

300. 0.4 0.7500 21. 0.720 0.250 0.050 1.07
 4.9 0.01 0.000 10. 2.

4

300. 0.4 0.7500 21. 0.720 0.250 0.050 0.96
 4.30 0.01 0.000 10. 2.

5

300. 0.4 0.7500 21. 0.720 0.250 0.050 0.87
 3.8 0.01 0.000 10. 2.

6

300. 0.4 0.7500 21. 0.720 0.250 0.050 0.80
 3.40 0.01 0.000 10. 2.

7

300. 0.4 0.6630 21. 0.720 0.124 0.020 0.63
 2.5 0.01 0.000 10. 2.

8

300. 0.4 0.7500 60. 0.000 0.124 0.020 1.58
 20.0 0.01 0.125 0. 2.

1

9

43

345 337 338 346 0 0 0 0

44

10 2 3 11 0 0 0 0

86

4 1 2 0.

346	338	339	347	0	0	0	0
87	4	1	3	8		0.	
11	3	4	12	0	0	0	0
129	4	1	3			0.	
347	339	340	348	0	0	0	0
130	4	1	4	8		0.	
12	4	5	13	0	0	0	0
172	4	1	4			0.	
348	340	341	349	0	0	0	0
173	4	1	5	8		0.	
13	5	6	14	0	0	0	0
215	4	1	5			0.	
349	341	342	350	0	0	0	0
216	4	1	6	8		0.	
14	6	7	15	0	0	0	0
258	4	1	6			0.	
350	342	343	351	0	0	0	0
259	4	1	7	8		0.	
15	7	8	16	0	0	0	0
301	4	1	7			0.	
351	343	344	352	0	0	0	0
302	4	1	8	0.01			
362	353	1	9	0	0	0	0
303	4	1	8	0.01			
371	362	9	17	0	0	0	0
304	4	1	8	0.01			
372	371	17	25	0	0	0	0
305	4	1	8	0.01			
373	372	25	33	0	0	0	0
306	4	1	8	0.01			
374	373	33	41	0	0	0	0
307	4	1	8	0.01			
375	374	41	49	0	0	0	0
308	4	1	8	0.01			
376	375	49	57	0	0	0	0
309	4	1	8	0.01			
377	376	57	65	0	0	0	0
310	4	1	8	0.01			
378	377	65	73	0	0	0	0
311	4	1	8	0.01			
379	378	73	81	0	0	0	0
312	4	1	8	0.01			
380	379	81	89	0	0	0	0
313	4	1	8	0.01			
381	380	89	97	0	0	0	0
314	4	1	8	0.01			
382	381	97	105	0	0	0	0
315	4	1	8	0.01			
383	382	105	113	0	0	0	0
316	4	1	8	0.01			
384	383	113	121	0	0	0	0
317	4	1	8	0.01			
385	384	121	129	0	0	0	0
318	4	1	8	0.01			
386	385	129	137	0	0	0	0
319	4	1	8	0.01			
387	386	137	145	0	0	0	0
320	4	1	8	0.01			
388	387	145	153	0	0	0	0
321	4	1	8	0.01			

388	388	153	161	0	0	0	0
322	4	1	8			1.	
363	354	353	362	0	0	0	0
323	4	1	8			1.	
390	363	362	371	0	0	0	0
324	4	1	8	1		1.	
391	390	371	372	0	0	0	0
337	4	1	8			1.	
404	403	384	385	0	0	0	0
338	4	1	8			1.	
389	404	385	386	0	0	0	0
339	4	1	8			1.	
389	389	386	387	0	0	0	0
340	4	1	8			2.	
364	355	354	363	0	0	0	0
341	4	1	8			2.	
406	364	363	390	0	0	0	0
342	4	1	8	1		2.	
407	406	390	391	0	0	0	0
353	4	1	8			2.	
418	417	401	402	0	0	0	0
354	4	1	8			2.	
405	418	402	403	0	0	0	0
355	4	1	8			2.	
405	405	403	404	0	0	0	0
356	4	1	8			3.	
365	356	355	364	0	0	0	0
357	4	1	8			3.	
420	365	364	406	0	0	0	0
358	4	1	8	1		3.	
421	420	406	407	0	0	0	0
367	4	1	8			3.	
430	429	415	416	0	0	0	0
368	4	1	8			3.	
419	430	416	417	0	0	0	0
369	4	1	8			3.	
419	419	417	418	0	0	0	0
370	4	1	8			4.	
366	357	356	365	0	0	0	0
371	4	1	8			4.	
432	366	365	420	0	0	0	0
372	4	1	8	1		4.	
433	432	420	421	0	0	0	0
379	4	1	8			4.	
440	439	427	428	0	0	0	0
380	4	1	8			4.	
431	440	428	429	0	0	0	0
381	4	1	8			4.	
431	431	429	430	0	0	0	0
382	4	1	8			5.	
367	358	357	366	0	0	0	0
383	4	1	8			5.	
442	367	366	432	0	0	0	0
384	4	1	8	1		5.	
443	442	432	433	0	0	0	0
389	4	1	8			5.	
448	447	437	438	0	0	0	0
390	4	1	8			5.	
441	448	438	439	0	0	0	0
391	4	1	8			5.	

441	441	439	440	0	0	0	0
392	4	1	8			6.	
368	359	358	367	0	0	0	0
393	4	1	8			6.	
450	368	367	442	0	0	0	0
394	4	1	8	1		6.	
451	450	442	443	0	0	0	0
397	4	1	8			6.	
454	453	445	446	0	0	0	0
398	4	1	8			6.	
449	454	446	447	0	0	0	0
399	4	1	8			6.	
449	449	447	448	0	0	0	0
400	4	1	8			7.	
369	360	359	368	0	0	0	0
401	4	1	8			7.	
456	369	368	450	0	0	0	0
402	4	1	8			7.	
457	456	450	451	0	0	0	0
403	4	1	8			7.	
458	457	451	452	0	0	0	0
404	4	1	8			7.	
455	458	452	453	0	0	0	0
405	4	1	8			7.	
455	455	453	454	0	0	0	0
406	4	1	8			8.	
370	361	360	369	0	0	0	0
407	4	1	8			8.	
460	370	369	456	0	0	0	0
408	4	1	8			8.	
459	460	456	457	0	0	0	0
409	4	1	8			8.	
459	459	457	458	0	0	0	0

Figure A.4.1 Case 3 - Contours of Horizontal Displacement

Figure A.4.2 Case 3 - Contours of Vertical Displacement

Depth (ft)

Contour Interval; 8

Distance from Centerline of Embankment (ft)

Figure A.4.3 Case 3 - Contours of Principal Stress Ratio

Figure A.4.4 Case 3 - Contours of Local Factors of Safety

Computer Program Input

Figure A.5.1 Case 4 - Contours of Horizontal Displacement

Figure A.5.2 Case 4 - Contours of Vertical Displacement

Figure A.5.3 Case 4 - Contours of Principal Stress Ratio

Figure A.5.4 Case 4 - Contours of Local Factors of Safety

State Route 1 over Ramsey Creek - Franklin County, Indiana Case 4

460	0	2	1	18	1
1	0	15	.01		
0.0	0.5	9.0			
1	1	0	0.000	0.000	
2	1	0	0.000	-7.500	1
7	1	0	0.000	-27.500	
8	1	1	0.000	-35.000	
9	0	0	12.500	0.000	
17	0	0	25.000	0.000	8
337	0	0	625.000	0.000	
345	1	0	640.000	0.000	
10	0	0	12.500	-7.500	1
15	0	0	12.500	-27.500	
16	0	1	12.500	-35.000	
18	0	0	25.000	-7.500	8
338	0	0	625.000	-7.500	
346	1	0	640.000	-7.500	1
351	1	0	640.000	-27.500	
352	1	1	640.000	-35.000	
19	0	0	25.000	-11.500	8
339	0	0	625.000	-11.500	
20	0	0	25.000	-15.500	8
340	0	0	625.000	-15.500	
21	0	0	25.000	-19.500	8
341	0	0	625.000	-19.500	
22	0	0	25.000	-23.500	8
342	0	0	625.000	-23.500	
23	0	0	25.000	-27.500	8
343	0	0	625.000	-27.500	
24	0	1	25.000	-35.000	8
344	0	1	625.000	-35.000	
353	1	0	0.000	10.000	1
361	1	0	0.000	90.000	
362	0	0	12.500	10.000	1
370	0	0	12.500	90.000	
371	0	0	25.000	10.000	1
387	0	0	265.000	10.000	
388	0	0	280.000	5.000	
389	0	0	250.000	15.000	
390	0	0	25.000	20.000	1
404	0	0	235.000	20.000	
405	0	0	220.000	25.000	
406	0	0	25.000	30.000	1
418	0	0	205.000	30.000	
419	0	0	190.000	35.000	
420	0	0	25.000	40.000	1
430	0	0	175.000	40.000	
431	0	0	160.000	45.000	
432	0	0	25.000	50.000	1
440	0	0	145.000	50.000	
441	0	0	130.000	55.000	
442	0	0	25.000	60.000	1
448	0	0	115.000	60.000	
449	0	0	100.000	65.000	
450	0	0	25.000	70.000	1
454	0	0	85.000	70.000	
455	0	0	70.000	75.000	
456	0	0	25.000	80.000	1
458	0	0	55.000	80.000	

459 0 0 40.000 85.000

460 0 0 25.000 90.000

8

1

0. 640. -7.5 0. 0. 0.
0. 0. 0. -11.5 0.125 0.0626 21. 8.0

1

0. 640. -11.5 -7.51 0. 0.
0. 0. 0. -11.5 0.120 0.0576 21. 7.20

2

0. 640. -15.5 -11.51 0. 0.
0. 0. 0. -11.5 0.120 0.0576 21. 4.9

2

0. 640. -19.5 -15.51 0. 0.
0. 0. 0. -11.5 0.120 0.0576 21. 4.30

2

0. 640. -23.5 -19.51 0. 0.
0. 0. 0. -11.5 0.120 0.0576 21. 3.8

2

0. 640. -27.5 -23.51 0. 0.
0. 0. 0. -11.5 0.120 0.0576 21. 3.40

2

0. 640. -35.0 -27.51 0. 0.
0. 0. 0. -11.5 0.130 0.0676 21. 2.5

3

0. 295. 0. 90. 0. 0.
0.1 0.50

0

0

0

2 409 4 10 8 13 4 2 16

1

300. 0.4 0.7500 21. 0.720 0.230 0.039 1.58
8.0 0.01 0.000 10. 2.

2

300. 0.4 0.7500 21. 0.720 0.250 0.050 1.46
7.20 0.01 0.000 10. 2.

3

300. 0.4 0.7500 21. 0.720 0.250 0.050 1.07
4.9 0.01 0.000 10. 2.

4

300. 0.4 0.7500 21. 0.720 0.250 0.050 0.96
4.30 0.01 0.000 10. 2.

5

300. 0.4 0.7500 21. 0.720 0.250 0.050 0.87
3.8 0.01 0.000 10. 2.

6

300. 0.4 0.7500 21. 0.720 0.250 0.050 0.80
3.40 0.01 0.000 10. 2.

7

300. 0.4 0.6630 21. 0.720 0.124 0.020 0.63
2.5 0.01 0.000 10. 2.

8

300. 0.4 0.7500 60. 0.000 0.124 0.020 1.58
20.0 0.01 0.125 0. 2.

1

9

43

345 337 338 346 0 0 0 0

44

10

86

4 1 2 10 0 0 0 0

346	338	339	347	0	0	0	0
87	4	1	3	8		0.	
11	3	4	12	0	0	0	0
129	4	1	3			0.	
347	339	340	348	0	0	0	0
130	4	1	4	8		0.	
12	4	5	13	0	0	0	0
172	4	1	4			0.	
348	340	341	349	0	0	0	0
173	4	1	5	8		0.	
13	5	6	14	0	0	0	0
215	4	1	5			0.	
349	341	342	350	0	0	0	0
216	4	1	6	8		0.	
14	6	7	15	0	0	0	0
258	4	1	6			0.	
350	342	343	351	0	0	0	0
259	4	1	7	8		0.	
15	7	8	16	0	0	0	0
301	4	1	7			0.	
351	343	344	352	0	0	0	0
302	4	1	8	0.01			
362	353	1	9	0	0	0	0
303	4	1	8	0.01			
371	362	9	17	0	0	0	0
304	4	1	8	0.01			
372	371	17	25	0	0	0	0
305	4	1	8	0.01			
373	372	25	33	0	0	0	0
306	4	1	8	0.01			
374	373	33	41	0	0	0	0
307	4	1	8	0.01			
375	374	41	49	0	0	0	0
308	4	1	8	0.01			
376	375	49	57	0	0	0	0
309	4	1	8	0.01			
377	376	57	65	0	0	0	0
310	4	1	8	0.01			
378	377	65	73	0	0	0	0
311	4	1	8	0.01			
379	378	73	81	0	0	0	0
312	4	1	8	0.01			
380	379	81	89	0	0	0	0
313	4	1	8	0.01			
381	380	89	97	0	0	0	0
314	4	1	8	0.01			
382	381	97	105	0	0	0	0
315	4	1	8	0.01			
383	382	105	113	0	0	0	0
316	4	1	8	0.01			
384	383	113	121	0	0	0	0
317	4	1	8	0.01			
385	384	121	129	0	0	0	0
318	4	1	8	0.01			
386	385	129	137	0	0	0	0
319	4	1	8	0.01			
387	386	137	145	0	0	0	0
320	4	1	8	0.01			
388	387	145	153	0	0	0	0
321	4	1	8	0.01			

388	388	153	161	0	0	0	0
322	4	1	8			1.	
363	354	353	362	0	0	0	0
323	4	1	8			1.	
390	363	362	371	0	0	0	0
324	4	1	8	1		1.	
391	390	371	372	0	0	0	0
337	4	1	8			1.	
404	403	384	385	0	0	0	0
338	4	1	8			1.	
389	404	385	386	0	0	0	0
339	4	1	8			1.	
389	389	386	387	0	0	0	0
340	4	1	8			2.	
364	355	354	363	0	0	0	0
341	4	1	8			2.	
406	364	363	390	0	0	0	0
342	4	1	8	1		2.	
407	406	390	391	0	0	0	0
353	4	1	8			2.	
418	417	401	402	0	0	0	0
354	4	1	8			2.	
405	418	402	403	0	0	0	0
355	4	1	8			2.	
405	405	403	404	0	0	0	0
356	4	1	8			3.	
365	356	355	364	0	0	0	0
357	4	1	8			3.	
420	365	364	406	0	0	0	0
358	4	1	8	1		3.	
421	420	406	407	0	0	0	0
367	4	1	8			3.	
430	429	415	416	0	0	0	0
368	4	1	8			3.	
419	430	416	417	0	0	0	0
369	4	1	8			3.	
419	419	417	418	0	0	0	0
370	4	1	8			4.	
366	357	356	365	0	0	0	0
371	4	1	8			4.	
432	366	365	420	0	0	0	0
372	4	1	8	1		4.	
433	432	420	421	0	0	0	0
379	4	1	8			4.	
440	439	427	428	0	0	0	0
380	4	1	8			4.	
431	440	428	429	0	0	0	0
381	4	1	8			4.	
431	431	429	430	0	0	0	0
382	4	1	8			5.	
367	358	357	366	0	0	0	0
383	4	1	8			5.	
442	367	366	432	0	0	0	0
384	4	1	8	1		5.	
443	442	432	433	0	0	0	0
389	4	1	8			5.	
448	447	437	438	0	0	0	0
390	4	1	8			5.	
441	448	438	439	0	0	0	0
391	4	1	8			5.	

441	441	439	440	0	0	0	0
392	4	1	8			6.	
368	359	358	367	0	0	0	0
393	4	1	8			6.	
450	368	367	442	0	0	0	0
394	4	1	8	1		6.	
451	450	442	443	0	0	0	0
397	4	1	8			6.	
454	453	445	446	0	0	0	0
398	4	1	8			6.	
449	454	446	447	0	0	0	0
399	4	1	8			6.	
449	449	447	448	0	0	0	0
400	4	1	8			7.	
369	360	359	368	0	0	0	0
401	4	1	8			7.	
456	369	368	450	0	0	0	0
402	4	1	8			7.	
457	456	450	451	0	0	0	0
403	4	1	8			7.	
458	457	451	452	0	0	0	0
404	4	1	8			7.	
455	458	452	453	0	0	0	0
405	4	1	8			7.	
455	455	453	454	0	0	0	0
406	4	1	8			8.	
370	361	360	369	0	0	0	0
407	4	1	8			8.	
460	370	369	456	0	0	0	0
408	4	1	8			8.	
459	460	456	457	0	0	0	0
409	4	1	8			8.	
459	459	457	458	0	0	0	0

1	20	2	2	1	8		
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1	0.						
	0.	.067	.083	.16	0.	.14	.16
							.2

1	1	9	0	1
2	9	17	0	1
3	17	25	0	1
4	25	33	0	1
5	33	41	0	1
6	41	49	0	1
7	49	57	0	1
8	57	65	0	1
9	65	73	0	1
10	73	81	0	1
11	81	89	0	1
12	89	97	0	1
13	97	105	0	1
14	105	113	0	1
15	113	121	0	1
16	121	129	0	1
17	129	137	0	1
18	137	145	0	1
19	145	153	0	1
20	153	161	0	1

Figure A.5.1 Case 4 - Contours of Horizontal Displacement

Figure A.5.2 Case 4 - Contours of Vertical Displacement

Depth (ft) Contour Interval; 4

Distance from Centerline of Embankment (ft)

Figure A.5.3 Case 4 - Contours of Principal Stress Ratio

Figure A.5.4 Case 4 - Contours of Local Factors of Safety

Computer Program Input

Figure A.6.1 Case 5 - Contours of Horizontal Displacement

Figure A.6.2 Case 5 - Contours of Vertical Displacement

Figure A.6.3 Case 5 - Contours of Principal Stress Ratio

Figure A.6.4 Case 5 - Contours of Local Factors of Safety

State Route 1 over Ramsey Creek - Franklin County, Indiana Case 5

460	0	2	1	18	1
1	0	15	.01		
0.0	0.5	9.0			
1	1	0	0.000	0.000	
2	1	0	0.000	-7.500	1
7	1	0	0.000	-27.500	
8	1	1	0.000	-35.000	
9	0	0	12.500	0.000	
17	0	0	25.000	0.000	8
337	0	0	625.000	0.000	
345	1	0	640.000	0.000	
10	0	0	12.500	-7.500	1
15	0	0	12.500	-27.500	
16	0	1	12.500	-35.000	
18	0	0	25.000	-7.500	8
338	0	0	625.000	-7.500	
346	1	0	640.000	-7.500	1
351	1	0	640.000	-27.500	
352	1	1	640.000	-35.000	
19	0	0	25.000	-11.500	8
339	0	0	625.000	-11.500	
20	0	0	25.000	-15.500	8
340	0	0	625.000	-15.500	
21	0	0	25.000	-19.500	8
341	0	0	625.000	-19.500	
22	0	0	25.000	-23.500	8
342	0	0	625.000	-23.500	
23	0	0	25.000	-27.500	8
343	0	0	625.000	-27.500	
24	0	1	25.000	-35.000	8
344	0	1	625.000	-35.000	
353	1	0	0.000	10.000	1
361	1	0	0.000	90.000	
362	0	0	12.500	10.000	1
370	0	0	12.500	90.000	
371	0	0	25.000	10.000	1
387	0	0	265.000	10.000	
388	0	0	280.000	5.000	
389	0	0	250.000	15.000	
390	0	0	25.000	20.000	1
404	0	0	235.000	20.000	
405	0	0	220.000	25.000	
406	0	0	25.000	30.000	1
418	0	0	205.000	30.000	
419	0	0	190.000	35.000	
420	0	0	25.000	40.000	1
430	0	0	175.000	40.000	
431	0	0	160.000	45.000	
432	0	0	25.000	50.000	1
440	0	0	145.000	50.000	
441	0	0	130.000	55.000	
442	0	0	25.000	60.000	1
448	0	0	115.000	60.000	
449	0	0	100.000	65.000	
450	0	0	25.000	70.000	1
454	0	0	85.000	70.000	
455	0	0	70.000	75.000	
456	0	0	25.000	80.000	1
458	0	0	55.000	80.000	

459	0	0	40.000	85.000				
460	0	0	25.000	90.000				
8								
1								
0.	640.		-7.5	0.	0.	0.		
0.	0.		0.	-11.5	0.125	0.0626	21.	8.0
1								
0.	640.		-11.5	-7.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	7.20
2								
0.	640.		-15.5	-11.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	4.9
2								
0.	640.		-19.5	-15.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	4.30
2								
0.	640.		-23.5	-19.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	3.8
2								
0.	640.		-27.5	-23.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	3.40
2								
0.	640.		-35.0	-27.51	0.	0.		
0.	0.		0.	-11.5	0.130	0.0676	21.	2.5
3								
0.	295.		0.	90.	0.	0.		
0.1	0.50							
0	0	0						
2	409	4	10	8	13	4	2	16
1								
300.	0.4		0.7500	21.	0.720	0.230	0.039	1.58
8.0	0.01		0.000	10.	2.			
2								
300.	0.4		0.7500	21.	0.720	0.250	0.050	1.46
7.20	0.01		0.000	10.	2.			
3								
300.	0.4		0.7500	21.	0.720	0.250	0.050	1.07
4.9	0.01		0.000	10.	2.			
4								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.96
4.30	0.01		0.000	10.	2.			
5								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.87
3.8	0.01		0.000	10.	2.			
6								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.80
3.40	0.01		0.000	10.	2.			
7								
300.	0.4		0.6630	21.	0.720	0.124	0.020	0.63
2.5	0.01		0.000	10.	2.			
8								
300.	0.4		0.7500	60.	0.000	0.124	0.020	0.73
3.0	0.01		0.125	0.	2.			
1	4	1	1	8	0.			
9	1	2	10	0	0	0	0	
43	4	1	1		0.			
345	337	338	346	0	0	0	0	
44	4	1	2	8	0.			
10	2	3	11	0	0	0	0	
86	4	1	2		0.			

346	338	339	347	0	0	0	0
87	4	1	3	8	0.		
11	3	4	12	0	0	0	0
129	4	1	3		0.		
347	339	340	348	0	0	0	0
130	4	1	4	8	0.		
12	4	5	13	0	0	0	0
172	4	1	4		0.		
348	340	341	349	0	0	0	0
173	4	1	5	8	0.		
13	5	6	14	0	0	0	0
215	4	1	5		0.		
349	341	342	350	0	0	0	0
216	4	1	6	8	0.		
14	6	7	15	0	0	0	0
258	4	1	6		0.		
350	342	343	351	0	0	0	0
259	4	1	7	8	0.		
15	7	8	16	0	0	0	0
301	4	1	7		0.		
351	343	344	352	0	0	0	0
302	4	1	8	0.01			
362	353	1	9	0	0	0	0
303	4	1	8	0.01			
371	362	9	17	0	0	0	0
304	4	1	8	0.01			
372	371	17	25	0	0	0	0
305	4	1	8	0.01			
373	372	25	33	0	0	0	0
306	4	1	8	0.01			
374	373	33	41	0	0	0	0
307	4	1	8	0.01			
375	374	41	49	0	0	0	0
308	4	1	8	0.01			
376	375	49	57	0	0	0	0
309	4	1	8	0.01			
377	376	57	65	0	0	0	0
310	4	1	8	0.01			
378	377	65	73	0	0	0	0
311	4	1	8	0.01			
379	378	73	81	0	0	0	0
312	4	1	8	0.01			
380	379	81	89	0	0	0	0
313	4	1	8	0.01			
381	380	89	97	0	0	0	0
314	4	1	8	0.01			
382	381	97	105	0	0	0	0
315	4	1	8	0.01			
383	382	105	113	0	0	0	0
316	4	1	8	0.01			
384	383	113	121	0	0	0	0
317	4	1	8	0.01			
385	384	121	129	0	0	0	0
318	4	1	8	0.01			
386	385	129	137	0	0	0	0
319	4	1	8	0.01			
387	386	137	145	0	0	0	0
320	4	1	8	0.01			
388	387	145	153	0	0	0	0
321	4	1	8	0.01			

388	388	153	161	0	0	0	0
322	4	1	8			1.	
363	354	353	362	0	0	0	0
323	4	1	8			1.	
390	363	362	371	0	0	0	0
324	4	1	8	1		1.	
391	390	371	372	0	0	0	0
337	4	1	8			1.	
404	403	384	385	0	0	0	0
338	4	1	8			1.	
389	404	385	386	0	0	0	0
339	4	1	8			1.	
389	389	386	387	0	0	0	0
340	4	1	8			2.	
364	355	354	363	0	0	0	0
341	4	1	8			2.	
406	364	363	390	0	0	0	0
342	4	1	8	1		2.	
407	406	390	391	0	0	0	0
353	4	1	8			2.	
418	417	401	402	0	0	0	0
354	4	1	8			2.	
405	418	402	403	0	0	0	0
355	4	1	8			2.	
405	405	403	404	0	0	0	0
356	4	1	8			3.	
365	356	355	364	0	0	0	0
357	4	1	8			3.	
420	365	364	406	0	0	0	0
358	4	1	8	1		3.	
421	420	406	407	0	0	0	0
367	4	1	8			3.	
430	429	415	416	0	0	0	0
368	4	1	8			3.	
419	430	416	417	0	0	0	0
369	4	1	8			3.	
419	419	417	418	0	0	0	0
370	4	1	8			4.	
366	357	356	365	0	0	0	0
371	4	1	8			4.	
432	366	365	420	0	0	0	0
372	4	1	8	1		4.	
433	432	420	421	0	0	0	0
379	4	1	8			4.	
440	439	427	428	0	0	0	0
380	4	1	8			4.	
431	440	428	429	0	0	0	0
381	4	1	8			4.	
431	431	429	430	0	0	0	0
382	4	1	8			5.	
367	358	357	366	0	0	0	0
383	4	1	8			5.	
442	367	366	432	0	0	0	0
384	4	1	8	1		5.	
443	442	432	433	0	0	0	0
389	4	1	8			5.	
448	447	437	438	0	0	0	0
390	4	1	8			5.	
441	448	438	439	0	0	0	0
391	4	1	8			5.	

441	441	439	440	0	0	0	0
392	4	1	8			6.	
368	359	358	367	0	0	0	0
393	4	1	8			6.	
450	368	367	442	0	0	0	0
394	4	1	8	1		6.	
451	450	442	443	0	0	0	0
397	4	1	8			6.	
454	453	445	446	0	0	0	0
398	4	1	8			6.	
449	454	446	447	0	0	0	0
399	4	1	8			6.	
449	449	447	448	0	0	0	0
400	4	1	8			7.	
369	360	359	368	0	0	0	0
401	4	1	8			7.	
456	369	368	450	0	0	0	0
402	4	1	8			7.	
457	456	450	451	0	0	0	0
403	4	1	8			7.	
458	457	451	452	0	0	0	0
404	4	1	8			7.	
455	458	452	453	0	0	0	0
405	4	1	8			7.	
455	455	453	454	0	0	0	0
406	4	1	8			8.	
370	361	360	369	0	0	0	0
407	4	1	8			8.	
460	370	369	456	0	0	0	0
408	4	1	8			8.	
459	460	456	457	0	0	0	0
409	4	1	8			8.	
459	459	457	458	0	0	0	0
1	42	2	2	3	8		

1	0.								
	0.	.067	.083	.16	0.	.14	.16	.2	
2	6.								
	0.	.067	.083	.16	0.	.14	.16	.2	
3	12.								
	0.	.067	.083	.16	0.	.14	.16	.2	
1	1	9	0	1					
2	9	17	0	1					
3	17	25	0	1					
4	25	33	0	1					
5	33	41	0	1					
6	41	49	0	1					
7	49	57	0	1					
8	57	65	0	1					
9	65	73	0	1					
10	73	81	0	1					
11	81	89	0	1					
12	89	97	0	1					
13	97	105	0	1					
14	105	113	0	1					
15	113	121	0	1					
16	121	129	0	1					
17	129	137	0	1					
18	137	145	0	1					
19	145	153	0	1					
20	153	161	0	1					

21	355	364	0	2
22	364	406	0	2
23	406	407	0	2
24	407	408	0	2
25	408	409	0	2
26	409	410	0	2
27	410	411	0	2
28	411	412	0	2
29	412	413	0	2
30	413	414	0	2
31	414	415	0	2
32	415	416	0	2
33	416	417	0	2
34	417	418	0	2
35	358	367	0	3
36	367	442	0	3
37	442	443	0	3
38	443	444	0	3
39	444	445	0	3
40	445	446	0	3
41	446	447	0	3
42	447	448	0	3

Figure A.6.2 Case 5 - Contours of Vertical Displacement

Figure A.6.3 Case 5 - Contours of Principal Stress Ratio

Figure A.6.4 Case 5 - Contours of Local Factors of Safety

Computer Program Input

- Figure A.7.1 Case 6 - Contours of Horizontal Displacement
- Figure A.7.2 Case 6 - Contours of Vertical Displacement
- Figure A.7.3 Case 6 - Contours of Principal Stress Ratio
- Figure A.7.4 Case 6 - Contours of Local Factors of Safety

State Route 1 over Ramsey Creek - Franklin County, Indiana Case 6

460	0	2	1	18	1
1	0	15	.01		
0.0		0.5	9.0		
1	1	0	0.000	0.000	
2	1	0	0.000	-7.500	1
7	1	0	0.000	-27.500	
8	1	1	0.000	-35.000	
9	0	0	12.500	0.000	
17	0	0	25.000	0.000	8
337	0	0	625.000	0.000	
345	1	0	640.000	0.000	
10	0	0	12.500	-7.500	1
15	0	0	12.500	-27.500	
16	0	1	12.500	-35.000	
18	0	0	25.000	-7.500	8
338	0	0	625.000	-7.500	
346	1	0	640.000	-7.500	1
351	1	0	640.000	-27.500	
352	1	1	640.000	-35.000	
19	0	0	25.000	-11.500	8
339	0	0	625.000	-11.500	
20	0	0	25.000	-15.500	8
340	0	0	625.000	-15.500	
21	0	0	25.000	-19.500	8
341	0	0	625.000	-19.500	
22	0	0	25.000	-23.500	8
342	0	0	625.000	-23.500	
23	0	0	25.000	-27.500	8
343	0	0	625.000	-27.500	
24	0	1	25.000	-35.000	8
344	0	1	625.000	-35.000	
353	1	0	0.000	10.000	1
361	1	0	0.000	90.000	
362	0	0	12.500	10.000	1
370	0	0	12.500	90.000	
371	0	0	25.000	10.000	1
387	0	0	265.000	10.000	
388	0	0	280.000	5.000	
389	0	0	250.000	15.000	
390	0	0	25.000	20.000	1
404	0	0	235.000	20.000	
405	0	0	220.000	25.000	
406	0	0	25.000	30.000	1
418	0	0	205.000	30.000	
419	0	0	190.000	35.000	
420	0	0	25.000	40.000	1
430	0	0	175.000	40.000	
431	0	0	160.000	45.000	
432	0	0	25.000	50.000	1
440	0	0	145.000	50.000	
441	0	0	130.000	55.000	
442	0	0	25.000	60.000	1
448	0	0	115.000	60.000	
449	0	0	100.000	65.000	
450	0	0	25.000	70.000	1
454	0	0	85.000	70.000	
455	0	0	70.000	75.000	
456	0	0	25.000	80.000	1
458	0	0	55.000	80.000	

459	0	0	40.000	85.000				
460	0	0	25.000	90.000				
8								
1								
0.	640.		-7.5	0.	0.			
0.	0.		0.	-7.5	0.125	0.0626	21.	8.0
1								
0.	640.		-11.5	-7.51	0.	0.		
0.	0.		0.	-7.5	0.120	0.0576	21.	7.20
2								
0.	640.		-15.5	-11.51	0.	0.		
0.	0.		0.	-7.5	0.120	0.0576	21.	4.9
2								
0.	640.		-19.5	-15.51	0.	0.		
0.	0.		0.	-7.5	0.120	0.0576	21.	4.30
2								
0.	640.		-23.5	-19.51	0.	0.		
0.	0.		0.	-7.5	0.120	0.0576	21.	3.8
2								
0.	640.		-27.5	-23.51	0.	0.		
0.	0.		0.	-7.5	0.120	0.0576	21.	3.40
2								
0.	640.		-35.0	-27.51	0.	0.		
0.	0.		0.	-7.5	0.130	0.0676	21.	2.5
3								
0.	295.		0.	90.	0.	0.		
0.1	0.50							
0	0	0						
2	409	4	10	8	13	4	2	16
1								
300.	0.4		0.7500	21.	0.720	0.230	0.039	1.58
8.0	0.01		0.000	10.	2.			
2								
300.	0.4		0.7500	21.	0.720	0.250	0.050	1.46
7.20	0.01		0.000	10.	2.			
3								
300.	0.4		0.7500	21.	0.720	0.250	0.050	1.07
4.9	0.01		0.000	10.	2.			
4								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.96
4.30	0.01		0.000	10.	2.			
5								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.87
3.8	0.01		0.000	10.	2.			
6								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.80
3.40	0.01		0.000	10.	2.			
7								
300.	0.4		0.6630	21.	0.720	0.124	0.020	0.63
2.5	0.01		0.000	10.	2.			
8								
300.	0.4		0.7500	60.	0.000	0.124	0.020	1.58
20.0	0.01		0.125	0.	2.			
1	4	1	1	8	0.			
9	1	2	10	0	0	0	0	0
43	4	1	1		0.			
345	337	338	346	0	0	0	0	0
44	4	1	2	8	0.			
10	2	3	11	0	0	0	0	0
86	4	1	2		0.			

346	338	339	347	0	0	0	0
87	4	1	3	8		0.	
11	3	4	12	0	0	0	0
129	4	1	3			0.	
347	339	340	348	0	0	0	0
130	4	1	4	8		0.	
12	4	5	13	0	0	0	0
172	4	1	4			0.	
348	340	341	349	0	0	0	0
173	4	1	5	8		0.	
13	5	6	14	0	0	0	0
215	4	1	5			0.	
349	341	342	350	0	0	0	0
216	4	1	6	8		0.	
14	6	7	15	0	0	0	0
258	4	1	6			0.	
350	342	343	351	0	0	0	0
259	4	1	7	8		0.	
15	7	8	16	0	0	0	0
301	4	1	7			0.	
351	343	344	352	0	0	0	0
302	4	1	8			0.01	
362	353	1	9	0	0	0	0
303	4	1	8			0.01	
371	362	9	17	0	0	0	0
304	4	1	8			0.01	
372	371	17	25	0	0	0	0
305	4	1	8			0.01	
373	372	25	33	0	0	0	0
306	4	1	8			0.01	
374	373	33	41	0	0	0	0
307	4	1	8			0.01	
375	374	41	49	0	0	0	0
308	4	1	8			0.01	
376	375	49	57	0	0	0	0
309	4	1	8			0.01	
377	376	57	65	0	0	0	0
310	4	1	8			0.01	
378	377	65	73	0	0	0	0
311	4	1	8			0.01	
379	378	73	81	0	0	0	0
312	4	1	8			0.01	
380	379	81	89	0	0	0	0
313	4	1	8			0.01	
381	380	89	97	0	0	0	0
314	4	1	8			0.01	
382	381	97	105	0	0	0	0
315	4	1	8			0.01	
383	382	105	113	0	0	0	0
316	4	1	8			0.01	
384	383	113	121	0	0	0	0
317	4	1	8			0.01	
385	384	121	129	0	0	0	0
318	4	1	8			0.01	
386	385	129	137	0	0	0	0
319	4	1	8			0.01	
387	386	137	145	0	0	0	0
320	4	1	8			0.01	
388	387	145	153	0	0	0	0
321	4	1	8			0.01	

388	388	153	161	0	0	0	0
322	4	1	8			1.	
363	354	353	362	0	0	0	0
323	4	1	8			1.	
390	363	362	371	0	0	0	0
324	4	1	8	1		1.	
391	390	371	372	0	0	0	0
337	4	1	8			1.	
404	403	384	385	0	0	0	0
338	4	1	8			1.	
389	404	385	386	0	0	0	0
339	4	1	8			1.	
389	389	386	387	0	0	0	0
340	4	1	8			2.	
364	355	354	363	0	0	0	0
341	4	1	8			2.	
406	364	363	390	0	0	0	0
342	4	1	8	1		2.	
407	406	390	391	0	0	0	0
353	4	1	8			2.	
418	417	401	402	0	0	0	0
354	4	1	8			2.	
405	418	402	403	0	0	0	0
355	4	1	8			2.	
405	405	403	404	0	0	0	0
356	4	1	8			3.	
365	356	355	364	0	0	0	0
357	4	1	8			3.	
420	365	364	406	0	0	0	0
358	4	1	8	1		3.	
421	420	406	407	0	0	0	0
367	4	1	8			3.	
430	429	415	416	0	0	0	0
368	4	1	8			3.	
419	430	416	417	0	0	0	0
369	4	1	8			3.	
419	419	417	418	0	0	0	0
370	4	1	8			4.	
366	357	356	365	0	0	0	0
371	4	1	8			4.	
432	366	365	420	0	0	0	0
372	4	1	8	1		4.	
433	432	420	421	0	0	0	0
379	4	1	8			4.	
440	439	427	428	0	0	0	0
380	4	1	8			4.	
431	440	428	429	0	0	0	0
381	4	1	8			4.	
431	431	429	430	0	0	0	0
382	4	1	8			5.	
367	358	357	366	0	0	0	0
383	4	1	8			5.	
442	367	366	432	0	0	0	0
384	4	1	8	1		5.	
443	442	432	433	0	0	0	0
389	4	1	8			5.	
448	447	437	438	0	0	0	0
390	4	1	8			5.	
441	448	438	439	0	0	0	0
391	4	1	8			5.	

441	441	439	440	0	0	0	0
392	4	1	8			6.	
368	359	358	367	0	0	0	0
393	4	1	8			6.	
450	368	367	442	0	0	0	0
394	4	1	8	1		6.	
451	450	442	443	0	0	0	0
397	4	1	8			6.	
454	453	445	446	0	0	0	0
398	4	1	8			6.	
449	454	446	447	0	0	0	0
399	4	1	8			6.	
449	449	447	448	0	0	0	0
400	4	1	8			7.	
369	360	359	368	0	0	0	0
401	4	1	8			7.	
456	369	368	450	0	0	0	0
402	4	1	8			7.	
457	456	450	451	0	0	0	0
403	4	1	8			7.	
458	457	451	452	0	0	0	0
404	4	1	8			7.	
455	458	452	453	0	0	0	0
405	4	1	8			7.	
455	455	453	454	0	0	0	0
406	4	1	8			8.	
370	361	360	369	0	0	0	0
407	4	1	8			8.	
460	370	369	456	0	0	0	0
408	4	1	8			8.	
459	460	456	457	0	0	0	0
409	4	1	8			8.	
459	459	457	458	0	0	0	0
1	20	2	2	1	8		

1	0.								
	0.	.067	.083	.16	0.	.14	.16	.2	
1	1	9	0	1					
2	9	17	0	1					
3	17	25	0	1					
4	25	33	0	1					
5	33	41	0	1					
6	41	49	0	1					
7	49	57	0	1					
8	57	65	0	1					
9	65	73	0	1					
10	73	81	0	1					
11	81	89	0	1					
12	89	97	0	1					
13	97	105	0	1					
14	105	113	0	1					
15	113	121	0	1					
16	121	129	0	1					
17	129	137	0	1					
18	137	145	0	1					
19	145	153	0	1					
20	153	161	0	1

Figure A.7.1 Case 6 - Contours of Horizontal Displacement

Figure A.7.2 Case 6 - Contours of Vertical Displacement

Figure A.7.3 Case 6 - Contours of Principal Stress Ratio

Figure A.7.4 Case 6 - Contours of Local Factors of Safety

Computer Program Input

- Figure A.8.1 Case 7 - Contours of Horizontal Displacement
- Figure A.8.2 Case 7 - Contours of Vertical Displacement
- Figure A.8.3 Case 7 - Contours of Principal Stress Ratio
- Figure A.8.4 Case 7 - Contours of Local Factors of Safety

State Route 1 over Ramsey Creek - Franklin County, Indiana Case 7

460	0	2	1	18	1
1	0	15	.01		
0.0		0.5	9.0		
1	1	0	0.000	0.000	
2	1	0	0.000	-7.500	1
7	1	0	0.000	-27.500	
8	1	1	0.000	-35.000	
9	0	0	12.500	0.000	
17	0	0	25.000	0.000	8
337	0	0	625.000	0.000	
345	1	0	640.000	0.000	
10	0	0	12.500	-7.500	1
15	0	0	12.500	-27.500	
16	0	1	12.500	-35.000	
18	0	0	25.000	-7.500	8
338	0	0	625.000	-7.500	
346	1	0	640.000	-7.500	1
351	1	0	640.000	-27.500	
352	1	1	640.000	-35.000	
19	0	0	25.000	-11.500	8
339	0	0	625.000	-11.500	
20	0	0	25.000	-15.500	8
340	0	0	625.000	-15.500	
21	0	0	25.000	-19.500	8
341	0	0	625.000	-19.500	
22	0	0	25.000	-23.500	8
342	0	0	625.000	-23.500	
23	0	0	25.000	-27.500	8
343	0	0	625.000	-27.500	
24	0	1	25.000	-35.000	8
344	0	1	625.000	-35.000	
353	1	0	0.000	10.000	1
361	1	0	0.000	90.000	
362	0	0	12.500	10.000	1
370	0	0	12.500	90.000	
371	0	0	25.000	10.000	1
387	0	0	265.000	10.000	
388	0	0	280.000	5.000	
389	0	0	250.000	15.000	
390	0	0	25.000	20.000	1
404	0	0	235.000	20.000	
405	0	0	220.000	25.000	
406	0	0	25.000	30.000	1
418	0	0	205.000	30.000	
419	0	0	190.000	35.000	
420	0	0	25.000	40.000	1
430	0	0	175.000	40.000	
431	0	0	160.000	45.000	
432	0	0	25.000	50.000	1
440	0	0	145.000	50.000	
441	0	0	130.000	55.000	
442	0	0	25.000	60.000	1
448	0	0	115.000	60.000	
449	0	0	100.000	65.000	
450	0	0	25.000	70.000	1
454	0	0	85.000	70.000	
455	0	0	70.000	75.000	
456	0	0	25.000	80.000	1
458	0	0	55.000	80.000	

459	0	0	40.000	85.000					
460	0	0	25.000	90.000					
8									
1									
0.	640.		-7.5	0.	0.	0.			
0.	0.		0.	-11.5	0.125	0.0626	21.	8.0	
1									
0.	640.		-11.5	-7.51	0.	0.			
0.	0.		0.	-11.5	0.120	0.0576	21.	7.20	
2									
0.	640.		-15.5	-11.51	0.	0.			
0.	0.		0.	-11.5	0.120	0.0576	21.	4.9	
2									
0.	640.		-19.5	-15.51	0.	0.			
0.	0.		0.	-11.5	0.120	0.0576	21.	4.30	
2									
0.	640.		-23.5	-19.51	0.	0.			
0.	0.		0.	-11.5	0.120	0.0576	21.	3.8	
2									
0.	640.		-27.5	-23.51	0.	0.			
0.	0.		0.	-11.5	0.120	0.0576	21.	3.40	
2									
0.	640.		-35.0	-27.51	0.	0.			
0.	0.		0.	-11.5	0.130	0.0676	21.	2.5	
3									
0.	295.		0.	90.	0.	0.			
0.1	0.50								
0	0	0							
2	409	4	10	8	13	4	2	16	
1									
300.	0.4		0.7500	21.	0.720	0.230	0.039	1.58	
8.0	0.01		0.000	10.	2.				
2									
300.	0.4		0.7500	21.	0.720	0.250	0.050	1.46	
7.20	0.01		0.000	10.	2.				
3									
300.	0.4		0.7500	21.	0.720	0.250	0.050	1.07	
4.9	0.01		0.000	10.	2.				
4									
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.96	
4.30	0.01		0.000	10.	2.				
5									
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.87	
3.8	0.01		0.000	10.	2.				
6									
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.80	
3.40	0.01		0.000	10.	2.				
7									
300.	0.4		0.6630	21.	0.720	0.124	0.020	0.63	
2.5	0.01		0.000	10.	2.				
8									
300.	0.4		0.7500	60.	0.000	0.124	0.020	1.58	
20.0	0.01		0.125	0.	2.				
1	4	1	1	8	0.				
9	1	2	10	0	0	0	0		
43	4	1	1		0.				
345	337	338	346	0	0	0	0		
44	4	1	2	8	0.				
10	2	3	11	0	0	0	0		
86	4	1	2		0.				

346	338	339	347	0	0	0	0
87	4	1	3	8		0.	
11	3	4	12	0	0	0	0
129	4	1	3			0.	
347	339	340	348	0	0	0	0
130	4	1	4	8		0.	
12	4	5	13	0	0	0	0
172	4	1	4			0.	
348	340	341	349	0	0	0	0
173	4	1	5	8		0.	
13	5	6	14	0	0	0	0
215	4	1	5			0.	
349	341	342	350	0	0	0	0
216	4	1	6	8		0.	
14	6	7	15	0	0	0	0
258	4	1	6			0.	
350	342	343	351	0	0	0	0
259	4	1	7	8		0.	
15	7	8	16	0	0	0	0
301	4	1	7			0.	
351	343	344	352	0	0	0	0
302	4	1	8		0.01		
362	353	1	9	0	0	0	0
303	4	1	8		0.01		
371	362	9	17	0	0	0	0
304	4	1	8		0.01		
372	371	17	25	0	0	0	0
305	4	1	8		0.01		
373	372	25	33	0	0	0	0
306	4	1	8		0.01		
374	373	33	41	0	0	0	0
307	4	1	8		0.01		
375	374	41	49	0	0	0	0
308	4	1	8		0.01		
376	375	49	57	0	0	0	0
309	4	1	8		0.01		
377	376	57	65	0	0	0	0
310	4	1	8		0.01		
378	377	65	73	0	0	0	0
311	4	1	8		0.01		
379	378	73	81	0	0	0	0
312	4	1	8		0.01		
380	379	81	89	0	0	0	0
313	4	1	8		0.01		
381	380	89	97	0	0	0	0
314	4	1	8		0.01		
382	381	97	105	0	0	0	0
315	4	1	8		0.01		
383	382	105	113	0	0	0	0
316	4	1	8		0.01		
384	383	113	121	0	0	0	0
317	4	1	8		0.01		
385	384	121	129	0	0	0	0
318	4	1	8		0.01		
386	385	129	137	0	0	0	0
319	4	1	8		0.01		
387	386	137	145	0	0	0	0
320	4	1	8		0.01		
388	387	145	153	0	0	0	0
321	4	1	8		0.01		

388	388	153	161	0	0	0	0
322	4	1	8			1.	
363	354	353	362	0	0	0	0
323	4	1	8			1.	
390	363	362	371	0	0	0	0
324	4	1	8	1		1.	
391	390	371	372	0	0	0	0
337	4	1	8			1.	
404	403	384	385	0	0	0	0
338	4	1	8			1.	
389	404	385	386	0	0	0	0
339	4	1	8			1.	
389	389	386	387	0	0	0	0
340	4	1	8			2.	
364	355	354	363	0	0	0	0
341	4	1	8			2.	
406	364	363	390	0	0	0	0
342	4	1	8	1		2.	
407	406	390	391	0	0	0	0
353	4	1	8			2.	
418	417	401	402	0	0	0	0
354	4	1	8			2.	
405	418	402	403	0	0	0	0
355	4	1	8			2.	
405	405	403	404	0	0	0	0
356	4	1	8			3.	
365	356	355	364	0	0	0	0
357	4	1	8			3.	
420	365	364	406	0	0	0	0
358	4	1	8	1		3.	
421	420	406	407	0	0	0	0
367	4	1	8			3.	
430	429	415	416	0	0	0	0
368	4	1	8			3.	
419	430	416	417	0	0	0	0
369	4	1	8			3.	
419	419	417	418	0	0	0	0
370	4	1	8			4.	
366	357	356	365	0	0	0	0
371	4	1	8			4.	
432	366	365	420	0	0	0	0
372	4	1	8	1		4.	
433	432	420	421	0	0	0	0
379	4	1	8			4.	
440	439	427	428	0	0	0	0
380	4	1	8			4.	
431	440	428	429	0	0	0	0
381	4	1	8			4.	
431	431	429	430	0	0	0	0
382	4	1	8			5.	
367	358	357	366	0	0	0	0
383	4	1	8			5.	
442	367	366	432	0	0	0	0
384	4	1	8	1		5.	
443	442	432	433	0	0	0	0
389	4	1	8			5.	
448	447	437	438	0	0	0	0
390	4	1	8			5.	
441	448	438	439	0	0	0	0
391	4	1	8			5.	

441	441	439	440	0	0	0	0
392	4	1	8			6.	
368	359	358	367	0	0	0	0
393	4	1	8			6.	
450	368	367	442	0	0	0	0
394	4	1	8	1		6.	
451	450	442	443	0	0	0	0
397	4	1	8			6.	
454	453	445	446	0	0	0	0
398	4	1	8			6.	
449	454	446	447	0	0	0	0
399	4	1	8			6.	
449	449	447	448	0	0	0	0
400	4	1	8			7.	
369	360	359	368	0	0	0	0
401	4	1	8			7.	
456	369	368	450	0	0	0	0
402	4	1	8			7.	
457	456	450	451	0	0	0	0
403	4	1	8			7.	
458	457	451	452	0	0	0	0
404	4	1	8			7.	
455	458	452	453	0	0	0	0
405	4	1	8			7.	
455	455	453	454	0	0	0	0
406	4	1	8			8.	
370	361	360	369	0	0	0	0
407	4	1	8			8.	
460	370	369	456	0	0	0	0
408	4	1	8			8.	
459	460	456	457	0	0	0	0
409	4	1	8			8.	
459	459	457	458	0	0	0	0
1	20	2	2	1	8		

1	0.								
	0.	.067	.083	.16	0.	1.68	1.92	2.4	
1	1	9	0	1					
2	9	17	0	1					
3	17	25	0	1					
4	25	33	0	1					
5	33	41	0	1					
6	41	49	0	1					
7	49	57	0	1					
8	57	65	0	1					
9	65	73	0	1					
10	73	81	0	1					
11	81	89	0	1					
12	89	97	0	1					
13	97	105	0	1					
14	105	113	0	1					
15	113	121	0	1					
16	121	129	0	1					
17	129	137	0	1					
18	137	145	0	1					
19	145	153	0	1					
20	153	161	0	1

Figure A.8.1 Case 7 - Contours of Horizontal Displacement

Figure A.8.2 Case 7 - Contours of Vertical Displacement

Figure A.8.3 Case 7 - Contours of Principal Stress Ratio

Figure A.8.4 Case 7 - Contours of Local Factors of Safety

Computer Program Input

- Figure A.9.1 Case 8 - Contours of Horizontal Displacement
- Figure A.9.2 Case 8 - Contours of Vertical Displacement
- Figure A.9.3 Case 8 - Contours of Principal Stress Ratio
- Figure A.9.4 Case 8 - Contours of Local Factors of Safety

State Route 1 over Ramsey Creek - Franklin County, Indiana Case 8

460	0	2	1	18	1
1	0	15	.01		
0.0	0.5	9.0			
1	1	0	0.000	0.000	
2	1	0	0.000	-7.500	1
7	1	0	0.000	-27.500	
8	1	1	0.000	-35.000	
9	0	0	12.500	0.000	
17	0	0	25.000	0.000	8
337	0	0	625.000	0.000	
345	1	0	640.000	0.000	
10	0	0	12.500	-7.500	1
15	0	0	12.500	-27.500	
16	0	1	12.500	-35.000	
18	0	0	25.000	-7.500	8
338	0	0	625.000	-7.500	
346	1	0	640.000	-7.500	1
351	1	0	640.000	-27.500	
352	1	1	640.000	-35.000	
19	0	0	25.000	-11.500	8
339	0	0	625.000	-11.500	
20	0	0	25.000	-15.500	8
340	0	0	625.000	-15.500	
21	0	0	25.000	-19.500	8
341	0	0	625.000	-19.500	
22	0	0	25.000	-23.500	8
342	0	0	625.000	-23.500	
23	0	0	25.000	-27.500	8
343	0	0	625.000	-27.500	
24	0	1	25.000	-35.000	8
344	0	1	625.000	-35.000	
353	1	0	0.000	10.000	1
361	1	0	0.000	90.000	
362	0	0	12.500	10.000	1
370	0	0	12.500	90.000	
371	0	0	25.000	10.000	1
387	0	0	265.000	10.000	
388	0	0	280.000	5.000	
389	0	0	250.000	15.000	
390	0	0	25.000	20.000	1
404	0	0	235.000	20.000	
405	0	0	220.000	25.000	
406	0	0	25.000	30.000	1
418	0	0	205.000	30.000	
419	0	0	190.000	35.000	
420	0	0	25.000	40.000	1
430	0	0	175.000	40.000	
431	0	0	160.000	45.000	
432	0	0	25.000	50.000	1
440	0	0	145.000	50.000	
441	0	0	130.000	55.000	
442	0	0	25.000	60.000	1
448	0	0	115.000	60.000	
449	0	0	100.000	65.000	
450	0	0	25.000	70.000	1
454	0	0	85.000	70.000	
455	0	0	70.000	75.000	
456	0	0	25.000	80.000	1
458	0	0	55.000	80.000	

459	0	0	40.000	85.000				
460	0	0	25.000	90.000				
8								
1								
0.	640.		-7.5	0.	0.	0.		
0.	0.		0.	-11.5	0.125	0.0626	21.	8.0
1								
0.	640.		-11.5	-7.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	7.20
2								
0.	640.		-15.5	-11.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	4.9
2								
0.	640.		-19.5	-15.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	4.30
2								
0.	640.		-23.5	-19.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	3.8
2								
0.	640.		-27.5	-23.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	3.40
2								
0.	640.		-35.0	-27.51	0.	0.		
0.	0.		0.	-11.5	0.130	0.0676	21.	2.5
3								
0.	295.		0.	90.	0.	0.		
0.1	0.50							
0	0	0						
2	409	4	10	8	13	4	2	16
1								
300.	0.4		0.7500	21.	0.720	0.230	0.039	1.58
8.0	0.01		0.000	10.	2.			
2								
300.	0.4		0.7500	21.	0.720	0.250	0.050	1.46
7.20	0.01		0.000	10.	2.			
3								
300.	0.4		0.7500	21.	0.720	0.250	0.050	1.07
4.9	0.01		0.000	10.	2.			
4								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.96
4.30	0.01		0.000	10.	2.			
5								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.87
3.8	0.01		0.000	10.	2.			
6								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.80
3.40	0.01		0.000	10.	2.			
7								
300.	0.4		0.6630	21.	0.720	0.124	0.020	0.63
2.5	0.01		0.000	10.	2.			
8								
300.	0.4		0.7500	60.	0.000	0.124	0.020	0.73
3.0	0.01		0.125	0.	2.			
1	4	1	1	8	0.			
9	1	2	10	0	0	0	0	
43	4	1	1		0.			
345	337	338	346	0	0	0	0	
44	4	1	2	8	0.			
10	2	3	11	0	0	0	0	
86	4	1	2		0.			

346	338	339	347	0	0	0	0
87	4	1	3	8		0.	
11	3	4	12	0	0	0	0
129	4	1	3			0.	
347	339	340	348	0	0	0	0
130	4	1	4	8		0.	
12	4	5	13	0	0	0	0
172	4	1	4			0.	
348	340	341	349	0	0	0	0
173	4	1	5	8		0.	
13	5	6	14	0	0	0	0
215	4	1	5			0.	
349	341	342	350	0	0	0	0
216	4	1	6	8		0.	
14	6	7	15	0	0	0	0
258	4	1	6			0.	
350	342	343	351	0	0	0	0
259	4	1	7	8		0.	
15	7	8	16	0	0	0	0
301	4	1	7			0.	
351	343	344	352	0	0	0	0
302	4	1	8		0.01		
362	353	1	9	0	0	0	0
303	4	1	8		0.01		
371	362	9	17	0	0	0	0
304	4	1	8		0.01		
372	371	17	25	0	0	0	0
305	4	1	8		0.01		
373	372	25	33	0	0	0	0
306	4	1	8		0.01		
374	373	33	41	0	0	0	0
307	4	1	8		0.01		
375	374	41	49	0	0	0	0
308	4	1	8		0.01		
376	375	49	57	0	0	0	0
309	4	1	8		0.01		
377	376	57	65	0	0	0	0
310	4	1	8		0.01		
378	377	65	73	0	0	0	0
311	4	1	8		0.01		
379	378	73	81	0	0	0	0
312	4	1	8		0.01		
380	379	81	89	0	0	0	0
313	4	1	8		0.01		
381	380	89	97	0	0	0	0
314	4	1	8		0.01		
382	381	97	105	0	0	0	0
315	4	1	8		0.01		
383	382	105	113	0	0	0	0
316	4	1	8		0.01		
384	383	113	121	0	0	0	0
317	4	1	8		0.01		
385	384	121	129	0	0	0	0
318	4	1	8		0.01		
386	385	129	137	0	0	0	0
319	4	1	8		0.01		
387	386	137	145	0	0	0	0
320	4	1	8		0.01		
388	387	145	153	0	0	0	0
321	4	1	8		0.01		

388	388	153	161	0	0	0	0
322	4	1	8			1.	
363	354	353	362	0	0	0	0
323	4	1	8			1.	
390	363	362	371	0	0	0	0
324	4	1	8	1		1.	
391	390	371	372	0	0	0	0
337	4	1	8			1.	
404	403	384	385	0	0	0	0
338	4	1	8			1.	
389	404	385	386	0	0	0	0
339	4	1	8			1.	
389	389	386	387	0	0	0	0
340	4	1	8			2.	
364	355	354	363	0	0	0	0
341	4	1	8			2.	
406	364	363	390	0	0	0	0
342	4	1	8	1		2.	
407	406	390	391	0	0	0	0
353	4	1	8			2.	
418	417	401	402	0	0	0	0
354	4	1	8			2.	
405	418	402	403	0	0	0	0
355	4	1	8			2.	
405	405	403	404	0	0	0	0
356	4	1	8			3.	
365	356	355	364	0	0	0	0
357	4	1	8			3.	
420	365	364	406	0	0	0	0
358	4	1	8	1		3.	
421	420	406	407	0	0	0	0
367	4	1	8			3.	
430	429	415	416	0	0	0	0
368	4	1	8			3.	
419	430	416	417	0	0	0	0
369	4	1	8			3.	
419	419	417	418	0	0	0	0
370	4	1	8			4.	
366	357	356	365	0	0	0	0
371	4	1	8			4.	
432	366	365	420	0	0	0	0
372	4	1	8	1		4.	
433	432	420	421	0	0	0	0
379	4	1	8			4.	
440	439	427	428	0	0	0	0
380	4	1	8			4.	
431	440	428	429	0	0	0	0
381	4	1	8			4.	
431	431	429	430	0	0	0	0
382	4	1	8			5.	
367	358	357	366	0	0	0	0
383	4	1	8			5.	
442	367	366	432	0	0	0	0
384	4	1	8	1		5.	
443	442	432	433	0	0	0	0
389	4	1	8			5.	
448	447	437	438	0	0	0	0
390	4	1	8			5.	
441	448	438	439	0	0	0	0
391	4	1	8			5.	

441	441	439	440	0	0	0	0				
392	4	1	8			6.					
368	359	358	367	0	0	0	0				
393	4	1	8			6.					
450	368	367	442	0	0	0	0				
394	4	1	8	1		6.					
451	450	442	443	0	0	0	0				
397	4	1	8			6.					
454	453	445	446	0	0	0	0				
398	4	1	8			6.					
449	454	446	447	0	0	0	0				
399	4	1	8			6.					
449	449	447	448	0	0	0	0				
400	4	1	8			7.					
369	360	359	368	0	0	0	0				
401	4	1	8			7.					
456	369	368	450	0	0	0	0				
402	4	1	8			7.					
457	456	450	451	0	0	0	0				
403	4	1	8			7.					
458	457	451	452	0	0	0	0				
404	4	1	8			7.					
455	458	452	453	0	0	0	0				
405	4	1	8			7.					
455	455	453	454	0	0	0	0				
406	4	1	8			8.					
370	361	360	369	0	0	0	0				
407	4	1	8			8.					
460	370	369	456	0	0	0	0				
408	4	1	8			8.					
459	460	456	457	0	0	0	0				
409	4	1	8			8.					
459	459	457	458	0	0	0	0				
1	42	2	2	3	8						
1	0.										
	0.	.067	.083	.16	0.	1.68	1.92	2.4			
2	6.										
	0.	.067	.083	.16	0.	1.68	1.92	2.4			
3	12.										
	0.	.067	.083	.16	0.	1.68	1.92	2.4			
1	1	9	0	1							
2	9	17	0	1							
3	17	25	0	1							
4	25	33	0	1							
5	33	41	0	1							
6	41	49	0	1							
7	49	57	0	1							
8	57	65	0	1							
9	65	73	0	1							
10	73	81	0	1							
11	81	89	0	1							
12	89	97	0	1							
13	97	105	0	1							
14	105	113	0	1							
15	113	121	0	1							
16	121	129	0	1							
17	129	137	0	1							
18	137	145	0	1							
19	145	153	0	1							
20	153	161	0	1							

21	355	364	0	2
22	364	406	0	2
23	406	407	0	2
24	407	408	0	2
25	408	409	0	2
26	409	410	0	2
27	410	411	0	2
28	411	412	0	2
29	412	413	0	2
30	413	414	0	2
31	414	415	0	2
32	415	416	0	2
33	416	417	0	2
34	417	418	0	2
35	358	367	0	3
36	367	442	0	3
37	442	443	0	3
38	443	444	0	3
39	444	445	0	3
40	445	446	0	3
41	446	447	0	3
42	447	448	0	3

Figure A.9.1 Case 8 - Contours of Horizontal Displacement

Figure A.9.2 Case 8 - Contours of Vertical Displacement

Depth (ft)

Contour Interval; 8

Distance from Centerline of Embankment (ft)

Figure A.9.3 Case 8 - Contours of Principal Stress Ratio

Figure A.9.4 Case 8 - Contours of Local Factors of Safety

Computer Program Input

Figure A.10.1 Case 9 - Contours of Horizontal Displacement

Figure A.10.2 Case 9 - Contours of Vertical Displacement

Figure A.10.3 Case 9 - Contours of Principal Stress Ratio

Figure A.10.4 Case 9 - Contours of Local Factors of Safety

State Route 1 over Ramsey Creek - Franklin County, Indiana Case 9

460	0	1	1	18	1
1	0	15	.01		
0.0	0.5	9.0			
1	1	0	0.000	0.000	
2	1	0	0.000	-7.500	1
7	1	0	0.000	-27.500	
8	1	1	0.000	-35.000	
9	0	0	12.500	0.000	
17	0	0	25.000	0.000	8
337	0	0	625.000	0.000	
345	1	0	640.000	0.000	
10	0	0	12.500	-7.500	1
15	0	0	12.500	-27.500	
16	0	1	12.500	-35.000	
18	0	0	25.000	-7.500	8
338	0	0	625.000	-7.500	
346	1	0	640.000	-7.500	1
351	1	0	640.000	-27.500	
352	1	1	640.000	-35.000	
19	0	0	25.000	-11.500	8
339	0	0	625.000	-11.500	
20	0	0	25.000	-15.500	8
340	0	0	625.000	-15.500	
21	0	0	25.000	-19.500	8
341	0	0	625.000	-19.500	
22	0	0	25.000	-23.500	8
342	0	0	625.000	-23.500	
23	0	0	25.000	-27.500	8
343	0	0	625.000	-27.500	
24	0	1	25.000	-35.000	8
344	0	1	625.000	-35.000	
353	1	0	0.000	10.000	1
361	1	0	0.000	90.000	
362	0	0	12.500	10.000	1
370	0	0	12.500	90.000	
371	0	0	25.000	10.000	1
387	0	0	265.000	10.000	
388	0	0	280.000	5.000	
389	0	0	250.000	15.000	
390	0	0	25.000	20.000	1
404	0	0	235.000	20.000	
405	0	0	220.000	25.000	
406	0	0	25.000	30.000	1
418	0	0	205.000	30.000	
419	0	0	190.000	35.000	
420	0	0	25.000	40.000	1
430	0	0	175.000	40.000	
431	0	0	160.000	45.000	
432	0	0	25.000	50.000	1
440	0	0	145.000	50.000	
441	0	0	130.000	55.000	
442	0	0	25.000	60.000	1
448	0	0	115.000	60.000	
449	0	0	100.000	65.000	
450	0	0	25.000	70.000	1
454	0	0	85.000	70.000	
455	0	0	70.000	75.000	
456	0	0	25.000	80.000	1
458	0	0	55.000	80.000	

459	0	0	40.000	85.000				
460	0	0	25.000	90.000				
8								
1								
0.	640.		-7.5	0.	0.	0.		
0.	0.		0.	-11.5	0.125	0.0626	21.	8.0
1								
0.	640.		-11.5	-7.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	7.20
2								
0.	640.		-15.5	-11.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	4.9
2								
0.	640.		-19.5	-15.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	4.30
2								
0.	640.		-23.5	-19.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	3.8
2								
0.	640.		-27.5	-23.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	3.40
2								
0.	640.		-35.0	-27.51	0.	0.		
0.	0.		0.	-11.5	0.130	0.0676	21.	2.5
3								
0.	295.		0.	90.	0.	0.		
0.1	0.50							
0	0	0						
2	409	4	10	8	13	4	2	16
1								
300.	0.4		0.7500	21.	0.720	0.230	0.039	1.58
8.0	0.01		0.000	10.	2.			
2								
300.	0.4		0.7500	21.	0.720	0.250	0.050	1.46
7.20	0.01		0.000	10.	2.			
3								
300.	0.4		0.7500	21.	0.720	0.250	0.050	1.07
4.9	0.01		0.000	10.	2.			
4								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.96
4.30	0.01		0.000	10.	2.			
5								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.87
3.8	0.01		0.000	10.	2.			
6								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.80
3.40	0.01		0.000	10.	2.			
7								
300.	0.4		0.6630	21.	0.720	0.124	0.020	0.63
2.5	0.01		0.000	10.	2.			
8								
300.	0.4		0.7500	60.	0.000	0.124	0.020	1.58
8.0	0.01		0.125	0.	2.			
1	4	1	1	8	0.			
9	1	2	10	0	0	0	0	
43	4	1	1		0.			
345	337	338	346	0	0	0	0	
44	4	1	2	8	0.			
10	2	3	11	0	0	0	0	
86	4	1	2		0.			

346	338	339	347	0	0	0	0
87	4	1	3	8		0.	
11	3	4	12	0	0	0	0
129	4	1	3			0.	
347	339	340	348	0	0	0	0
130	4	1	4	8		0.	
12	4	5	13	0	0	0	0
172	4	1	4			0.	
348	340	341	349	0	0	0	0
173	4	1	5	8		0.	
13	5	6	14	0	0	0	0
215	4	1	5			0.	
349	341	342	350	0	0	0	0
216	4	1	6	8		0.	
14	6	7	15	0	0	0	0
258	4	1	6			0.	
350	342	343	351	0	0	0	0
259	4	1	7	8		0.	
15	7	8	16	0	0	0	0
301	4	1	7			0.	
351	343	344	352	0	0	0	0
302	4	1	8	0.01			
362	353	1	9	0	0	0	0
303	4	1	8	0.01			
371	362	9	17	0	0	0	0
304	4	1	8	0.01			
372	371	17	25	0	0	0	0
305	4	1	8	0.01			
373	372	25	33	0	0	0	0
306	4	1	8	0.01			
374	373	33	41	0	0	0	0
307	4	1	8	0.01			
375	374	41	49	0	0	0	0
308	4	1	8	0.01			
376	375	49	57	0	0	0	0
309	4	1	8	0.01			
377	376	57	65	0	0	0	0
310	4	1	8	0.01			
378	377	65	73	0	0	0	0
311	4	1	8	0.01			
379	378	73	81	0	0	0	0
312	4	1	8	0.01			
380	379	81	89	0	0	0	0
313	4	1	8	0.01			
381	380	89	97	0	0	0	0
314	4	1	8	0.01			
382	381	97	105	0	0	0	0
315	4	1	8	0.01			
383	382	105	113	0	0	0	0
316	4	1	8	0.01			
384	383	113	121	0	0	0	0
317	4	1	8	0.01			
385	384	121	129	0	0	0	0
318	4	1	8	0.01			
386	385	129	137	0	0	0	0
319	4	1	8	0.01			
387	386	137	145	0	0	0	0
320	4	1	8	0.01			
388	387	145	153	0	0	0	0
321	4	1	8	0.01			

388	388	153	161	0	0	0	0
322	4	1	8			1.	
363	354	353	362	0	0	0	0
323	4	1	8			1.	
390	363	362	371	0	0	0	0
324	4	1	8	1		1.	
391	390	371	372	0	0	0	0
337	4	1	8			1.	
404	403	384	385	0	0	0	0
338	4	1	8			1.	
389	404	385	386	0	0	0	0
339	4	1	8			1.	
389	389	386	387	0	0	0	0
340	4	1	8			2.	
364	355	354	363	0	0	0	0
341	4	1	8			2.	
406	364	363	390	0	0	0	0
342	4	1	8	1		2.	
407	406	390	391	0	0	0	0
353	4	1	8			2.	
418	417	401	402	0	0	0	0
354	4	1	8			2.	
405	418	402	403	0	0	0	0
355	4	1	8			2.	
405	405	403	404	0	0	0	0
356	4	1	8			3.	
365	356	355	364	0	0	0	0
357	4	1	8			3.	
420	365	364	406	0	0	0	0
358	4	1	8	1		3.	
421	420	406	407	0	0	0	0
367	4	1	8			3.	
430	429	415	416	0	0	0	0
368	4	1	8			3.	
419	430	416	417	0	0	0	0
369	4	1	8			3.	
419	419	417	418	0	0	0	0
370	4	1	8			4.	
366	357	356	365	0	0	0	0
371	4	1	8			4.	
432	366	365	420	0	0	0	0
372	4	1	8	1		4.	
433	432	420	421	0	0	0	0
379	4	1	8			4.	
440	439	427	428	0	0	0	0
380	4	1	8			4.	
431	440	428	429	0	0	0	0
381	4	1	8			4.	
431	431	429	430	0	0	0	0
382	4	1	8			5.	
367	358	357	366	0	0	0	0
383	4	1	8			5.	
442	367	366	432	0	0	0	0
384	4	1	8	1		5.	
443	442	432	433	0	0	0	0
389	4	1	8			5.	
448	447	437	438	0	0	0	0
390	4	1	8			5.	
441	448	438	439	0	0	0	0
391	4	1	8			5.	

441	441	439	440	0	0	0	0
392	4	1	8			6.	
368	359	358	367	0	0	0	0
393	4	1	8			6.	
450	368	367	442	0	0	0	0
394	4	1	8	1		6.	
451	450	442	443	0	0	0	0
397	4	1	8			6.	
454	453	445	446	0	0	0	0
398	4	1	8			6.	
449	454	446	447	0	0	0	0
399	4	1	8			6.	
449	449	447	448	0	0	0	0
400	4	1	8			7.	
369	360	359	368	0	0	0	0
401	4	1	8			7.	
456	369	368	450	0	0	0	0
402	4	1	8			7.	
457	456	450	451	0	0	0	0
403	4	1	8			7.	
458	457	451	452	0	0	0	0
404	4	1	8			7.	
455	458	452	453	0	0	0	0
405	4	1	8			7.	
455	455	453	454	0	0	0	0
406	4	1	8			8.	
370	361	360	369	0	0	0	0
407	4	1	8			8.	
460	370	369	456	0	0	0	0
408	4	1	8			8.	
459	460	456	457	0	0	0	0
409	4	1	8			8.	
459	459	457	458	0	0	0	0

Figure A.10.1 Case 9 - Contours of Horizontal Displacement

Figure A.10.2 Case 9 - Contours of Vertical Displacement

Figure A.10.3 Case 9 - Contours of Principal Stress Ratio

Figure A.10.4 Case 9 - Contours of Local Factors of Safety

Computer Program Input

Figure A.11.1 Case 10 - Contours of Horizontal Displacement

Figure A.11.2 Case 10 - Contours of Vertical Displacement

Figure A.11.3 Case 10 - Contours of Principal Stress Ratio

Figure A.11.4 Case 10 - Contours of Local Factors of Safety

State Route 1 over Ramsey Creek - Franklin County, Indiana Case 10

460	0	1	1	18	1
1	0	15	.01		
0.0	0.5	9.0			
1	1	0	0.000	0.000	
2	1	0	0.000	-7.500	1
7	1	0	0.000	-27.500	
8	1	1	0.000	-35.000	
9	0	0	12.500	0.000	
17	0	0	25.000	0.000	8
337	0	0	625.000	0.000	
345	1	0	640.000	0.000	
10	0	0	12.500	-7.500	1
15	0	0	12.500	-27.500	
16	0	1	12.500	-35.000	
18	0	0	25.000	-7.500	8
338	0	0	625.000	-7.500	
346	1	0	640.000	-7.500	1
351	1	0	640.000	-27.500	
352	1	1	640.000	-35.000	
19	0	0	25.000	-11.500	8
339	0	0	625.000	-11.500	
20	0	0	25.000	-15.500	8
340	0	0	625.000	-15.500	
21	0	0	25.000	-19.500	8
341	0	0	625.000	-19.500	
22	0	0	25.000	-23.500	8
342	0	0	625.000	-23.500	
23	0	0	25.000	-27.500	8
343	0	0	625.000	-27.500	
24	0	1	25.000	-35.000	8
344	0	1	625.000	-35.000	
353	1	0	0.000	10.000	1
361	1	0	0.000	90.000	
362	0	0	12.500	10.000	1
370	0	0	12.500	90.000	
371	0	0	25.000	10.000	1
387	0	0	265.000	10.000	
388	0	0	280.000	5.000	
389	0	0	250.000	15.000	
390	0	0	25.000	20.000	1
404	0	0	235.000	20.000	
405	0	0	220.000	25.000	
406	0	0	25.000	30.000	1
418	0	0	205.000	30.000	
419	0	0	190.000	35.000	
420	0	0	25.000	40.000	1
430	0	0	175.000	40.000	
431	0	0	160.000	45.000	
432	0	0	25.000	50.000	1
440	0	0	145.000	50.000	
441	0	0	130.000	55.000	
442	0	0	25.000	60.000	1
448	0	0	115.000	60.000	
449	0	0	100.000	65.000	
450	0	0	25.000	70.000	1
454	0	0	85.000	70.000	
455	0	0	70.000	75.000	
456	0	0	25.000	80.000	1
458	0	0	55.000	80.000	

459	0	0	40.000	85.000				
460	0	0	25.000	90.000				
8								
1								
0.	640.		-7.5	0.	0.	0.		
0.	0.		0.	-11.5	0.125	0.0626	21.	10.0
1								
0.	640.		-11.5	-7.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	7.20
2								
0.	640.		-15.5	-11.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	4.9
2								
0.	640.		-19.5	-15.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	4.30
2								
0.	640.		-23.5	-19.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	3.8
2								
0.	640.		-27.5	-23.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	3.40
2								
0.	640.		-35.0	-27.51	0.	0.		
0.	0.		0.	-11.5	0.130	0.0676	21.	2.5
3								
0.	295.		0.	90.	0.	0.		
0.1	0.50							
0	0	0						
2	409	4	10	8	13	4	2	16
1								
300.	0.4		0.7500	21.	0.720	0.230	0.039	1.90
10.0	0.01		0.000	10.	2.			
2								
300.	0.4		0.7500	21.	0.720	0.250	0.050	1.46
7.20	0.01		0.000	10.	2.			
3								
300.	0.4		0.7500	21.	0.720	0.250	0.050	1.07
4.9	0.01		0.000	10.	2.			
4								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.96
4.30	0.01		0.000	10.	2.			
5								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.87
3.8	0.01		0.000	10.	2.			
6								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.80
3.40	0.01		0.000	10.	2.			
7								
300.	0.4		0.6630	21.	0.720	0.124	0.020	0.63
2.5	0.01		0.000	10.	2.			
8								
300.	0.4		0.7500	60.	0.000	0.124	0.020	1.58
8.0	0.01		0.125	0.	2.			
1	4	1	1	8	0.			
9	1	2	10	0	0	0	0	
43	4	1	1		0.			
345	337	338	346	0	0	0	0	
44	4	1	2	8	0.			
10	2	3	11	0	0	0	0	
86	4	1	2		0.			

346	338	339	347	0	0	0	0
87	4	1	3	8		0.	
11	3	4	12	0	0	0	0
129	4	1	3			0.	
347	339	340	348	0	0	0	0
130	4	1	4	8		0.	
12	4	5	13	0	0	0	0
172	4	1	4			0.	
348	340	341	349	0	0	0	0
173	4	1	5	8		0.	
13	5	6	14	0	0	0	0
215	4	1	5			0.	
349	341	342	350	0	0	0	0
216	4	1	6	8		0.	
14	6	7	15	0	0	0	0
258	4	1	6			0.	
350	342	343	351	0	0	0	0
259	4	1	7	8		0.	
15	7	8	16	0	0	0	0
301	4	1	7			0.	
351	343	344	352	0	0	0	0
302	4	1	8		0.01		
362	353	1	9	0	0	0	0
303	4	1	8		0.01		
371	362	9	17	0	0	0	0
304	4	1	8		0.01		
372	371	17	25	0	0	0	0
305	4	1	8		0.01		
373	372	25	33	0	0	0	0
306	4	1	8		0.01		
374	373	33	41	0	0	0	0
307	4	1	8		0.01		
375	374	41	49	0	0	0	0
308	4	1	8		0.01		
376	375	49	57	0	0	0	0
309	4	1	8		0.01		
377	376	57	65	0	0	0	0
310	4	1	8		0.01		
378	377	65	73	0	0	0	0
311	4	1	8		0.01		
379	378	73	81	0	0	0	0
312	4	1	8		0.01		
380	379	81	89	0	0	0	0
313	4	1	8		0.01		
381	380	89	97	0	0	0	0
314	4	1	8		0.01		
382	381	97	105	0	0	0	0
315	4	1	8		0.01		
383	382	105	113	0	0	0	0
316	4	1	8		0.01		
384	383	113	121	0	0	0	0
317	4	1	8		0.01		
385	384	121	129	0	0	0	0
318	4	1	8		0.01		
386	385	129	137	0	0	0	0
319	4	1	8		0.01		
387	386	137	145	0	0	0	0
320	4	1	8		0.01		
388	387	145	153	0	0	0	0
321	4	1	8		0.01		

388	388	153	161	0	0	0	0
322	4	1	8			1.	
363	354	353	362	0	0	0	0
323	4	1	8			1.	
390	363	362	371	0	0	0	0
324	4	1	8	1		1.	
391	390	371	372	0	0	0	0
337	4	1	8			1.	
404	403	384	385	0	0	0	0
338	4	1	8			1.	
389	404	385	386	0	0	0	0
339	4	1	8			1.	
389	389	386	387	0	0	0	0
340	4	1	8			2.	
364	355	354	363	0	0	0	0
341	4	1	8			2.	
406	364	363	390	0	0	0	0
342	4	1	8	1		2.	
407	406	390	391	0	0	0	0
353	4	1	8			2.	
418	417	401	402	0	0	0	0
354	4	1	8			2.	
405	418	402	403	0	0	0	0
355	4	1	8			2.	
405	405	403	404	0	0	0	0
356	4	1	8			3.	
365	356	355	364	0	0	0	0
357	4	1	8			3.	
420	365	364	406	0	0	0	0
358	4	1	8	1		3.	
421	420	406	407	0	0	0	0
367	4	1	8			3.	
430	429	415	416	0	0	0	0
368	4	1	8			3.	
419	430	416	417	0	0	0	0
369	4	1	8			3.	
419	419	417	418	0	0	0	0
370	4	1	8			4.	
366	357	356	365	0	0	0	0
371	4	1	8			4.	
432	366	365	420	0	0	0	0
372	4	1	8	1		4.	
433	432	420	421	0	0	0	0
379	4	1	8			4.	
440	439	427	428	0	0	0	0
380	4	1	8			4.	
431	440	428	429	0	0	0	0
381	4	1	8			4.	
431	431	429	430	0	0	0	0
382	4	1	8			5.	
367	358	357	366	0	0	0	0
383	4	1	8			5.	
442	367	366	432	0	0	0	0
384	4	1	8	1		5.	
443	442	432	433	0	0	0	0
389	4	1	8			5.	
448	447	437	438	0	0	0	0
390	4	1	8			5.	
441	448	438	439	0	0	0	0
391	4	1	8			5.	

441	441	439	440	0	0	0	0
392	4	1	8			6.	
368	359	358	367	0	0	0	0
393	4	1	8			6.	
450	368	367	442	0	0	0	0
394	4	1	8		1	6.	
451	450	442	443	0	0	0	0
397	4	1	8			6.	
454	453	445	446	0	0	0	0
398	4	1	8			6.	
449	454	446	447	0	0	0	0
399	4	1	8			6.	
449	449	447	448	0	0	0	0
400	4	1	8			7.	
369	360	359	368	0	0	0	0
401	4	1	8			7.	
456	369	368	450	0	0	0	0
402	4	1	8			7.	
457	456	450	451	0	0	0	0
403	4	1	8			7.	
458	457	451	452	0	0	0	0
404	4	1	8			7.	
455	458	452	453	0	0	0	0
405	4	1	8			7.	
455	455	453	454	0	0	0	0
406	4	1	8			8.	
370	361	360	369	0	0	0	0
407	4	1	8			8.	
460	370	369	456	0	0	0	0
408	4	1	8			8.	
459	460	456	457	0	0	0	0
409	4	1	8			8.	
459	459	457	458	0	0	0	0

Figure A.11.1 Case 10 - Contours of Horizontal Displacement

Figure A.11.2 Case 10 - Contours of Vertical Displacement

Depth (ft)

Contour Interval; 8

Distance from Centerline of Embankment (ft)

Figure A.11.3 Case 10 - Contours of Principal Stress Ratio

Contour Interval; 0.5

99

Computer Program Input

Figure A.12.1 Case 11 - Contours of Horizontal Displacement

Figure A.12.2 Case 11 - Contours of Vertical Displacement

Figure A.12.3 Case 11 - Contours of Principal Stress Ratio

Figure A.12.4 Case 11 - Contours of Local Factors of Safety

State Route 1 over Ramsey Creek - Franklin County, Indiana Case 11

460	0	1	1	18	1	
1	0	15	.01			
0.0	0.5	9.0				
1	1	0	0.000	0.000		
2	1	0	0.000	-7.500	1	
7	1	0	0.000	-27.500		
8	1	1	0.000	-35.000		
9	0	0	12.500	0.000		
17	0	0	25.000	0.000	8	
337	0	0	625.000	0.000		
345	1	0	640.000	0.000		
10	0	0	12.500	-7.500	1	
15	0	0	12.500	-27.500		
16	0	1	12.500	-35.000		
18	0	0	25.000	-7.500	8	
338	0	0	625.000	-7.500		
346	1	0	640.000	-7.500	1	
351	1	0	640.000	-27.500		
352	1	1	640.000	-35.000		
19	0	0	25.000	-11.500	8	
339	0	0	625.000	-11.500		
20	0	0	25.000	-15.500	8	
340	0	0	625.000	-15.500		
21	0	0	25.000	-19.500	8	
341	0	0	625.000	-19.500		
22	0	0	25.000	-23.500	8	
342	0	0	625.000	-23.500		
23	0	0	25.000	-27.500	8	
343	0	0	625.000	-27.500		
24	0	1	25.000	-35.000	8	
344	0	1	625.000	-35.000		
353	1	0	0.000	10.000	1	
361	1	0	0.000	90.000		
362	0	0	12.500	10.000	1	
370	0	0	12.500	90.000		
371	0	0	25.000	10.000	1	
387	0	0	265.000	10.000		
388	0	0	280.000	5.000		
389	0	0	250.000	15.000		
390	0	0	25.000	20.000	1	
404	0	0	235.000	20.000		
405	0	0	220.000	25.000		
406	0	0	25.000	30.000	1	
418	0	0	205.000	30.000		
419	0	0	190.000	35.000		
420	0	0	25.000	40.000	1	
430	0	0	175.000	40.000		
431	0	0	160.000	45.000		
432	0	0	25.000	50.000	1	
440	0	0	145.000	50.000		
441	0	0	130.000	55.000		
442	0	0	25.000	60.000	1	
448	0	0	115.000	60.000		
449	0	0	100.000	65.000		
450	0	0	25.000	70.000	1	
454	0	0	85.000	70.000		
455	0	0	70.000	75.000		
456	0	0	25.000	80.000	1	
458	0	0	55.000	80.000		

459 0 0 40.000 85.000
 460 0 0 25.000 90.000

8

1

0. 640. -7.5 0. 0. 0.
 0. 0. 0. -11.5 0.125 0.0626 21. 10.0

1

0. 640. -11.5 -7.51 0. 0.
 0. 0. 0. -11.5 0.120 0.0576 21. 3.00

2

0. 640. -15.5 -11.51 0. 0.
 0. 0. 0. -11.5 0.120 0.0576 21. 2.5

2

0. 640. -19.5 -15.51 0. 0.
 0. 0. 0. -11.5 0.120 0.0576 21. 2.00

2

0. 640. -23.5 -19.51 0. 0.
 0. 0. 0. -11.5 0.120 0.0576 21. 1.50

2

0. 640. -27.5 -23.51 0. 0.
 0. 0. 0. -11.5 0.120 0.0576 21. 1.50

2

0. 640. -35.0 -27.51 0. 0.
 0. 0. 0. -11.5 0.130 0.0676 21. 1.02

3

0. 295. 0. 90. 0. 0.
 0.1 0.50

0 0 0

2 409 4 10 8 13 4 2 16

1

300. 0.4 0.7500 21. 0.720 0.230 0.039 1.90
 10.0 0.01 0.000 10. 2.

2

300. 0.4 0.7500 21. 0.720 0.250 0.050 0.73
 3.00 0.01 0.000 10. 2.

3

300. 0.4 0.7500 21. 0.720 0.250 0.050 0.63
 2.50 0.01 0.000 10. 2.

4

300. 0.4 0.7500 21. 0.720 0.250 0.050 0.53
 2.00 0.01 0.000 10. 2.

5

300. 0.4 0.7500 21. 0.720 0.250 0.050 0.42
 1.50 0.01 0.000 10. 2.

6

300. 0.4 0.7500 21. 0.720 0.250 0.050 0.42
 1.50 0.01 0.000 10. 2.

7

300. 0.4 0.6630 21. 0.720 0.124 0.020 0.30
 1.02 0.01 0.000 10. 2.

8

300. 0.4 0.7500 60. 0.000 0.124 0.020 1.58
 8.0 0.01 0.125 0. 2.

1

4 1 1 8 0.

9

1 2 10 0 0 0 0

43

4 1 1 0.

345 337 338 346 0 0 0 0

44

4 1 2 8 0.

10

2 3 11 0 0 0 0

86

4 1 2 0.

346	338	339	347	0	0	0	0
87	4	1	3	8	0.		
11	3	4	12	0	0	0	0
129	4	1	3		0.		
347	339	340	348	0	0	0	0
130	4	1	4	8	0.		
12	4	5	13	0	0	0	0
172	4	1	4		0.		
348	340	341	349	0	0	0	0
173	4	1	5	8	0.		
13	5	6	14	0	0	0	0
215	4	1	5		0.		
349	341	342	350	0	0	0	0
216	4	1	6	8	0.		
14	6	7	15	0	0	0	0
258	4	1	6		0.		
350	342	343	351	0	0	0	0
259	4	1	7	8	0.		
15	7	8	16	0	0	0	0
301	4	1	7		0.		
351	343	344	352	0	0	0	0
302	4	1	8	0.01			
362	353	1	9	0	0	0	0
303	4	1	8	0.01			
371	362	9	17	0	0	0	0
304	4	1	8	0.01			
372	371	17	25	0	0	0	0
305	4	1	8	0.01			
373	372	25	33	0	0	0	0
306	4	1	8	0.01			
374	373	33	41	0	0	0	0
307	4	1	8	0.01			
375	374	41	49	0	0	0	0
308	4	1	8	0.01			
376	375	49	57	0	0	0	0
309	4	1	8	0.01			
377	376	57	65	0	0	0	0
310	4	1	8	0.01			
378	377	65	73	0	0	0	0
311	4	1	8	0.01			
379	378	73	81	0	0	0	0
312	4	1	8	0.01			
380	379	81	89	0	0	0	0
313	4	1	8	0.01			
381	380	89	97	0	0	0	0
314	4	1	8	0.01			
382	381	97	105	0	0	0	0
315	4	1	8	0.01			
383	382	105	113	0	0	0	0
316	4	1	8	0.01			
384	383	113	121	0	0	0	0
317	4	1	8	0.01			
385	384	121	129	0	0	0	0
318	4	1	8	0.01			
386	385	129	137	0	0	0	0
319	4	1	8	0.01			
387	386	137	145	0	0	0	0
320	4	1	8	0.01			
388	387	145	153	0	0	0	0
321	4	1	8	0.01			

388	388	153	161	0	0	0	0
322	4	1	8			1.	
363	354	353	362	0	0	0	0
323	4	1	8			1.	
390	363	362	371	0	0	0	0
324	4	1	8	1		1.	
391	390	371	372	0	0	0	0
337	4	1	8			1.	
404	403	384	385	0	0	0	0
338	4	1	8			1.	
389	404	385	386	0	0	0	0
339	4	1	8			1.	
389	389	386	387	0	0	0	0
340	4	1	8			2.	
364	355	354	363	0	0	0	0
341	4	1	8			2.	
406	364	363	390	0	0	0	0
342	4	1	8	1		2.	
407	406	390	391	0	0	0	0
353	4	1	8			2.	
418	417	401	402	0	0	0	0
354	4	1	8			2.	
405	418	402	403	0	0	0	0
355	4	1	8			2.	
405	405	403	404	0	0	0	0
356	4	1	8			3.	
365	356	355	364	0	0	0	0
357	4	1	8			3.	
420	365	364	406	0	0	0	0
358	4	1	8	1		3.	
421	420	406	407	0	0	0	0
367	4	1	8			3.	
430	429	415	416	0	0	0	0
368	4	1	8			3.	
419	430	416	417	0	0	0	0
369	4	1	8			3.	
419	419	417	418	0	0	0	0
370	4	1	8			4.	
366	357	356	365	0	0	0	0
371	4	1	8			4.	
432	366	365	420	0	0	0	0
372	4	1	8	1		4.	
433	432	420	421	0	0	0	0
379	4	1	8			4.	
440	439	427	428	0	0	0	0
380	4	1	8			4.	
431	440	428	429	0	0	0	0
381	4	1	8			4.	
431	431	429	430	0	0	0	0
382	4	1	8			5.	
367	358	357	366	0	0	0	0
383	4	1	8			5.	
442	367	366	432	0	0	0	0
384	4	1	8	1		5.	
443	442	432	433	0	0	0	0
389	4	1	8			5.	
448	447	437	438	0	0	0	0
390	4	1	8			5.	
441	448	438	439	0	0	0	0
391	4	1	8			5.	

441	441	439	440	0	0	0	0
392	4	1	8			6.	
368	359	358	367	0	0	0	0
393	4	1	8			6.	
450	368	367	442	0	0	0	0
394	4	1	8	1		6.	
451	450	442	443	0	0	0	0
397	4	1	8			6.	
454	453	445	446	0	0	0	0
398	4	1	8			6.	
449	454	446	447	0	0	0	0
399	4	1	8			6.	
449	449	447	448	0	0	0	0
400	4	1	8			7.	
369	360	359	368	0	0	0	0
401	4	1	8			7.	
456	369	368	450	0	0	0	0
402	4	1	8			7.	
457	456	450	451	0	0	0	0
403	4	1	8			7.	
458	457	451	452	0	0	0	0
404	4	1	8			7.	
455	458	452	453	0	0	0	0
405	4	1	8			7.	
455	455	453	454	0	0	0	0
406	4	1	8			8.	
370	361	360	369	0	0	0	0
407	4	1	8			8.	
460	370	369	456	0	0	0	0
408	4	1	8			8.	
459	460	456	457	0	0	0	0
409	4	1	8			8.	
459	459	457	458	0	0	0	0

Figure A.12.1 Case 11 - Contours of Horizontal Displacement

Figure A.12.2 Case 11 - Contours of Vertical Displacement

Depth (ft)

Contour Interval; 4

Distance from Centerline of Embankment (ft)

Figure A.12.3 Case 11 - Contours of Principal Stress Ratio

Figure A.12.4 Case 11 - Contours of Local Factors of Safety

Computer Program Input

Figure A.13.1 Case 12 - Contours of Horizontal Displacement

Figure A.13.2 Case 12 - Contours of Vertical Displacement

Figure A.13.3 Case 12 - Contours of Principal Stress Ratio

Figure A.13.4 Case 12 - Contours of Local Factors of Safety

State Route 1 over Ramsey Creek - Franklin County, Indiana Case 12

460	0	2	1	18	1
1	0	15	.01		
0.0	0.5	9.0			
1	1	0	0.000	0.000	
2	1	0	0.000	-7.500	1
7	1	0	0.000	-27.500	
8	1	1	0.000	-35.000	
9	0	0	12.500	0.000	
17	0	0	25.000	0.000	8
337	0	0	625.000	0.000	
345	1	0	640.000	0.000	
10	0	0	12.500	-7.500	1
15	0	0	12.500	-27.500	
16	0	1	12.500	-35.000	
18	0	0	25.000	-7.500	8
338	0	0	625.000	-7.500	
346	1	0	640.000	-7.500	1
351	1	0	640.000	-27.500	
352	1	1	640.000	-35.000	
19	0	0	25.000	-11.500	8
339	0	0	625.000	-11.500	
20	0	0	25.000	-15.500	8
340	0	0	625.000	-15.500	
21	0	0	25.000	-19.500	8
341	0	0	625.000	-19.500	
22	0	0	25.000	-23.500	8
342	0	0	625.000	-23.500	
23	0	0	25.000	-27.500	8
343	0	0	625.000	-27.500	
24	0	1	25.000	-35.000	8
344	0	1	625.000	-35.000	
353	1	0	0.000	10.000	1
361	1	0	0.000	90.000	
362	0	0	12.500	10.000	1
370	0	0	12.500	90.000	
371	0	0	25.000	10.000	1
387	0	0	265.000	10.000	
388	0	0	280.000	5.000	
389	0	0	250.000	15.000	
390	0	0	25.000	20.000	1
404	0	0	235.000	20.000	
405	0	0	220.000	25.000	
406	0	0	25.000	30.000	1
418	0	0	205.000	30.000	
419	0	0	190.000	35.000	
420	0	0	25.000	40.000	1
430	0	0	175.000	40.000	
431	0	0	160.000	45.000	
432	0	0	25.000	50.000	1
440	0	0	145.000	50.000	
441	0	0	130.000	55.000	
442	0	0	25.000	60.000	1
448	0	0	115.000	60.000	
449	0	0	100.000	65.000	
450	0	0	25.000	70.000	1
454	0	0	85.000	70.000	
455	0	0	70.000	75.000	
456	0	0	25.000	80.000	1
458	0	0	55.000	80.000	

459	0	0	40.000	85.000				
460	0	0	25.000	90.000				
8								
1								
0.	640.		-7.5	0.	0.	0.		
0.	0.		0.	-11.5	0.125	0.0626	21.	8.0
1								
0.	640.		-11.5	-7.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	7.20
2								
0.	640.		-15.5	-11.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	4.9
2								
0.	640.		-19.5	-15.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	4.30
2								
0.	640.		-23.5	-19.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	3.8
2								
0.	640.		-27.5	-23.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	3.40
2								
0.	640.		-35.0	-27.51	0.	0.		
0.	0.		0.	-11.5	0.130	0.0676	21.	2.5
3								
0.	295.		0.	90.	0.	0.		
0.1	0.50							
0	0	0						
2	409	4	10	8	13	4	2	16
1								
300.	0.4		0.7500	21.	0.720	0.230	0.039	1.58
8.0	0.01		0.000	10.	2.			
2								
300.	0.4		0.7500	21.	0.720	0.250	0.050	1.46
7.20	0.01		0.000	10.	2.			
3								
300.	0.4		0.7500	21.	0.720	0.250	0.050	1.07
4.9	0.01		0.000	10.	2.			
4								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.96
4.30	0.01		0.000	10.	2.			
5								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.87
3.8	0.01		0.000	10.	2.			
6								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.80
3.40	0.01		0.000	10.	2.			
7								
300.	0.4		0.6630	21.	0.720	0.124	0.020	0.63
2.5	0.01		0.000	10.	2.			
8								
300.	0.4		0.7500	60.	0.000	0.124	0.020	1.58
8.0	0.01		0.125	0.	2.			
1	4	1	1	8	0.			
9	1	2	10	0	0	0	0	
43	4	1	1		0.			
345	337	338	346	0	0	0	0	
44	4	1	2	8	0.			
10	2	3	11	0	0	0	0	
86	4	1	2		0.			

346	338	339	347	0	0	0	0
87	4	1	3	8	0.		
11	3	4	12	0	0	0	0
129	4	1	3		0.		
347	339	340	348	0	0	0	0
130	4	1	4	8	0.		
12	4	5	13	0	0	0	0
172	4	1	4		0.		
348	340	341	349	0	0	0	0
173	4	1	5	8	0.		
13	5	6	14	0	0	0	0
215	4	1	5		0.		
349	341	342	350	0	0	0	0
216	4	1	6	8	0.		
14	6	7	15	0	0	0	0
258	4	1	6		0.		
350	342	343	351	0	0	0	0
259	4	1	7	8	0.		
15	7	8	16	0	0	0	0
301	4	1	7		0.		
351	343	344	352	0	0	0	0
302	4	1	8	0.01			
362	353	1	9	0	0	0	0
303	4	1	8	0.01			
371	362	9	17	0	0	0	0
304	4	1	8	0.01			
372	371	17	25	0	0	0	0
305	4	1	8	0.01			
373	372	25	33	0	0	0	0
306	4	1	8	0.01			
374	373	33	41	0	0	0	0
307	4	1	8	0.01			
375	374	41	49	0	0	0	0
308	4	1	8	0.01			
376	375	49	57	0	0	0	0
309	4	1	8	0.01			
377	376	57	65	0	0	0	0
310	4	1	8	0.01			
378	377	65	73	0	0	0	0
311	4	1	8	0.01			
379	378	73	81	0	0	0	0
312	4	1	8	0.01			
380	379	81	89	0	0	0	0
313	4	1	8	0.01			
381	380	89	97	0	0	0	0
314	4	1	8	0.01			
382	381	97	105	0	0	0	0
315	4	1	8	0.01			
383	382	105	113	0	0	0	0
316	4	1	8	0.01			
384	383	113	121	0	0	0	0
317	4	1	8	0.01			
385	384	121	129	0	0	0	0
318	4	1	8	0.01			
386	385	129	137	0	0	0	0
319	4	1	8	0.01			
387	386	137	145	0	0	0	0
320	4	1	8	0.01			
388	387	145	153	0	0	0	0
321	4	1	8	0.01			

388	388	153	161	0	0	0	0
322	4	1	8			1.	
363	354	353	362	0	0	0	0
323	4	1	8			1.	
390	363	362	371	0	0	0	0
324	4	1	8	1		1.	
391	390	371	372	0	0	0	0
337	4	1	8			1.	
404	403	384	385	0	0	0	0
338	4	1	8			1.	
389	404	385	386	0	0	0	0
339	4	1	8			1.	
389	389	386	387	0	0	0	0
340	4	1	8			2.	
364	355	354	363	0	0	0	0
341	4	1	8			2.	
406	364	363	390	0	0	0	0
342	4	1	8	1		2.	
407	406	390	391	0	0	0	0
353	4	1	8			2.	
418	417	401	402	0	0	0	0
354	4	1	8			2.	
405	418	402	403	0	0	0	0
355	4	1	8			2.	
405	405	403	404	0	0	0	0
356	4	1	8			3.	
365	356	355	364	0	0	0	0
357	4	1	8			3.	
420	365	364	406	0	0	0	0
358	4	1	8	1		3.	
421	420	406	407	0	0	0	0
367	4	1	8			3.	
430	429	415	416	0	0	0	0
368	4	1	8			3.	
419	430	416	417	0	0	0	0
369	4	1	8			3.	
419	419	417	418	0	0	0	0
370	4	1	8			4.	
366	357	356	365	0	0	0	0
371	4	1	8			4.	
432	366	365	420	0	0	0	0
372	4	1	8	1		4.	
433	432	420	421	0	0	0	0
379	4	1	8			4.	
440	439	427	428	0	0	0	0
380	4	1	8			4.	
431	440	428	429	0	0	0	0
381	4	1	8			4.	
431	431	429	430	0	0	0	0
382	4	1	8			5.	
367	358	357	366	0	0	0	0
383	4	1	8			5.	
442	367	366	432	0	0	0	0
384	4	1	8	1		5.	
443	442	432	433	0	0	0	0
389	4	1	8			5.	
448	447	437	438	0	0	0	0
390	4	1	8			5.	
441	448	438	439	0	0	0	0
391	4	1	8			5.	

441	441	439	440	0	0	0	0
392	4	1	8			6.	
368	359	358	367	0	0	0	0
393	4	1	8			6.	
450	368	367	442	0	0	0	0
394	4	1	8	1		6.	
451	450	442	443	0	0	0	0
397	4	1	8			6.	
454	453	445	446	0	0	0	0
398	4	1	8			6.	
449	454	446	447	0	0	0	0
399	4	1	8			6.	
449	449	447	448	0	0	0	0
400	4	1	8			7.	
369	360	359	368	0	0	0	0
401	4	1	8			7.	
456	369	368	450	0	0	0	0
402	4	1	8			7.	
457	456	450	451	0	0	0	0
403	4	1	8			7.	
458	457	451	452	0	0	0	0
404	4	1	8			7.	
455	458	452	453	0	0	0	0
405	4	1	8			7.	
455	455	453	454	0	0	0	0
406	4	1	8			8.	
370	361	360	369	0	0	0	0
407	4	1	8			8.	
460	370	369	456	0	0	0	0
408	4	1	8			8.	
459	460	456	457	0	0	0	0
409	4	1	8			8.	
459	459	457	458	0	0	0	0
1	20	2	2	1	8		

1 0.

	0.	.067	.083	.16	0.	1.68	1.92	2.4
1	1	9	0	1				
2	9	17	0	1				
3	17	25	0	1				
4	25	33	0	1				
5	33	41	0	1				
6	41	49	0	1				
7	49	57	0	1				
8	57	65	0	1				
9	65	73	0	1				
10	73	81	0	1				
11	81	89	0	1				
12	89	97	0	1				
13	97	105	0	1				
14	105	113	0	1				
15	113	121	0	1				
16	121	129	0	1				
17	129	137	0	1				
18	137	145	0	1				
19	145	153	0	1				
20	153	161	0	1

Figure A.13.1 Case 12 - Contours of Horizontal Displacement

Contour Interval; 4

Figure A.13.3 Case 12 - Contours of Principal Stress Ratio

Contour Interval; 1

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Computer Program Input

Figure A.14.1 Case 13 - Contours of Horizontal Displacement

Figure A.14.2 Case 13 - Contours of Vertical Displacement

Figure A.14.3 Case 13 - Contours of Principal Stress Ratio

Figure A.14.4 Case 13 - Contours of Local Factors of Safety

State Route 1 over Ramsey Creek - Franklin County, Indiana Case 13

460	0	2	1	18	1	
1	0	15	.01			
0.0		0.5	9.0			
1	1	0	0.000	0.000		
2	1	0	0.000	-7.500	1	
7	1	0	0.000	-27.500		
8	1	1	0.000	-35.000		
9	0	0	12.500	0.000		
17	0	0	25.000	0.000	8	
337	0	0	625.000	0.000		
345	1	0	640.000	0.000		
10	0	0	12.500	-7.500	1	
15	0	0	12.500	-27.500		
16	0	1	12.500	-35.000		
18	0	0	25.000	-7.500	8	
338	0	0	625.000	-7.500		
346	1	0	640.000	-7.500	1	
351	1	0	640.000	-27.500		
352	1	1	640.000	-35.000		
19	0	0	25.000	-11.500	8	
339	0	0	625.000	-11.500		
20	0	0	25.000	-15.500	8	
340	0	0	625.000	-15.500		
21	0	0	25.000	-19.500	8	
341	0	0	625.000	-19.500		
22	0	0	25.000	-23.500	8	
342	0	0	625.000	-23.500		
23	0	0	25.000	-27.500	8	
343	0	0	625.000	-27.500		
24	0	1	25.000	-35.000	8	
344	0	1	625.000	-35.000		
353	1	0	0.000	10.000	1	
361	1	0	0.000	90.000		
362	0	0	12.500	10.000	1	
370	0	0	12.500	90.000		
371	0	0	25.000	10.000	1	
387	0	0	265.000	10.000		
388	0	0	280.000	5.000		
389	0	0	250.000	15.000		
390	0	0	25.000	20.000	1	
404	0	0	235.000	20.000		
405	0	0	220.000	25.000		
406	0	0	25.000	30.000	1	
418	0	0	205.000	30.000		
419	0	0	190.000	35.000		
420	0	0	25.000	40.000	1	
430	0	0	175.000	40.000		
431	0	0	160.000	45.000		
432	0	0	25.000	50.000	1	
440	0	0	145.000	50.000		
441	0	0	130.000	55.000		
442	0	0	25.000	60.000	1	
448	0	0	115.000	60.000		
449	0	0	100.000	65.000		
450	0	0	25.000	70.000	1	
454	0	0	85.000	70.000		
455	0	0	70.000	75.000		
456	0	0	25.000	80.000	1	
458	0	0	55.000	80.000		

459	0	0	40.000	85.000				
460	0	0	25.000	90.000				
8								
1								
0.	640.		-7.5	0.	0.	0.		
0.	0.		0.	-11.5	0.125	0.0626	21.	8.0
1								
0.	640.		-11.5	-7.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	7.20
2								
0.	640.		-15.5	-11.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	4.9
2								
0.	640.		-19.5	-15.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	4.30
2								
0.	640.		-23.5	-19.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	3.8
2								
0.	640.		-27.5	-23.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	3.40
2								
0.	640.		-35.0	-27.51	0.	0.		
0.	0.		0.	-11.5	0.130	0.0676	21.	2.5
3								
0.	295.		0.	90.	0.	0.		
0.1	0.50							
0	0	0						
2	409	4	10	8	13	4	2	16
1								
300.	0.4		0.7500	21.	0.720	0.230	0.039	1.58
8.0	0.01		0.000	10.	2.			
2								
300.	0.4		0.7500	21.	0.720	0.250	0.050	1.46
7.20	0.01		0.000	10.	2.			
3								
300.	0.4		0.7500	21.	0.720	0.250	0.050	1.07
4.9	0.01		0.000	10.	2.			
4								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.96
4.30	0.01		0.000	10.	2.			
5								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.87
3.8	0.01		0.000	10.	2.			
6								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.80
3.40	0.01		0.000	10.	2.			
7								
300.	0.4		0.6630	21.	0.720	0.124	0.020	0.63
2.5	0.01		0.000	10.	2.			
8								
300.	0.4		0.7500	60.	0.000	0.124	0.020	1.58
8.0	0.01		0.125	0.	2.			
1	4	1	1	8	0.			
9	1	2	10	0	0	0	0	
43	4	1	1		0.			
345	337	338	346	0	0	0	0	
44	4	1	2	8	0.			
10	2	3	11	0	0	0	0	
86	4	1	2		0.			

346	338	339	347	0	0	0	0
87	4	1	3	8		0.	
11	3	4	12	0	0	0	0
129	4	1	3			0.	
347	339	340	348	0	0	0	0
130	4	1	4	8		0.	
12	4	5	13	0	0	0	0
172	4	1	4			0.	
348	340	341	349	0	0	0	0
173	4	1	5	8		0.	
13	5	6	14	0	0	0	0
215	4	1	5			0.	
349	341	342	350	0	0	0	0
216	4	1	6	8		0.	
14	6	7	15	0	0	0	0
258	4	1	6			0.	
350	342	343	351	0	0	0	0
259	4	1	7	8		0.	
15	7	8	16	0	0	0	0
301	4	1	7			0.	
351	343	344	352	0	0	0	0
302	4	1	8	0.01			
362	353	1	9	0	0	0	0
303	4	1	8	0.01			
371	362	9	17	0	0	0	0
304	4	1	8	0.01			
372	371	17	25	0	0	0	0
305	4	1	8	0.01			
373	372	25	33	0	0	0	0
306	4	1	8	0.01			
374	373	33	41	0	0	0	0
307	4	1	8	0.01			
375	374	41	49	0	0	0	0
308	4	1	8	0.01			
376	375	49	57	0	0	0	0
309	4	1	8	0.01			
377	376	57	65	0	0	0	0
310	4	1	8	0.01			
378	377	65	73	0	0	0	0
311	4	1	8	0.01			
379	378	73	81	0	0	0	0
312	4	1	8	0.01			
380	379	81	89	0	0	0	0
313	4	1	8	0.01			
381	380	89	97	0	0	0	0
314	4	1	8	0.01			
382	381	97	105	0	0	0	0
315	4	1	8	0.01			
383	382	105	113	0	0	0	0
316	4	1	8	0.01			
384	383	113	121	0	0	0	0
317	4	1	8	0.01			
385	384	121	129	0	0	0	0
318	4	1	8	0.01			
386	385	129	137	0	0	0	0
319	4	1	8	0.01			
387	386	137	145	0	0	0	0
320	4	1	8	0.01			
388	387	145	153	0	0	0	0
321	4	1	8	0.01			

388	388	153	161	0	0	0	0
322	4	1	8			1.	
363	354	353	362	0	0	0	0
323	4	1	8			1.	
390	363	362	371	0	0	0	0
324	4	1	8	1		1.	
391	390	371	372	0	0	0	0
337	4	1	8			1.	
404	403	384	385	0	0	0	0
338	4	1	8			1.	
389	404	385	386	0	0	0	0
339	4	1	8			1.	
389	389	386	387	0	0	0	0
340	4	1	8			2.	
364	355	354	363	0	0	0	0
341	4	1	8			2.	
406	364	363	390	0	0	0	0
342	4	1	8	1		2.	
407	406	390	391	0	0	0	0
353	4	1	8			2.	
418	417	401	402	0	0	0	0
354	4	1	8			2.	
405	418	402	403	0	0	0	0
355	4	1	8			2.	
405	405	403	404	0	0	0	0
356	4	1	8			3.	
365	356	355	364	0	0	0	0
357	4	1	8			3.	
420	365	364	406	0	0	0	0
358	4	1	8	1		3.	
421	420	406	407	0	0	0	0
367	4	1	8			3.	
430	429	415	416	0	0	0	0
368	4	1	8			3.	
419	430	416	417	0	0	0	0
369	4	1	8			3.	
419	419	417	418	0	0	0	0
370	4	1	8			4.	
366	357	356	365	0	0	0	0
371	4	1	8			4.	
432	366	365	420	0	0	0	0
372	4	1	8	1		4.	
433	432	420	421	0	0	0	0
379	4	1	8			4.	
440	439	427	428	0	0	0	0
380	4	1	8			4.	
431	440	428	429	0	0	0	0
381	4	1	8			4.	
431	431	429	430	0	0	0	0
382	4	1	8			5.	
367	358	357	366	0	0	0	0
383	4	1	8			5.	
442	367	366	432	0	0	0	0
384	4	1	8	1		5.	
443	442	432	433	0	0	0	0
389	4	1	8			5.	
448	447	437	438	0	0	0	0
390	4	1	8			5.	
441	448	438	439	0	0	0	0
391	4	1	8			5.	

441	441	439	440	0	0	0	0
392	4	1	8			6.	
368	359	358	367	0	0	0	0
393	4	1	8			6.	
450	368	367	442	0	0	0	0
394	4	1	8			6.	
451	450	442	443	0	0	0	0
397	4	1	8			6.	
454	453	445	446	0	0	0	0
398	4	1	8			6.	
449	454	446	447	0	0	0	0
399	4	1	8			6.	
449	449	447	448	0	0	0	0
400	4	1	8			7.	
369	360	359	368	0	0	0	0
401	4	1	8			7.	
456	369	368	450	0	0	0	0
402	4	1	8			7.	
457	456	450	451	0	0	0	0
403	4	1	8			7.	
458	457	451	452	0	0	0	0
404	4	1	8			7.	
455	458	452	453	0	0	0	0
405	4	1	8			7.	
455	455	453	454	0	0	0	0
406	4	1	8			8.	
370	361	360	369	0	0	0	0
407	4	1	8			8.	
460	370	369	456	0	0	0	0
408	4	1	8			8.	
459	460	456	457	0	0	0	0
409	4	1	8			8.	
459	459	457	458	0	0	0	0

1	20	2	2	1	8				
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1	0.								
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1	0.	.067	.083	.16	0.	.14	.16	.2	
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1	1	9	0	1					
2	9	17	0	1					
3	17	25	0	1					
4	25	33	0	1					
5	33	41	0	1					
6	41	49	0	1					
7	49	57	0	1					
8	57	65	0	1					
9	65	73	0	1					
10	73	81	0	1					
11	81	89	0	1					
12	89	97	0	1					
13	97	105	0	1					
14	105	113	0	1					
15	113	121	0	1					
16	121	129	0	1					
17	129	137	0	1					
18	137	145	0	1					
19	145	153	0	1					
20	153	161	0	1

Figure A.14.1 Case 13 - Contours of Horizontal Displacement

Figure A.14.2 Case 13 - Contours of Vertical Displacement

Depth (ft)

Contour Interval; 4

Distance from Centerline of Embankment (ft)

Figure A.14.3 Case 13 - Contours of Principal Stress Ratio

Contour Interval; 0.5

Figure A.14.4 Case 13 - Contours of Local Factors of Safety

Computer Program Input

Figure A.15.1 Case 14 - Contours of Horizontal Displacement

Figure A.15.2 Case 14 - Contours of Vertical Displacement

Figure A.15.3 Case 14 - Contours of Principal Stress Ratio

Figure A.15.4 Case 14 - Contours of Local Factors of Safety

State Route 1 over Ramsey Creek - Franklin County, Indiana Case 14

460	0	2	1	18	1	
1	0	15	.01			
0.0		0.5		9.0		
1	1	0		0.000	0.000	
2	1	0		0.000	-7.500	1
7	1	0		0.000	-27.500	
8	1	1		0.000	-35.000	
9	0	0		12.500	0.000	
17	0	0		25.000	0.000	8
337	0	0		625.000	0.000	
345	1	0		640.000	0.000	
10	0	0		12.500	-7.500	1
15	0	0		12.500	-27.500	
16	0	1		12.500	-35.000	
18	0	0		25.000	-7.500	8
338	0	0		625.000	-7.500	
346	1	0		640.000	-7.500	1
351	1	0		640.000	-27.500	
352	1	1		640.000	-35.000	
19	0	0		25.000	-11.500	8
339	0	0		625.000	-11.500	
20	0	0		25.000	-15.500	8
340	0	0		625.000	-15.500	
21	0	0		25.000	-19.500	8
341	0	0		625.000	-19.500	
22	0	0		25.000	-23.500	8
342	0	0		625.000	-23.500	
23	0	0		25.000	-27.500	8
343	0	0		625.000	-27.500	
24	0	1		25.000	-35.000	8
344	0	1		625.000	-35.000	
353	1	0		0.000	10.000	1
361	1	0		0.000	90.000	
362	0	0		12.500	10.000	1
370	0	0		12.500	90.000	
371	0	0		25.000	10.000	1
387	0	0		265.000	10.000	
388	0	0		280.000	5.000	
389	0	0		250.000	15.000	
390	0	0		25.000	20.000	1
404	0	0		235.000	20.000	
405	0	0		220.000	25.000	
406	0	0		25.000	30.000	1
418	0	0		205.000	30.000	
419	0	0		190.000	35.000	
420	0	0		25.000	40.000	1
430	0	0		175.000	40.000	
431	0	0		160.000	45.000	
432	0	0		25.000	50.000	1
440	0	0		145.000	50.000	
441	0	0		130.000	55.000	
442	0	0		25.000	60.000	1
448	0	0		115.000	60.000	
449	0	0		100.000	65.000	
450	0	0		25.000	70.000	1
454	0	0		85.000	70.000	
455	0	0		70.000	75.000	
456	0	0		25.000	80.000	1
458	0	0		55.000	80.000	

459 0 0 40.000 85.000
 460 0 0 25.000 90.000

8

1

0. 640. -7.5 0. 0. 0.
 0. 0. 0. -11.5 0.125 0.0626 21. 10.0

1

0. 640. -11.5 -7.51 0. 0.
 0. 0. 0. -11.5 0.120 0.0576 21. 7.20

2

0. 640. -15.5 -11.51 0. 0.
 0. 0. 0. -11.5 0.120 0.0576 21. 4.9

2

0. 640. -19.5 -15.51 0. 0.
 0. 0. 0. -11.5 0.120 0.0576 21. 4.30

2

0. 640. -23.5 -19.51 0. 0.
 0. 0. 0. -11.5 0.120 0.0576 21. 3.8

2

0. 640. -27.5 -23.51 0. 0.
 0. 0. 0. -11.5 0.120 0.0576 21. 3.40

2

0. 640. -35.0 -27.51 0. 0.
 0. 0. 0. -11.5 0.130 0.0676 21. 2.5

3

0. 295. 0. 90. 0. 0.
 0.1 0.50
 0 0 0

2

2 409 4 10 8 13 4 2 16

1

300. 0.4 0.7500 21. 0.720 0.230 0.039 1.90
 10.0 0.01 0.000 10. 2.

2

300. 0.4 0.7500 21. 0.720 0.250 0.050 1.46
 7.20 0.01 0.000 10. 2.

3

300. 0.4 0.7500 21. 0.720 0.250 0.050 1.07
 4.9 0.01 0.000 10. 2.

4

300. 0.4 0.7500 21. 0.720 0.250 0.050 0.96
 4.30 0.01 0.000 10. 2.

5

300. 0.4 0.7500 21. 0.720 0.250 0.050 0.87
 3.8 0.01 0.000 10. 2.

6

300. 0.4 0.7500 21. 0.720 0.250 0.050 0.80
 3.40 0.01 0.000 10. 2.

7

300. 0.4 0.6630 21. 0.720 0.124 0.020 0.63
 2.5 0.01 0.000 10. 2.

8

300. 0.4 0.7500 60. 0.000 0.124 0.020 1.58
 20.0 0.01 0.125 0. 2.

1

1 4 1 1 8 0.

9

9 1 2 10 0 0 0 0

43

43 4 1 1 0.

345

345 337 338 346 0 0 0 0

44

44 4 1 2 8 0.

10

10 2 3 11 0 0 0 0

86

86 4 1 2 0.

346	338	339	347	0	0	0	0
87	4	1	3	8	0.		
11	3	4	12	0	0	0	0
129	4	1	3		0.		
347	339	340	348	0	0	0	0
130	4	1	4	8	0.		
12	4	5	13	0	0	0	0
172	4	1	4		0.		
348	340	341	349	0	0	0	0
173	4	1	5	8	0.		
13	5	6	14	0	0	0	0
215	4	1	5		0.		
349	341	342	350	0	0	0	0
216	4	1	6	8	0.		
14	6	7	15	0	0	0	0
258	4	1	6		0.		
350	342	343	351	0	0	0	0
259	4	1	7	8	0.		
15	7	8	16	0	0	0	0
301	4	1	7		0.		
351	343	344	352	0	0	0	0
302	4	1	8	0.01			
362	353	1	9	0	0	0	0
303	4	1	8	0.01			
371	362	9	17	0	0	0	0
304	4	1	8	0.01			
372	371	17	25	0	0	0	0
305	4	1	8	0.01			
373	372	25	33	0	0	0	0
306	4	1	8	0.01			
374	373	33	41	0	0	0	0
307	4	1	8	0.01			
375	374	41	49	0	0	0	0
308	4	1	8	0.01			
376	375	49	57	0	0	0	0
309	4	1	8	0.01			
377	376	57	65	0	0	0	0
310	4	1	8	0.01			
378	377	65	73	0	0	0	0
311	4	1	8	0.01			
379	378	73	81	0	0	0	0
312	4	1	8	0.01			
380	379	81	89	0	0	0	0
313	4	1	8	0.01			
381	380	89	97	0	0	0	0
314	4	1	8	0.01			
382	381	97	105	0	0	0	0
315	4	1	8	0.01			
383	382	105	113	0	0	0	0
316	4	1	8	0.01			
384	383	113	121	0	0	0	0
317	4	1	8	0.01			
385	384	121	129	0	0	0	0
318	4	1	8	0.01			
386	385	129	137	0	0	0	0
319	4	1	8	0.01			
387	386	137	145	0	0	0	0
320	4	1	8	0.01			
388	387	145	153	0	0	0	0
321	4	1	8	0.01			

388	388	153	161	0	0	0	0
322	4	1	8			1.	
363	354	353	362	0	0	0	0
323	4	1	8			1.	
390	363	362	371	0	0	0	0
324	4	1	8	1		1.	
391	390	371	372	0	0	0	0
337	4	1	8			1.	
404	403	384	385	0	0	0	0
338	4	1	8			1.	
389	404	385	386	0	0	0	0
339	4	1	8			1.	
389	389	386	387	0	0	0	0
340	4	1	8			2.	
364	355	354	363	0	0	0	0
341	4	1	8			2.	
406	364	363	390	0	0	0	0
342	4	1	8	1		2.	
407	406	390	391	0	0	0	0
353	4	1	8			2.	
418	417	401	402	0	0	0	0
354	4	1	8			2.	
405	418	402	403	0	0	0	0
355	4	1	8			2.	
405	405	403	404	0	0	0	0
356	4	1	8			3.	
365	356	355	364	0	0	0	0
357	4	1	8			3.	
420	365	364	406	0	0	0	0
358	4	1	8	1		3.	
421	420	406	407	0	0	0	0
367	4	1	8			3.	
430	429	415	416	0	0	0	0
368	4	1	8			3.	
419	430	416	417	0	0	0	0
369	4	1	8			3.	
419	419	417	418	0	0	0	0
370	4	1	8			4.	
366	357	356	365	0	0	0	0
371	4	1	8			4.	
432	366	365	420	0	0	0	0
372	4	1	8	1		4.	
433	432	420	421	0	0	0	0
379	4	1	8			4.	
440	439	427	428	0	0	0	0
380	4	1	8			4.	
431	440	428	429	0	0	0	0
381	4	1	8			4.	
431	431	429	430	0	0	0	0
382	4	1	8			5.	
367	358	357	366	0	0	0	0
383	4	1	8			5.	
442	367	366	432	0	0	0	0
384	4	1	8	1		5.	
443	442	432	433	0	0	0	0
389	4	1	8			5.	
448	447	437	438	0	0	0	0
390	4	1	8			5.	
441	448	438	439	0	0	0	0
391	4	1	8			5.	

441	441	439	440	0	0	0	0
392	4	1	8			6.	
368	359	358	367	0	0	0	0
393	4	1	8			6.	
450	368	367	442	0	0	0	0
394	4	1	8	1		6.	
451	450	442	443	0	0	0	0
397	4	1	8			6.	
454	453	445	446	0	0	0	0
398	4	1	8			6.	
449	454	446	447	0	0	0	0
399	4	1	8			6.	
449	449	447	448	0	0	0	0
400	4	1	8			7.	
369	360	359	368	0	0	0	0
401	4	1	8			7.	
456	369	368	450	0	0	0	0
402	4	1	8			7.	
457	456	450	451	0	0	0	0
403	4	1	8			7.	
458	457	451	452	0	0	0	0
404	4	1	8			7.	
455	458	452	453	0	0	0	0
405	4	1	8			7.	
455	455	453	454	0	0	0	0
406	4	1	8			8.	
370	361	360	369	0	0	0	0
407	4	1	8			8.	
460	370	369	456	0	0	0	0
408	4	1	8			8.	
459	460	456	457	0	0	0	0
409	4	1	8			8.	
459	459	457	458	0	0	0	0

1	20	2	2	1	8						
1	0.										
	0.	.067	.083	.16	0.	1.68	1.92	2.4			

1	1	9	0	1
2	9	17	0	1
3	17	25	0	1
4	25	33	0	1
5	33	41	0	1
6	41	49	0	1
7	49	57	0	1
8	57	65	0	1
9	65	73	0	1
10	73	81	0	1
11	81	89	0	1
12	89	97	0	1
13	97	105	0	1
14	105	113	0	1
15	113	121	0	1
16	121	129	0	1
17	129	137	0	1
18	137	145	0	1
19	145	153	0	1
20	153	161	0	1

Figure A.15.1 Case 14 - Contours of Horizontal Displacement

Figure A.15.2 Case 14 - Contours of Vertical Displacement

Contour Interval; 4

134

Contour Interval; 0.5

Distance from Centerline of Embankment (ft)

Figure A.15.4 Case 14 - Contours of Local Factors of Safety

Computer Program Input

Figure A.16.1 Case 15 - Contours of Horizontal Displacement

Figure A.16.2 Case 15 - Contours of Vertical Displacement

Figure A.16.3 Case 15 - Contours of Principal Stress Ratio

Figure A.16.4 Case 15 - Contours of Local Factors of Safety

State Route 1 over Ramsey Creek - Franklin County, Indiana Case 15

460	0	2	1	18	1
1	0	15	.01		
0.0	0.5	9.0			
1	1	0	0.000	0.000	
2	1	0	0.000	-7.500	1
7	1	0	0.000	-27.500	
8	1	1	0.000	-35.000	
9	0	0	12.500	0.000	
17	0	0	25.000	0.000	8
337	0	0	625.000	0.000	
345	1	0	640.000	0.000	
10	0	0	12.500	-7.500	1
15	0	0	12.500	-27.500	
16	0	1	12.500	-35.000	
18	0	0	25.000	-7.500	8
338	0	0	625.000	-7.500	
346	1	0	640.000	-7.500	1
351	1	0	640.000	-27.500	
352	1	1	640.000	-35.000	
19	0	0	25.000	-11.500	8
339	0	0	625.000	-11.500	
20	0	0	25.000	-15.500	8
340	0	0	625.000	-15.500	
21	0	0	25.000	-19.500	8
341	0	0	625.000	-19.500	
22	0	0	25.000	-23.500	8
342	0	0	625.000	-23.500	
23	0	0	25.000	-27.500	8
343	0	0	625.000	-27.500	
24	0	1	25.000	-35.000	8
344	0	1	625.000	-35.000	
353	1	0	0.000	10.000	1
361	1	0	0.000	90.000	
362	0	0	12.500	10.000	1
370	0	0	12.500	90.000	
371	0	0	25.000	10.000	1
387	0	0	265.000	10.000	
388	0	0	280.000	5.000	
389	0	0	250.000	15.000	
390	0	0	25.000	20.000	1
404	0	0	235.000	20.000	
405	0	0	220.000	25.000	
406	0	0	25.000	30.000	1
418	0	0	205.000	30.000	
419	0	0	190.000	35.000	
420	0	0	25.000	40.000	1
430	0	0	175.000	40.000	
431	0	0	160.000	45.000	
432	0	0	25.000	50.000	1
440	0	0	145.000	50.000	
441	0	0	130.000	55.000	
442	0	0	25.000	60.000	1
448	0	0	115.000	60.000	
449	0	0	100.000	65.000	
450	0	0	25.000	70.000	1
454	0	0	85.000	70.000	
455	0	0	70.000	75.000	
456	0	0	25.000	80.000	1
458	0	0	55.000	80.000	

459	0	0	40.000	85.000				
460	0	0	25.000	90.000				
8								
1								
0.	640.	-7.5	0.	0.	0.			
0.	0.	0.	-11.5	0.125	0.0626	21.	10.0	
1								
0.	640.	-11.5	-7.51	0.	0.			
0.	0.	0.	-11.5	0.120	0.0576	21.	7.20	
2								
0.	640.	-15.5	-11.51	0.	0.			
0.	0.	0.	-11.5	0.120	0.0576	21.	4.9	
2								
0.	640.	-19.5	-15.51	0.	0.			
0.	0.	0.	-11.5	0.120	0.0576	21.	4.30	
2								
0.	640.	-23.5	-19.51	0.	0.			
0.	0.	0.	-11.5	0.120	0.0576	21.	3.8	
2								
0.	640.	-27.5	-23.51	0.	0.			
0.	0.	0.	-11.5	0.120	0.0576	21.	3.40	
2								
0.	640.	-35.0	-27.51	0.	0.			
0.	0.	0.	-11.5	0.130	0.0676	21.	2.5	
3								
0.	295.	0.	90.	0.	0.			
0.1	0.50							
0	0	0						
2	409	4	10	8	13	4	2	16
1								
300.	0.4	0.7500	21.	0.720	0.230	0.039	1.90	
10.0	0.01	0.000	10.	2.				
2								
300.	0.4	0.7500	21.	0.720	0.250	0.050	1.46	
7.20	0.01	0.000	10.	2.				
3								
300.	0.4	0.7500	21.	0.720	0.250	0.050	1.07	
4.9	0.01	0.000	10.	2.				
4								
300.	0.4	0.7500	21.	0.720	0.250	0.050	0.96	
4.30	0.01	0.000	10.	2.				
5								
300.	0.4	0.7500	21.	0.720	0.250	0.050	0.87	
3.8	0.01	0.000	10.	2.				
6								
300.	0.4	0.7500	21.	0.720	0.250	0.050	0.80	
3.40	0.01	0.000	10.	2.				
7								
300.	0.4	0.6630	21.	0.720	0.124	0.020	0.63	
2.5	0.01	0.000	10.	2.				
8								
300.	0.4	0.7500	60.	0.000	0.124	0.020	1.58	
20.0	0.01	0.125	0.	2.				
1	4	1	1	8	0.			
9	1	2	10	0	0	0	0	
43	4	1	1		0.			
345	337	338	346	0	0	0	0	
44	4	1	2	8	0.			
10	2	3	11	0	0	0	0	
86	4	1	2		0.			

346	338	339	347	0	0	0	0
87	4	1	3	8		0.	
11	3	4	12	0	0	0	0
129	4	1	3			0.	
347	339	340	348	0	0	0	0
130	4	1	4	8		0.	
12	4	5	13	0	0	0	0
172	4	1	4			0.	
348	340	341	349	0	0	0	0
173	4	1	5	8		0.	
13	5	6	14	0	0	0	0
215	4	1	5			0.	
349	341	342	350	0	0	0	0
216	4	1	6	8		0.	
14	6	7	15	0	0	0	0
258	4	1	6			0.	
350	342	343	351	0	0	0	0
259	4	1	7	8		0.	
15	7	8	16	0	0	0	0
301	4	1	7			0.	
351	343	344	352	0	0	0	0
302	4	1	8	0.01			
362	353	1	9	0	0	0	0
303	4	1	8	0.01			
371	362	9	17	0	0	0	0
304	4	1	8	0.01			
372	371	17	25	0	0	0	0
305	4	1	8	0.01			
373	372	25	33	0	0	0	0
306	4	1	8	0.01			
374	373	33	41	0	0	0	0
307	4	1	8	0.01			
375	374	41	49	0	0	0	0
308	4	1	8	0.01			
376	375	49	57	0	0	0	0
309	4	1	8	0.01			
377	376	57	65	0	0	0	0
310	4	1	8	0.01			
378	377	65	73	0	0	0	0
311	4	1	8	0.01			
379	378	73	81	0	0	0	0
312	4	1	8	0.01			
380	379	81	89	0	0	0	0
313	4	1	8	0.01			
381	380	89	97	0	0	0	0
314	4	1	8	0.01			
382	381	97	105	0	0	0	0
315	4	1	8	0.01			
383	382	105	113	0	0	0	0
316	4	1	8	0.01			
384	383	113	121	0	0	0	0
317	4	1	8	0.01			
385	384	121	129	0	0	0	0
318	4	1	8	0.01			
386	385	129	137	0	0	0	0
319	4	1	8	0.01			
387	386	137	145	0	0	0	0
320	4	1	8	0.01			
388	387	145	153	0	0	0	0
321	4	1	8	0.01			

388	388	153	161	0	0	0	0
322	4	1	8			1.	
363	354	353	362	0	0	0	0
323	4	1	8			1.	
390	363	362	371	0	0	0	0
324	4	1	8	1		1.	
391	390	371	372	0	0	0	0
337	4	1	8			1.	
404	403	384	385	0	0	0	0
338	4	1	8			1.	
389	404	385	386	0	0	0	0
339	4	1	8			1.	
389	389	386	387	0	0	0	0
340	4	1	8			2.	
364	355	354	363	0	0	0	0
341	4	1	8			2.	
406	364	363	390	0	0	0	0
342	4	1	8	1		2.	
407	406	390	391	0	0	0	0
353	4	1	8			2.	
418	417	401	402	0	0	0	0
354	4	1	8			2.	
405	418	402	403	0	0	0	0
355	4	1	8			2.	
405	405	403	404	0	0	0	0
356	4	1	8			3.	
365	356	355	364	0	0	0	0
357	4	1	8			3.	
420	365	364	406	0	0	0	0
358	4	1	8	1		3.	
421	420	406	407	0	0	0	0
367	4	1	8			3.	
430	429	415	416	0	0	0	0
368	4	1	8			3.	
419	430	416	417	0	0	0	0
369	4	1	8			3.	
419	419	417	418	0	0	0	0
370	4	1	8			4.	
366	357	356	365	0	0	0	0
371	4	1	8			4.	
432	366	365	420	0	0	0	0
372	4	1	8	1		4.	
433	432	420	421	0	0	0	0
379	4	1	8			4.	
440	439	427	428	0	0	0	0
380	4	1	8			4.	
431	440	428	429	0	0	0	0
381	4	1	8			4.	
431	431	429	430	0	0	0	0
382	4	1	8			5.	
367	358	357	366	0	0	0	0
383	4	1	8			5.	
442	367	366	432	0	0	0	0
384	4	1	8	1		5.	
443	442	432	433	0	0	0	0
389	4	1	8			5.	
448	447	437	438	0	0	0	0
390	4	1	8			5.	
441	448	438	439	0	0	0	0
391	4	1	8			5.	

441	441	439	440	0	0	0	0				
392	4	1	8			6.					
368	359	358	367	0	0	0	0				
393	4	1	8			6.					
450	368	367	442	0	0	0	0				
394	4	1	8	1		6.					
451	450	442	443	0	0	0	0				
397	4	1	8			6.					
454	453	445	446	0	0	0	0				
398	4	1	8			6.					
449	454	446	447	0	0	0	0				
399	4	1	8			6.					
449	449	447	448	0	0	0	0				
400	4	1	8			7.					
369	360	359	368	0	0	0	0				
401	4	1	8			7.					
456	369	368	450	0	0	0	0				
402	4	1	8			7.					
457	456	450	451	0	0	0	0				
403	4	1	8			7.					
458	457	451	452	0	0	0	0				
404	4	1	8			7.					
455	458	452	453	0	0	0	0				
405	4	1	8			7.					
455	455	453	454	0	0	0	0				
406	4	1	8			8.					
370	361	360	369	0	0	0	0				
407	4	1	8			8.					
460	370	369	456	0	0	0	0				
408	4	1	8			8.					
459	460	456	457	0	0	0	0				
409	4	1	8			8.					
459	459	457	458	0	0	0	0				
1	20	2	2	1	8						
1	0.										
	0.	.067		.083		.16		0.	.14	.16	.2
1	1	9	0	1							
2	9	17	0	1							
3	17	25	0	1							
4	25	33	0	1							
5	33	41	0	1							
6	41	49	0	1							
7	49	57	0	1							
8	57	65	0	1							
9	65	73	0	1							
10	73	81	0	1							
11	81	89	0	1							
12	89	97	0	1							
13	97	105	0	1							
14	105	113	0	1							
15	113	121	0	1							
16	121	129	0	1							
17	129	137	0	1							
18	137	145	0	1							
19	145	153	0	1							
20	153	161	0	1

Figure A.16.1 Case 15 - Contours of Horizontal Displacement

Figure A.16.2 Case 15 - Contours of Vertical Displacement

Computer Program Input

Figure A.17.1 Case 16 - Contours of Horizontal Displacement

Figure A.17.2 Case 16 - Contours of Vertical Displacement

Figure A.17.3 Case 16 - Contours of Principal Stress Ratio

Figure A.17.4 Case 16 - Contours of Local Factors of Safety

State Route 1 over Ramsey Creek - Franklin County, Indiana Case 16

460	0	2	1	18	1
1	0	15	.01		
0.0	0.5	9.0			
1	1	0	0.000	0.000	
2	1	0	0.000	-7.500	1
7	1	0	0.000	-27.500	
8	1	1	0.000	-35.000	
9	0	0	12.500	0.000	
17	0	0	25.000	0.000	8
337	0	0	625.000	0.000	
345	1	0	640.000	0.000	
10	0	0	12.500	-7.500	1
15	0	0	12.500	-27.500	
16	0	1	12.500	-35.000	
18	0	0	25.000	-7.500	8
338	0	0	625.000	-7.500	
346	1	0	640.000	-7.500	1
351	1	0	640.000	-27.500	
352	1	1	640.000	-35.000	
19	0	0	25.000	-11.500	8
339	0	0	625.000	-11.500	
20	0	0	25.000	-15.500	8
340	0	0	625.000	-15.500	
21	0	0	25.000	-19.500	8
341	0	0	625.000	-19.500	
22	0	0	25.000	-23.500	8
342	0	0	625.000	-23.500	
23	0	0	25.000	-27.500	8
343	0	0	625.000	-27.500	
24	0	1	25.000	-35.000	8
344	0	1	625.000	-35.000	
353	1	0	0.000	10.000	1
361	1	0	0.000	90.000	
362	0	0	12.500	10.000	1
370	0	0	12.500	90.000	
371	0	0	25.000	10.000	1
387	0	0	265.000	10.000	
388	0	0	280.000	5.000	
389	0	0	250.000	15.000	
390	0	0	25.000	20.000	1
404	0	0	235.000	20.000	
405	0	0	220.000	25.000	
406	0	0	25.000	30.000	1
418	0	0	205.000	30.000	
419	0	0	190.000	35.000	
420	0	0	25.000	40.000	1
430	0	0	175.000	40.000	
431	0	0	160.000	45.000	
432	0	0	25.000	50.000	1
440	0	0	145.000	50.000	
441	0	0	130.000	55.000	
442	0	0	25.000	60.000	1
448	0	0	115.000	60.000	
449	0	0	100.000	65.000	
450	0	0	25.000	70.000	1
454	0	0	85.000	70.000	
455	0	0	70.000	75.000	
456	0	0	25.000	80.000	1
458	0	0	55.000	80.000	

459	0	0	40.000	85.000				
460	0	0	25.000	90.000				
8								
1								
0.	640.		-7.5	0.	0.	0.		
0.	0.		0.	-11.5	0.125	0.0626	21.	10.0
1								
0.	640.		-11.5	-7.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	3.00
2								
0.	640.		-15.5	-11.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	2.50
2								
0.	640.		-19.5	-15.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	2.00
2								
0.	640.		-23.5	-19.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	1.50
2								
0.	640.		-27.5	-23.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	1.50
2								
0.	640.		-35.0	-27.51	0.	0.		
0.	0.		0.	-11.5	0.130	0.0676	21.	1.02
3								
0.	295.		0.	90.	0.	0.		
0.1	0.50							
0	0	0						
2	409	4	10	8	13	4	2	16
1								
300.	0.4		0.7500	21.	0.720	0.230	0.039	1.90
10.0	0.01		0.000	10.	2.			
2								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.73
3.00	0.01		0.000	10.	2.			
3								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.63
2.50	0.01		0.000	10.	2.			
4								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.53
2.00	0.01		0.000	10.	2.			
5								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.42
1.50	0.01		0.000	10.	2.			
6								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.42
1.50	0.01		0.000	10.	2.			
7								
300.	0.4		0.6630	21.	0.720	0.124	0.020	0.30
1.02	0.01		0.000	10.	2.			
8								
300.	0.4		0.7500	60.	0.000	0.124	0.020	1.58
8.0	0.01		0.125	0.	2.			
1	4	1	1	8	0.			
9	1	2	10	0	0	0	0	
43	4	1	1		0.			
345	337	338	346	0	0	0	0	
44	4	1	2	8	0.			
10	2	3	11	0	0	0	0	
86	4	1	2		0.			

346	338	339	347	0	0	0	0
87	4	1	3	8		0.	
11	3	4	12	0	0	0	0
129	4	1	3			0.	
347	339	340	348	0	0	0	0
130	4	1	4	8		0.	
12	4	5	13	0	0	0	0
172	4	1	4			0.	
348	340	341	349	0	0	0	0
173	4	1	5	8		0.	
13	5	6	14	0	0	0	0
215	4	1	5			0.	
349	341	342	350	0	0	0	0
216	4	1	6	8		0.	
14	6	7	15	0	0	0	0
258	4	1	6			0.	
350	342	343	351	0	0	0	0
259	4	1	7	8		0.	
15	7	8	16	0	0	0	0
301	4	1	7			0.	
351	343	344	352	0	0	0	0
302	4	1	8	0.01			
362	353	1	9	0	0	0	0
303	4	1	8	0.01			
371	362	9	17	0	0	0	0
304	4	1	8	0.01			
372	371	17	25	0	0	0	0
305	4	1	8	0.01			
373	372	25	33	0	0	0	0
306	4	1	8	0.01			
374	373	33	41	0	0	0	0
307	4	1	8	0.01			
375	374	41	49	0	0	0	0
308	4	1	8	0.01			
376	375	49	57	0	0	0	0
309	4	1	8	0.01			
377	376	57	65	0	0	0	0
310	4	1	8	0.01			
378	377	65	73	0	0	0	0
311	4	1	8	0.01			
379	378	73	81	0	0	0	0
312	4	1	8	0.01			
380	379	81	89	0	0	0	0
313	4	1	8	0.01			
381	380	89	97	0	0	0	0
314	4	1	8	0.01			
382	381	97	105	0	0	0	0
315	4	1	8	0.01			
383	382	105	113	0	0	0	0
316	4	1	8	0.01			
384	383	113	121	0	0	0	0
317	4	1	8	0.01			
385	384	121	129	0	0	0	0
318	4	1	8	0.01			
386	385	129	137	0	0	0	0
319	4	1	8	0.01			
387	386	137	145	0	0	0	0
320	4	1	8	0.01			
388	387	145	153	0	0	0	0
321	4	1	8	0.01			

388	388	153	161	0	0	0	0
322	4	1	8			1.	
363	354	353	362	0	0	0	0
323	4	1	8			1.	
390	363	362	371	0	0	0	0
324	4	1	8	1		1.	
391	390	371	372	0	0	0	0
337	4	1	8			1.	
404	403	384	385	0	0	0	0
338	4	1	8			1.	
389	404	385	386	0	0	0	0
339	4	1	8			1.	
389	389	386	387	0	0	0	0
340	4	1	8			2.	
364	355	354	363	0	0	0	0
341	4	1	8			2.	
406	364	363	390	0	0	0	0
342	4	1	8	1		2.	
407	406	390	391	0	0	0	0
353	4	1	8			2.	
418	417	401	402	0	0	0	0
354	4	1	8			2.	
405	418	402	403	0	0	0	0
355	4	1	8			2.	
405	405	403	404	0	0	0	0
356	4	1	8			3.	
365	356	355	364	0	0	0	0
357	4	1	8			3.	
420	365	364	406	0	0	0	0
358	4	1	8	1		3.	
421	420	406	407	0	0	0	0
367	4	1	8			3.	
430	429	415	416	0	0	0	0
368	4	1	8			3.	
419	430	416	417	0	0	0	0
369	4	1	8			3.	
419	419	417	418	0	0	0	0
370	4	1	8			4.	
366	357	356	365	0	0	0	0
371	4	1	8			4.	
432	366	365	420	0	0	0	0
372	4	1	8	1		4.	
433	432	420	421	0	0	0	0
379	4	1	8			4.	
440	439	427	428	0	0	0	0
380	4	1	8			4.	
431	440	428	429	0	0	0	0
381	4	1	8			4.	
431	431	429	430	0	0	0	0
382	4	1	8			5.	
367	358	357	366	0	0	0	0
383	4	1	8			5.	
442	367	366	432	0	0	0	0
384	4	1	8	1		5.	
443	442	432	433	0	0	0	0
389	4	1	8			5.	
448	447	437	438	0	0	0	0
390	4	1	8			5.	
441	448	438	439	0	0	0	0
391	4	1	8			5.	

441	441	439	440	0	0	0	0
392	4	1	8			6.	
368	359	358	367	0	0	0	0
393	4	1	8			6.	
450	368	367	442	0	0	0	0
394	4	1	8	1		6.	
451	450	442	443	0	0	0	0
397	4	1	8			6.	
454	453	445	446	0	0	0	0
398	4	1	8			6.	
449	454	446	447	0	0	0	0
399	4	1	8			6.	
449	449	447	448	0	0	0	0
400	4	1	8			7.	
369	360	359	368	0	0	0	0
401	4	1	8			7.	
456	369	368	450	0	0	0	0
402	4	1	8			7.	
457	456	450	451	0	0	0	0
403	4	1	8			7.	
458	457	451	452	0	0	0	0
404	4	1	8			7.	
455	458	452	453	0	0	0	0
405	4	1	8			7.	
455	455	453	454	0	0	0	0
406	4	1	8			8.	
370	361	360	369	0	0	0	0
407	4	1	8			8.	
460	370	369	456	0	0	0	0
408	4	1	8			8.	
459	460	456	457	0	0	0	0
409	4	1	8			8.	
459	459	457	458	0	0	0	0
1	20	2	2	1	8		

1	0.								
	0.	.067	.083	.16	0.	1.68	1.92	2.4	
1	1	9	0	1					
2	9	17	0	1					
3	17	25	0	1					
4	25	33	0	1					
5	33	41	0	1					
6	41	49	0	1					
7	49	57	0	1					
8	57	65	0	1					
9	65	73	0	1					
10	73	81	0	1					
11	81	89	0	1					
12	89	97	0	1					
13	97	105	0	1					
14	105	113	0	1					
15	113	121	0	1					
16	121	129	0	1					
17	129	137	0	1					
18	137	145	0	1					
19	145	153	0	1					
20	153	161	0	1

Figure A.17.1 Case 16 - Contours of Horizontal Displacement

Figure A.17.2 Case 16 - Contours of Vertical Displacement

Depth (ft) Contour Interval; 8

Distance from Centerline of Embankment (ft)

Figure A.17.3 Case 16 - Contours of Principal Stress Ratio

Figure A.17.4 Case 16 - Contours of Local Factors of Safety

Computer Program Input

Figure A.18.1 Case 17 - Contours of Horizontal Displacement

Figure A.18.2 Case 17 - Contours of Vertical Displacement

Figure A.18.3 Case 17 - Contours of Principal Stress Ratio

Figure A.18.4 Case 17 - Contours of Local Factors of Safety

State Route 1 over Ramsey Creek - Franklin County, Indiana Case 17

460	0	2	1	18	1
1	0	15	.01		
0.0	0.5	9.0			
1	1	0	0.000	0.000	
2	1	0	0.000	-7.500	1
7	1	0	0.000	-27.500	
8	1	1	0.000	-35.000	
9	0	0	12.500	0.000	
17	0	0	25.000	0.000	8
337	0	0	625.000	0.000	
345	1	0	640.000	0.000	
10	0	0	12.500	-7.500	1
15	0	0	12.500	-27.500	
16	0	1	12.500	-35.000	
18	0	0	25.000	-7.500	8
338	0	0	625.000	-7.500	
346	1	0	640.000	-7.500	1
351	1	0	640.000	-27.500	
352	1	1	640.000	-35.000	
19	0	0	25.000	-11.500	8
339	0	0	625.000	-11.500	
20	0	0	25.000	-15.500	8
340	0	0	625.000	-15.500	
21	0	0	25.000	-19.500	8
341	0	0	625.000	-19.500	
22	0	0	25.000	-23.500	8
342	0	0	625.000	-23.500	
23	0	0	25.000	-27.500	8
343	0	0	625.000	-27.500	
24	0	1	25.000	-35.000	8
344	0	1	625.000	-35.000	
353	1	0	0.000	10.000	1
361	1	0	0.000	90.000	
362	0	0	12.500	10.000	1
370	0	0	12.500	90.000	
371	0	0	25.000	10.000	1
387	0	0	265.000	10.000	
388	0	0	280.000	5.000	
389	0	0	250.000	15.000	
390	0	0	25.000	20.000	1
404	0	0	235.000	20.000	
405	0	0	220.000	25.000	
406	0	0	25.000	30.000	1
418	0	0	205.000	30.000	
419	0	0	190.000	35.000	
420	0	0	25.000	40.000	1
430	0	0	175.000	40.000	
431	0	0	160.000	45.000	
432	0	0	25.000	50.000	1
440	0	0	145.000	50.000	
441	0	0	130.000	55.000	
442	0	0	25.000	60.000	1
448	0	0	115.000	60.000	
449	0	0	100.000	65.000	
450	0	0	25.000	70.000	1
454	0	0	85.000	70.000	
455	0	0	70.000	75.000	
456	0	0	25.000	80.000	1
458	0	0	55.000	80.000	

459 0 0 40.000 85.000
 460 0 0 25.000 90.000

8
 1
 0. 640. -7.5 0. 0. 0.
 0. 0. 0. -11.5 0.125 0.0626 21. 10.
 1
 0. 640. -11.5 -7.51 0. 0.
 0. 0. 0. -11.5 0.120 0.0576 21. 3.00
 2
 0. 640. -15.5 -11.51 0. 0.
 0. 0. 0. -11.5 0.120 0.0576 21. 2.5
 2
 0. 640. -19.5 -15.51 0. 0.
 0. 0. 0. -11.5 0.120 0.0576 21. 2.00
 2
 0. 640. -23.5 -19.51 0. 0.
 0. 0. 0. -11.5 0.120 0.0576 21. 1.5
 2
 0. 640. -27.5 -23.51 0. 0.
 0. 0. 0. -11.5 0.120 0.0576 21. 1.50
 2
 0. 640. -35.0 -27.51 0. 0.
 0. 0. 0. -11.5 0.130 0.0676 21. 1.02

3
 0. 295. 0. 90. 0. 0.
 0.1 0.50
 0 0 0
 2 409 4 10 8 13 4 2 16
 1
 300. 0.4 0.7500 21. 0.720 0.230 0.039 1.90
 10.0 0.01 0.000 10. 2.
 2
 300. 0.4 0.7500 21. 0.720 0.250 0.050 0.73
 3.00 0.01 0.000 10. 2.
 3
 300. 0.4 0.7500 21. 0.720 0.250 0.050 0.63
 2.50 0.01 0.000 10. 2.
 4
 300. 0.4 0.7500 21. 0.720 0.250 0.050 0.53
 2.00 0.01 0.000 10. 2.
 5
 300. 0.4 0.7500 21. 0.720 0.250 0.050 0.42
 1.50 0.01 0.000 10. 2.
 6
 300. 0.4 0.7500 21. 0.720 0.250 0.050 0.42
 1.50 0.01 0.000 10. 2.
 7
 300. 0.4 0.6630 21. 0.720 0.124 0.020 0.30
 1.02 0.01 0.000 10. 2.
 8
 300. 0.4 0.7500 60. 0.000 0.124 0.020 1.58
 8.0 0.01 0.125 0. 2.
 1 4 1 1 8 0.
 9 1 2 10 0 0 0 0
 43 4 1 1 0.
 345 337 338 346 0 0 0 0
 44 4 1 2 8 0.
 10 2 3 11 0 0 0 0
 86 4 1 2 0.

346	338	339	347	0	0	0	0
87	4	1	3	8		0.	
11	3	4	12	0	0	0	0
129	4	1	3			0.	
347	339	340	348	0	0	0	0
130	4	1	4	8		0.	
12	4	5	13	0	0	0	0
172	4	1	4			0.	
348	340	341	349	0	0	0	0
173	4	1	5	8		0.	
13	5	6	14	0	0	0	0
215	4	1	5			0.	
349	341	342	350	0	0	0	0
216	4	1	6	8		0.	
14	6	7	15	0	0	0	0
258	4	1	6			0.	
350	342	343	351	0	0	0	0
259	4	1	7	8		0.	
15	7	8	16	0	0	0	0
301	4	1	7			0.	
351	343	344	352	0	0	0	0
302	4	1	8	0.01			
362	353	1	9	0	0	0	0
303	4	1	8	0.01			
371	362	9	17	0	0	0	0
304	4	1	8	0.01			
372	371	17	25	0	0	0	0
305	4	1	8	0.01			
373	372	25	33	0	0	0	0
306	4	1	8	0.01			
374	373	33	41	0	0	0	0
307	4	1	8	0.01			
375	374	41	49	0	0	0	0
308	4	1	8	0.01			
376	375	49	57	0	0	0	0
309	4	1	8	0.01			
377	376	57	65	0	0	0	0
310	4	1	8	0.01			
378	377	65	73	0	0	0	0
311	4	1	8	0.01			
379	378	73	81	0	0	0	0
312	4	1	8	0.01			
380	379	81	89	0	0	0	0
313	4	1	8	0.01			
381	380	89	97	0	0	0	0
314	4	1	8	0.01			
382	381	97	105	0	0	0	0
315	4	1	8	0.01			
383	382	105	113	0	0	0	0
316	4	1	8	0.01			
384	383	113	121	0	0	0	0
317	4	1	8	0.01			
385	384	121	129	0	0	0	0
318	4	1	8	0.01			
386	385	129	137	0	0	0	0
319	4	1	8	0.01			
387	386	137	145	0	0	0	0
320	4	1	8	0.01			
388	387	145	153	0	0	0	0
321	4	1	8	0.01			

388	388	153	161	0	0	0	0
322	4	1	8			1.	
363	354	353	362	0	0	0	0
323	4	1	8			1.	
390	363	362	371	0	0	0	0
324	4	1	8	1		1.	
391	390	371	372	0	0	0	0
337	4	1	8			1.	
404	403	384	385	0	0	0	0
338	4	1	8			1.	
389	404	385	386	0	0	0	0
339	4	1	8			1.	
389	389	386	387	0	0	0	0
340	4	1	8			2.	
364	355	354	363	0	0	0	0
341	4	1	8			2.	
406	364	363	390	0	0	0	0
342	4	1	8	1		2.	
407	406	390	391	0	0	0	0
353	4	1	8			2.	
418	417	401	402	0	0	0	0
354	4	1	8			2.	
405	418	402	403	0	0	0	0
355	4	1	8			2.	
405	405	403	404	0	0	0	0
356	4	1	8			3.	
365	356	355	364	0	0	0	0
357	4	1	8			3.	
420	365	364	406	0	0	0	0
358	4	1	8	1		3.	
421	420	406	407	0	0	0	0
367	4	1	8			3.	
430	429	415	416	0	0	0	0
368	4	1	8			3.	
419	430	416	417	0	0	0	0
369	4	1	8			3.	
419	419	417	418	0	0	0	0
370	4	1	8			4.	
366	357	356	365	0	0	0	0
371	4	1	8			4.	
432	366	365	420	0	0	0	0
372	4	1	8	1		4.	
433	432	420	421	0	0	0	0
379	4	1	8			4.	
440	439	427	428	0	0	0	0
380	4	1	8			4.	
431	440	428	429	0	0	0	0
381	4	1	8			4.	
431	431	429	430	0	0	0	0
382	4	1	8			5.	
367	358	357	366	0	0	0	0
383	4	1	8			5.	
442	367	366	432	0	0	0	0
384	4	1	8	1		5.	
443	442	432	433	0	0	0	0
389	4	1	8			5.	
448	447	437	438	0	0	0	0
390	4	1	8			5.	
441	448	438	439	0	0	0	0
391	4	1	8			5.	

441	441	439	440	0	0	0	0
392	4	1	8			6.	
368	359	358	367	0	0	0	0
393	4	1	8			6.	
450	368	367	442	0	0	0	0
394	4	1	8	1		6.	
451	450	442	443	0	0	0	0
397	4	1	8			6.	
454	453	445	446	0	0	0	0
398	4	1	8			6.	
449	454	446	447	0	0	0	0
399	4	1	8			6.	
449	449	447	448	0	0	0	0
400	4	1	8			7.	
369	360	359	368	0	0	0	0
401	4	1	8			7.	
456	369	368	450	0	0	0	0
402	4	1	8			7.	
457	456	450	451	0	0	0	0
403	4	1	8			7.	
458	457	451	452	0	0	0	0
404	4	1	8			7.	
455	458	452	453	0	0	0	0
405	4	1	8			7.	
455	455	453	454	0	0	0	0
406	4	1	8			8.	
370	361	360	369	0	0	0	0
407	4	1	8			8.	
460	370	369	456	0	0	0	0
408	4	1	8			8.	
459	460	456	457	0	0	0	0
409	4	1	8			8.	
459	459	457	458	0	0	0	0

1	20	2	2	1	8		
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1	0.						
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0.	.067	.083	.16	0.	.14	.16	.2
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1	1	9	0	1
2	9	17	0	1
3	17	25	0	1
4	25	33	0	1
5	33	41	0	1
6	41	49	0	1
7	49	57	0	1
8	57	65	0	1
9	65	73	0	1
10	73	81	0	1
11	81	89	0	1
12	89	97	0	1
13	97	105	0	1
14	105	113	0	1
15	113	121	0	1
16	121	129	0	1
17	129	137	0	1
18	137	145	0	1
19	145	153	0	1
20	153	161	0	1

Figure A.18.1 Case 17 - Contours of Horizontal Displacement

Figure A.18.2 Case 17 - Contours of Vertical Displacement

Depth (ft)

Contour Interval; 8

Distance from Centerline of Embankment (ft)

Figure A.18.3 Case 17 - Contours of Principal Stress Ratio

Figure A.18.4 Case 17 - Contours of Local Factors of Safety

Computer Program Input

Figure A.19.1 Case 18 - Contours of Horizontal Displacement

Figure A.19.2 Case 18 - Contours of Vertical Displacement

Figure A.19.3 Case 18 - Contours of Principal Stress Ratio

Figure A.19.4 Case 18 - Contours of Local Factors of Safety

State Route 1 over Ramsey Creek - Franklin County, Indiana Case 18

460	0	1	1	18	1
1	0	15	.01		
0.0	0.5	9.0			
1	1	0	0.000	0.000	
2	1	0	0.000	-7.500	1
7	1	0	0.000	-27.500	
8	1	1	0.000	-35.000	
9	0	0	12.500	0.000	
17	0	0	25.000	0.000	8
337	0	0	625.000	0.000	
345	1	0	640.000	0.000	
10	0	0	12.500	-7.500	1
15	0	0	12.500	-27.500	
16	0	1	12.500	-35.000	
18	0	0	25.000	-7.500	8
338	0	0	625.000	-7.500	
346	1	0	640.000	-7.500	1
351	1	0	640.000	-27.500	
352	1	1	640.000	-35.000	
19	0	0	25.000	-11.500	8
339	0	0	625.000	-11.500	
20	0	0	25.000	-15.500	8
340	0	0	625.000	-15.500	
21	0	0	25.000	-19.500	8
341	0	0	625.000	-19.500	
22	0	0	25.000	-23.500	8
342	0	0	625.000	-23.500	
23	0	0	25.000	-27.500	8
343	0	0	625.000	-27.500	
24	0	1	25.000	-35.000	8
344	0	1	625.000	-35.000	
353	1	0	0.000	10.000	1
361	1	0	0.000	90.000	
362	0	0	12.500	10.000	1
370	0	0	12.500	90.000	
371	0	0	25.000	10.000	1
387	0	0	265.000	10.000	
388	0	0	280.000	5.000	
389	0	0	250.000	15.000	
390	0	0	25.000	20.000	1
404	0	0	235.000	20.000	
405	0	0	220.000	25.000	
406	0	0	25.000	30.000	1
418	0	0	205.000	30.000	
419	0	0	190.000	35.000	
420	0	0	25.000	40.000	1
430	0	0	175.000	40.000	
431	0	0	160.000	45.000	
432	0	0	25.000	50.000	1
440	0	0	145.000	50.000	
441	0	0	130.000	55.000	
442	0	0	25.000	60.000	1
448	0	0	115.000	60.000	
449	0	0	100.000	65.000	
450	0	0	25.000	70.000	1
454	0	0	85.000	70.000	
455	0	0	70.000	75.000	
456	0	0	25.000	80.000	1
458	0	0	55.000	80.000	

459	0	0	40.000	85.000				
460	0	0	25.000	90.000				
8								
1								
0.	640.		-7.5	0.	0.	0.		
0.	0.		0.	-11.5	0.125	0.0626	21.	10.0
1								
0.	640.		-11.5	-7.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	3.00
2								
0.	640.		-15.5	-11.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	2.5
2								
0.	640.		-19.5	-15.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	2.00
2								
0.	640.		-23.5	-19.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	1.50
2								
0.	640.		-27.5	-23.51	0.	0.		
0.	0.		0.	-11.5	0.120	0.0576	21.	1.50
2								
0.	640.		-35.0	-27.51	0.	0.		
0.	0.		0.	-11.5	0.130	0.0676	21.	1.02
3								
0.	295.		0.	90.	0.	0.		
0.1	0.50							
0	0	0						
2	409	4	10	8	13	4	2	16
1								
300.	0.4		0.7500	21.	0.720	0.230	0.039	1.90
10.0	0.01		0.000	10.	2.			
2								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.73
3.00	0.01		0.000	10.	2.			
3								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.63
2.50	0.01		0.000	10.	2.			
4								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.53
2.00	0.01		0.000	10.	2.			
5								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.42
1.50	0.01		0.000	10.	2.			
6								
300.	0.4		0.7500	21.	0.720	0.250	0.050	0.42
1.50	0.01		0.000	10.	2.			
7								
300.	0.4		0.6630	21.	0.720	0.124	0.020	0.30
1.02	0.01		0.000	10.	2.			
8								
300.	0.4		0.7500	60.	0.000	0.124	0.020	0.73
3.0	0.01		0.125	0.	2.			
1	4	1	1	8	0.			
9	1	2	10	0	0	0	0	
43	4	1	1		0.			
345	337	338	346	0	0	0	0	
44	4	1	2	8	0.			
10	2	3	11	0	0	0	0	
86	4	1	2		0.			

346	338	339	347	0	0	0	0
87	4	1	3	8		0.	
11	3	4	12	0	0	0	0
129	4	1	3			0.	
347	339	340	348	0	0	0	0
130	4	1	4	8		0.	
12	4	5	13	0	0	0	0
172	4	1	4			0.	
348	340	341	349	0	0	0	0
173	4	1	5	8		0.	
13	5	6	14	0	0	0	0
215	4	1	5			0.	
349	341	342	350	0	0	0	0
216	4	1	6	8		0.	
14	6	7	15	0	0	0	0
258	4	1	6			0.	
350	342	343	351	0	0	0	0
259	4	1	7	8		0.	
15	7	8	16	0	0	0	0
301	4	1	7			0.	
351	343	344	352	0	0	0	0
302	4	1	8	0.01			
362	353	1	9	0	0	0	0
303	4	1	8	0.01			
371	362	9	17	0	0	0	0
304	4	1	8	0.01			
372	371	17	25	0	0	0	0
305	4	1	8	0.01			
373	372	25	33	0	0	0	0
306	4	1	8	0.01			
374	373	33	41	0	0	0	0
307	4	1	8	0.01			
375	374	41	49	0	0	0	0
308	4	1	8	0.01			
376	375	49	57	0	0	0	0
309	4	1	8	0.01			
377	376	57	65	0	0	0	0
310	4	1	8	0.01			
378	377	65	73	0	0	0	0
311	4	1	8	0.01			
379	378	73	81	0	0	0	0
312	4	1	8	0.01			
380	379	81	89	0	0	0	0
313	4	1	8	0.01			
381	380	89	97	0	0	0	0
314	4	1	8	0.01			
382	381	97	105	0	0	0	0
315	4	1	8	0.01			
383	382	105	113	0	0	0	0
316	4	1	8	0.01			
384	383	113	121	0	0	0	0
317	4	1	8	0.01			
385	384	121	129	0	0	0	0
318	4	1	8	0.01			
386	385	129	137	0	0	0	0
319	4	1	8	0.01			
387	386	137	145	0	0	0	0
320	4	1	8	0.01			
388	387	145	153	0	0	0	0
321	4	1	8	0.01			

388	388	153	161	0	0	0	0
322	4	1	8			1.	
363	354	353	362	0	0	0	0
323	4	1	8			1.	
390	363	362	371	0	0	0	0
324	4	1	8	1		1.	
391	390	371	372	0	0	0	0
337	4	1	8			1.	
404	403	384	385	0	0	0	0
338	4	1	8			1.	
389	404	385	386	0	0	0	0
339	4	1	8			1.	
389	389	386	387	0	0	0	0
340	4	1	8			2.	
364	355	354	363	0	0	0	0
341	4	1	8			2.	
406	364	363	390	0	0	0	0
342	4	1	8	1		2.	
407	406	390	391	0	0	0	0
353	4	1	8			2.	
418	417	401	402	0	0	0	0
354	4	1	8			2.	
405	418	402	403	0	0	0	0
355	4	1	8			2.	
405	405	403	404	0	0	0	0
356	4	1	8			3.	
365	356	355	364	0	0	0	0
357	4	1	8			3.	
420	365	364	406	0	0	0	0
358	4	1	8	1		3.	
421	420	406	407	0	0	0	0
367	4	1	8			3.	
430	429	415	416	0	0	0	0
368	4	1	8			3.	
419	430	416	417	0	0	0	0
369	4	1	8			3.	
419	419	417	418	0	0	0	0
370	4	1	8			4.	
366	357	356	365	0	0	0	0
371	4	1	8			4.	
432	366	365	420	0	0	0	0
372	4	1	8	1		4.	
433	432	420	421	0	0	0	0
379	4	1	8			4.	
440	439	427	428	0	0	0	0
380	4	1	8			4.	
431	440	428	429	0	0	0	0
381	4	1	8			4.	
431	431	429	430	0	0	0	0
382	4	1	8			5.	
367	358	357	366	0	0	0	0
383	4	1	8			5.	
442	367	366	432	0	0	0	0
384	4	1	8	1		5.	
443	442	432	433	0	0	0	0
389	4	1	8			5.	
448	447	437	438	0	0	0	0
390	4	1	8			5.	
441	448	438	439	0	0	0	0
391	4	1	8			5.	

441	441	439	440	0	0	0	0
392	4	1	8			6.	
368	359	358	367	0	0	0	0
393	4	1	8			6.	
450	368	367	442	0	0	0	0
394	4	1	8	1		6.	
451	450	442	443	0	0	0	0
397	4	1	8			6.	
454	453	445	446	0	0	0	0
398	4	1	8			6.	
449	454	446	447	0	0	0	0
399	4	1	8			6.	
449	449	447	448	0	0	0	0
400	4	1	8			7.	
369	360	359	368	0	0	0	0
401	4	1	8			7.	
456	369	368	450	0	0	0	0
402	4	1	8			7.	
457	456	450	451	0	0	0	0
403	4	1	8			7.	
458	457	451	452	0	0	0	0
404	4	1	8			7.	
455	458	452	453	0	0	0	0
405	4	1	8			7.	
455	455	453	454	0	0	0	0
406	4	1	8			8.	
370	361	360	369	0	0	0	0
407	4	1	8			8.	
460	370	369	456	0	0	0	0
408	4	1	8			8.	
459	460	456	457	0	0	0	0
409	4	1	8			8.	
459	459	457	458	0	0	0	0

Depth (ft)

Contour Interval; 0.4'

Distance from Centerline of Embankment (ft)

Figure A.19.1 Case 18 - Contours of Horizontal Displacement

Figure A.19.2 Case 18 - Contours of Vertical Displacement

Depth (ft)

Contour Interval; 8

Distance from Centerline of Embankment (ft)

Figure A.19.3 Case 18 - Contours of Principal Stress Ratio

Figure A.19.4 Case 18 - Contours of Local Factors of Safety

APPENDIX B

Computer disk with input data files

FOR A COPY OF THE INPUT DATA FILES,
CONTACT SCOTT LUDLOW AT PURDUE UNIVERSITY

APPENDIX C

List of References

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