30-18276

OhioEPA

State of Ohio Environmental Protection Agency

Northeast District Office
2110 E. Aurora Road
Twinsburg, Ohio 44087-1969
(330) 425-9171 **FAX** (330) 487-0769

George V. Voinovich
Governor

May 18, 1998

RE: Bert Avenue Landfill
Cuyahoga County
Waste Stabilization Report
Notice of Deficiency

Mr. Theodore G. Adams, Vice-President
B. Koh & Associates, Inc.
11 West Main Street
Springville, NY 14141-1012

Dear Mr. Adams:

On March 26, 1998, the Ohio Environmental Protection Agency (OEPA)-Division of Solid and Infectious Waste Management (DSIWM)-Northeast District Office (NEDO), received a Waste Stabilization Study Report for the Bert Avenue Landfill in the Village of Newburgh Heights, Cuyahoga County, Ohio. Due to the problems encountered in complying with Condition Ten (10) of the closure plan approval dated July 24, 1996, Dames & Moore prepared the report on behalf of B. Koh and Associates as alternative to the closure plan condition.

After a cursory review of the report by the OEPA-DSIWM, a conference call was conducted between representatives of the following organizations: the OEPA-DSIWM, the Nuclear Regulatory Agency (NRC), B. Koh and Associates and Dames & Moore. Due to the issues that were discussed during this call, an addendum to this reported was created by Dames & Moore and submitted to the OEPA-DSIWM-NEDO on April 13, 1998. The OEPA-DSIWM has completed a review of the original report and the addendum. A copy of the review is enclosed.

If you have any questions, I can be contacted at (330) 963-1186.

Sincerely,

Jerry L. Parker, R.S., E.I.T.
Division of Solid and Infectious
Waste Management

NL10

130041

enclosure

cc: Mr. Kurt Princic, DSIWM-NEDO
Mr. Doug Evans, DSIWM-CO
Mr. John Romano, CCHD
Mr. Tim Johnson, NRC
Mr. Bruce Jorgensen, NRC
Mr. Doug Perisutti, Solar Testing
Mr. Rich Lacey, Geotechnics
FILE:[LAND/Bert Avenue LF/ COR/18]
Mr. Herb Davidson, AWSR
Mr. Brien Kilkenny, AWSR
Mr. Steve Kilper, AWS
Mr. Larry Chintella, Dames&Moore
Mr. Fred Erdman, Dames&Moore
Mr. Pete Smith, Dames&Moore
Mayor Ed Kohlar, Newburgh Heights

TECIP NMSS/DWM

OhioEPA
State of Ohio Environmental Protection Agency

STREET ADDRESS:
1800 WaterMark Drive
Columbus, OH 43215-1099

TELE: (614) 644-3020 FAX: (614) 644-2329

MAILING ADDRESS:
P.O. Box 1049
Columbus, OH 43216-1049

INTEROFFICE COMMUNICATION

TO: Jerry Parker, DSIWM-NEDO

FROM: Doug Evans, DSIWM-CO

SUBJECT: Slope Stability Comments for Bert Avenue Site

DATE: May 18, 1998

Pursuant to your request, I have reviewed the slope stability analysis portion of the report titled, *Waste Stabilization Study Report*, dated March 27, 1998, and the report titled *Eastern Slope Stability Evaluation-Addendum*, dated April 13, 1998. Both reports were prepared by Dames & Moore and address stability issues with the proposed design of the waste containment cell at the Bert Avenue Site.

BACKGROUND

Ohio EPA evaluates the adequacy of slope stability factors of safety (FOS) based on the consequences of a slope failure and the confidence in the slope stability analysis (SSA) input parameters. The following table is a condensed version of the performance criteria contained in DSIWM Guidance # 180 *Factors of Safety For Slope Stability Analysis*. The guidance document is included as Attachment 1.

Recommended Minimum Factors of Safety

Consequences of Failure	Input Parameter Uncertainty Small	Input Parameter Uncertainty Large
Limited danger or environmental impact	1.25 (1.2)*	1.5 (1.3)
Potential danger or environmental impact	1.5 (1.3)	2.0 (1.7)

* Numbers with parentheses are for dynamic conditions

The design of the containment cell incorporates a geosynthetic clay liner (GCL) as part of the liner system. Ohio EPA has issued an advisory regarding these products, *Advisory on Structural Considerations for Incorporating Geosynthetic Clay Liners In Solid Waste Facility Design,* and is included as Attachment 2. The advisory provides owners, operators and consultants with detailed concerns over the use of these products and specific recommendations to alleviate the concerns. The recommendations include specific testing procedures and performance criteria.

The specific contents of a SSA are sensitive to the particular conditions present at an individual site. However, there are a number of items that should "typically" be included in any SSA in order for DSIWM to determine the appropriateness and adequacy of the evaluation. The SSA should contain both a narrative and supporting information.

- The narrative should include:
    - The scope, extent, and findings of the subsurface investigation;
    - The scope, extent, and findings of the laboratory material testing program;
    - Logic and rationale for the selection of the analysis input parameters;
    - Logic and rationale for the selection of the critical cross-sections;
    - Graphical depictions of the plan and profile views of the critical cross-sections;
    - A discussion of the failure modes and conditions analyzed;
    - The results of the evaluation for the most critical cases of both static and dynamic conditions for both deep-seated and shallow failures mechanisms.

- The supporting data and information should include:
    - Field data from the subsurface investigation;
    - Laboratory data from the material testing program;
    - The actual calculations and/or computer output.

COMMENTS

1. A. Due to the close proximity of homes, a roadway, and the possible use of the area as a park, the potential danger to human life from a deep-seated failure cannot be disregarded. In addition, due to the presence of the "groundwater conveyance layer" and its connectivity to the storm sewer system, the potential exists for contaminates to be rapidly transported off-site if a deep-seated slope failure were to occur. Finally, most of the SSA strength parameters have been assumed using correlative information or generic manufacturer supplied data. These types of strength data are considered to have a large degree of uncertainty associated with their use for the purposes of SSA. Based on the available information, the recommended minimum FOS for deep-seated slope failures at this site are 2.0 and 1.7 for static and dynamic conditions, respectively.

B. Due to the limited danger or environmental impact that would likely occur from a slope failure of the cover system and the large degree of uncertainty in SSA strength parameters, the recommended minimum FOS for shallow slope failures are 1.5 and 1.3 for static and dynamic conditions, respectively.

[Note: By obtaining highly accurate project-specific strength data on project-specific waste, soils and geosynthetics, the recommended minimum FOS can be reduced to 1.5 and 1.3 for deep-seated failures, and 1.25 and 1.2 for shallow failures, for static and dynamic conditions, respectively.]

2. The incorporation of a GCL in the facility's design significantly heightens DSIWM's concerns over slope stability. It is recommended to test the GCL in accordance with the attached GCL advisory to alleviate these concerns. By following the advisory it is possible to reduce the FOS for failure surfaces passing through or along the GCL to 1.3 and 1.1 for static and dynamic conditions, respectively. Should the advisory not be followed, the appropriate FOS for failure surfaces involving the GCL are 2.0 and 1.7 for static and dynamic conditions, respectively, using a shear strength parameter equivalent to hydrated bentonite.

3. The scope, extent and a summary of the findings of the laboratory material testing program as it pertains to the slope stability of the proposed facility should be provided in the SSA narrative. The actual laboratory data should be included in an appendix.

4. The logic, rationale and specific data used for the selection of the analysis input parameters should be documented in the SSA narrative.

A. The waste material has been shown to be very weak at high moisture contents (approximately greater than 17%). As a result, the stabilization report proposes to control moisture content of the waste by the addition of an admixture. Unconfined compression tests on amended waste specimens yielded a minimum undrained shear strength of 4300 psf. 2000 psf was assumed for this layer in the SSA. However, the 2000 psf value may be unconservative at low normal stresses because of the non-linear stress-dependent shear behavior of many soils.

The shear strength of the amended and unamended waste should be determined using a consolidated undrained triaxial procedure, and should be tested over the entire range of normal stresses that will be present in the field due to the design. In addition, the laboratory shear specimens should adequately model and be representative of field fill placement, including material composition, moisture content, and unit weight.

B. The slope stability addendum evaluates the stability of the GCL using a generic shear strength of 500 psf supplied by the manufacturer. Shear strength test data submitted to Ohio EPA on comparable material indicates this value may be unconservative at low normal stresses (see Attachment 3). As previously stated, it is recommended that the project-specific GCL and the materials that it interfaces with, be tested for shear strength in accordance with the GCL advisory.

C. The interface friction angle between the textured geomembrane and the recompacted clay barrier layer of the cover system has been assumed to be 27° from generic manufacturer data. The submittal also indicates this value may be as low as 25°. Since 27° is at the lower range of acceptability, e.g. FOS = 1.53, this value should be verified through project-specific testir.g.

D. The shear strength of the compacted clay in the liner and cap systems has been estimated from textbook literature to be 1600 psf. This value may be unconservative at low normal stresses and should be verified through project-specific testing.

E. The shear strength of the select backfill of the cover system has been selected from the literature to be 1600 psf. This value appears to be unconservative. The material will be exposed to winter freeze/thaw and summer desiccation. Thus, contributions to shear strength from "cohesion" will be negligible due to cracking of the soil. The shear strength of the select backfill should be changed to a frictional rather than a cohesional base, and a crack zone should be specified in the computer model.

5. A discussion of the following failure modes should be included in the SSA narrative. The supporting calculations and data should be included in an appendix.

A. It is not clear if deep-seated static and dynamic rotational failures within the waste have been analyzed. Information should be included in the proposal addressing this failure mode.

B. The hand calculations for static and dynamic translational failures involving the GCL may not adequately model the complex stability issues. It is recommended to include the GCL into the computer analysis model.

To illustrate the failure modes requested by comments 5A. and B., a rudimentary SSA depicting possible failure surfaces is included as Attachment 4. In addition, the analysis utilizes minimal parameter values that will probably be exceeded by the testing requested in comment 4, based on our experiences with the materials in question. Also note that, pending parameter verification, the analysis should meet the performance criteria outlined in comments 1 and 2. Please note that Attachment 4 is offered for illustrative purposes only, and the accuracy of the calculations is neither expressed or implied.

C. The potential for seepage-induced slides of the cover system has not been evaluated. A significant number of failures have occurred across the nation due to inadequate evaluation and design of these systems. At the facilities where these failures occurred, the drainage layers were unable to adequately relieve the pore water pressure that can build-up in the cover system during heavy downpours. Potential pore water pressure build up in the drainage layer must be taken into account when investigating the stability of the final cover system. Consideration of seepage forces should include an investigation of the maximum pore water pressure that may build up in the drainage layer of the cover system based on the maximum fluid flux through the cover soils that could occur during saturated conditions and a major rain event.

If you have comments or questions, please call me at (614) 728-5371.

DE/dk

Attachment
1

OhioEPA

State of Ohio Environmental Protection Agency

STREET ADDRESS:
1800 WaterMark Drive
Columbus, OH 43215-1099

TELE: (614) 644-3020 FAX: (614) 644-2329

MAILING ADDRESS:
P.O. Box 1049
Columbus, OH 43216-1049

DSIWM GUIDANCE DOCUMENT
(614) 644-2621
FAX: (614) 728-5315

SUBJECT: Factors of Safety For Slope Stability Analysis

GUIDANCE #: 0180

REFERENCES:

Municipal Solid Waste	Industrial Solid Waste	Residual Solid Waste
OAC 3745-27-06(C)(4)(i)	OAC 3745-29-06(C)(4)(i)	OAC 3745-29-05(C)(5)(i)
OAC 3745-29-06(C)(4)(j)	OAC 3745-29-06(C)(4)(j)	OAC 3745-29-05(C)(5)(i)

CROSS REFERENCES: "Location Restriction Demonstrations: Unstable Areas", Ohio EPA guidance # 0133, issued June 1, 1994.
"Location Restriction Demonstrations: Seismic Impact Zones", Ohio EPA guidance # 0129, issued May 24, 1994.

DATE: November 24, 1995
(Supersedes document titled "Slope Stability Analysis" dated Feb. 6, 1995)

TOTAL NUMBER OF PAGES: 3

I. PURPOSE

The purpose of this document is to provide guidance on the factors of safety for slope stability analyses for both static and earthquake conditions.

II. APPLICABILITY

This guidance applies to permit applicants of municipal, industrial, and residual solid waste facilities who must present an analysis for slope stability.

III. BACKGROUND

Since the 1990 rules a slope stability analysis has been included in the permit application process as part of the engineering design. The analysis includes both static and earthquake conditions for areas in seismic impact zones, and only static conditions outside of seismic impact zones. Even with the advent of location restriction demonstrations addressing seismic impact zones and unstable areas due to RCRA Subtitle D, DSIWM's engineering design requirements for slope stability did not change in the 1994 rules. However, the factor of safety required is not specified in rule.

Theoretically a factor of safety (FS) < 1 is unstable, a FS > 1 is stable, and a FS = 1 is at equilibrium. This FS is developed from many components affecting the stability of a slope. These components include: failure plane geometry, anisotropy of soil, tension cracks, dynamic loading or earthquakes, and pore water pressure. The differing combinations of these elements produce a degree of uncertainty which cannot be fully accounted for in the slope stability analysis. Therefore due to uncertainties with the quantity and quality of data, the accuracy of the assumptions, and the risks to public health & safety and/or the environment associated with a slope failure, **DSIWM recommends** a FS ≥ 1.5 for static conditions and a FS ≥ 1.3 for seismic conditions. These recommended values were obtained from the U.S. EPA Guide to Technical Resources for the Design of Land Disposal Facilities, see Table 1. Alternative values will be evaluated if the owner or operator can satisfactorily show that lower factors of safety are based on the quality of data, conservative assumptions, and consequences of a slope failure. However, it should be noted that if the slope being analyzed presents **imminent** danger to human life or the environment and the quality of soil data is poor, DSIWM may choose to increase the FS to at least 2.0 for static conditions and at least 1.7 for seismic conditions, as depicted in Table 1.

Additionally, in Ohio EPA guidance #0133, "Location Restriction Demonstration: Unstable Areas", the recommended FS is 1.5 for static conditions and 1.3 for earthquake conditions. Also, in Ohio EPA guidance #0129, "Location Restriction Demonstration: Seismic Impact Zones", the recommended FS is 1.3 for earthquake conditions.

IV. PROCEDURE

For Facilities Outside of Seismic Impact Zones--Only static conditions need be addressed in the slope stability analysis. Each side of the landfill may be investigated separately. If the FS is less than the recommended 1.5 for static conditions, the owner or operator can propose an alternative FS based on the quality of data, conservative assumptions, and consequences of failure. However, if an **imminent** danger to human life or the environment is present and the quality of data is poor, DSIWM may choose to either increase the FS to at least 2.0, or request that the owner or operator improve the quality of data.

For Facilities Located in Seismic Impact Zones--Both static and earthquake conditions must be addressed in the slope stability analysis. Each side of the landfill may be investigated separately. If the FS is less than the recommended 1.5 for static conditions or if the FS is less than the recommended 1.3 for earthquake conditions, the owner or operator can propose an alternative FS based on the quality of data, conservative assumptions, and consequences of failure. However, if an **imminent** danger to human life or the environment is present and the quality of data is poor, DSIWM may choose to either increase the FS to at least 2.0 for static conditions and to increase the FS to at least 1.7 for earthquake conditions, or request that the owner or operator improve the quality of data.

V. POINT OF CONTACT

Engineering - Policy Unit, Supervisor, (614) 728-5373

Table 1.

Recommended Minimum Values of Factor of Safety for Slope Stability Analyses

Consequences of Slope Failure	Uncertainty of Strength Measurements Small[1]	Large[2]
No imminent danger to human life or major environmental impact if slope fails	1.25 (1.2)*	1.5 (1.3)
Imminent danger to human life or major environmental impact if slope fails	1.5 (1.3)	$FS \geq 2.0$ ($FS \geq 1.7$)

1 The uncertainty of strength measurements is smallest when the soil conditions are uniform and high quality strength test data provide a consistent, complete, and logical picture of the strength characteristics.

2 The uncertainty of the strength measurements is greatest when the soil conditions are complex and when strength data do not provide a consistent, complete, and logical picture of the strength characteristics.

* Numbers without parentheses apply to static conditions and those within parentheses apply to seismic conditions.

Source: EPA Guide to Technical Resources for the Design of Land Disposal Facilities.

Attachment
2

OhioEPA

State of Ohio Environmental Protection Agency

STREET ADDRESS:
1800 WaterMark Drive
Columbus, OH 43215-1099

TELE: (614) 644-3020 FAX: (614) 644-2329

MAILING ADDRESS:
P.O. Box 1049
Columbus, OH 43216-1049

MEMORANDUM

TO: All Solid Waste Landfill Facility Owners/Operators, Approved Health Departments, and Design Engineers

FROM: Doug Evans, Division of Solid and Infectious Waste Management (DSIWM)

SUBJECT: Advisory on Structural Integrity Considerations for Incorporating Geosynthetic Clay Liners In Solid Waste Landfill Facility Design

DATE: September 17, 1997

1.0 Introduction

Ohio's solid waste landfill regulations allow a geosynthetic clay liner (GCL) to be used in lieu of the recompacted soil barrier layer of the composite cap system or in lieu of a portion of the recompacted soil layer of the composite bottom liner system. Nevertheless, GCLs are a relative newcomer to the evolving field of waste containment, and significant concerns remain over their ability to be appropriately incorporated into waste containment designs. These concerns include inherent stability shortcomings, hydraulic equivalency, and long term performance. Many of these issues continue to be investigated by manufacturers and researchers alike who have, over time, offered changing, conflicting, and ambiguous information on GCLs, thus creating uncertainty regarding the appropriate use of these products.

Recent information suggests that there are special considerations which must be taken into account when utilizing a GCL in certain applications, including use on side slopes and in areas of landfills where localized non-uniform stresses may be encountered.

The purpose of this document is to provide owners, operators, and consultants with the detailed concerns that DSIWM has for the use of GCLs in solid waste landfill design, as well as specific recommendations to allay these concerns.

2.0 Background

Initially, issues regarding GCLs centered on hydraulic conductivity, equivalence to compacted clay liners, and internal shear strength. More recently, interface shear strength, bearing capacity, and overall long term performance have come to the forefront of concern. Ohio's solid waste regulations have addressed the hydraulic conductivity and equivalence issue by setting forth specific criteria regarding the thickness of clay which a GCL can replace, based on its specific mass of bentonite. However, significant issues remain regarding stability and long term performance associated with use of GCLs in landfill design.

The use of a GCL is a double edged sword; the bentonite contained in the GCL provides low hydraulic conductivity, and yet it probably has the least shear resistance or bearing capacity of any soil. Add to this a significant number of engineering failures and a lack of long term performance data, and concern regarding designs incorporating GCLs is heightened. It is our thought that by sharing our concerns and recommendations with owners, operators, and consultants, that GCLs can be properly incorporated in landfill designs and a considerable amount of time and energy can be conserved by all involved in the DSIWM permitting process.

While the advantages of GCLs are numerous, they are beyond the intended scope of this advisory. This document is intended to make our concerns about GCLs known and to provide design and testing recommendations to alleviate these concerns. This document will explain DSIWM's concerns regarding GCLs in more detail, provide recommendations for incorporating GCLs in landfill design, and offer guidance for determining appropriate strength parameters to use in the necessary design calculations. The concerns contained in this advisory must be addressed by owner/operators proposing to use GCLs. The recommendations made in this document should be considered the preferred method for alleviating the listed concerns, but should not be interpreted as regulatory requirements. By following the recommendations of this advisory, owner/operators will benefit from a straightforward review which will be less likely delayed by revisions during the review process. Conversely, if alternative procedures are used to address the concerns outlined in this document, the alternative procedures will have to be evaluated on a case by case basis for their technical merit, and will probably result in a longer review period.

Please note that although this information is being provided to interested parties in a proactive effort to clarify regulatory concerns and expedite permit review, these issues are exceedingly complex and research is ongoing. Therefore, the information is subject to update and revision as more research is conducted and more issues arise.

For the purposes of this document, GCLs can be grouped into two broad categories, reinforced and unreinforced. Reinforced GCLs are basically comprised of three components, a bentonitic clay soil sandwiched between two geotextiles, with reinforcement to provide additional strength. The reinforcement is accomplished by intermittently stitching the three components together (stitch bonding), or by punching fibers throughout the three components (needle punching). Both types of reinforcement provide additional bonding and strength qualities to the product. Unreinforced GCLs consist of a bentonitic clay soil sandwiched between two geotextiles with no reinforcement, or bentonitic clay soil adhered to a geomembrane.

Stability characteristics are unique to each GCL. This is due to the differing geosynthetic components which are combined in individual GCLs and the methods by which the components are joined. Reinforced GCLs have greater shear strength characteristics than unreinforced GCLs. In addition, reinforced GCLs constructed with non-woven geotextiles are more stable over a larger range of applications than those constructed with a woven geotextile. This is because the woven geotextiles allow bentonite to extrude more readily than non-woven textiles. The extruded bentonite essentially lubricates the interface(s) between the GCL and adjacent materials, greatly reducing the shear resistance of the composite system.

3.0 Regulatory Considerations

The following Ohio Administrative Code (OAC) references are useful for the purposes of this advisory.

The municipal, industrial, and residual solid waste (MSW, ISW, RSW) regulations require that a permit applicant demonstrate the stability of the landfill. OAC 3745-27-06(C)(4)(j) in the MSW regulations states;

> *"(C) The following information shall be presented in narrative form in a report divided according to paragraphs (C)(1) to (C)(9) of this rule.*
>
> > *(4) The following design calculations with references to equations used, showing site specific input and assumptions:*
> >
> > > *(j) Slope stability analysis".*

Requirements identical to these in the MSW rules are found in OAC 3745-29-06(C)(4)(j) and OAC 3745-30-05(C)(5)(j) for ISW and RSW facilities, respectively.

The MSW and ISW regulations require that a GCL be negligibly permeable to fluid migration and contain a specific mass of bentonite per area. OAC 3745-27-08(C)(3)(a) and (c) and OAC 3745-29-08(C)(3)(a) and (c) state, respectively, for the MSW and ISW regulations;

> *"(3) A Geosynthetic clay liner used in lieu of part of the recompacted soil liner pursuant to paragraph (C)(1)(j) of this rule, or in lieu of part of the recompacted soil barrier layer, pursuant to paragraph (C)(15) or (C)(16) of this rule, shall have the following characteristics:*
>
> > *(a) Be negligibly permeable to fluid migration; and*
> >
> > *(c) Have a bentonite mass per unit area of at least one pound per square foot".*

4.0 Concerns and Recommendations

DSIWM has two main areas of concern with incorporating a GCL in a landfill design:

- Defining performance standards which can account for uncertainties associated with the use of a relatively new and developing product without a proven long term performance record; and

- Determining accurate and appropriate design parameters to fully account for the exceptionally weak nature of hydrated bentonite.

These two main areas of concern have a number of specific concerns which are discussed in the following sub-sections.

4.1 Assuring Long Term Performance

Very little is known about the long term performance of GCLs. This issue is discussed at length in U.S. EPA's recently released *Report of 1995 Workshop on Geosynthetic Clay Liners*, dated June 1996, and also in the American Society for Testing and Materials (ASTM) Special Testing Publication No. 1308, *Testing and Acceptance Criteria for Geosynthetic Clay Liners*, published in January of 1997. Additionally, there appears to be a growing opinion among eminent researchers in the GCL arena that it may be more prudent to evaluate post-peak strength conditions than peak conditions. This is due to uncertainties surrounding the processes that may initiate deformations in composite lining systems during construction, waste placement, and the waste's subsequent settlement. These processes may result in the development of post-peak or residual shear strength conditions which are weaker than peak strength values.

Ohio EPA guidance document number 180, *Factors of Safety for Slope Stability Analysis*, dated November 24, 1995, explains the methodology that DSIWM uses for the selection of an appropriate recommended factor of safety for a solid waste landfill, based on imminent danger to human life or major environmental impact if the slope were to fail and the degree of certainty in the assumed parameters. However, the incorporation of a GCL in the solid waste landfill design adds an additional unknown to the factor of safety selection process. Therefore, due to uncertainties and a lack of long term performance data, DSIWM recommends designing for post-peak conditions with a 1.3 static factor of safety and a 1.1 dynamic factor of safety for designs incorporating GCLs, see Table 1.

Table 1

Recommended Minimum Factors of Safety

Post-Peak Static Stability [1,2]	1.30
Post-Peak Pseudo-Static Stability [2,3]	1.10

1. Potential pore water pressure build up in the drainage layer must be taken into account when investigating the stability of the final cover system. Consideration of seepage forces should include an investigation of the maximum pore water pressure that may build up in the drainage layer of the cover system based on the maximum fluid flux through the cover soils which could occur during saturated conditions and a major rain event.

   [Comment: Seepage forces are important because a significant number of landfill final cover failures have occurred across the nation due to inadequate design of the drainage layer. Drainage layers have been unable to adequately relieve the pore water pressure that can build up in cap systems during heavy downpours. The design inadequacies include underestimating the volume of water that can permeate through the cover soils during a major rain event and/or inadequate controls for keeping the drainage layer from becoming partially or completely clogged throughout the life and post closure of the landfill.]

2. Post-peak shear strength should be determined utilizing a shear displacement of at least 50 mm (2 in).

3. Should a deformational approach be chosen over a pseudo-static analysis, deformation in the composite cap system should not exceed 15 cm (6 in) and deformation in the composite liner system should not exceed 10 cm (4 in).

4.2 Accounting for the Weak Nature of Hydrated Bentonite

The bentonite component of the GCL usually controls the strength characteristics of the composite bottom liner and cap system. Hydrated bentonite has the lowest peak and residual shear strengths of any soil. Bentonitic soils also have an extremely high affinity for moisture and will wick significant amounts of moisture from even the driest subgrade. In other words, GCLs <u>will</u> hydrate. Bentonite's affinity for moisture results in extraordinarily large swell pressures which can cause the hydrated bentonite to extrude from the GCL into the interfaces between the GCL and adjacent materials, essentially lubricating these interfaces, thereby weakening the structural integrity of the composite system.

Hydrated bentonite also exhibits an extremely low bearing capacity. Thus localized non-uniform stresses can cause the bentonite in GCLs to flow or migrate away from higher stress concentrations allowing the GCL to thin in localized areas. This bentonite thinning results in GCLs no longer meeting the regulatory requirements on specific mass per unit area, and greatly increases fluid flux through the GCL.

It is the low hydraulic conductivity of hydrated bentonite that makes the GCL useful and it is also the hydrated bentonite that makes the GCL so weak. Focusing on the weakness issue, some designers have suggested encapsulating the GCL between two geomembranes to prevent hydration. While this will minimize widespread hydration, localized zones of hydrated bentonite and ensuing weakened conditions are still a possibility owing to imperfections in geomembrane installation. A U.S. EPA sponsored test section of an encapsulated GCL recently failed due to such localized zones of hydration.

4.2.1 Determining Shear Strength Characteristics

Many times in the past, slope stability calculations required in the permitting process have been submitted to DSIWM utilizing manufacturer-supplied generic shear strength data. While this data may be useful in preliminary design evaluations, it is inadequate for the stability calculations required in the DSIWM permitting process. Typically, manufacturer's data is accompanied by disclaimers which state that the information should not be relied upon to determine final design parameters and that project-specific shear testing should be conducted for this purpose. DSIWM emphatically recommends testing the shear strength of project-specific materials under appropriate conditions, including normal stress, moisture content, and shearing procedure.

Currently, no established or otherwise universally accepted test method exists for determining the internal shear strength and interface shear strength of a GCL. "Appropriate" shear testing has proven to be a highly subjective and controversial issue around the state and nation. This is to be expected when one considers the array of products, each with distinctively different characteristics, and the reality that any inaccuracies inadvertently introduced into sample selection, sample preparation, or actual shearing may falsely increase the measured shear resistance.

With this in mind, DSIWM is outlining some of the more pertinent aspects of shear testing a GCL and recommending the following specific testing procedures.

A. Sample Selection

Ideally the shear samples should be selected from rolls that are delivered to the site. However, this is often impractical. The next best alternative is to obtain identical product samples from another site. If either of the preceding options are unavailable, samples from the manufacturer may be used, if the manufacturer will certify that the samples are representative of materials shipped to the field. This is important because the amount of reinforcement can vary significantly in the manufacturing process.

B. Hydration

- According to U.S. EPA (1996), GCLs will hydrate when placed in contact with typical construction subgrade soils and will probably hydrate significantly within the first few days (moisture contents as high as 50 % were measured after 10 days). Stark (1997a) reports that this hydration typically occurs under a free swell condition and that the swell pressure of a reinforced GCL can be on the order of 35 to 40 kPa (730 - 835 psf). A confining stress of this magnitude, equivalent 2.1 to 2.5 m (7 - 8 ft) of soil, is typically never applied to a cap system and it is usually a number of weeks if not months before a confining stress capable of preventing GCL swell is applied to the composite bottom liner system. In addition, this swell pressure is capable of destroying the reinforcement of GCLs and/or forcing hydrated bentonite into the interfaces, thereby greatly decreasing the integrity of the bottom liner or cap system. Consequently, DSIWM recommends that project-specific GCLs and adjacent materials be allowed to fully hydrate, as a single unit, in a free swell condition until vertical expansion has essentially ceased (an inconsequential confining stress of no more than 0.5 psi to prevent sample deterioration or to provide a founding for displacement measurement is acceptable). The vertical expansion should be determined by monitoring vertical displacement until swelling has reached 100% primary as determined by ASTM 4546 and moisture samples should be taken from the hydrated GCL after the shear test to verify the degree of hydration.

C. Normal Stress

DSIWM recommends that project-specific materials including soils and geosynthetics be tested for internal and interface shear strength over the entire range of normal stresses which will be encountered in the particular design.

- For cap systems, this includes the low normal stresses associated with these applications and any additional stresses which may be induced by surface water diversion benches, roads, equipment, or other structures constructed above the composite cap system.

- For composite bottom liner systems, the range of normal stresses which needs to be evaluated can be extensive, varying from low values at the perimeter of the fill to extremely high values under the deepest areas of the fill.

D. Shear Displacement Rate

Gilbert et al. (1997) and Stark (1997) show that the rate of shear displacement can greatly affect the measured shear strength of GCLs. Shear strength values from tests using a displacement rate of 1 mm/min, the industry norm, have been shown to be in significant excess of those values using slower displacement rates. Stark (1997) reports that rates equal to or less than 0.04 mm/min (.0016 in/min) do not seem to have a detrimental affect on measured shear strength values of one reinforced GCL. Gilbert et al. (1997) and U.S. EPA (1996) recommend ASTM D-3080 for determining the appropriate direct shear rate. DSIWM recommends following the ASTM D-3080 procedure for determining the appropriate direct shear rate for GCLs; and that the direct shear rate should not exceed 0.04 mm/min.

E. Test Method

Currently the most common method used for determining internal shear strengths and interface shear strengths of GCLs is ASTM D-5321 utilizing a 300 mm square shear box. DSIWM recommends this procedure for determining the shear strength of Geosynthetic/Geosynthetic or Geosynthetic/soil interfaces, and the internal shear strength of GCLs.

4.2.2 Avoiding GCL Thinning

After GCLs have hydrated and stresses have been applied, the bentonite has been observed to migrate away from high stress concentrations, resulting in localized thinning of the GCL. This phenomenon is especially likely to occur in areas of composite bottom lining systems where non-uniform stress concentrations typically develop. This includes areas in the immediate proximity of wrinkles, in and around sumps, and beneath leachate collection piping. Thinning of the GCL due to migration of the bentonite has been observed at one facility here in Ohio.

One-dimensional compression tests show that the thickness of a hydrated GCL can decrease significantly due to bentonite migration. This phenomenon has been evidenced in exhumed GCLs and has been noted by numerous authors including Fox et al. (1997), Richardson (1997), Anderson (1996), Koerner and Narejo (1995), and Anderson and Allen (1995). According to Fox et al. (1997), bentonite migration seems to be more pronounced in unreinforced GCLs than in reinforced GCLs. Anderson and Allen (1995) and Anderson (1996) also show that the thickness of a GCL can be significantly reduced in the vicinity of a wrinkle in the overlying geomembrane due to hydrated bentonite flowing up into the air space of the wrinkle, which may change shape but does not necessarily disappear according to Koerner (1996).

Thinning of the GCL has serious implications for meeting the regulatory requirements, which include criteria for specific mass of bentonite per unit area and hydraulic performance. GCLs are allowed to replace a portion of the recompacted soil layer based on their hydraulic performance. However, the hydraulic performance or fluid flux through a GCL is directly related to the thickness or specific mass of bentonite per unit area. Thus, if the bentonite thins, the fluid flux through the GCL will increase, and the requirements for hydraulic performance and specific mass of bentonite per unit area may no longer be satisfied. It is therefore recommended that the sump areas and areas directly beneath leachate collection piping not incorporate GCLs, and that wrinkling of the geomembrane be kept to an absolute minimum. DSIWM recognizes that there will be design and construction difficulties associated with this recommendation and that there are alternative approaches. Unfortunately, insufficient information currently exists for DSIWM to make any other recommendation.

5.0 Concerns and Recommendations Unique to Unreinforced GCLs:

Unreinforced GCLs lack any added reinforcement to resist shear stresses, such as needle punching or stitch bonding. As a consequence, these products have internal shear strength and bearing capacity characteristics approximately equivalent to hydrated bentonite. USEPA (1996) comments that shear data on unreinforced GCLs show friction angles of about 10 degrees. Richardson (1997) estimates the bearing capacity of a hydrated unreinforced GCL to be 40 kPa (825 psf) and the internal shear strength to be less than 5 kPa (100 psf) for low normal stresses such as those associated with caps.

For low normal stresses such as those in cap systems, unreinforced GCLs will hydrate fully under confining stresses significantly less than the swell pressure of the GCL. Furthermore, these products have a severely limited shear resistance which essentially corresponds to hydrated bentonite. These products may also undergo significant creep due to the time-dependent deformational characteristics of hydrated bentonite, resulting in extremely low post-peak or residual strength conditions. Additionally, the extremely low bearing capacity of unreinforced GCLs may result in thinning of the GCL from bentonite migration due to non-uniform stress concentrations, such as wheel loads, that may be applied to a cap during closure and post closure. For these reasons, it is recommended that composite cap system designs do not incorporate unreinforced GCLs and that unreinforced GCLs be restricted to use on bottom lining slopes of less than 10%.

6.0 Procedural Considerations

The recommended testing procedures and factors of safety for GCLs are a component of the slope stability analysis required in the DSIWM permitting process. The first Ohio Administrative Code cited in Section 3.0, Regulatory Considerations points out that a slope stability analysis is to be included in the narrative section of the permit to install application. This requirement applies to all permit applications or alteration requests proposing to use a GCL, initially; and may apply to alterations or other changes proposing to exchange one GCL for another. Additionally, this requirement may also apply to permit applications, alteration requests, or other changes already incorporating a GCL, but proposing to change materials or thicknesses of materials for individual components of the composite bottom liner and composite cap system, or any other circumstance that may cause uncertainty in the validity of previously submitted slope stability calculations.

The specific contents of a slope stability analysis can be sensitive to particular conditions present at an individual site and often need to be assessed on a case by case basis. However, in general, a slope stability analysis for a landfill should include the following:

A. The rationale, cross-sections, and plan views, for critical slope conditions* which may occur during the excavation and construction of the landfill**.

B. The rationale, cross-sections, and plan views, for critical slope conditions* which may occur during the operation and filling of the landfill**.

C. The rationale, cross-sections, and plan views, for critical slope conditions* which may occur during final closure and post closure care of the landfill.

D. The rationale for the selection of soil and geosynthetic strength characteristics, including detailed information from a site specific subsurface exploration, and detailed information from a project specific materials shear strength testing program.

E. A discussion of the methodology used for the determination of the factors of safety.

F. The physical calculations and/or computer output for the critical conditions of the excavation, intermediate or interim waste slopes, and final slopes.

* Determining critical slope conditions includes investigating both static and dynamic cases for both deep-seated and shallow failure surfaces for both rotational and translational modes of failure.

** Operational and construction practices can have a profound impact upon the integrity of the engineered components of waste containment facilities and should not be overlooked in the design process. Recommendations for operational and construction practices relating to geosynthetics have been provided in a previous memorandum titled *Unstable Slopes Advisory for Solid Waste Landfill Facilities*, dated December 2, 1996. Specific terms and conditions of a permit to install may be necessary in order to limit waste placement to a maximum slope height and inclination during the filling of a phase or unit to maintain the integrity of the engineered components of the landfill.

7.0 Summary

In summary, Ohio's solid waste regulations allow a GCL to be used in lieu of the recompacted soil layer of the composite final cap system or for a portion of the recompacted soil layer of the composite bottom liner system. However, any liner or cap system utilizing one of these products must perform adequately. DSIWM has significant reservations regarding the ability of GCLs to perform as safely and durably as compacted clay soils in some applications. These concerns are due to the inherent low strength characteristics of bentonitic soils and a lack of long term performance data on these products. The low strength characteristics of bentonite preclude GCLs from being used on some slopes and allow GCLs to thin when subjected to non-uniform stresses. In an effort to provide direction to interested parties in alleviating DSIWM's concerns and to expedite review of proposals incorporating these products, DSIWM offers the following recommendations:

- Project-specific geosynthetics and soils should be tested appropriately for internal and interface shear strengths over the entire range of normal stresses which will be encountered for a particular application, and the results incorporated into the required slope stability calculations.

- The recommended minimum factors of safety for GCLs are listed below and should be satisfied using a post-peak shear strength with a shear displacement of at least 50 mm (2 in).

Post-Peak Static Stability	1.30
Post-Peak Pseudo-Static Stability	1.10

- Prior to shearing, the GCL should be allowed to fully hydrate in a free swell condition until primary swell is complete. The moisture content should be verified upon completion of the shear test.

- DSIWM recommends that the rate of shear for direct shear tests on GCLs be determined using ASTM D-3080, and that it not exceed 0.04mm/min.

- DSIWM recommends determining internal and interface shear strengths of GCLs by ASTM D-5321 utilizing a 300 mm square shear box.

- Wrinkling of the geomembrane should be kept to an absolute minimum, and any sump areas and areas directly beneath leachate collection piping should not incorporate GCLs.

- Unreinforced GCLs should only be used on slopes with a grade of less than 10%, and should not be used in composite cap systems.

The recommendations made above apply to all permit applications or alteration requests initially proposing to use a GCL, and may apply to alterations or other changes proposing to exchange one GCL for another. Additionally, these recommendations may apply to permit applications, alteration requests, or other changes already incorporating a GCL, but proposing to change materials or thicknesses of materials for individual components of the composite bottom liner or composite cap system, or any other circumstance that may cause uncertainty in the validity of previously submitted slope stability calculations.

A substantial portion of the information contained in this advisory will be incorporated into a comprehensive policy statement on slope stability. A draft copy of the policy will be distributed to interested parties for review and comment. If you have any comments or questions concerning the information contained in this advisory or would like information regarding the forthcoming slope stability policy, please contact me at (614) 728-5371. If you would like to be included on the interested party list for the slope stability policy please fax me your name, address, company/affiliation, telephone and fax numbers at (614) 728-5315.

DE/dk

Attachment: References

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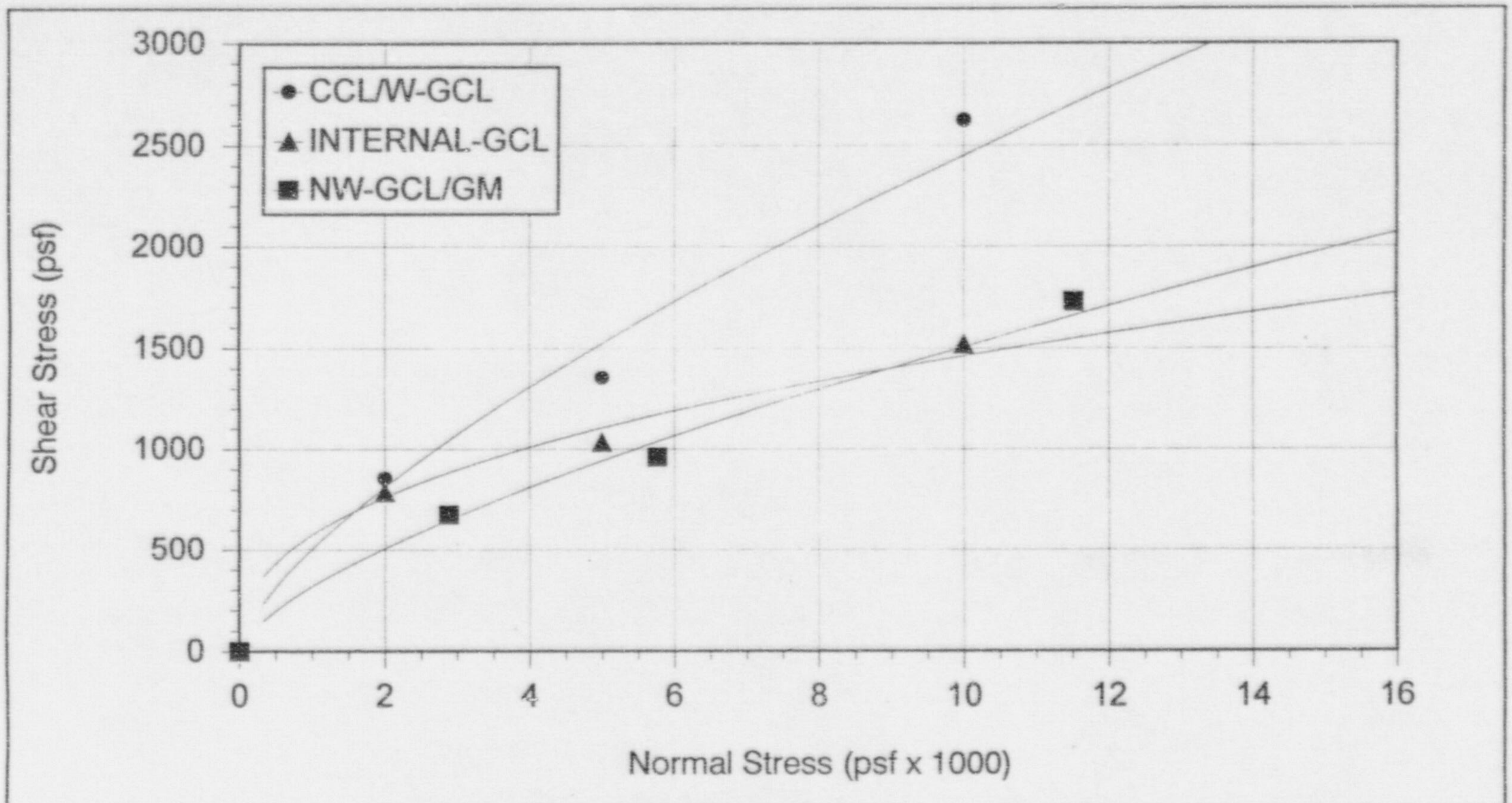
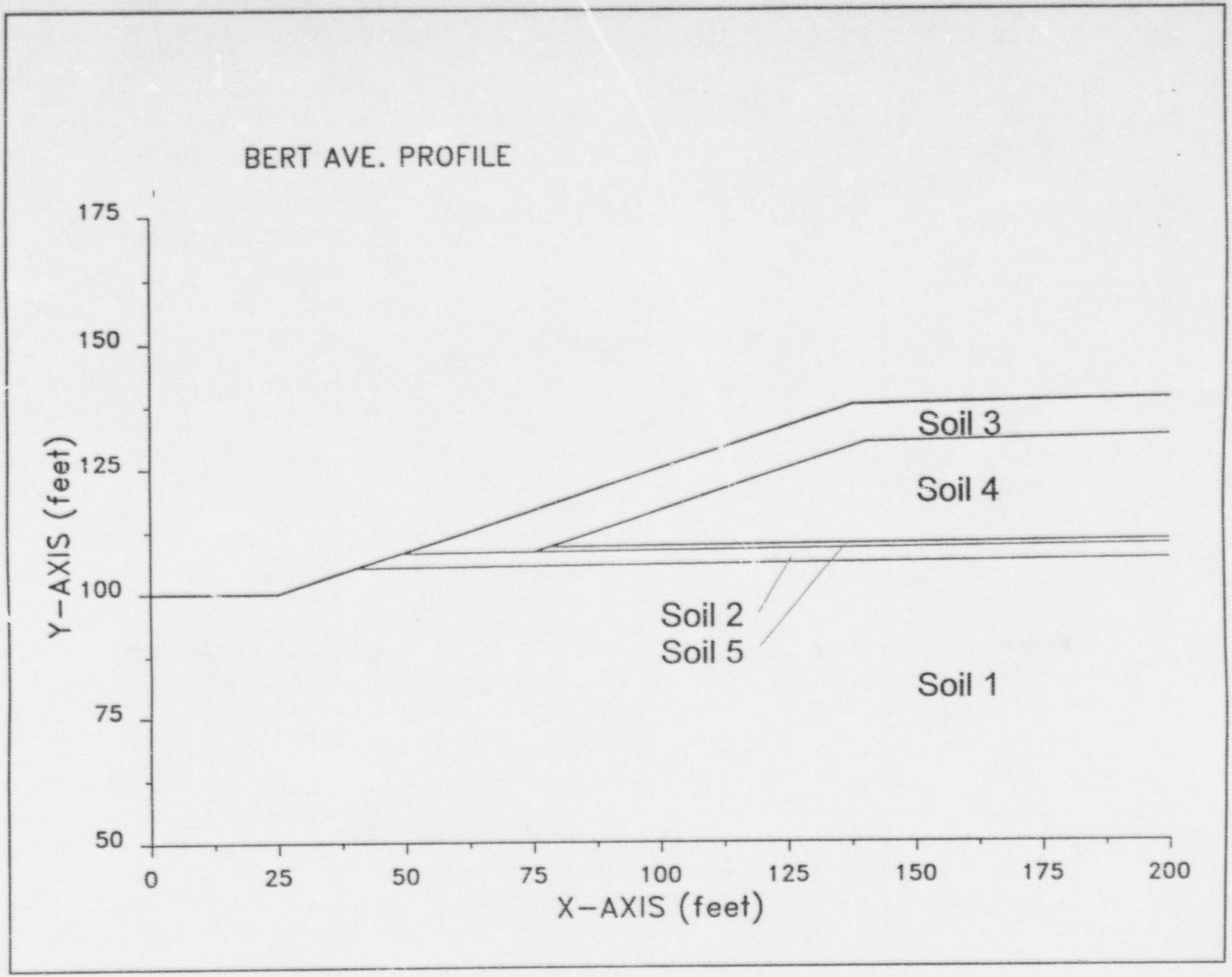
Attachment
3

GCL Failure Envelope



CCL - compacted clay liner, W - woven, GCL - geosynthetic clay liner,
NW - non-woven, GM - geomembrane.

Attachment 4


BERT AVE. PROFILE
Soil 3
Soil 4
Soil 2
Soil 5
Soil 1
Y-AXIS (feet)
X-AXIS (feet)
175
150
125
100
75
50
0
25
50
75
100
125
150
175
200

Soil #	Material Type	"Wet" Unit Weight (psf)	Cohesion	Friction Angle
1	Ground Water Conveyance Material	130	0	30
2	Recompacted Soil Liner	137	0	27
3	Recompacted Soil Barrier and Protective Material	137	0	27
4	Waste	135	0	25
5	GCL	110	non-linear	non-linear

GCL Non-Linear Strength Parameters	
Normal Stress (psf)	Shear Stress (psf)
0	0
150	100
300	150
500	200
3000	500
6000	800

X S T A B L

Slope Stability Analysis
using the
Method of Slices

Ver. 5.202 96 á 1605

Problem Description : BERT AVE. Static Rotational

SEGMENT BOUNDARY COORDINATES

5 SURFACE boundary segments

Segment No.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)	Soil Unit Below Segment
1	.0	100.0	25.0	100.0	1
2	25.0	100.0	40.0	105.0	1
3	40.0	105.0	49.0	108.0	2
4	49.0	108.0	137.6	137.5	3
5	137.6	137.5	200.0	138.7	3

7 SUBSURFACE boundary segments

Segment No.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)	Soil Unit Below Segment
1	49.0	108.0	75.0	108.3	2
2	75.0	108.3	78.0	109.3	5
3	78.0	109.3	140.1	130.0	4
4	140.1	130.0	200.0	131.2	4
5	78.0	109.3	200.0	110.5	5
6	75.0	108.3	200.0	109.6	2
7	40.0	105.0	200.0	106.6	1

A CRACKED ZONE HAS BEEN SPECIFIED

Depth of crack below ground surface = 3.00 (feet)

Maximum depth of water in crack = .00 (feet)
Unit weight of water in crack = 62.40 (pcf)

Failure surfaces will have a vertical side equal to the specified depth of crack and be affected by a hydrostatic force according to the specified depth of water in the crack

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ISOTROPIC Soil Parameters

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5 Soil unit(s) specified

Soil Unit No.	Unit Weight Moist (pcf)	Unit Weight Sat. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Parameter Ru	Pore Pressure Constant (psf)	Water Surface No.
1	130.0	130.0	.0	30.00	.000	.0	0
2	137.0	137.0	.0	27.00	.000	.0	0
3	137.0	137.0	.0	27.00	.000	.0	0
4	135.0	135.0	.0	25.00	.000	.0	0
5	110.0	110.0	.0	.00	.000	.0	0

NON-LINEAR MOHR-COULOMB envelope has been specified for 1 soil(s)

Soil Unit # 5

Point No.	Normal Stress (psf)	Shear Stress (psf)
1	.0	.0
2	150.0	100.0
3	300.0	150.0
4	500.0	200.0
5	3000.0	500.0
6	6000.0	800.0

A SINGLE FAILURE SURFACE HAS BEEN SPECIFIED FOR ANALYSIS

Trial failure surface is CIRCULAR, with a radius of 57.89 feet

Center at x = 90.99 ; y = 166.76 ; Seg. Length = 4.00 feet

The CIRCULAR failure surface was estimated by the following 23 coordinate points :

Point No.	x-surf (ft)	y-surf (ft)
1	67.11	114.03
2	70.81	112.51

3	74.60	111.24
4	78.48	110.24
5	82.41	109.51
6	86.38	109.06
7	90.38	108.88
8	94.38	108.97
9	98.36	109.35
10	102.31	109.99
11	106.20	110.91
12	110.02	112.09
13	113.75	113.54
14	117.37	115.24
15	120.87	117.18
16	124.22	119.36
17	127.42	121.77
18	130.44	124.39
19	133.27	127.22
20	135.90	130.23
21	138.31	133.42
22	139.03	134.53
23	139.03	137.53

****

SELECTED METHOD OF ANALYSIS: Spencer (1973)

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****

SUMMARY OF INDIVIDUAL SLICE INFORMATION

****

Slice	x-base (ft)	y-base (ft)	height (ft)	width (ft)	alpha	beta	weight (lb)
1	68.96	113.27	1.38	3.70	-22.38	18.42	698.
2	72.71	111.87	4.02	3.79	-18.42	18.42	2089.
3	76.54	110.74	6.43	3.87	-14.46	18.42	3410.
4	79.23	110.10	7.96	1.51	-10.50	18.42	1650.
5	81.20	109.74	8.98	2.42	-10.50	18.42	2976.
6	83.09	109.44	9.92	1.37	-6.54	18.42	1858.
7	85.08	109.21	10.81	2.60	-6.54	18.42	3831.
8	88.38	108.97	12.14	4.00	-2.58	18.42	6575.
9	92.38	108.93	13.52	4.00	1.38	18.42	7313.
10	96.37	109.16	14.61	3.98	5.34	18.42	7891.
11	98.86	109.43	15.17	1.00	9.30	18.42	2072.
12	100.84	109.75	15.51	2.94	9.30	18.42	6211.
13	104.25	110.45	15.95	3.89	13.26	18.42	8447.
14	108.11	111.50	16.18	3.82	17.22	18.42	8410.
15	111.89	112.82	16.12	3.73	21.18	18.42	8181.
16	115.56	114.39	15.78	3.62	25.14	18.42	7772.
17	119.12	116.21	15.14	3.50	29.10	18.42	7201.
18	122.55	118.27	14.21	3.35	33.06	18.42	6489.
19	125.82	120.57	13.01	3.19	37.02	18.42	5663.
20	128.93	123.08	11.53	3.02	40.98	18.42	4750.
21	131.85	125.81	9.78	2.83	44.94	18.42	3785.
22	133.58	127.57	8.59	.62	48.90	18.42	726.

23	134.89	129.08	7.52	2.01	48.90	18.42	2072.
24	136.75	131.36	5.86	1.70	52.86	18.42	1367.
25	137.96	132.95	4.55	.71	52.86	1.10	444.
26	138.67	133.98	3.55	.72	56.82	1.10	351.

Nonlinear M-C Iteration Number - 1

ITERATIONS FOR SPENCER'S METHOD

Iter #	Theta	FOS_force	FOS_moment
2	14.4273	1.5562	1.6010
3	14.7116	1.5595	1.5562
4	14.7008	1.5594	1.5595

ITERATIONS FOR SPENCER'S METHOD

Iter #	Theta	FOS_force	FOS_moment
1	14.7008	1.5594	1.5595

SLICE INFORMATION ... continued :

Slice	Sigma (psf)	c-value (psf)	phi	U-base (lb)	U-top (lb)	P-top (lb)	Delta
1	280.9	.0	27.00	0.	0.	0.	.00
2	766.7	.0	27.00	0.	0.	0.	.00
3	1155.0	.0	27.00	0.	0.	0.	.00
4	1355.1	.0	27.00	0.	0.	0.	.00
5	1503.9	.0	25.00	0.	0.	0.	.00
6	1581.4	.0	25.00	0.	0.	0.	.00
7	1599.9	140.0	6.84	0.	0.	0.	.00
8	1734.5	140.0	6.84	0.	0.	0.	.00
9	1872.6	140.0	6.84	0.	0.	0.	.00
10	1973.8	140.0	6.84	0.	0.	0.	.00
11	2001.4	140.0	6.84	0.	0.	0.	.00
12	2082.4	.0	25.00	0.	0.	0.	.00
13	2058.8	.0	25.00	0.	0.	0.	.00
14	2009.2	.0	25.00	0.	0.	0.	.00
15	1925.6	.0	25.00	0.	0.	0.	.00
16	1811.3	.0	25.00	0.	0.	0.	.00
17	1669.7	.0	25.00	0.	0.	0.	.00
18	1504.1	.0	25.00	0.	0.	0.	.00
19	1318.3	.0	25.00	0.	0.	0.	.00
20	1116.3	.0	25.00	0.	0.	0.	.00
21	902.2	.0	25.00	0.	0.	0.	.00
22	751.3	.0	25.00	0.	0.	0.	.00
23	647.9	.0	27.00	0.	0.	0.	.00
24	474.3	.0	27.00	0.	0.	0.	.00
25	368.7	.0	27.00	0.	0.	0.	.00
26	267.6	.0	27.00	0.	0.	0.	.00

SPENCER'S (1973) - TOTAL Stresses at center of slice base

Slice #	Base x-coord (ft)	Normal Stress (psf)	Vertical Stress (psf)	Pore Water Pressure (psf)	Shear Stress (psf)
1	68.96	280.9	188.7	.0	91.8
2	72.71	766.7	550.5	.0	250.5
3	76.54	1155.0	880.5	.0	377.4
4	79.23	1355.1	1090.9	.0	442.8
5	81.20	1503.9	1229.3	.0	449.7
6	83.09	1581.4	1355.4	.0	472.9
7	85.08	1599.9	1471.5	.0	212.9
8	88.38	1734.5	1645.3	.0	223.3
9	92.38	1872.6	1828.7	.0	233.9
10	96.37	1973.8	1981.3	.0	241.7
11	98.86	2001.4	2063.3	.0	243.8
12	100.84	2082.4	2110.3	.0	622.7
13	104.25	2058.8	2169.5	.0	615.6
14	108.11	2009.2	2201.1	.0	600.8
15	111.89	1925.6	2193.3	.0	575.8
16	115.56	1811.3	2146.3	.0	541.6
17	119.12	1669.7	2060.3	.0	499.3
18	122.55	1504.1	1935.6	.0	449.8
19	125.82	1318.3	1772.9	.0	394.2
20	128.93	1116.3	1573.0	.0	333.8
21	131.85	902.2	1336.7	.0	269.8
22	133.58	751.3	1175.8	.0	224.7
23	134.89	647.9	1029.5	.0	211.7
24	136.75	474.3	802.5	.0	155.0
25	137.96	368.7	623.9	.0	120.5
26	138.67	267.6	485.7	.0	87.5

SPENCER'S (1973) - Magnitude & Location of Interslice Forces

Slice #	Right x-coord (ft)	Force Angle (degrees)	Interslice Force (lb)	Force Height (ft)	Boundary Height (ft)	Height Ratio
1	70.81	14.70	793.	1.25	2.75	.452
2	74.60	14.70	2778.	1.81	5.28	.342
3	78.48	14.70	5482.	2.43	7.57	.321
4	79.99	14.70	6568.	2.65	8.35	.318
5	82.41	14.70	8391.	3.04	9.61	.317
6	83.78	14.70	9318.	3.23	10.22	.316
7	86.38	14.70	10385.	3.83	11.39	.336
8	90.38	14.70	11631.	4.58	12.90	.355
9	94.38	14.70	12412.	5.22	14.14	.369
10	98.36	14.70	12648.	5.78	15.09	.383
11	99.37	14.70	12561.	5.92	15.26	.388
12	102.31	14.70	13418.	5.83	15.76	.370
13	106.20	14.70	13944.	5.71	16.14	.354
14	110.02	14.70	13858.	5.56	16.22	.343
15	113.75	14.70	13202.	5.36	16.02	.334
16	117.37	14.70	12048.	5.09	15.53	.327
17	120.87	14.70	10495.	4.73	14.75	.321

18	124.22	14.70	8661.	4.29	13.68	.314
19	127.42	14.70	6681.	3.76	12.34	.305
20	130.44	14.70	4696.	3.14	10.72	.293
21	133.27	14.70	2850.	2.41	8.84	.273
22	133.88	14.70	2444.	2.22	8.33	.266
23	135.90	14.70	1340.	1.54	6.70	.229
24	137.60	14.70	510.	.77	5.02	.153
25	138.31	14.70	240.	.46	4.09	.112
26	139.03	.00	0.	1.19	3.00	.398

AVERAGE VALUES ALONG FAILURE SURFACE

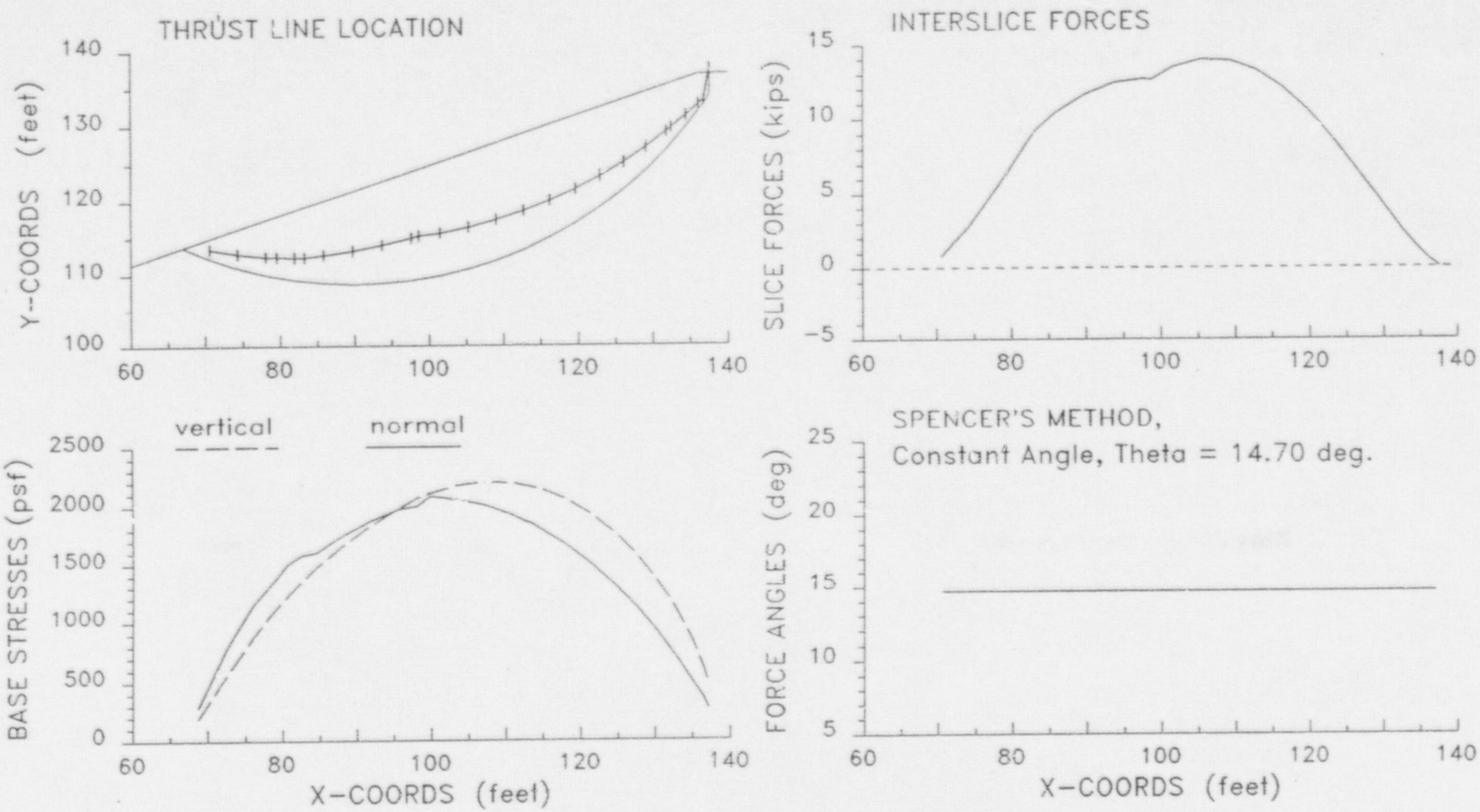
Total Normal Stress = 1397.19 (psf)
Pore Water Pressure = .00 (psf)
Shear Stress = 362.20 (psf)

Total Length of failure surface = 81.32 feet

For the single specified surface and the assumed angle of the interslice forces, the SPENCER'S (1973) procedure gives a

FACTOR OF SAFETY = 1.559

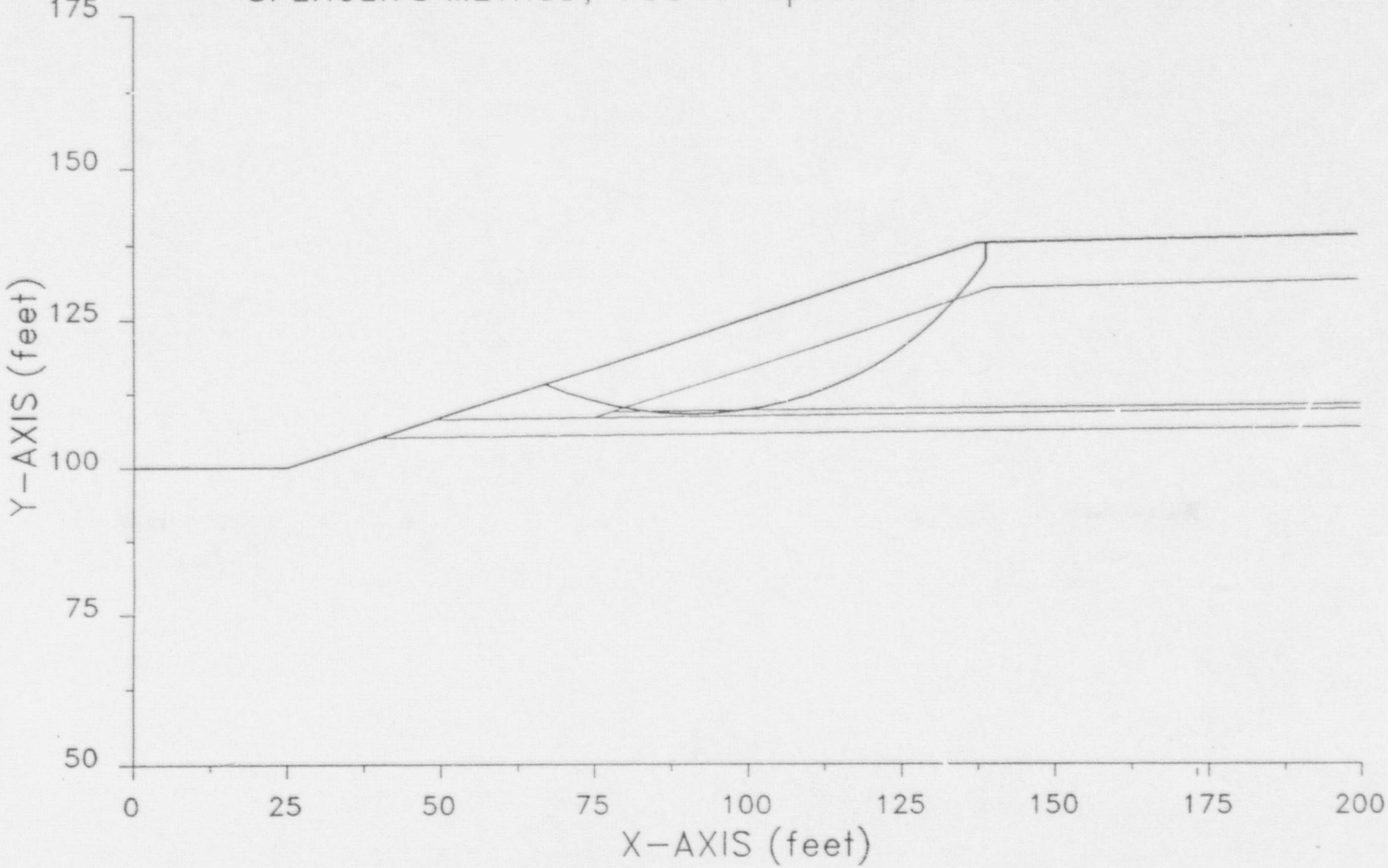
Total shear strength available
along specified failure surface = 459.31E+02 lb



BERT AVE. Static Rotational
SPENCER'S METHOD, FOS for Specified Surface = 1.559

BERT AVE. Static Rotational

SPENCER'S METHOD, FOS for Specified Surface = 1.559



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*               X S T A B L            *
*                                      *
*        Slope Stability Analysis      *
*               using the              *
*            Method of Slices          *
*                                      *
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Problem Description : BERT AVE. Dynamic Rotational

SEGMENT BOUNDARY COORDINATES

5 SURFACE boundary segments

Segment No.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)	Soil Unit Below Segment
1	.0	100.0	25.0	100.0	1
2	25.0	100.0	40.0	105.0	1
3	40.0	105.0	49.0	108.0	2
4	49.0	108.0	137.6	137.5	3
5	137.6	137.5	200.0	138.7	3

7 SUBSURFACE boundary segments

Segment No.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)	Soil Unit Below Segment
1	49.0	108.0	75.0	108.3	2
2	75.0	108.3	78.0	109.3	5
3	78.0	109.3	140.1	130.0	4
4	140.1	130.0	200.0	131.2	4
5	78.0	109.3	200.0	110.5	5
6	75.0	108.3	200.0	109.6	2
7	40.0	105.0	200.0	106.6	1

A CRACKED ZONE HAS BEEN SPECIFIED

Depth of crack below ground surface = 3.00 (feet)

Maximum depth of water in crack = .00 (feet)
Unit weight of water in crack = 62.40 (pcf)

Failure surfaces will have a vertical side equal to the specified depth of crack and be affected by a hydrostatic force according to the specified depth of water in the crack

---

ISOTROPIC Soil Parameters

---

5 Soil unit(s) specified

Soil Unit No.	Unit Weight Moist (pcf)	Unit Weight Sat. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Parameter Ru	Pore Pressure Constant (psf)	Water Surface No.
1	130.0	130.0	.0	30.00	.000	.0	0
2	137.0	137.0	.0	27.00	.000	.0	0
3	137.0	137.0	.0	27.00	.000	.0	0
4	135.0	135.0	.0	25.00	.000	.0	0
5	110.0	110.0	.0	.00	.000	.0	0

NON-LINEAR MOHR-COULOMB envelope has been specified for 1 soil(s)

Soil Unit # 5

Point No.	Normal Stress (psf)	Shear Stress (psf)
1	.0	.0
2	150.0	100.0
3	300.0	150.0
4	500.0	200.0
5	3000.0	500.0
6	6000.0	800.0

A horizontal earthquake loading coefficient of .150 has been assigned

A vertical earthquake loading coefficient of .000 has been assigned

---

A SINGLE FAILURE SURFACE HAS BEEN SPECIFIED FOR ANALYSIS

Trial failure surface is CIRCULAR, with a radius of 57.89 feet

Center at x = 90.99 ; y = 166.76 ; Seg. Length = 4.00 feet

The CIRCULAR failure surface was estimated by
the following 23 coordinate points :

Point No.	x-surf (ft)	y-surf (ft)
1	67.11	114.03
2	70.81	112.51
3	74.60	111.24
4	78.48	110.24
5	82.41	109.51
6	8[illegible].38	109.06
7	90.38	108.88
8	94.38	108.97
9	98.36	109.35
10	102.31	109.99
11	106.20	110.91
12	110.02	112.09
13	113.75	113.54
14	117.37	115.24
15	120.87	117.18
16	124.22	119.36
17	127.42	121.77
18	130.44	124.39
19	133.27	127.22
20	135.90	130.23
21	138.31	133.42
22	139.03	134.53
23	139.03	137.53

****
SELECTED METHOD OF ANALYSIS: Spencer (1973)
****

****
SUMMARY OF INDIVIDUAL SLICE INFORMATION
****

Slice	x-base (ft)	y-base (ft)	height (ft)	width (ft)	alpha	beta	weight (lb)
1	68.96	113.27	1.38	3.70	-22.38	18.42	698.
2	72.71	111.87	4.02	3.79	-18.42	18.42	2089.
3	76.54	110.74	6.43	3.87	-14.46	18.42	3410.
4	79.23	110.10	7.96	1.51	-10.50	18.42	1650.
5	81.20	109.74	8.98	2.42	-10.50	18.42	2976.
6	83.09	109.44	9.92	1.37	-6.54	18.42	1858.
7	85.08	109.21	10.81	2.60	-6.54	18.42	3831.
8	88.38	108.97	12.14	4.00	-2.58	18.42	6575.
9	92.38	108.93	13.52	4.00	1.38	18.42	7313.
10	96.37	109.16	14.61	3.98	5.34	18.42	7891.
11	98.86	109.43	15.17	1.00	9.30	18.42	2072.
12	100.84	109.75	15.51	2.94	9.30	18.42	6211.
13	104.25	110.45	15.95	3.89	13.26	18.42	8447.
14	108.11	111.50	16.18	3.82	17.22	18.42	8410.

15	111.89	112.82	16.12	3.73	21.18	18.42	8181.
16	115.56	114.39	15.78	3.62	25.14	18.42	7772.
17	119.12	116.21	15.14	3.50	29.10	18.42	7201.
18	122.55	118.27	14.21	3.35	33.06	18.42	6489.
19	125.82	120.57	13.01	3.19	37.02	18.42	5663.
20	128.93	123.08	11.53	3.02	40.98	18.42	4750.
21	131.85	125.81	9.78	2.83	44.94	18.42	3785.
22	133.58	127.57	8.59	.62	48.90	18.42	726.
23	134.89	129.08	7.52	2.01	48.90	18.42	2072.
24	136.75	131.36	5.86	1.70	52.86	18.42	1367.
25	137.96	132.95	4.55	.71	52.86	1.10	444.
26	138.67	133.98	3.55	.72	56.82	1.10	351.

Nonlinear M-C Iteration Number - 1

ITERATIONS FOR SPENCER'S METHOD

Iter #	Theta	FOS_force	FOS_moment
2	22.2930	1.0518	1.0182
3	21.3175	-----	1.0518
3	21.8053	1.0476	-----
4	21.4355	1.0444	1.0476
5	21.4797	1.0449	1.0444

ITERATIONS FOR SPENCER'S METHOD

Iter #	Theta	FOS_force	FOS_moment
1	21.4797	1.0449	1.0444

SLICE INFORMATION ... continued :

Slice	Sigma (psf)	c-value (psf)	phi	U-base (lb)	U-top (lb)	P-top (lb)	Delta
1	398.8	.0	27.00	0.	0.	0.	.00
2	1006.7	.0	27.00	0.	0.	0.	.00
3	1426.4	.0	27.00	0.	0.	0.	.00
4	1592.2	.0	27.00	0.	0.	0.	.00
5	1730.0	.0	25.00	0.	0.	0.	.00
6	1751.9	.0	25.00	0.	0.	0.	.00
7	1620.5	140.0	6.84	0.	0.	0.	.00
8	1724.5	140.0	6.84	0.	0.	0.	.00
9	1830.6	140.0	6.84	0.	0.	0.	.00
10	1900.2	140.0	6.84	0.	0.	0.	.00
11	1900.1	140.0	6.84	0.	0.	0.	.00
12	2064.5	.0	25.00	0.	0.	0.	.00
13	1997.1	.0	25.00	0.	0.	0.	.00
14	1909.6	.0	25.00	0.	0.	0.	.00
15	1795.1	.0	25.00	0.	0.	0.	.00
16	1657.6	.0	25.00	0.	0.	0.	.00
17	1500.9	.0	25.00	0.	0.	0.	.00
18	1328.6	.0	25.00	0.	0.	0.	.00
19	1144.6	.0	25.00	0.	0.	0.	.00

20	952.6	.0	25.00	0.	0.	0.	.00
21	756.7	.0	25.00	0.	0.	0.	.00
22	619.1	.0	25.00	0.	0.	0.	.00
23	533.0	.0	27.00	0.	0.	0.	.00
24	383.1	.0	27.00	0.	0.	0.	.00
25	297.8	.0	27.00	0.	0.	0.	.00
26	212.6	.0	27.00	0.	0.	0.	.00

SPENCER'S (1973) - TOTAL Stresses at center of slice base

Slice #	Base x-coord (ft)	Normal Stress (psf)	Vertical Stress (psf)	Pore Water Pressure (psf)	Shear Stress (psf)
1	68.96	398.8	188.7	.0	194.5
2	72.71	1006.7	550.5	.0	490.9
3	76.54	1426.4	880.5	.0	695.6
4	79.23	1592.2	1090.9	.0	776.4
5	81.20	1730.0	1229.3	.0	772.1
6	83.09	1751.9	1355.4	.0	781.8
7	85.08	1620.5	1471.5	.0	320.1
8	88.38	1724.5	1645.3	.0	332.0
9	92.38	1830.6	1828.7	.0	344.2
10	96.37	1900.2	1981.3	.0	352.2
11	98.86	1900.1	2063.3	.0	352.2
12	100.84	2064.5	2110.3	.0	921.4
13	104.25	1997.1	2169.5	.0	891.3
14	108.11	1909.6	2201.1	.0	852.2
15	111.89	1795.1	2193.3	.0	801.2
16	115.56	1657.6	2146.3	.0	739.8
17	119.12	1500.9	2060.3	.0	669.8
18	122.55	1328.6	1935.6	.0	592.9
19	125.82	1144.6	1772.9	.0	510.8
20	128.93	952.6	1573.0	.0	425.1
21	131.85	756.7	1336.7	.0	337.7
22	133.58	619.1	1175.8	.0	276.3
23	134.89	533.0	1029.8	.0	259.9
24	136.75	383.1	802.5	.0	186.8
25	137.96	297.8	623.9	.0	145.2
26	138.67	212.6	485.7	.0	103.7

SPENCER'S (1973) - Magnitude & Location of Interslice Forces

Slice #	Right x-coord (ft)	Force Angle (degrees)	Interslice Force (lb)	Force Height (ft)	Boundary Height (ft)	Height Ratio
1	70.81	21.48	1313.	1.43	2.75	.520
2	74.60	21.48	4346.	2.07	5.28	.393
3	78.48	21.48	8223.	2.81	7.57	.371
4	79.99	21.48	9699.	3.08	8.35	.369
5	82.41	21.48	12062.	3.57	9.61	.371
6	83.78	21.48	13210.	3.81	10.22	.373
7	86.38	21.48	14008.	4.64	11.39	.408

8	90.38	21.48	14709.	5.69	12.90	.441
9	94.38	21.48	14820.	6.58	14.14	.466
10	98.36	21.48	14296.	7.39	15.09	.490
11	99.37	21.48	14007.	7.59	15.26	.498
12	102.31	21.48	14851.	7.30	15.76	,463
13	106.20	21.48	15250.	7.00	16.14	.434
14	110.02	21.48	14964.	6.72	16.22	.414
15	113.75	21.48	14069.	6.42	16.02	.401
16	117.37	21.48	12669.	6.06	15.53	.390
17	120.87	21.48	10887.	5.62	14.75	.381
18	124.22	21.48	8862.	5.11	13.68	.373
19	127.42	21.48	6740.	4.50	12.34	.365
20	130.44	21.48	4669.	3.80	10.72	.355
21	133.27	21.48	2789.	3.01	8.84	.340
22	133.88	21.48	2384.	2.80	8.33	.336
23	135.90	21.48	1292.	2.04	6.70	.305
24	137.60	21.48	487.	1.21	5.02	.242
25	138.31	21.48	226.	.85	4.09	.208
26	139.03	.00	-2.	-.07	3.00	-.023

AVERAGE VALUES ALONG FAILURE SURFACE

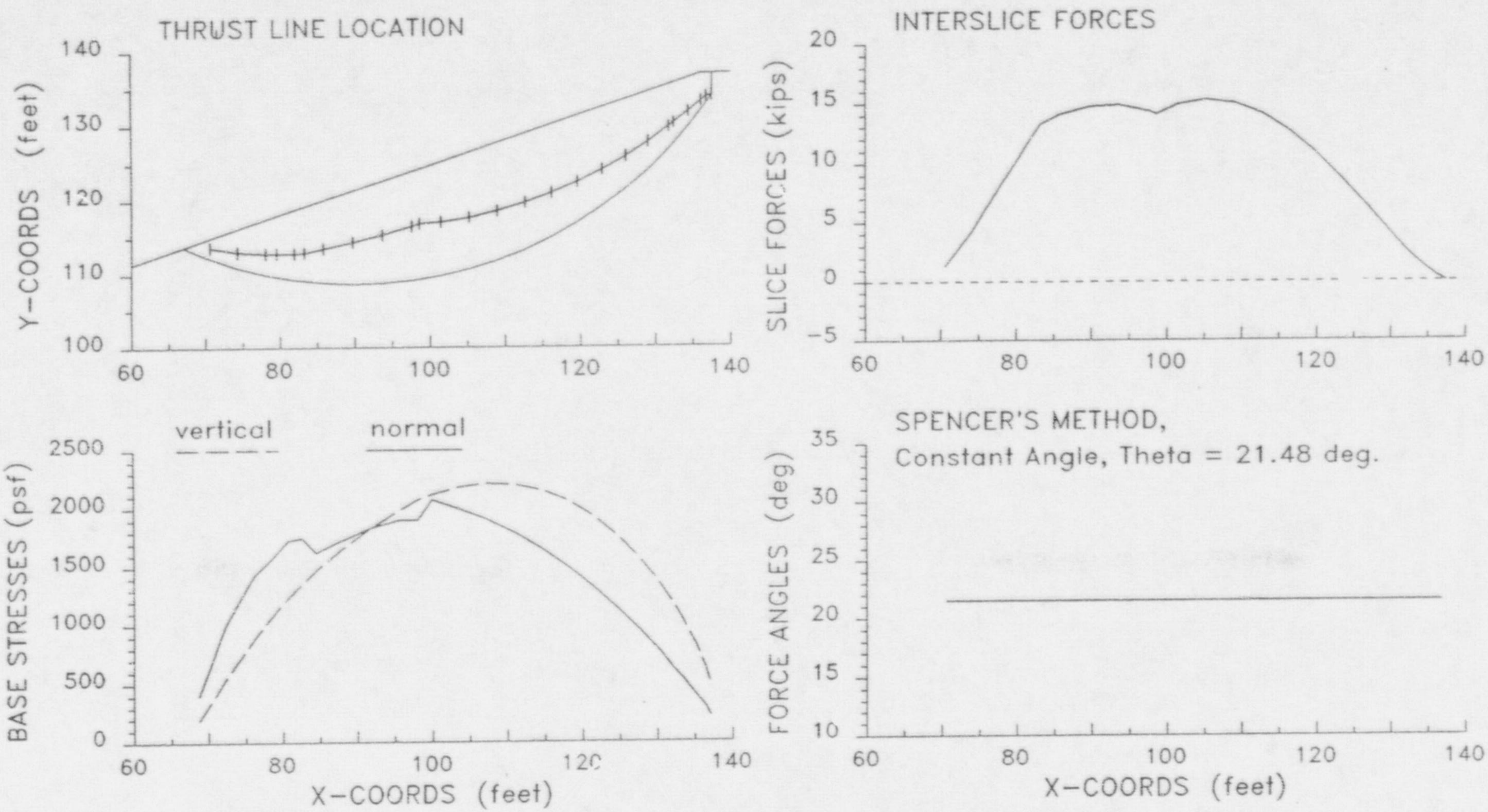
Total Normal Stress = 1361.38 (psf)
Pore Water Pressure = .00 (psf)
Shear Stress = 527.92 (psf)

Total Length of failure surface = 81.32 feet

For the single specified surface and the assumed angle of the interslice forces, the SPENCER'S (1973) procedure gives a

FACTOR OF SAFETY = 1.045

Total shear strength available
along specified failure surface = 448.56E+02 lb



BERT AVE. Dynamic Rotational
SPENCER'S METHOD, FOS for Specified Surface = 1.045

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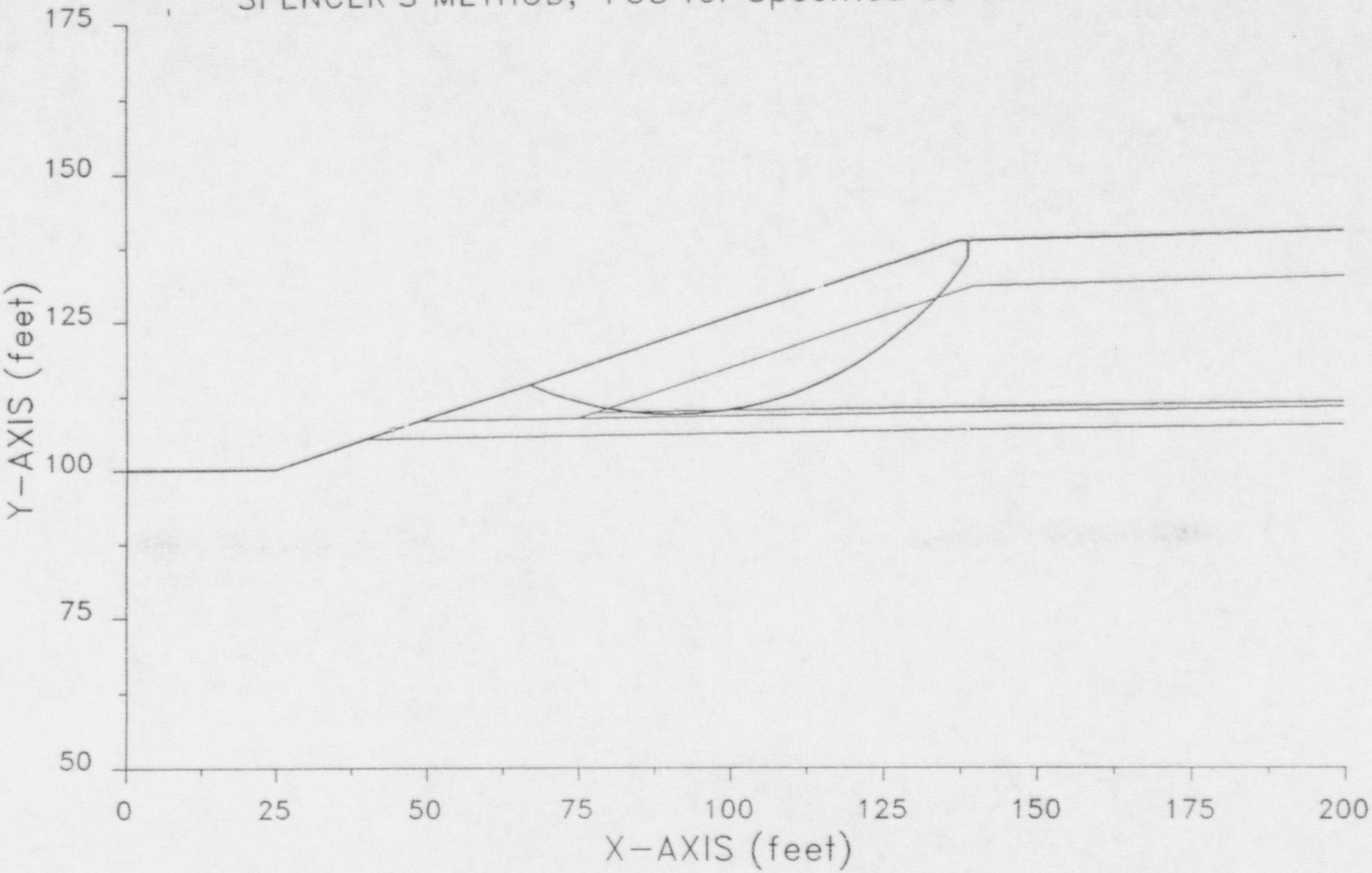
BERT AVE. Dynamic Rotational

SPENCER'S METHOD, FOS for Specified Surface = 1.045



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*                                       *
*       Slope Stability Analysis        *
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*           Method of Slices            *
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Problem Description : BERT AVE. Rotational Yield Co

SEGMENT BOUNDARY COORDINATES
----

5 SURFACE boundary segments

Segment No.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)	Soil Unit Below Segment
1	.0	100.0	25.0	100.0	1
2	25.0	100.0	40.0	105.0	1
3	40.0	105.0	49.0	108.0	2
4	49.0	108.0	137.6	137.5	3
5	137.6	137.5	200.0	138.7	3

7 SUBSURFACE boundary segments

Segment No.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)	Soil Unit Below Segment
1	49.0	108.0	75.0	108.3	2
2	75.0	108.3	78.0	109.3	5
3	78.0	109.3	140.1	130.0	4
4	140.1	130.0	200.0	131.2	4
5	78.0	109.3	200.0	110.5	5
6	75.0	108.3	200.0	109.6	2
7	40.0	105.0	200.0	106.6	1

A CRACKED ZONE HAS BEEN SPECIFIED

Depth of crack below ground surface = 3.00 (feet)

Maximum depth of water in crack = .00 (feet)
Unit weight of water in crack = 62.40 (pcf)

Failure surfaces will have a vertical side equal to the specified depth of crack and be affected by a hydrostatic force according to the specified depth of water in the crack .

ISOTROPIC Soil Parameters

---

5 Soil unit(s) specified

Soil Unit No.	Unit Weight Moist (pcf)	Sat. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Parameter Ru	Constant (psf)	Water Surface No.
1	130.0	130.0	.0	30.00	.000	.0	0
2	137.0	137.0	.0	27.00	.000	.0	0
3	137.0	137.0	.0	27.00	.000	.0	0
4	135.0	135.0	.0	25.00	.000	.0	0
5	110.0	110.0	.0	.00	.000	.0	0

NON-LINEAR MOHR-COULOMB envelope has been specified for 1 soil(s)

Soil Unit # 5

Point No.	Normal Stress (psf)	Shear Stress (psf)
1	.0	.0
2	150.0	100.0
3	300.0	150.0
4	500.0	200.0
5	3000.0	500.0
6	6000.0	800.0

A horizontal earthquake loading coefficient of .170 has been assigned

A vertical earthquake loading coefficient of .000 has been assigned

---

A SINGLE FAILURE SURFACE HAS BEEN SPECIFIED FOR ANALYSIS

Trial failure surface is CIRCULAR, with a radius of 57.89 feet

Center at x = 90.99 ; y = 166.76 ; Seg. Length = 4.00 feet

The CIRCULAR failure surface was estimated by the following 23 coordinate points :

Point No.	x-surf (ft)	y-surf (ft)
1	67.11	114.03
2	70.81	112.51
3	74.60	111.24
4	78.48	110.24
5	82.41	109.51
6	86.38	109.06
7	90.38	108.88
8	94.38	108.97
9	98.36	109.35
10	102.31	109.99
11	106.20	110.91
12	110.02	112.09
13	113.75	113.54
14	117.37	115.24
15	120.87	117.18
16	124.22	119.36
17	127.42	121.77
18	130.44	124.39
19	133.27	127.22
20	135.90	130.23
21	138.31	133.42
22	139.03	134.53
23	139.03	137.53

****

SELECTED METHOD OF ANALYSIS: Spencer (1973)

****

****

SUMMARY OF INDIVIDUAL SLICE INFORMATION

****

Slice	x-base (ft)	y-base (ft)	height (ft)	width (ft)	alpha	beta	weight (lb)
1	68.96	113.27	1.38	3.70	-22.38	18.42	698.
2	72.71	111.87	4.02	3.79	-18.42	18.42	2089.
3	76.54	110.74	6.43	3.87	-14.46	18.42	3410.
4	79.23	110.10	7.96	1.51	-10.50	18.42	1650.
5	81.20	109.74	8.98	2.42	-10.50	18.42	2976.
6	83.09	109.44	9.92	1.37	-6.54	18.42	1858.
7	85.08	109.21	10.81	2.60	-6.54	18.42	3831.
8	88.38	108.97	12.14	4.00	-2.58	18.42	6575.
9	92.38	108.93	13.52	4.00	1.38	18.42	7313.
10	96.37	109.16	14.61	3.98	5.34	18.42	7891.
11	98.86	109.43	15.17	1.00	9.30	18.42	2072.
12	100.84	109.75	15.51	2.94	9.30	18.42	6211.
13	104.25	110.45	15.95	3.89	13.26	18.42	8447.
14	108.11	111.50	16.18	3.82	17.22	18.42	8410.

15	111.89	112.82	16.12	3.73	21.18	18.42	8181.
16	115.56	114.39	15.78	3.62	25.14	18.42	7772.
17	119.12	116.21	15.14	3.50	29.10	18.42	7201.
18	122.55	118.27	14.21	3.35	33.06	18.42	6489.
19	125.82	120.57	13.01	3.19	37.02	18.42	5663.
20	128.93	123.08	11.53	3.02	40.98	18.42	4750.
21	131.85	125.81	9.78	2.83	44.94	18.42	3785.
22	133.58	127.57	8.59	.62	48.90	18.42	726.
23	134.89	129.08	7.52	2.01	48.90	18.42	2072.
24	136.75	131.36	5.86	1.70	52.86	18.42	1367.
25	137.96	132.95	4.55	.71	52.86	1.10	444.
26	138.67	133.98	3.55	.72	56.82	1.10	351.

Nonlinear M-C Iteration Number - 1

ITERATIONS FOR SPENCER'S METHOD

Iter #	Theta	FOS_force	FOS_moment
2	23.1835	1.0108	.9706
3	21.8511	-----	1.0108
3	22.5173	1.0050	-----
4	22.0338	1.0009	1.0050
5	22.0992	1.0015	1.0009

ITERATIONS FOR SPENCER'S METHOD

Iter #	Theta	FOS_force	FOS_moment
1	22.0992	1.0015	1.0009

SLICE INFORMATION ... continued :

Slice	Sigma (psf)	c-value (psf)	phi	U-base (lb)	U-top (lb)	P-top (lb)	Delta
1	421.6	.0	27.00	0.	0.	0.	.00
2	1048.8	.0	27.00	0.	0.	0.	.00
3	1470.5	.0	27.00	0.	0.	0.	.00
4	1628.2	.0	27.00	0.	0.	0.	.00
5	1762.6	.0	25.00	0.	0.	0.	.00
6	1775.0	.0	25.00	0.	0.	0.	.00
7	1619.2	140.0	6.84	0.	0.	0.	.00
8	1719.3	140.0	6.84	0.	0.	0.	.00
9	1821.5	140.0	6.84	0.	0.	0.	.00
10	1887.4	140.0	6.84	0.	0.	0.	.00
11	1884.2	140.0	6.84	0.	0.	0.	.00
12	2060.2	.0	25.00	0.	0.	0.	.00
13	1987.4	.0	25.00	0.	0.	0.	.00
14	1895.6	.0	25.00	0.	0.	0.	.00
15	1777.8	.0	25.00	0.	0.	0.	.00
16	1638.0	.0	25.00	0.	0.	0.	.00
17	1480.0	.0	25.00	0.	0.	0.	.00
18	1307.5	.0	25.00	0.	0.	0.	.00
19	1124.2	.0	25.00	0.	0.	0.	.00

20	933.9	.0	25.00	0.	0.	0.	.00
21	740.5	.0	25.00	0.	0.	0.	.00
22	604.8	.0	25.00	0.	0.	0.	.00
23	520.5	.0	27.00	0.	0.	0.	.00
24	373.4	.0	27.00	0.	0.	0.	.00
25	290.3	.0	27.00	0.	0.	0.	.00
26	207.2	.0	27.00	0.	0.	0.	.00

SPENCER'S (1973) - TOTAL Stresses at center of slice base

Slice #	Base x-coord (ft)	Normal Stress (psf)	Vertical Stress (psf)	Pore Water Pressure (psf)	Shear Stress (psf)
1	68.96	421.6	188.7	.0	214.5
2	72.71	1048.8	550.5	.0	533.6
3	76.54	1470.5	880.5	.0	748.1
4	79.23	1628.2	1090.9	.0	828.4
5	81.20	1762.6	1229.3	.0	820.7
6	83.09	1775.0	1355.4	.0	826.5
7	85.08	1619.2	1471.5	.0	333.8
8	88.38	1719.3	1645.3	.0	345.8
9	92.38	1821.5	1828.7	.0	358.0
10	96.37	1887.4	1981.3	.0	365.9
11	98.86	1884.2	2063.3	.0	365.6
12	100.84	2060.2	2110.3	.0	959.3
13	104.25	1987.4	2169.5	.0	925.4
14	108.11	1895.6	2201.1	.0	882.6
15	111.89	1777.8	2193.3	.0	827.8
16	115.56	1638.0	2146.3	.0	762.7
17	119.12	1480.0	2060.3	.0	689.1
18	122.55	1307.5	1935.6	.0	608.8
19	125.82	1124.2	1772.9	.0	523.5
20	128.93	933.9	1573.0	.0	434.9
21	131.85	740.5	1336.7	.0	344.8
22	133.58	604.8	1175.8	.0	281.6
23	134.89	520.5	1029.8	.0	264.8
24	136.75	373.4	802.5	.0	190.0
25	137.96	290.3	623.9	.0	147.7
26	138.67	207.2	485.7	.0	105.4

SPENCER'S (1973) - Magnitude & Location of Interslice Forces

Slice #	Right x-coord (ft)	Force Angle (degrees)	Interslice Force (lb)	Force Height (ft)	Boundary Height (ft)	Height Ratio
1	70.81	22.10	1421.	1.45	2.75	.527
2	74.60	22.10	4655.	2.11	5.28	.400
3	78.48	22.10	8742.	2.87	7.57	.379
4	79.99	22.10	10284.	3.15	8.35	.377
5	82.41	22.10	12736.	3.65	9.61	.379
6	83.78	22.10	13919.	3.90	10.22	.381
7	86.38	22.10	14676.	4.76	11.39	.418

8	90.38	22.10	15296.	5.86	12.90	.454
9	94.38	22.10	15311.	6.78	14.14	.480
10	98.36	22.10	14678.	7.62	15.09	.505
11	99.37	22.10	14360.	7.84	15.26	.514
12	102.31	22.10	15196.	7.52	15.76	.477
13	106.20	22.10	15568.	7.20	16.14	.446
14	110.02	22.10	15242.	6.91	16.22	.426
15	113.75	22.10	14301.	6.59	16.02	.411
16	117.37	22.10	12852.	6.21	15.53	.400
17	120.87	22.10	11024.	5.77	14.75	.391
18	124.22	22.10	8957.	5.24	13.68	.383
19	127.42	22.10	6801.	4.62	12.34	.375
20	130.44	22.10	4702.	3.91	10.72	.364
21	133.27	22.10	2804.	3.10	8.84	.350
22	133.88	22.10	2396.	2.89	8.33	.347
23	135.90	22.10	1296.	2.12	6.70	.316
24	137.60	22.10	488.	1.28	5.02	.254
25	138.31	22.10	225.	.91	4.09	.221
26	139.03	.00	-4.	.10	3.00	.033

AVERAGE VALUES ALONG FAILURE SURFACE

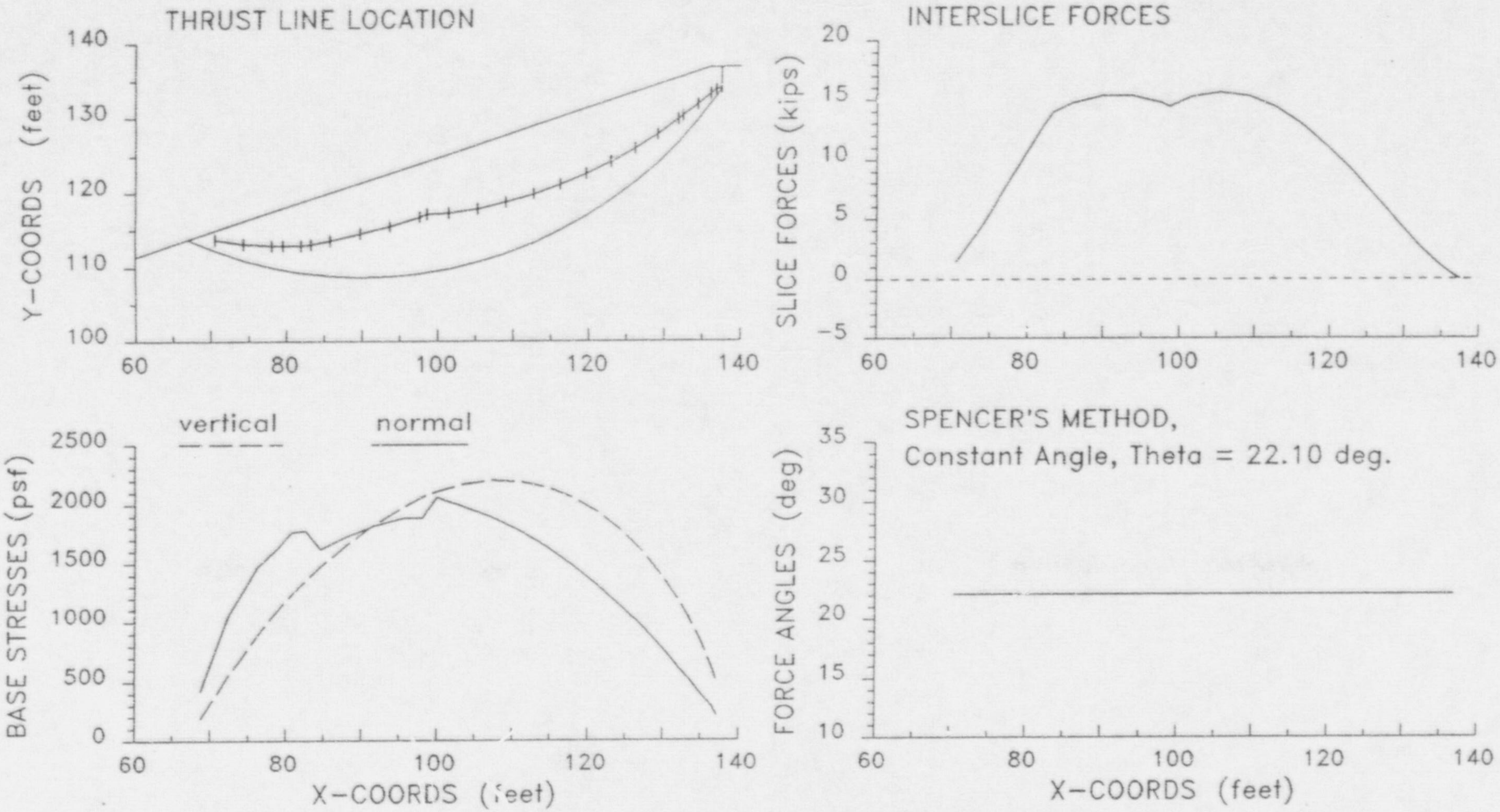
Total Normal Stress = 1358.13 (psf)
Pore Water Pressure = .00 (psf)
Shear Stress = 550.03 (psf)

Total Length of failure surface = 81.32 feet

For the single specified surface and the assumed angle of the interslice forces, the SPENCER'S (1973) procedure gives a

FACTOR OF SAFETY = 1.001

Total shear strength available
along specified failure surface = 447.94E+02 lb



BERT AVE. Rotational Yield Co
SPENCER'S METHOD, FOS for Specified Surface = 1.001

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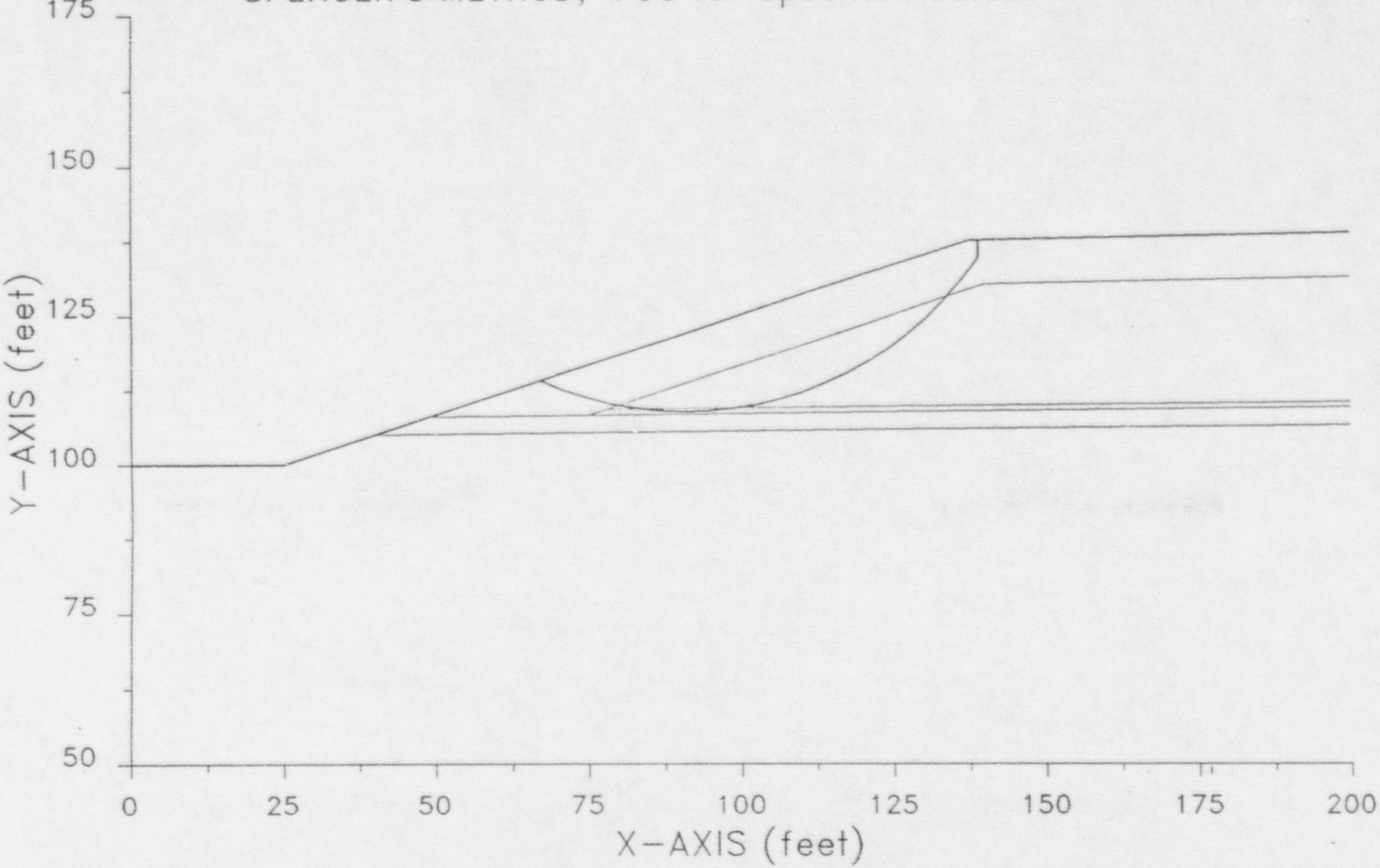
BERT AVE. Rotational Yield Co

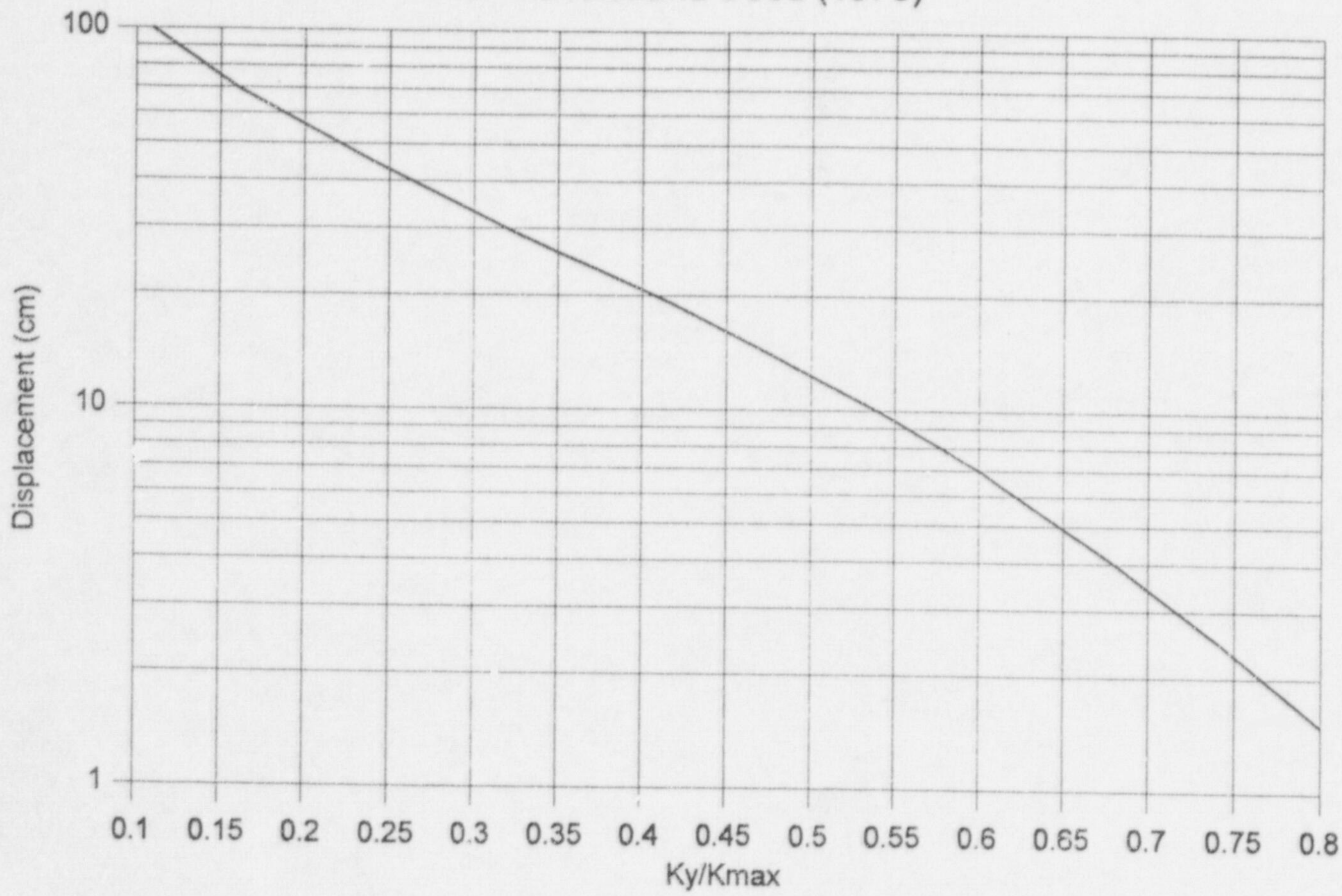
SPENCER'S METHOD, FOS for Specified Surface = 1.001



The FOS (1.05) for deep-seated rotational failures through the waste is less than the regulatory minimum recommended value of 1.3. However, a simplified Newmark deformation analysis indicates that no deformation will occur due to the design earthquake.

Earthquake-Induced Deformations (6.5 M)

After Makdisi and Seed (1978)



- The yield acceleration for this failure scenario is calculated to be 0.17g.
- The maximum seismic acceleration has been estimated to be 0.16g.

Since the maximum acceleration for the design earthquake is less than the acceleration required to induce displacement, no deformation should occur in the event of an earthquake equal to or less than the magnitude of the design earthquake.

- XSTABL File: SPECNC 5-18-98 9:31

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*****************************************
*              X S T A B L              *
*                                       *
*       Slope Stability Analysis        *
*             using the                 *
*          Method of Slices             *
*                                       *
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Problem Description : BERT AVE. Static Translational

SEGMENT BOUNDARY COORDINATES

5 SURFACE boundary segments

Segment No.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)	Soil Unit Below Segment
1	.0	100.0	25.0	100.0	1
2	25.0	100.0	40.0	105.0	1
3	40.0	105.0	49.0	108.0	2
4	49.0	108.0	137.6	137.5	3
5	137.6	137.5	200.0	138.7	3

7 SUBSURFACE boundary segments

Segment No.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)	Soil Unit Below Segment
1	49.0	108.0	75.0	108.3	2
2	75.0	108.3	78.0	109.3	5
3	78.0	109.3	140.1	130.0	4
4	140.1	130.0	200.0	131.2	4
5	78.0	109.3	200.0	110.5	5
6	75.0	108.3	200.0	109.6	2
7	40.0	105.0	200.0	106.6	1

A CRACKED ZONE HAS BEEN SPECIFIED

Depth of crack below ground surface = 3.00 (feet)

Maximum depth of water in crack = .00 (feet)
Unit weight of water in crack = 62.40 (pcf)

Failure surfaces will have a vertical side equal to the specified depth of crack and be affected by a hydrostatic force according to the specified depth of water in the crack

---

ISOTROPIC Soil Parameters

---

5 Soil unit(s) specified

Soil Unit No.	Unit Weight Moist (pcf)	Unit Weight Sat. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Parameter Ru	Pore Pressure Constant (psf)	Water Surface No.
1	130.0	130.0	.0	30.00	.000	.0	0
2	137.0	137.0	.0	27.00	.000	.0	0
3	137.0	137.0	.0	27.00	.000	.0	0
4	135.0	135.0	.0	25.00	.000	.0	0
5	110.0	110.0	.0	.00	.000	.0	0

NON-LINEAR MOHR-COULOMB envelope has been specified for 1 soil(s)

Soil Unit # 5

Point No.	Normal Stress (psf)	Shear Stress (psf)
1	.0	.0
2	150.0	100.0
3	300.0	150.0
4	500.0	200.0
5	3000.0	500.0
6	6000.0	800.0

A SINGLE FAILURE SURFACE HAS BEEN SPECIFIED FOR ANALYSIS

Trial failure surface specified by the following 11 coordinate points :

Point No.	x-surf (ft)	y-surf (ft)
1	68.04	114.34
2	68.35	114.15
3	76.88	108.93
4	77.00	108.80
5	125.43	109.27
6	125.93	109.77

7	131.30	118.21
8	136.68	126.64
9	138.47	129.46
10	141.61	134.58
11	141.61	137.58

****
SELECTED METHOD OF ANALYSIS: Spencer (1973)
****

****
SUMMARY OF INDIVIDUAL SLICE INFORMATION
****

Slice	x-base (ft)	y-base (ft)	height (ft)	width (ft)	alpha	beta	weight (lb)
1	68.19	114.24	.15	.31	-31.44	18.42	6.
2	72.62	111.54	4.32	8.53	-31.46	18.42	5054.
3	76.94	108.86	8.44	.12	-47.29	18.42	136.
4	77.50	108.80	8.68	1.00	.56	18.42	1181.
5	101.71	109.04	16.51	47.43	.56	18.42	105934.
6	125.68	109.52	24.01	.50	45.00	18.42	1629.
7	128.62	113.99	20.52	5.37	57.53	18.42	14961.
8	133.99	122.42	13.87	5.38	57.45	18.42	10166.
9	137.14	127.36	9.98	.92	57.59	18.42	1255.
10	138.04	128.77	8.73	.87	57.59	1.10	1040.
11	140.04	132.02	5.53	3.14	58.48	1.10	2377.

Nonlinear M-C Iteration Number - 1

ITERATIONS FOR SPENCER'S METHOD

Iter #	Theta	FOS_force	FOS_moment
2	11.4295	1.4047	1.5744
3	11.9726	1.4179	1.4047
4	11.9497	1.4173	1.4179

ITERATIONS FOR SPENCER'S METHOD

Iter #	Theta	FOS_force	FOS_moment
1	11.9497	1.4173	1.4179

SLICE INFORMATION ... continued :

Slice	Sigma (psf)	c-value (psf)	phi	U-base (lb)	U-top (lb)	P-top (lb)	Delta

1	34.9	.0	27.00	0.	0.	0.	.00
2	1031.3	.0	27.00	0.	0.	0.	.00
3	1939.0	140.0	6.84	0.	0.	0.	.00
4	1219.2	140.0	6.84	0.	0.	0.	.00
5	2287.8	140.0	6.84	0.	0.	0.	.00
6	2482.9	140.0	6.84	0.	0.	0.	.00
7	1565.4	.0	25.00	0.	0.	0.	.00
8	1063.0	.0	25.00	0.	0.	0.	.00
9	765.5	.0	25.00	0.	0.	0.	.00
10	670.8	.0	25.00	0.	0.	0.	.00
11	408.3	.0	27.00	0.	0.	0.	.00

SPENCER'S (1973) - TOTAL Stresses at center of slice base

Slice #	Base x-coord (ft)	Normal Stress (psf)	Vertical Stress (psf)	Pore Water Pressure (psf)	Shear Stress (psf)
1	68.19	34.9	20.1	.0	12.5
2	72.62	1031.3	592.4	.0	370.7
3	76.94	1939.0	1154.0	.0	262.9
4	77.50	1219.2	1180.9	.0	202.0
5	101.71	2287.8	2233.5	.0	292.5
6	125.68	2482.9	3251.9	.0	309.0
7	128.62	1565.4	2786.6	.0	515.0
8	133.99	1063.0	1889.5	.0	349.7
9	137.14	765.5	1364.3	.0	251.9
10	138.04	670.8	1195.4	.0	220.7
11	140.04	408.3	757.4	.0	146.8

SPENCER'S (1973) - Magnitude & Location of Interslice Forces

Slice #	Right x-coord (ft)	Force Angle (degrees)	Interslice Force (lb)	Force Height (ft)	Boundary Height (ft)	Height Ratio
1	68.35	11.95	11.	.13	.29	.436
2	76.88	11.95	8748.	3.52	8.36	.421
3	77.00	11.95	9032.	3.56	8.52	.417
4	78.00	11.95	9227.	3.68	8.85	.416
5	125.43	11.95	22330.	8.23	24.18	.342
6	125.93	11.95	21217.	8.30	23.84	.348
7	131.30	11.95	10541.	5.71	17.19	.332
8	136.68	11.95	3304.	2.94	10.55	.278
9	137.60	11.95	2407.	2.54	9.41	.270
10	138.47	11.95	1664.	2.23	8.06	.276
11	141.61	.00	-1.	-.04	3.00	-.012

AVERAGE VALUES ALONG FAILURE SURFACE

Total Normal Stress = 1722.00 (psf)

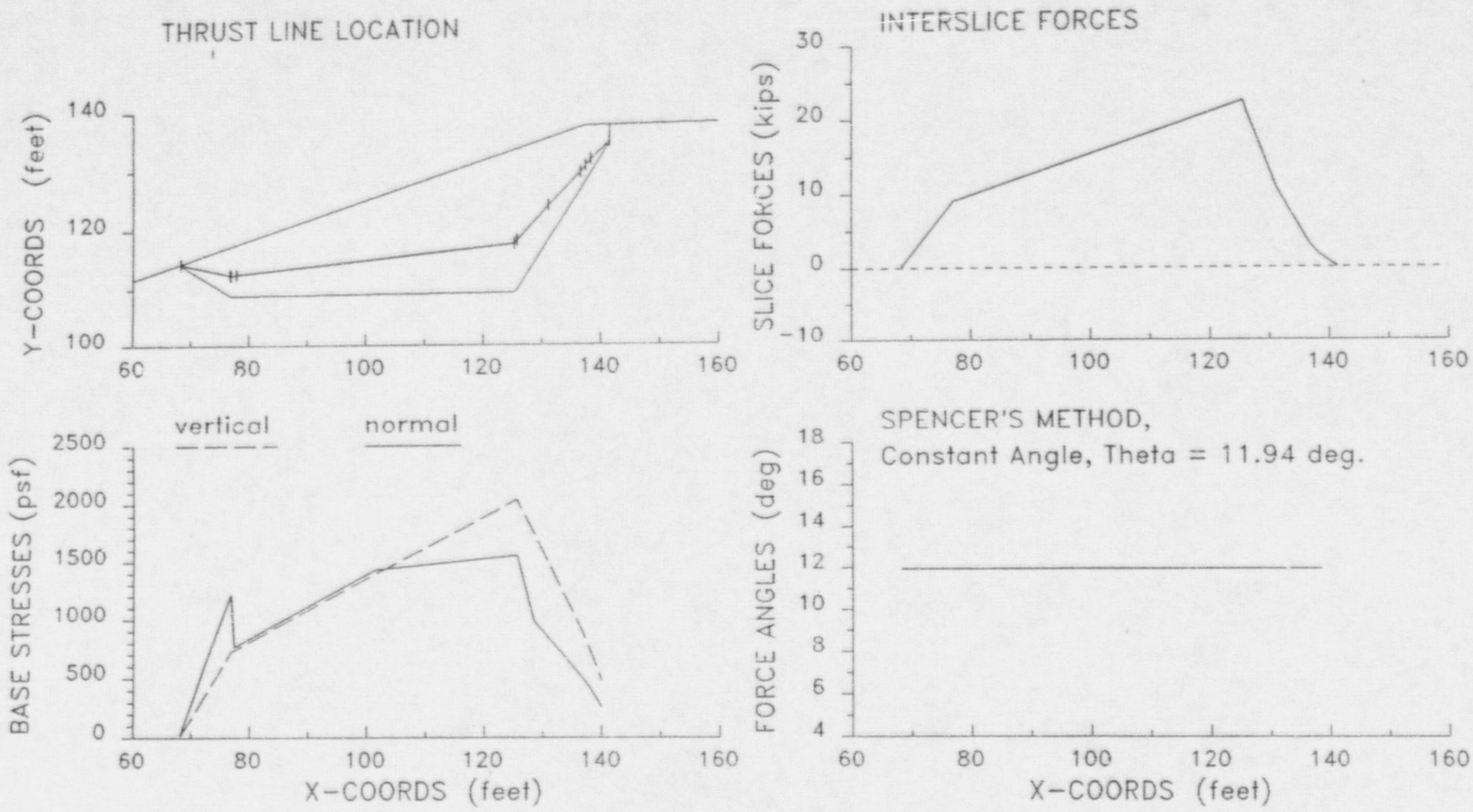
Pore Water Pressure = .00 (psf)
Shear Stress = 318.71 (psf)

Total Length of failure surface = 89.03 feet

For the single specified surface and the assumed angle of the interslice forces, the SPENCER'S (1973) procedure gives a

FACTOR OF SAFETY = 1.417

Total shear strength available
along specified failure surface = 402.14E+02 lb



BERT AVE. Static Translational
SPENCER'S METHOD, FOS for Specified Surface = 1.417

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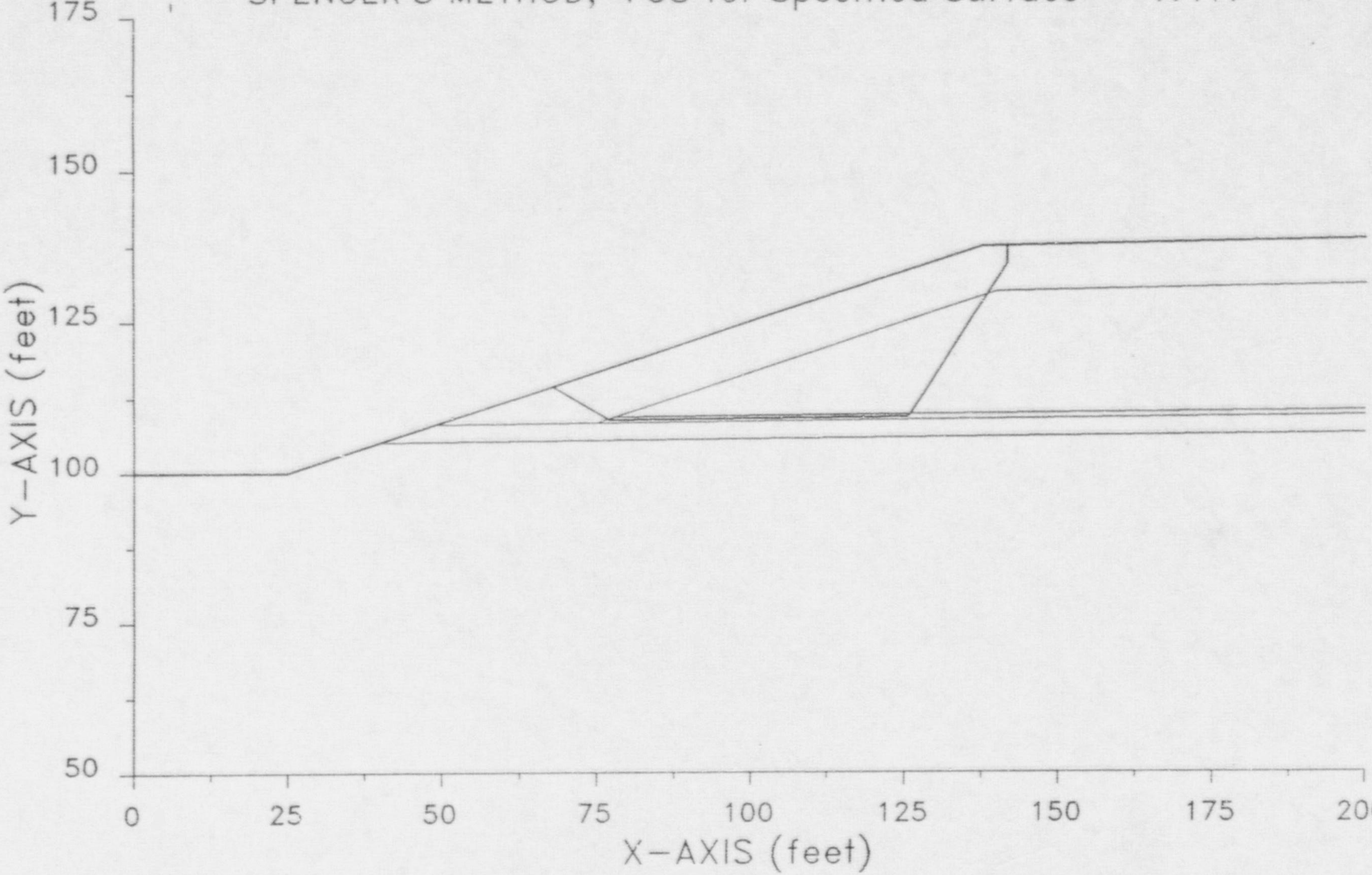
BERT AVE. Static Translational

SPENCER'S METHOD, FOS for Specified Surface = 1.417



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******************************************
*              X S T A B L               *
*                                        *
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*               using the                *
*            Method of Slices            *
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Problem Description : BERT AVE. Dynamic Translational

SEGMENT BOUNDARY COORDINATES

---

5 SURFACE boundary segments

Segment No.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)	Soil Unit Below Segment
1	.0	100.0	25.0	100.0	1
2	25.0	100.0	40.0	105.0	1
3	40.0	105.0	49.0	108.0	2
4	49.0	108.0	137.6	137.5	3
5	137.6	137.5	200.0	138.7	3

7 SUBSURFACE boundary segments

Segment No.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)	Soil Unit Below Segment
1	49.0	108.0	75.0	108.3	2
2	75.0	108.3	78.0	109.3	5
3	78.0	109.3	140.1	130.0	4
4	140.1	130.0	200.0	131.2	4
5	78.0	109.3	200.0	110.5	5
6	75.0	108.3	200.0	109.6	2
7	40.0	105.0	200.0	106.6	1

A CRACKED ZONE HAS BEEN SPECIFIED

Depth of crack below ground surface = 3.00   (feet)

Maximum depth of water in crack = .00 (feet)
Unit weight of water in crack = 62.40 (pcf)

Failure surfaces will have a vertical side equal to the specified depth of crack and be affected by a hydrostatic force according to the specified depth of water in the crack

---

ISOTROPIC Soil Parameters

---

5 Soil unit(s) specified

Soil Unit No.	Unit Weight Moist (pcf)	Unit Weight Sat. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Parameter Ru	Pore Pressure Constant (psf)	Water Surface No.
1	130.0	130.0	.0	30.00	.000	.0	0
2	137.0	137.0	.0	27.00	.000	.0	0
3	137.0	137.0	.0	27.00	.000	.0	0
4	135.0	135.0	.0	25.00	.000	.0	0
5	110.0	110.0	.0	.00	.000	.0	0

NON-LINEAR MOHR-COULOMB envelope has been specified for 1 soil(s)

Soil Unit # 5

Point No.	Normal Stress (psf)	Shear Stress (psf)
1	.0	.0
2	150.0	100.0
3	300.0	150.0
4	500.0	200.0
5	3000.0	500.0
6	6000.0	800.0

A horizontal earthquake loading coefficient of .150 has been assigned

A vertical earthquake loading coefficient of .000 has been assigned

---

A SINGLE FAILURE SURFACE HAS BEEN SPECIFIED FOR ANALYSIS

Trial failure surface specified by the following 11 coordinate points :

Point x-surf y-surf

No.	(ft)	(ft)
1	68.04	114.34
2	68.35	114.15
3	76.88	108.93
4	77.00	108.80
5	125.43	109.27
6	125.93	109.77
7	131.30	118.21
8	136.68	126.64
9	138.47	129.46
10	141.61	134.58
11	141.61	137.58

****
SELECTED METHOD OF ANALYSIS: Spencer (1973)
****

****
SUMMARY OF INDIVIDUAL SLICE INFORMATION
****

Slice	x-base (ft)	y-base (ft)	height (ft)	width (ft)	alpha	beta	weight (lb)
1	68.19	114.24	.15	.31	-31.44	18.42	6.
2	72.62	111.54	4.32	8.53	-31.46	18.42	5054.
3	76.94	108.86	8.44	.12	-47.29	18.42	136.
4	77.50	108.80	8.68	1.00	.56	18.42	1181.
5	101.71	109.04	16.51	47.43	.56	18.42	105934.
6	125.68	109.52	24.01	.50	45.00	18.42	1629.
7	128.62	113.99	20.52	5.37	57.53	18.42	14961.
8	133.99	122.42	13.87	5.38	57.45	18.42	10166.
9	137.14	127.36	9.98	.92	57.59	18.42	1255.
10	138.04	128.77	8.73	.87	57.59	1.10	1040.
11	140.04	132.02	5.53	3.14	58.48	1.10	2377.

Nonlinear M-C Iteration Number - 1

ITERATIONS FOR SPENCER'S METHOD

Iter #	Theta	FOS_force	FOS_moment
2	16.6972	.9503	.9662
3	16.9370	.9548	.9503
4	16.9024	.9542	.9548

ITERATIONS FOR SPENCER'S METHOD

Iter #	Theta	FOS_force	FOS_moment

1	16.9024	.9542	.9548

SLICE INFORMATION ... continued :

Slice	Sigma (psf)	c-value (psf)	phi	U-base (lb)	U-top (lb)	P-top (lb)	Delta
1	58.8	.0	27.00	0.	0.	0.	.00
2	1739.4	.0	27.00	0.	0.	0.	.00
3	2628.8	140.0	6.84	0.	0.	-0.	.00
4	1211.5	140.0	6.84	0.	0.	0.	.00
5	2251.5	140.0	6.84	0.	0.	0.	.00
6	2157.2	140.0	6.84	0.	0.	0.	.00
7	1268.2	.0	25.00	0.	0.	0.	.00
8	861.5	.0	25.00	0.	0.	0.	.00
9	620.0	.0	25.00	0.	0.	0.	.00
10	543.3	.0	25.00	0.	0.	0.	.00
11	328.4	.0	27.00	0.	0.	0.	.00

SPENCER'S (1973) - TOTAL Stresses at center of slice base

Slice #	Base x-coord (ft)	Normal Stress (psf)	Vertical Stress (psf)	Pore Water Pressure (psf)	Shear Stress (psf)
1	68.19	58.8	20.1	.0	31.4
2	72.62	1739.4	592.4	.0	928.8
3	76.94	2628.8	1154.0	.0	477.3
4	77.50	1211.5	1180.9	.0	299.1
5	101.71	2251.5	2233.5	.0	429.8
6	125.68	2157.2	3251.9	.0	418.0
7	128.62	1268.2	2786.6	.0	619.7
8	133.99	861.5	1889.5	.0	421.0
9	137.14	620.0	1364.3	.0	303.0
10	138.04	543.3	1195.4	.0	265.5
11	140.04	328.4	757.4	.0	175.4

SPENCER'S (1973) - Magnitude & Location of Interslice Forces

Slice #	Right x-coord (ft)	Force Angle (degrees)	Interslice Force (lb)	Force Height (ft)	Boundary Height (ft)	Height Ratio
1	68.35	16.90	21.	.14	.29	.473
2	76.88	16.90	17003.	3.81	8.36	.456
3	77.00	16.90	17391.	3.88	8.52	.455
4	78.00	16.90	17506.	4.10	8.85	.464
5	125.43	16.90	21124.	9.59	24.18	.397
6	125.93	16.90	19958.	9.64	23.84	.404
7	131.30	16.90	9905.	6.71	17.19	.390
8	136.68	16.90	3088.	3.61	10.55	.342
9	137.60	16.90	2243.	3.14	9.41	.334
10	138.47	16.90	1544.	2.75	8.06	.341

11	141.61	.00	-10.	-.20	3.00	-.067

AVERAGE VALUES ALONG FAILURE SURFACE

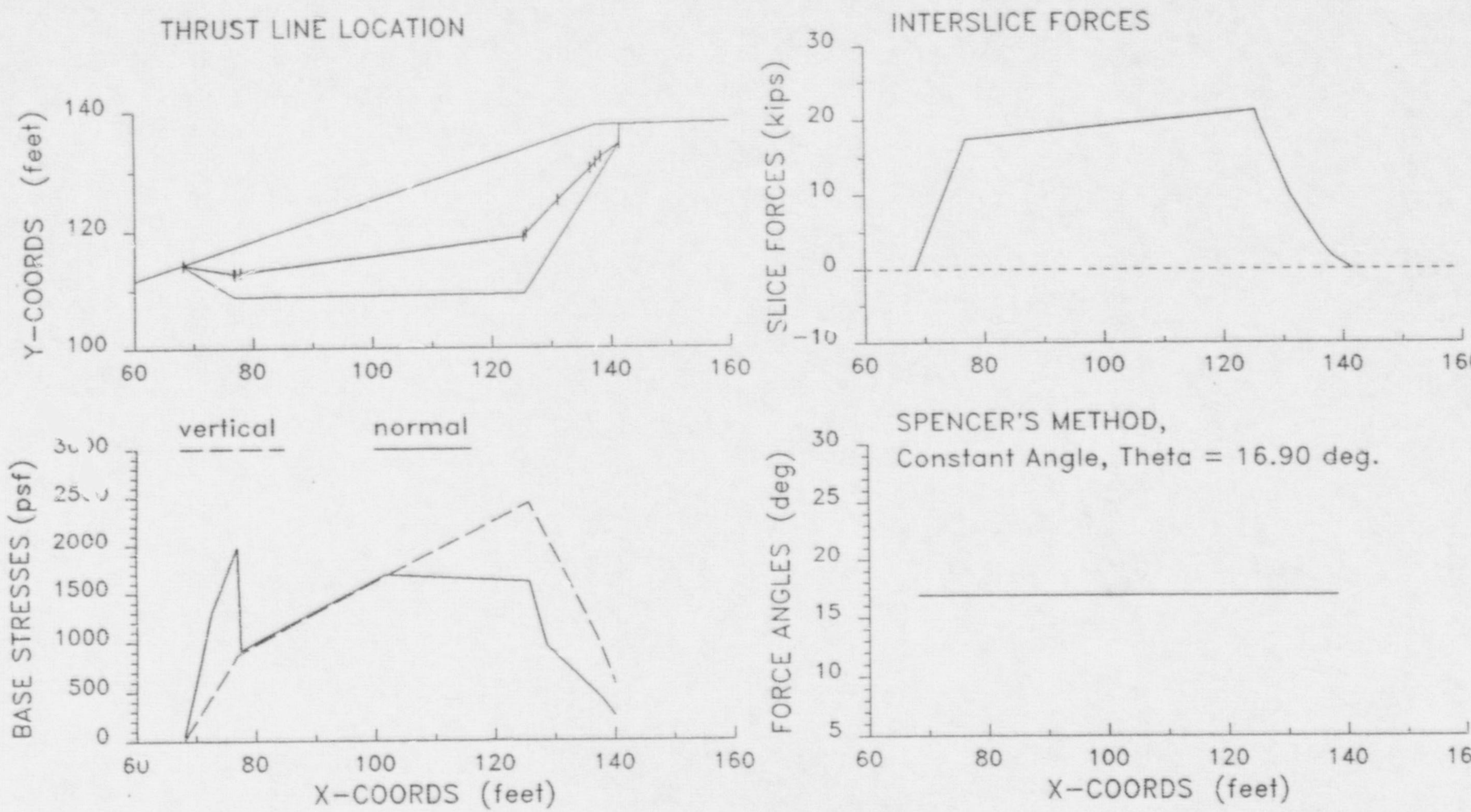
Total Normal Stress = 1714.41 (psf)
Pore Water Pressure = .00 (psf)
Shear Stress = 480.55 (psf)

Total Length of failure surface = 89.03 feet

For the single specified surface and the assumed angle of the interslice forces, the SPENCER'S (1973) procedure gives a

FACTOR OF SAFETY = .954

Total shear strength available along specified failure surface = 408.23E+02 lb



BERT AVE. Dynamic Translational
SPENCER'S METHOD, FOS for Specified Surface = .954

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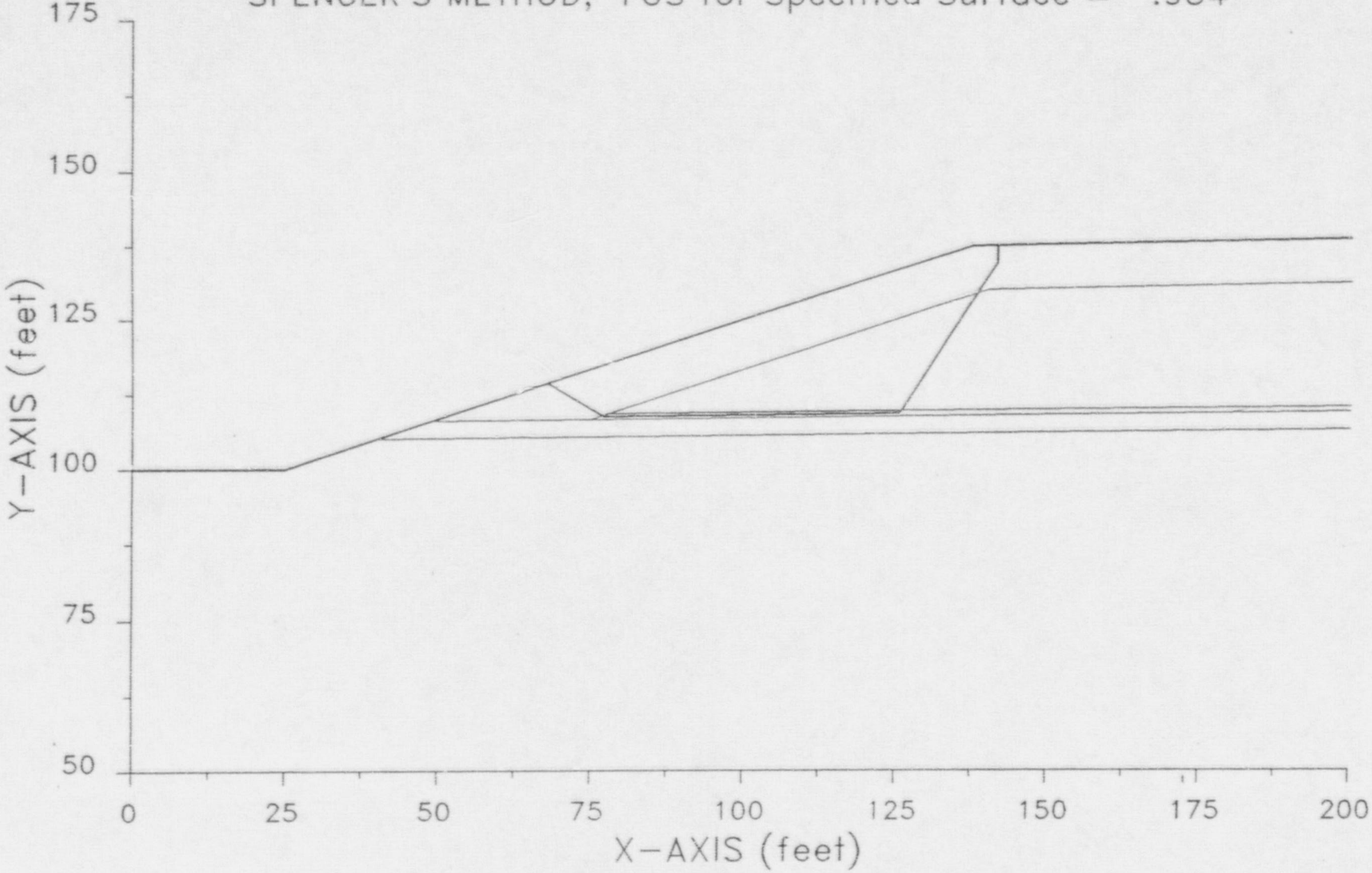
BERT AVE. Dynamic Translational

SPENCER'S METHOD, FOS for Specified Surface = .954



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****************************************
*              X S T A B L             *
*                                      *
*       Slope Stability Analysis       *
*              using the               *
*           Method of Slices           *
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Problem Description : BERT AVE. Translational Yield Co.

SEGMENT BOUNDARY COORDINATES

5 SURFACE boundary segments

Segment No.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)	Soil Unit Below Segment
1	.0	100.0	25.0	100.0	1
2	25.0	100.0	40.0	105.0	1
3	40.0	105.0	49.0	108.0	2
4	49.0	108.0	137.6	137.5	3
5	137.6	137.5	200.0	138.7	3

7 SUBSURFACE boundary segments

Segment No.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)	Soil Unit Below Segment
1	49.0	108.0	75.0	108.3	2
2	75.0	108.3	78.0	109.3	5
3	78.0	109.3	140.1	130.0	4
4	140.1	130.0	200.0	131.2	4
5	78.0	109.3	200.0	110.5	5
6	75.0	108.3	200.0	109.6	2
7	40.0	105.0	200.0	106.6	1

A CRACKED ZONE HAS BEEN SPECIFIED

Depth of crack below ground surface = 3.00 (feet)

Maximum depth of water in crack = .00 (feet)
Unit weight of water in crack = 62.40 (pcf)

Failure surfaces will have a vertical side equal to the specified depth of crack and be affected by a hydrostatic force according to the specified depth of water in the crack

---

ISOTROPIC Soil Parameters

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5 Soil unit(s) specified

Soil Unit No.	Unit Weight Moist (pcf)	Unit Weight Sat. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Parameter Ru	Pore Pressure Constant (psf)	Water Surface No.
1	130.0	130.0	.0	30.00	.000	.0	0
2	137.0	137.0	.0	27.00	.000	.0	0
3	137.0	137.0	.0	27.00	.000	.0	0
4	135.0	135.0	.0	25.00	.000	.0	0
5	110.0	110.0	.0	.00	.000	.0	0

NON-LINEAR MOHR-COULOMB envelope has been specified for 1 soil(s)

Soil Unit # 5

Point No.	Normal Stress (psf)	Shear Stress (psf)
1	.0	.0
2	150.0	100.0
3	300.0	150.0
4	500.0	200.0
5	3000.0	500.0
6	6000.0	800.0

A horizontal earthquake loading coefficient of .125 has been assigned

A vertical earthquake loading coefficient of .000 has been assigned

---

A SINGLE FAILURE SURFACE HAS BEEN SPECIFIED FOR ANALYSIS

Trial failure surface specified by the following 11 coordinate points :

Point x-surf y-surf

No.	(ft)	(ft)
1	68.04	114.34
2	68.35	114.15
3	76.88	108.92
4	77.00	108.80
5	125.43	109.27
6	125.93	109.77
7	131.30	118.21
8	136.68	126.64
9	138.47	129.46
10	141.61	134.58
11	141.61	137.58

****
SELECTED METHOD OF ANALYSIS: Spencer (1973)
****

****
SUMMARY OF INDIVIDUAL SLICE INFORMATION
****

Slice	x-base (ft)	y-base (ft)	height (ft)	width (ft)	alpha	beta	weight (lb)
1	68.19	114.24	.15	.31	-31.44	18.42	6.
2	72.62	111.54	4.32	8.53	-31.46	18.42	5054.
3	76.94	108.86	8.44	.12	-47.29	18.42	136.
4	77.50	108.80	8.68	1.00	.56	18.42	1181.
5	101.71	109.04	16.51	47.43	.56	18.42	105934.
6	125.68	109.52	24.01	.50	45.00	18.42	1629.
7	128.62	113.99	20.52	5.37	57.53	18.42	14961.
8	133.99	122.42	13.87	5.38	57.45	18.42	10166.
9	137.14	127.36	9.98	.92	57.59	18.42	1255.
10	138.04	128.77	8.73	.87	57.59	1.10	1040.
11	140.04	132.02	5.53	3.14	58.48	1.10	2377.

Nonlinear M-C Iteration Number - 1

ITERATIONS FOR SPENCER'S METHOD

Iter #	Theta	FOS_force	FOS_moment
2	16.1150	.9974	1.0247
3	16.4692	-----	.9974
3	16.2921	1.0007	-----
4	16.4251	1.0032	1.0007
5	16.4085	1.0029	1.0032

ITERATIONS FOR SPENCER'S METHOD

Iter #	Theta	FOS_force	FOS_moment
1	16.4085	1.0029	1.0032

SLICE INFORMATION ... continued :

Slice	Sigma (psf)	c-value (psf)	phi	U-base (lb)	U-top (lb)	P-top (lb)	Delta
1	53.7	.0	27.00	0.	0.	0.	.00
2	1588.1	.0	27.00	0.	0.	0.	.00
3	2526.1	140.0	6.84	0.	0.	0.	.00
4	1215.1	140.0	6.84	0.	0.	0.	.00
5	2261.6	140.0	6.84	0.	0.	0.	.00
6	2200.1	140.0	6.84	0.	0.	0.	.00
7	1305.0	.0	25.00	0.	0.	0.	.00
8	886.5	.0	25.00	0.	0.	0.	.00
9	638.0	.0	25.00	0.	0.	0.	.00
10	559.1	.0	25.00	0.	0.	0.	.00
11	338.0	.0	27.00	0.	0.	0.	.00

SPENCER'S (1973) - TOTAL Stresses at center of slice base

Slice #	Base x-coord (ft)	Normal Stress (psf)	Vertical Stress (psf)	Pore Water Pressure (psf)	Shear Stress (psf)
1	68.19	53.7	20.1	.0	27.3
2	72.62	1588.1	592.4	.0	806.8
3	76.94	2526.1	1154.0	.0	441.8
4	77.50	1215.1	1180.9	.0	285.0
5	101.71	2261.6	2233.5	.0	410.2
6	125.68	2200.1	3251.9	.0	402.8
7	128.62	1305.0	2786.6	.0	606.8
8	133.99	886.5	1889.5	.0	412.2
9	137.14	638.0	1364.3	.0	296.7
10	138.04	559.1	1195.4	.0	260.0
11	140.04	338.0	757.4	.0	171.7

SPENCER'S (1973) - Magnitude & Location of Interslice Forces

Slice #	Right x-coord (ft)	Force Angle (degrees)	Interslice Force (lb)	Force Height (ft)	Boundary Height (ft)	Height Ratio
1	68.35	16.41	19.	.14	.29	.469
2	76.86	16.41	15181.	3.78	8.36	.452
3	77.00	16.41	15553.	3.84	8.52	.451
4	78.00	16.41	15684.	4.05	8.85	.458
5	125.43	16.41	21077.	9.33	24.18	.386
6	125.93	16.41	19926.	9.37	23.84	.393
7	131.30	16.41	9893.	6.51	17.19	.379
8	136.68	16.41	3089.	3.49	10.55	.330

9	137.60	16.41	2246.	3.03	9.41	.322
10	138.47	16.41	1548.	2.65	8.06	.329
11	141.61	.00	-3.	.12	3.00	.038

AVERAGE VALUES ALONG FAILURE SURFACE

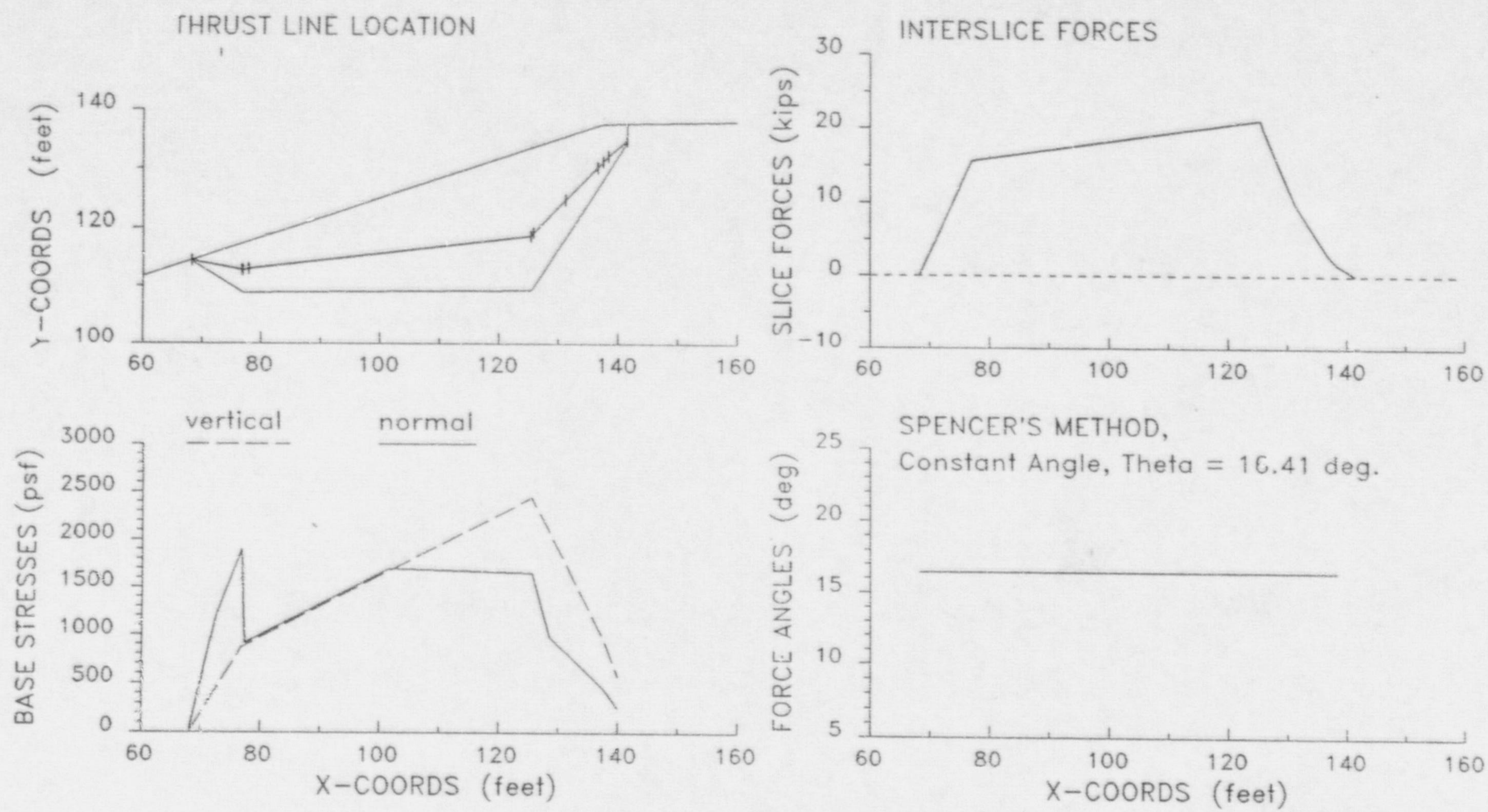
Total Normal Stress = 1711.20 (psf)
Pore Water Pressure = .00 (psf)
Shear Stress = 453.10 (psf)

Total Length of failure surface = 89.03 feet

For the single specified surface and the assumed angle of the interslice forces, the SPENCER'S (1973) procedure gives a

FACTOR OF SAFETY = 1.003

Total shear strength available
along specified failure surface = 404.55E+02 lb



BERT AVE. Translational Yield Co.
SPENCER'S METHOD, FOS for Specified Surface = 1.003

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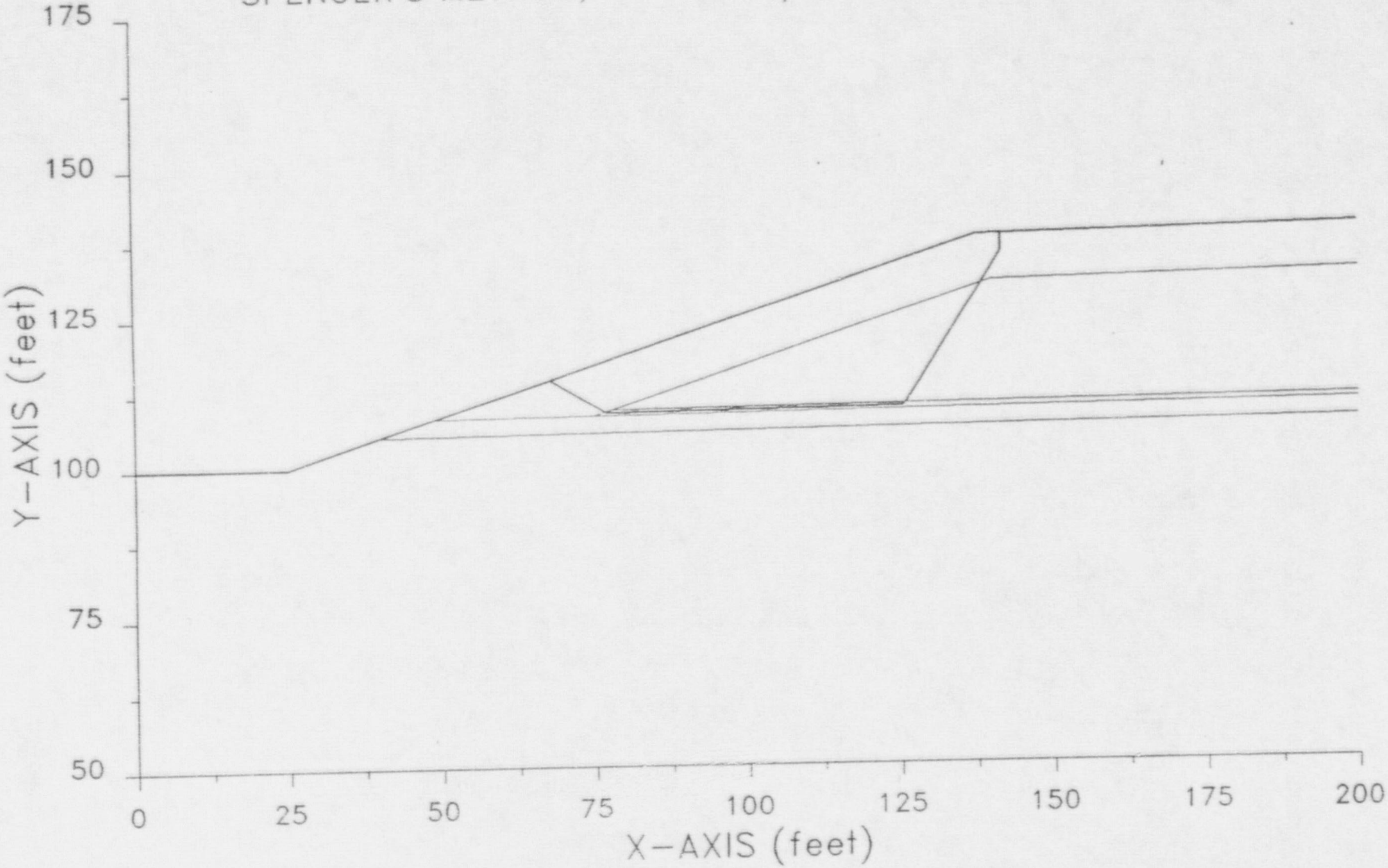
BERT AVE. Translational Yield Co.

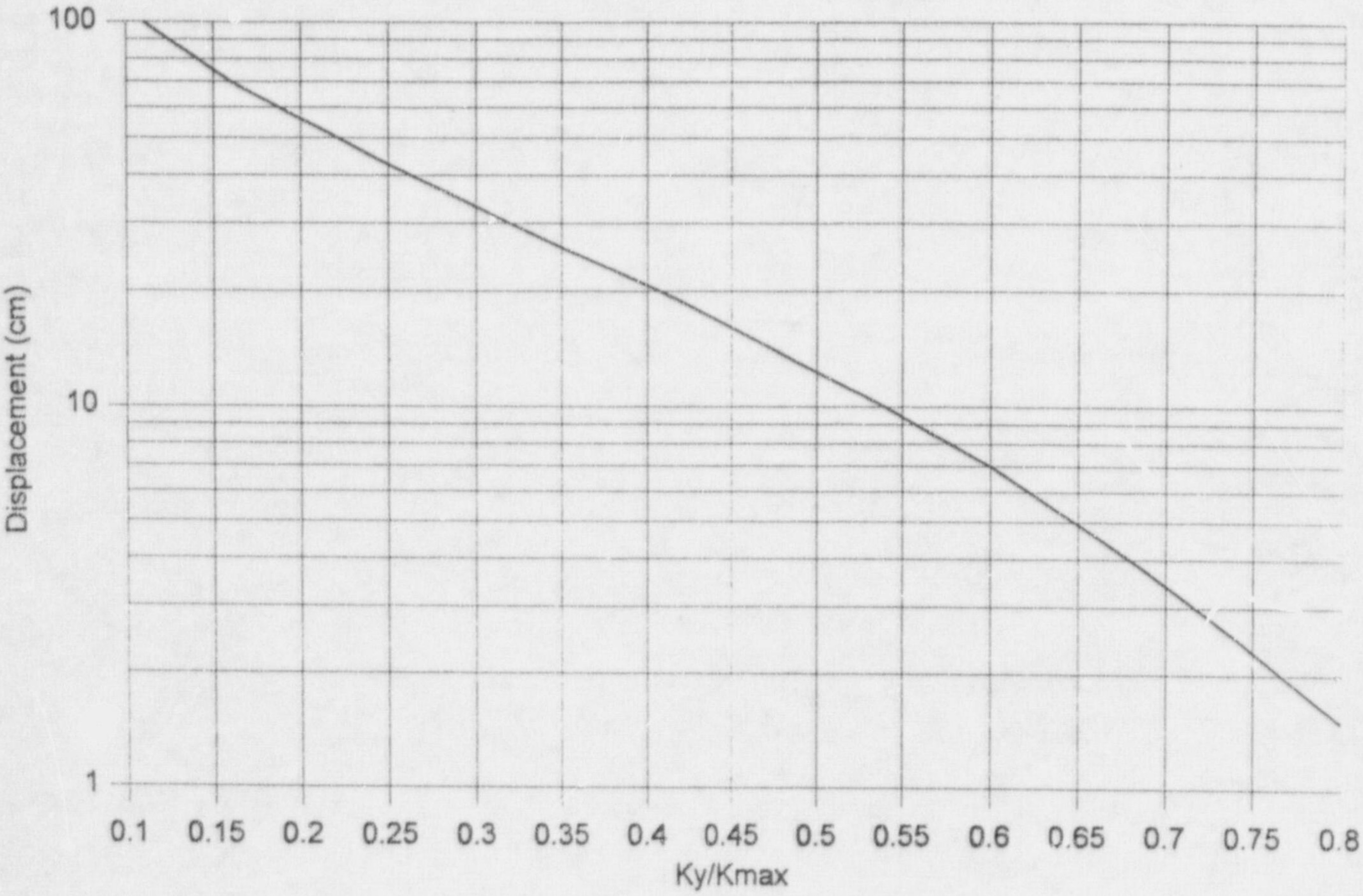
SPENCER'S METHOD, FOS for Specified Surface = 1.003



The FOS (0.95) for deep-seated translational failures involving the GCL is less than the regulatory minimum recommended value of 1.1. However, a simplified Newmark deformation analysis indicates that minimal deformation will occur due to the design earthquake.

Earthquake-Induced Deformations (6.5 M)

After Makdisi and Seed (1978)



- The yield acceleration for this failure scenario is calculated to be 0.125g.
- The maximum seismic acceleration has been estimated to be 0.16g.

Ky/Kmax= 0.83.

Less than 2 cm of displacement can be expected to occur from an earthquake equal to or less than the magnitude of the design earthquake. This exceeds the criteria outlined in Ohio EPA's GCL advisory.