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GOVERNMENT OF THE PEOPLE'S REPUBLIC OF BANGLADESH
MINISTRY OF IRRIGATION, WATER DEVELOPMENT AND FLOOD CONTROL
BANGLADESH WATER DEVELOPMENT BOARD

DHAKA INTEGRATED FLOOD PROTECTION PROJECT
MID-TERM CONSULTANCY SERVICES
ADB LOAN - 1124-BAN (SF)

83

FINAL REPORT

VOLUME - I

ANNEXURE - I : DESIGN OF EMBANKMENT REMEDIAL MEASURES AND REINFORCED EARTH EMBANKMENT ALONG LEFT BANK OF BURIGANGA RIVER

TECHNOCONSULT INTERNATIONAL LIMITED, BANGLADESH
in association with
ASSOCIATED CONSULTING ENGINEERS LIMITED, BANGLADESH
DESH UPODESH LIMITED, BANGLADESH
and
Individual Consultants from LOUIS BERGER INTERNATIONAL, INC., USA

MAY 1993

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1. INTRODUCTION

1.1 Terms of Reference

GeoSyntec Consultants has prepared a Remedial Design Report for the Dhaka Integrated Flood Protection Project (DIFPP). The report was prepared at the request of Mr. A. Barik Bhuiyan of Desh Upodesh Limited (DUL), the Project Team Leader, and Mr. Max G. Williams, P.E., of Louis Berger International, Inc. (LBII), the Co-Team Leader of the Project Team for the DIFPP Bridging Period. The Project Team for the Bridging Period consisted of Technoconsult International Limited, Desh Upodesh Limited, Associated Consulting Engineers (ACE) Limited, all of Bangladesh, and Individual Consultants from Louis Berger International, Inc. of the USA. GeoSyntec Consultants worked as a subconsultant to Louis Berger International, Inc.

The Remedial Design Report describes the field and laboratory investigations, stability analyses, remedial design of the Class I and II areas, design of the embankment extension, and construction quality assurance of the remedial measures. The Remedial Design Report has been prepared by Dr. Kwasi Badu-Tweneboah, P.E., Mr. Roger B. North, P.E., and Dr. Neil D. Williams, P.E., all of GeoSyntec Consultants.

1.2 Background

Severe flooding occurred in Bangladesh, including the Greater Dhaka area, during the 1987 and 1988 Monsoon seasons. Vast areas of the country, were flooded to an unprecedented degree with flood levels about 1.5 m (5 ft) higher than normal. In Dhaka city alone, it is estimated that about 200 sq km (125 sq mi), or 77 percent of the total area of 260 sq km (163 sq mi), were submerged with water to depths over 4.5 m (15 ft) and that about 2.5 million people (which is 60 percent of the city's population) were directly affected by these floods. City life was totally disrupted during this period, and it is estimated that flood damages, which average about Tk. 250 million ($7,140,000) in normal years, exceeded Tk. 700 million ($20,000,000) in 1988.

In the wake of these floods, the Government of Bangladesh established a Committee for Flood Control and Drainage of Greater Dhaka (the Committee) in October 1988. The primary objective of the Committee was the preparation of a flood control plan for the Greater Dhaka area, based primarily on the 1987 Japan International Corporation Agency (JICA) study on storm drainage system improvements for Dhaka city, and the 1988 "Jansen Report" on causes of the 1988 flood and recommended solutions. In January 1989, the Committee submitted a detailed plan for phased investments in flood protection and drainage for Dhaka, Tongi, Narayangonj, and Savar, which was approved by the Government of Bangladesh in March 1989.

In view of the high priority assigned to the Dhaka flood protection project, the Government of Bangladesh immediately initiated Phase I of the recommended works on a crash program basis using their own resources. These works, which were designed to provide protection for about 137 sq km (85 sq mi) in the highly urbanized western part of the city of Dhaka, included:

- construction of a 29.2 km (18 mi) of embankment along the west side of the city;
- construction of an 8.5 km (5.3 mi) of reinforced concrete wall bordering the densely populated south-western side of the city;
- construction of flood protection embankments around Zia International Airport;
- construction of 29.8 km (18.6 mi) of flood protection wall around the Dhaka-Narayanganji-Demra area south of the city;

- construction of 2.0 km (1.2 mi) of new roads and raising of an additional 8.5 km (5.3 mi) of existing roads to an elevation corresponding to the 1988 flood level;

- cleaning and repair of the drainage khals (canals) and stormwater systems; and

- construction of drainage sluices to allow discharge of rainfall run-off through the flood protection structures.

These projects were undertaken as a coordinated effort by the Bangladesh Water Development Board (WDB), Dhaka Water and Sewage Authority (DWASA), Dhaka City Corporation (DCC), Rajdhani Unnayan Kattripaka (Capital Development Authority) (RAJUK), and the Bangladesh Army.

Soft soils are present along the embankment alignment and catastrophic failure of the soil embankment occurred at several locations due to the inability of the underlying soils to support the embankment. The embankment also sustained considerable damage in other areas resulting from erosion, vector damage, poor compaction, and poor construction quality control and quality assurance. In addition, the retaining walls suffered distress and there was concern regarding the adequacy of the design and construction of the retaining walls.

Several sections of the embankment were repaired. However, due to low shear strength of the subgrade soils, poor compaction and quality control during the repair efforts, and inadequate design, failure of the repaired sections has occurred on several occasions. In fact, some sections of the embankment have been repaired more than ten times.

A damage survey (Figure 1-1) was performed by LBII in conjunction with GeoSyntec Consultants in May 1991 [LBII, 1991] and GeoSyntec Consultants [1991]. The damage survey included a review of earlier preand post-construction site investigations carried out by the Bangladesh University of Engineering and Technology (BUET) and by the WDB. The primary findings of the survey were as follows:

- Parts of the embankment, totalling about 4.7 km (2.9 mi) were subject to sudden failure resulting from inadequate subgrade shear strength. As stated previously, in many locations failure had already occurred. A stability analysis for these areas based on soil strength parameters obtained from a previous site investigation indicated that the factor of safety against deep-seated failure was much less than unity. These areas were classified as Class I areas requiring Immediate (I) remedial action.

- An additional 3.1 km (1.9 mi) of the embankment was extensively damaged and required repair, but was not likely to fail catastrophically. These areas of the embankment were classified as Class II that required short-term (S) remedial action.

- More than 3.1 km (1.9 mi) of the embankment was damaged through wave erosion. These damaged areas were characterized as Class III that required long-term (M) remedial action.

- Poor quality control and quality assurance during construction of the reinforced concrete wall made it difficult to determine whether the wall was constructed as it was designed. The damage survey identified areas where the wall had cracked and where erosion (piping) had occurred under the wall.

The survey concluded that the extensive and extreme damage to the embankment was due primarily to:

- Poor compaction of the embankment soils. This poor compaction was caused by a lack of or improper moisture conditioning of the soil prior to placement, placement in lifts that were too high to allow adequate compaction during placement, and inadequate compaction methods for the types of soils used in construction.

- Inadequate subgrade improvement. The subgrade soils in many areas consisted of very soft organic clays and high plasticity silts and/or non-plastic silts. The shear strengths and bearing capacities of these soils were very low and the soils were incapable of supporting the embankment loads without improvement.

- Lack of erosion protection. Some areas of the embankment were subject to wave loading prior to establishment of an adequate vegetative cover. The poor compaction conditions in the embankment exacerbated the erosion damage. This damage could have been minimized if the embankment had been properly compacted and if the sod and jute method of slope protection had been employed in adequate time for the vegetative cover to develop.

- Inappropriate soils. Parts of the embankment were constructed with high plasticity organic clays. These soils are subject to large volume changes, strain softening, which could contribute to failure of the embankment. Other portions of the embankment were constructed with non-plastic silts, which spontaneously disperse in water and were highly susceptible to erosion.

A subsequent damage survey was conducted by GeoSyntec Consultants in August 1991, near the end of the Monsoon season. The survey confirmed the initial findings and conclusions, with the additional observation that severe wave erosion damage had occurred along approximately 11.5 km (7.15 mi) of the river side of the embankment.

At the request of the WDB, meetings were held between LBII, GeoSyntec Consultants, and the WDB in August 1991 to discuss remedial technologies. Based on these meetings, the following design recommendations were developed for the damaged areas:

- excavation of the top portion of the existing embankment in the Class I areas to an elevation to be determined during the remedial design;
- installation of vertical drains in the Class I and parts of the Class II areas;
- recompaction of the upper portion of the embankment in the Class I areas using good construction techniques to assure adequate compaction; and
- reconstruction of the embankment in the erosion damaged (Class III) areas and placement of concrete block for erosion protection.

The remedial design approach developed by GeoSyntec Consultants in conjunction with LBII and the WDB was summarized in the "Revised Remedial Design Approach, Dhaka Integrated Flood Protection Project, FAP-8B" [GeoSyntec Consultants, 1991].

1.3 Scope and Objectives

The purpose of the Remedial Design Report is to present and discuss the remedial design of the existing west embankment of the Dhaka Integrated Flood Protection Project in Dhaka, Bangladesh. The report summarizes work performed under the Contract for Consultancy Services for the Dhaka Integrated Flood Protection Project, FAP-8B, ADB Loan No. 1124 BAN(SF). The objectives of the remedial design program are as follows:

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- Obtain the embankment and subgrade soil properties in the Class I and II areas of the western embankment. For the purpose of the Remedial Design Report, the Class I areas are defined as those areas of the existing west embankment which are subject to imminent risk of catastrophic failure. Class II areas are defined as those areas which could potentially fail, but are not likely to fail catastrophically.

- Refine estimates of the Class I and II areas based on the results of the field and laboratory investigations and continuing monitoring programs.

- Perform a laboratory investigation to obtain the soil properties required for remedial design.

- Perform laboratory and field investigations along the proposed alignment of the embankment extension from Keller Morh to the Mittford Hospital.

- Prepare engineering drawings and construction specifications for the remedial design of the existing embankment in the Class I and II areas.

- Prepare conceptual designs for the embankment extension.

1.4 Organization

The remainder of the Remedial Design Report is organized as follows:

- Section 2 describes the field investigation;

- Section 3 discusses the laboratory testing program;

- Section 4 discusses the stability analyses;

- Section 5 presents the remedial design;

- Section 6 presents a conceptual design of the embankment extension from Keller Morh to the Mittford Hospital in Dhaka, Bangladesh;

- Section 7 discusses the construction of the remedial measures and presents the construction quality assurance plan; and

- Section 8 presents the conclusions and recommendations.

2. FIELD INVESTIGATION

2.1 Introduction

A geotechnical site investigation was conducted along the embankment alignment to determine material properties required for the remedial design of the western embankment. Additional site investigation was conducted along the Burhi Ganga River to obtain geotechnical properties for the design of the embankment extension from Keller Morh to the Mittford Hospital. The field investigation consisted of soil borings, in-situ testing, soil sampling, and piezometer installation. This section describes the field investigation program.

2.2 Soil Boring Program

2.2.1 Western Embankment

Eighty-three soil borings were drilled along the western embankment alignment at the stations and locations shown on Table 2-1. The borings were drilled from the crest of the embankment and from the side slopes of the embankment, and ranged in depth from 8 m (26 ft) to 40 m (130 ft). The borings were drilled by M. Ahmed & Associates, Ltd. (Ahmed), Dhaka, Bangladesh, and Foundation Consultants Ltd., Dhaka, Bangladesh, between 12 February 1992 and 10 June 1992, as indicated in Table 2-1. All soil borings were performed under the supervision of an engineer from the Project Team.

The borings were drilled vertically using a wash boring technique and equipment capable of pushing undisturbed Shelby-tube samples by hydraulic pressure. Casings were installed in each boring as they were advanced to stabilize the upper part of the boring and to provide a drilling fluid return conduit. In all cases, water was used as the drilling fluid.

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Disturbed and undisturbed samples were collected in each boring at nominal 0.45-m (1.5-ft) intervals. The undisturbed samples were obtained using 76-mm (3-in.) diameter Shelby-tube samplers inserted at selected depths of each of the borings. A split-spoon sampler was used to obtain the disturbed samples in conjunction with the standard penetration test (SPT). The SPT was conducted in general accordance with procedures outlined in ASTM D 1586-84, "Standard Method for Penetration Test and Split-Barrel Sampling of Soils". The SPT consisted of dropping a 620-N (140-b) weight free-falling hammer from a height of 760 mm (30 in.) to drive a standard 50-mm (2-in.) outside diameter split-spoon sampler a distance of 460 mm (18 in.). The blows per 150 mm (6 in.) increments of penetration of the split-spoon were recorded for each sample. The "standard penetration resistance", or the N-value", is the sum of the blows from the second and third (150-mm) increments. This resistance to penetration is usually used to aid in characterizing the engineering properties of in-situ soil materials or the lithostratigraphy. The SPT results and the field sample descriptions are presented in the soil boring logs in Appendix A of this report. The SPT results are discussed in Section 2.2.2.

Soil samples obtained from the split-spoon samplers were described in the field using the procedures outlined in ASTM D 1586-84. Samples were obtained from the split-spoon sampler and stored in containers marked with the project number, boring number, sample number, sample depth, and recovery. The samples were transported to various laboratories for further testing and classification. The samples were classified following the procedures outlined in ASTM D 2488-84, "Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)". Sampling intervals and soil descriptions are shown on the boring logs presented in Appendix A. The subsurface stratigraphy and the embankment profile as inferred from the soil borings are shown on Figures 2-1 through 2-20.

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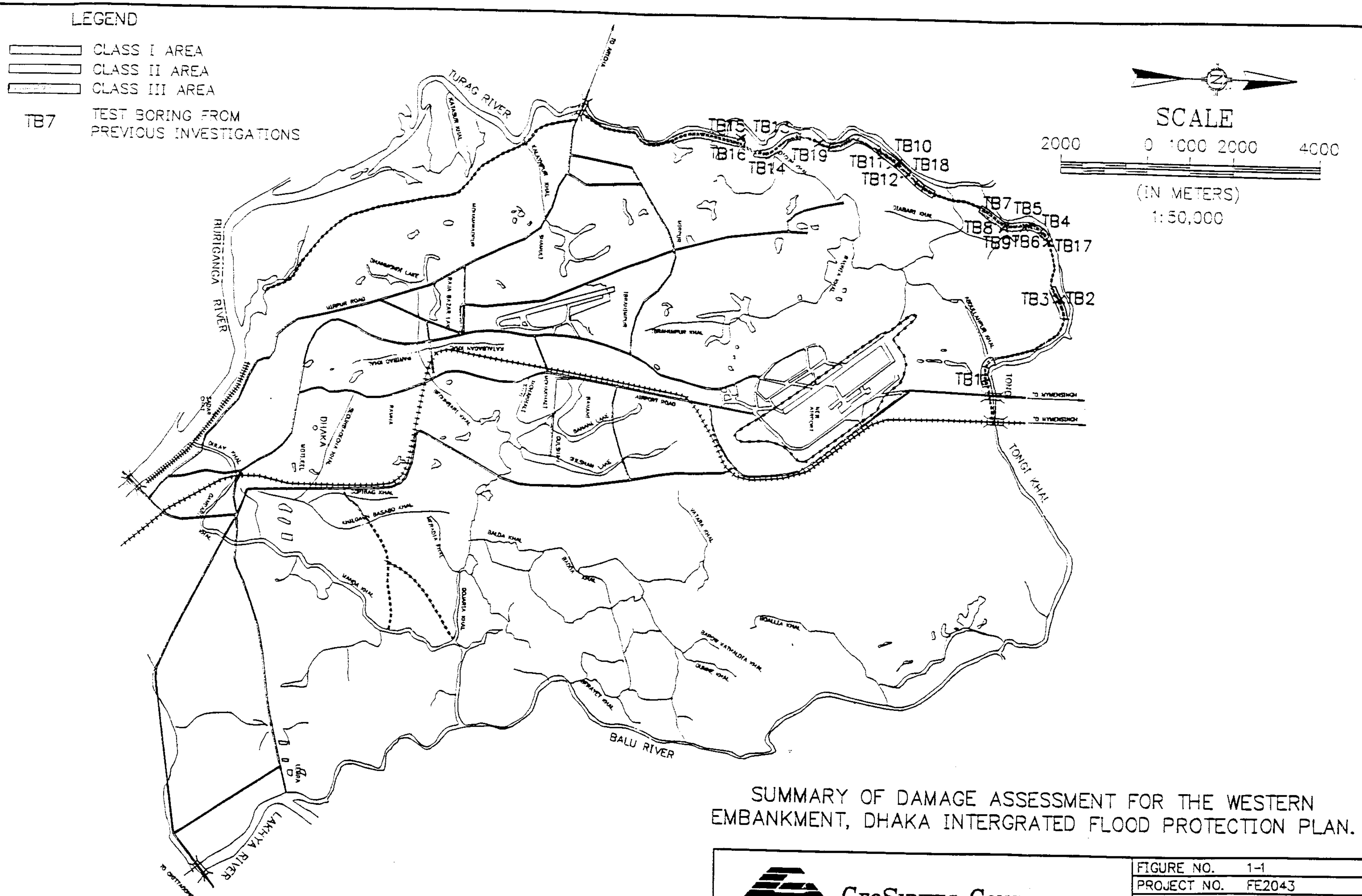

SUMMARY OF DAMAGE ASSESSMENT FOR THE WESTERN EMBANKMENT, DHAKA INTERGRATED FLOOD PROTECTION PLAN.

GEOSYNTEC CONSULTANTS

FIGURE NO.	1-1
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	8-A

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The locations and sample identification numbers of each Shelby-tube sample and split-spoon sample are identified on the boring logs in Appendix A. All samples collected from the soil borings were transported to geotechnical laboratories for testing (see Section 3).

In addition to the SPT, other in-situ tests were performed in some of the borings. These other tests are discussed in Section **2.3**.

Ground-water level measurements were obtained in each boring during the drilling operation and at the end of each day's drilling. A horizontal and vertical survey was performed by the Project Team to accurately determine the coordinates and elevations of the soil borings. These survey data are summarized in Table 1.

2.2.2 SPT Test Results

As discussed previously, SPT tests were performed as part of the soil borings conducted along the alignment of the western embankment. The SPT is the most commonly used in-situ test for obtaining the subsurface information of a site. The N-value, when properly interpreted, is an index for the soil depth and density. Several empirical correlations between the N-value and soil parameters, such as relative density, friction angle, shear strength, unit weight, etc., are available in the geotechnical literature.

The results of the SPT N-values obtained from the soil borings are presented for each boring log in Appendix A. Figures 2-1 through 2-20 show the SPT profiles along nine different cross-sections of the embankment and along the center line of Stations 1+000 to 18+500 m of the western embankment. The SPT N-values in the embankment are generally very low and range from 2 to about 10, indicating a very soft to soft material (assuming the embankment soils are predominantly silts and clays). These low values of penetration resistance would suggest that the embankment soils are poorly compacted. In general, well compacted fill materials for embankments should show relatively uniform and consistent SPT N-values, and should have greater than medium penetration resistance.

2.3 Field Testing

In addition to the SPT tests conducted in each soil boring, hand-operated miniature shear vane tests and pocket penetrometer tests were performed on some of the undisturbed Shelby-tube samples. The purpose of these tests was to determine the shear strength of the undisturbed samples. The hand-operated miniature vane test was performed in general accordance with ASTM D-4648, "Standard Test Method for Laboratory Miniature Vane Shear Test for Saturated Fine-Grained Clayey Soil".

2.4 Piezometer Installation

Twelve hydraulic piezometers were installed in various sections of the embankment to measure the excess pore pressures generated in the embankment. Additionally, seven dummy open tube piezometers were installed outside the influence area of the embankment to provide information on the pore water pressure profile of the subsoil. Table 2-2 presents a summary of the locations and depths of the piezometers.

Measurements are being conducted for each piezometer to establish the variations in the piezometric and ground-water levels, as well as any excess pore pressures within the embankment. Weekly measurements taken from 11 June 1992 to 23 July 1992 are summarized in Table 2-2.

2.5 Quality Assurance

In general, the field investigations were conducted following standard test protocols, in general accordance with applicable ASTM test methods. To ensure that this was done, a Subsurface Investigation Services and Field Investigation Plan was prepared and provided to each of the drilling contractors. A copy of this plan is provided in Attachment A. The contents of this document, inclusive of all the terms and conditions, were discussed with, and accepted by each drilling contractor. Additionally, strict quality assurance monitoring methods were employed in the field to ensure that reliable and reproducible data were obtained. This included inspection and monitoring of all equipment to ensure that they were maintained in good and proper working condition.

Table 2-1 DHAKA INTEGRATED FLOOD PROTECTION PROJECT
SUMMARY OF BORINGS DRILLED AT WESTERN EMBANKMENT

BORING NUMBER	DRILLING COMPANY	START DATE	FINISH DATE	STATION (m)	POSITION FROM CL (m)	BORING DEPTH (m)	GROUND ELEVN. (m)
201	FOUNDATION CONSULT	13 FEB 92	14 FEB 92	1050.0	-1.2	16.00	10.0
203	FOUNDATION CONSULT	14 FEB 92	15 FEB 92	1150.0	1.5	12.45	10.0
206	M. AHMED & ASSOC LTD	10 FEB 92	12 FEB 92	1215.0	-1.2	15.30	9.8
209	M. AHMED & ASSOC LTD	12 FEB 92	13 FEB 92	1315.0	-15.0	14.45	5.1
210	M. AHMED & ASSOC LTD	15 FEB 92	15 FEB 92	1315.0	25.5	7.95	2.2
211	M. AHMED & ASSOC LTD	12 FEB 92	13 FEB 92	1315.0	-0.5	16.95	9.6
212	FOUNDATION CONSULT	17 FEB 92	17 FEB 92	1600.0	-1.2	15.60	9.9
213	M. AHMED & ASSOC LTD	14 FEB 92	14 FEB 92	1415.0	-1.1	16.95	9.9
214	FOUNDATION CONSULT	20 FEB 92	20 FEB 92	4230.0	-1.2	12.45	9.1
215	FOUNDATION CONSULT	21 FEB 92	22 FEB 92	4550.0	-1.2	16.25	9.1
216	M. AHMED & ASSOC LTD	17 FEB 92	17 FEB 92	4750.0	-0.9	15.95	9.6
217	M. AHMED & ASSOC LTD	18 FEB 92	18 FEB 92	4750.0	15.8	10.45	5.1
218	M. AHMED & ASSOC LTD	19 FEB 92	20 FEB 92	4850.0	-1.1	17.45	9.4
219	FOUNDATION CONSULT	22 FEB 92	24 FEB 92	6360.0	-1.2	24.95	9.1
220	FOUNDATION CONSULT	25 FEB 92	26 FEB 92	6475.0	-0.6	24.95	8.9
221	FOUNDATION CONSULT	26 FEB 92	27 FEB 92	6750.0	-1.2	19.45	8.9
222	FOUNDATION CONSULT	27 FEB 92	1 MAR 92	6850.0	-1.0	28.15	8.3
223	M. AHMED & ASSOC LTD	21 FEB 92	23 FEB 92	6890.0	-0.7	26.95	8.6
224	M. AHMED & ASSOC LTD	24 FEB 92	25 FEB 92	6890.0	-13.4	23.65	6.6
225	M. AHMED & ASSOC LTD	26 FEB 92	27 FEB 92	6890.0	-22.9	19.95	4.6
226	M. AHMED & ASSOC LTD	27 FEB 92	28 FEB 92	6890.0	-32.7	19.45	1.7
227	M. AHMED & ASSOC LTD	25 FEB 92	26 FEB 92	6890.0	14.0	22.95	5.0
228	M. AHMED & ASSOC LTD	26 FEB 92	27 FEB 92	6890.0	21.0	20.95	2.9
229	FOUNDATION CONSULT	26 FEB 92	27 FEB 92	6940.0	-1.5	25.00	8.6
230	M. AHMED & ASSOC LTD	4 MAR 92	6 MAR 92	6990.0	0.8	34.45	8.1
231	M. AHMED & ASSOC LTD	2 MAR 92	3 MAR 92	6990.0	-11.9	25.95	5.9
232	M. AHMED & ASSOC LTD	1 MAR 92	2 MAR 92	6990.0	-32.0	19.95	4.6
233	M. AHMED & ASSOC LTD	29 FEB 92	29 FEB 92	6990.0	-39.0	18.95	4.0
234	M. AHMED & ASSOC LTD	2 MAR 92	4 MAR 92	6990.0	21.0	30.95	4.6
235	M. AHMED & ASSOC LTD	28 FEB 92	1 MAR 92	6990.0	45.1	28.95	2.2
236	FOUNDATION CONSULT	27 FEB 92	3 MAR 92	7250.0	1.2	37.40	9.3
237	FOUNDATION CONSULT	5 MAR 92	7 MAR 92	7050.0	1.0	39.00	9.2
238	FOUNDATION CONSULT	4 MAR 92	5 MAR 92	7350.0	1.2	27.40	9.3
239	FOUNDATION CONSULT	6 MAR 92	7 MAR 92	7450.0	1.1	22.25	9.7
240	FOUNDATION CONSULT	10 MAR 92	11 MAR 92	7700.0	0.9	32.45	9.1
241	FOUNDATION CONSULT	8 MAR 92	10 MAR 92	7800.0	1.1	29.50	9.0
242	FOUNDATION CONSULT	11 MAR 92	12 MAR 92	8000.0	1.2	25.75	9.2

POSITION FROM CL: + IS RIVER SIDE; - IS CITY SIDE

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Table 2-1 DHAKA INTEGRATED FLOOD PROTECTION PROJECT
SUMMARY OF BORINGS DRILLED AT WESTERN EMBANKMENT

BORING NUMBER	DRILLING COMPANY	START DATE	FINISH DATE	STATION (m)	POSITION FROM CL (m)	BORING DEPTH (m)	GROUND ELEVN. (m)
243	FOUNDATION CONSULT	13 MAR 92	13 MAR 92	8150.0	0.7	25.45	9.2
244	M. AHMED & ASSOC LTD	7 MAR 92	8 MAR 92	9250.0	0.3	25.45	8.6
245	M. AHMED & ASSOC LTD	6 MAR 92	8 MAR 92	9500.0	0.0	32.95	9.0
246	M. AHMED & ASSOC LTD	9 MAR 92	11 MAR 92	9900.0	0.0	34.95	9.2
247	M. AHMED & ASSOC LTD	10 MAR 92	11 MAR 92	10700.0	1.0	23.45	9.1
248	FOUNDATION CONSULT	15 MAR 92	15 MAR 92	10800.0	1.5	15.45	7.8
249	M. AHMED & ASSOC LTD	11 MAR 92	12 MAR 92	11000.0	1.0	20.45	7.9
250	M. AHMED & ASSOC LTD	12 MAR 92	12 MAR 92	11000.0	-6.5	16.95	6.9
251	M. AHMED & ASSOC LTD	13 MAR 92	13 MAR 92	11000.0	-9.0	17.95	4.4
252	M. AHMED & ASSOC LTD	14 MAR 92	14 MAR 92	11000.0	-13.5	15.45	1.8
253	M. AHMED & ASSOC LTD	13 MAR 92	13 MAR 92	11000.0	13.0	18.95	3.3
254	M. AHMED & ASSOC LTD	14 MAR 92	14 MAR 92	11000.0	19.0	15.95	1.7
255	FOUNDATION CONSULT	16 MAR 92	17 MAR 92	11100.0	0.8	25.45	9.4
256	FOUNDATION CONSULT	13 MAR 92	14 MAR 92	11220.0	-3.5	17.75	8.0
257	FOUNDATION CONSULT	15 MAR 92	15 MAR 92	11300.0	1.3	22.65	9.3
258	FOUNDATION CONSULT	18 MAR 92	20 MAR 92	13850.0	0.3	40.00	9.3
259	FOUNDATION CONSULT	23 MAR 92	25 MAR 92	13925.0	6.0	37.00	7.6
260	FOUNDATION CONSULT	17 MAR 92	18 MAR 92	13925.0	-6.7	37.75	6.6
261	FOUNDATION CONSULT	19 MAR 92	21 MAR 92	13925.0	-34.5	35.00	4.2
262	FOUNDATION CONSULT	25 MAR 92	26 MAR 92	13925.0	19.6	35.00	4.5
263	FOUNDATION CONSULT	26 MAR 92	27 MAR 92	13925.0	36.0	33.45	1.7
264	M. AHMED & ASSOC LTD	16 MAR 92	17 MAR 92	14320.0	-5.5	31.95	5.4
265	M. AHMED & ASSOC LTD	18 MAR 92	19 MAR 92	14320.0	-15.3	22.95	2.9
266	M. AHMED & ASSOC LTD	16 MAR 92	17 MAR 92	14320.0	13.6	22.95	3.9
267	M. AHMED & ASSOC LTD	17 MAR 92	18 MAR 92	14320.0	19.9	21.45	2.5
268	M. AHMED & ASSOC LTD	19 MAR 92	20 MAR 92	14720.0	-1.9	26.00	8.0
269	M. AHMED & ASSOC LTD	21 MAR 92	22 MAR 92	15035.0	-0.8	28.95	7.8
270	M. AHMED & ASSOC LTD	24 MAR 92	25 MAR 92	15035.0	-13.1	25.45	6.6
271	M. AHMED & ASSOC LTD	25 MAR 92	26 MAR 92	15035.0	-21.6	24.45	4.2
272	M. AHMED & ASSOC LTD	21 MAR 92	22 MAR 92	15035.0	14.2	27.45	4.3
273	M. AHMED & ASSOC LTD	23 MAR 92	23 MAR 92	15035.0	22.2	23.95	2.9
274	M. AHMED & ASSOC LTD	26 MAR 92	27 MAR 92	15035.0	29.4	22.45	1.3
275	M. AHMED & ASSOC LTD	7 JUN 92	8 JUN 92	15250.0	-4.3	22.95	7.7
276	M. AHMED & ASSOC LTD	4 JUN 92	5 JUN 92	15750.0	-1.0	24.95	7.9
277	M. AHMED & ASSOC LTD	6 JUN 92	7 JUN 92	15900.0	-0.8	22.45	9.8
278	FOUNDATION CONSULT	29 MAR 92	30 MAR 92	16610.0	0.5	18.00	10.1
279	FOUNDATION CONSULT	28 MAR 92	29 MAR 92	16610.0	-7.4	14.50	7.6

POSITION FROM CL: + IS RIVER SIDE; - IS CITY SIDE

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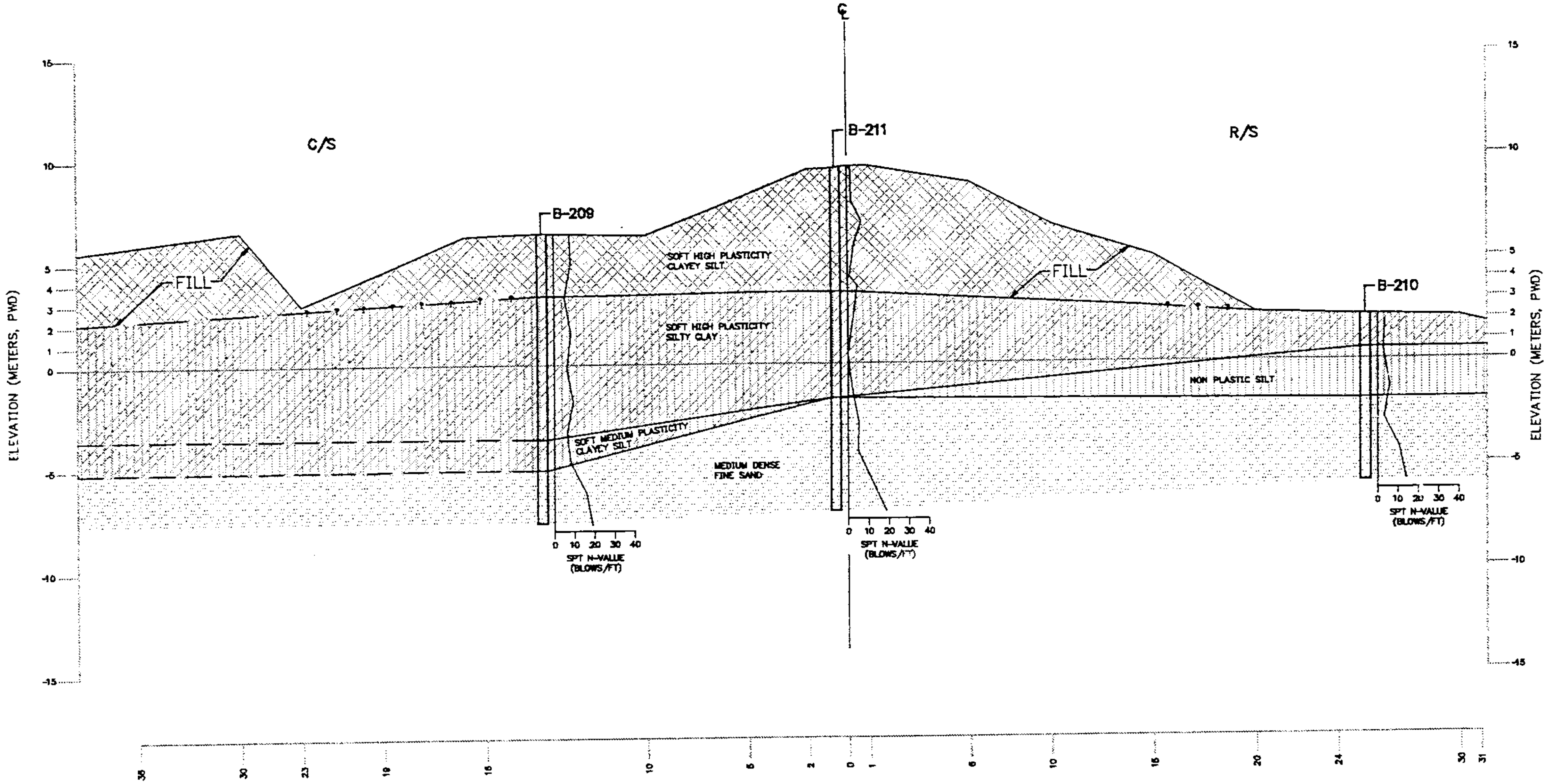
Table 2-1 DHAKA INTEGRATED FLOOD PROTECTION PROJECT
SUMMARY OF BORINGS DRILLED AT WESTERN EMBANKMENT

BORING NUMBER	DRILLING COMPANY	START DATE	FINISH DATE	STATION (m)	POSITION FROM CL (m)	BORING DEPTH (m)	GROUND ELEVN. (m)
280	FOUNDATION CONSULT	31 MAR 92	31 MAR 92	16610.0	16.3	13.55	5.9
281	FOUNDATION CONSULT	31 MAR 92	1 APR 92	16610.0	32.4	12.20	2.3
282	M. AHMED & ASSOC LTD	28 MAR 92	28 MAR 92	17140.0	1.1	26.45	8.3
283	M. AHMED & ASSOC LTD	28 MAR 92	29 MAR 92	17140.0	-12.6	24.95	7.0
284	M. AHMED & ASSOC LTD	29 MAR 92	29 MAR 92	17140.0	-27.1	16.45	1.5
285	M. AHMED & ASSOC LTD	30 MAR 92	30 MAR 92	17140.0	22.0	24.95	4.6
286	M. AHMED & ASSOC LTD	31 MAR 92	31 MAR 92	17140.0	30.0	22.95	1.7
287	M. AHMED & ASSOC LTD	30 MAR 92	31 MAR 92	17260.0	-1.6	24.45	8.0
288	M. AHMED & ASSOC LTD	31 MAR 92	1 APR 92	17400.0	-2.3	23.95	7.8

Table 2-2. SUMMARY OF PIEZOMETER DATA

Piezometer No.	Ch (m)	Depth (m)	Ref. R.L. (m)	Water Level On 11-6-92	On 18-6-92	On 25-6-92	On 2-7-92	On 9-7-92	On 16-7-92	On 23-7-92
. R/S	13+925	8.0	(+) 7.87	(+) 1.75	(+) 2.42	(+) 2.53	(+) 2.87	(+) 3.17	(+) 1.57	(+) 3.53
1/A C/S	13+925	12.5	(+) 4.517					(-) 1.38 *	(-) 1.18 •	(-) 1.12 •
. R/S	13+925	12.5	(+) 7.81	(+) 2.46	(+) 3.38	(+) 3.81	(+) 3.86	(+) 4.08	(+) 4.21	(+) 4.23
. R/S	6+990	13.0	(+) 8.16	(+) 0.23	(+) 3.61	(+) 5.01	(+) 5.62	(+) 5.89	(+) 5.91	(+) 5.96
/A. C/S	6+990	13.0	(+) 3.882					(-) 1.02 *	(-) 0.77 •	(-) 0.65 •
. C/S	6+990	10.0	(+) 8.21	(+) 2.91	(+) 6.21	(+) 5.71	(+) 5.63	(+) 5.58	(+) 5.66	(+) 5.79
. R/S	1+315	15.0	(+) 9.90	(-) 4.3	(-) 1.70	(-) 1.97	(-) 1.52	(-) 1.37	(+) 3.60	(+) 1.10
/A. C/S	1+315	15.0	(+) 5.195					(-) 5.30 *	(-) 4.50 •	(-) 4.89 •
. C/S	14+320	10.0	(+) 7.31	(+) 5.86	(+) 3.88	(+) 3.88	(+) 4.01	(+) 4.35	(+) 4.56	(+) 4.60
/A. C/S	14+320	14.5	+ 4.78					(-) 1.07 *	(-) 1.05 •	(-) 0.90 •
. C/S	14+320	14.5	(+) 7.24	(-) 5.71	(-) 4.22	(-) 3.21	(-) 2.51	(-) 1.68	(-) 0.76	(-) 0.36
. R/S	6+890	10.0	(+) 8.67	(-) 0.3	(+) 1.36	(+) 1.92	(+) 2.59	(+) 3.10	(+) 3.42	(+) 3.62
/A. C/S	6+890	15.0	(+) 4.68					(-) 2.95 *	(-) 2.20 •	(-) 2.12 •
. R/S	11+000	6.0	(+) 7.96	(+) 6.51	(+) 6.49	(+) 6.51	(+) 6.46	(+) 6.56	(+) 5.76	(+) 6.77
/A. C/S	11+000	10.0	(+) 4.64					(-) 1.55 *	(-) 1.50 •	(-) 1.37 •
10. R/S	11+000	10.0	(+) 7.97	(-) 0.28	(+) 0.01	(+) 0.09	(+) 0.20	(+) 0.47	(+) 0.82	(+) 1.04
11. C/S	6+890	15.0	(+) 8.75	(-) 2.90	(+) 0.42	(+) 2.45	~~(+) 3.64~~	(+) 4.4	(+) 4.80	(+) 4.95
12. R/S	16+610	12.0	(+) 8.82	(-) 1.23	(-) 0.50	(+) 0.10	(+) 0.62	(+) 2.5	(+) 2.52	(+) 2.56
12/A- C/S	16+610	12.0	(+) 6.123					(-) 3.27 *	(-) 3.15 •	(-) 13.05 •

NOTE : * Water level refference to top of the G.I. Pipe.



SECTION
CS.NO-4 CH.-1315M

GeoSyntec Consultants	FIGURE NO.	2-1
	PROJECT NO.	FE2043
	DOCUMENT NO.	
	PAGE NO.	18

DWG: 2043F001 199209101109 SB

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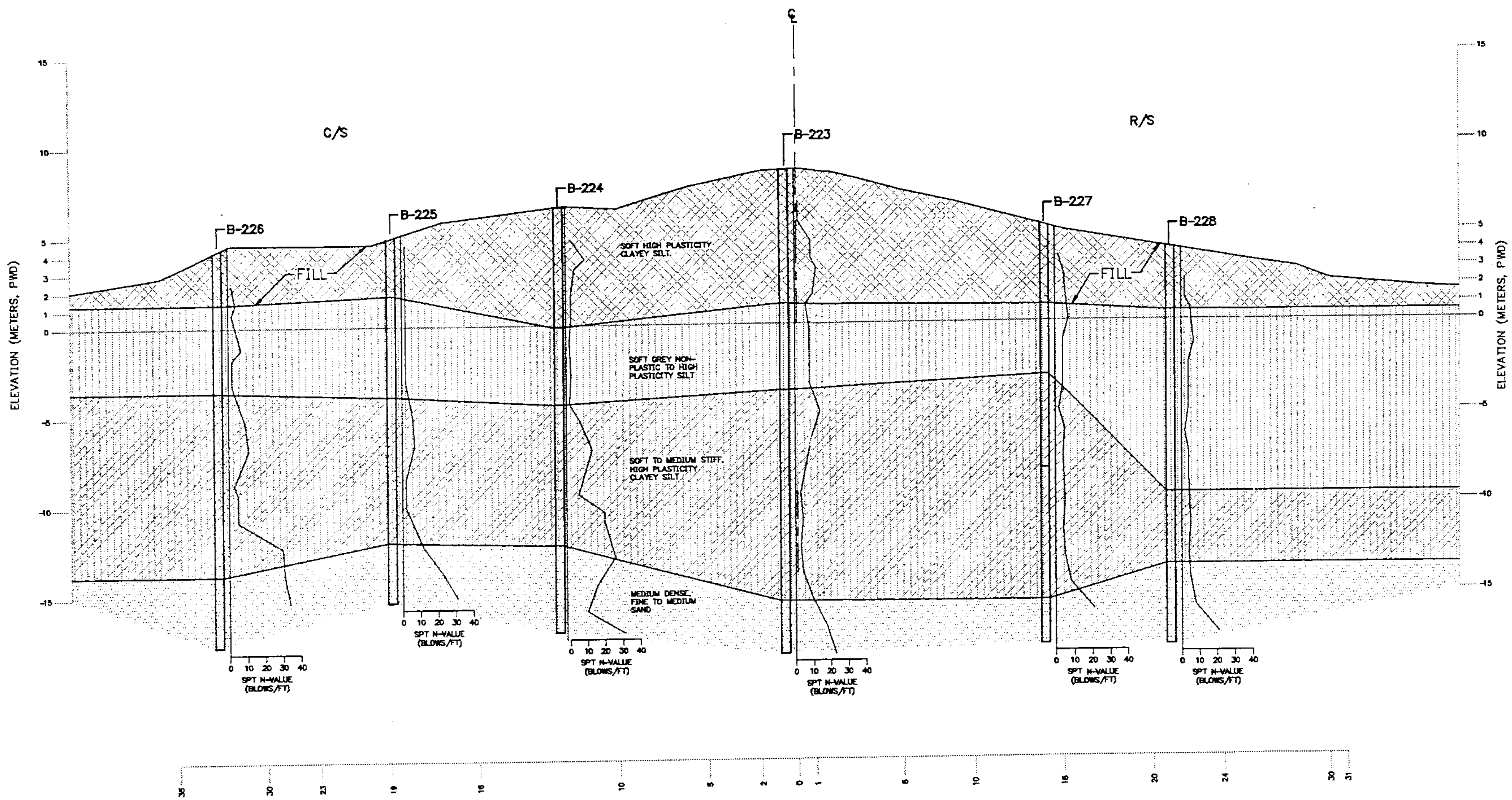


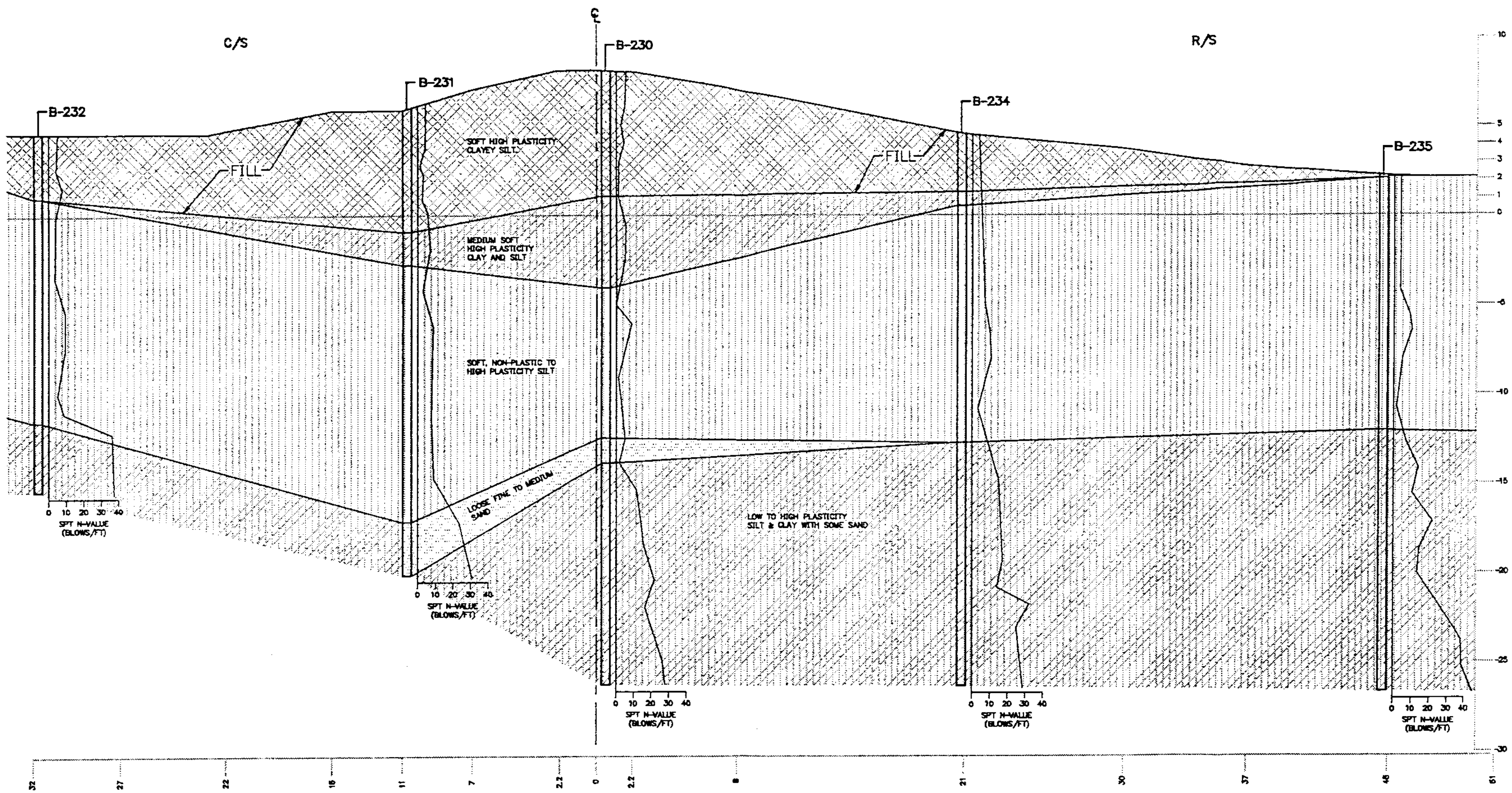
SECTION
CS.NO-4 CH.-6890M
SCALE: HOR.: 1CM=1M
SCALE: VERT.: 1CM=6M

GEOSYNTEC CONSULTANTS

FIGURE NO.	2-2
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	19

DWG: 2043F0D2 199209101111 SB

27



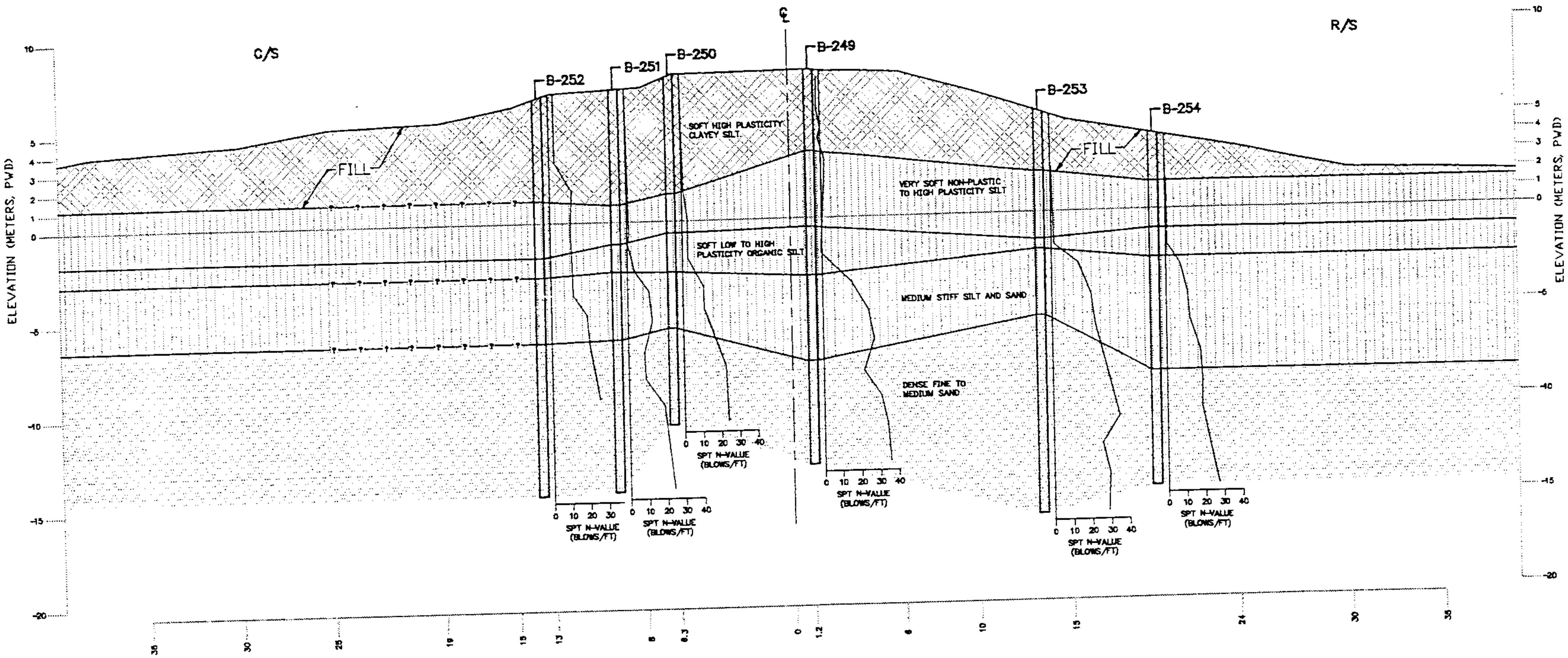
SECTION
CS.NO-17 CH.-6990M

SCALE: HOR.: 1CM=1M
SCALE: VERT.: 1CM=8M

GEOSYNTEC CONSULTANTS

FIGURE NO.	2-3
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	20

DWG: 2043F003 199209101119 SB



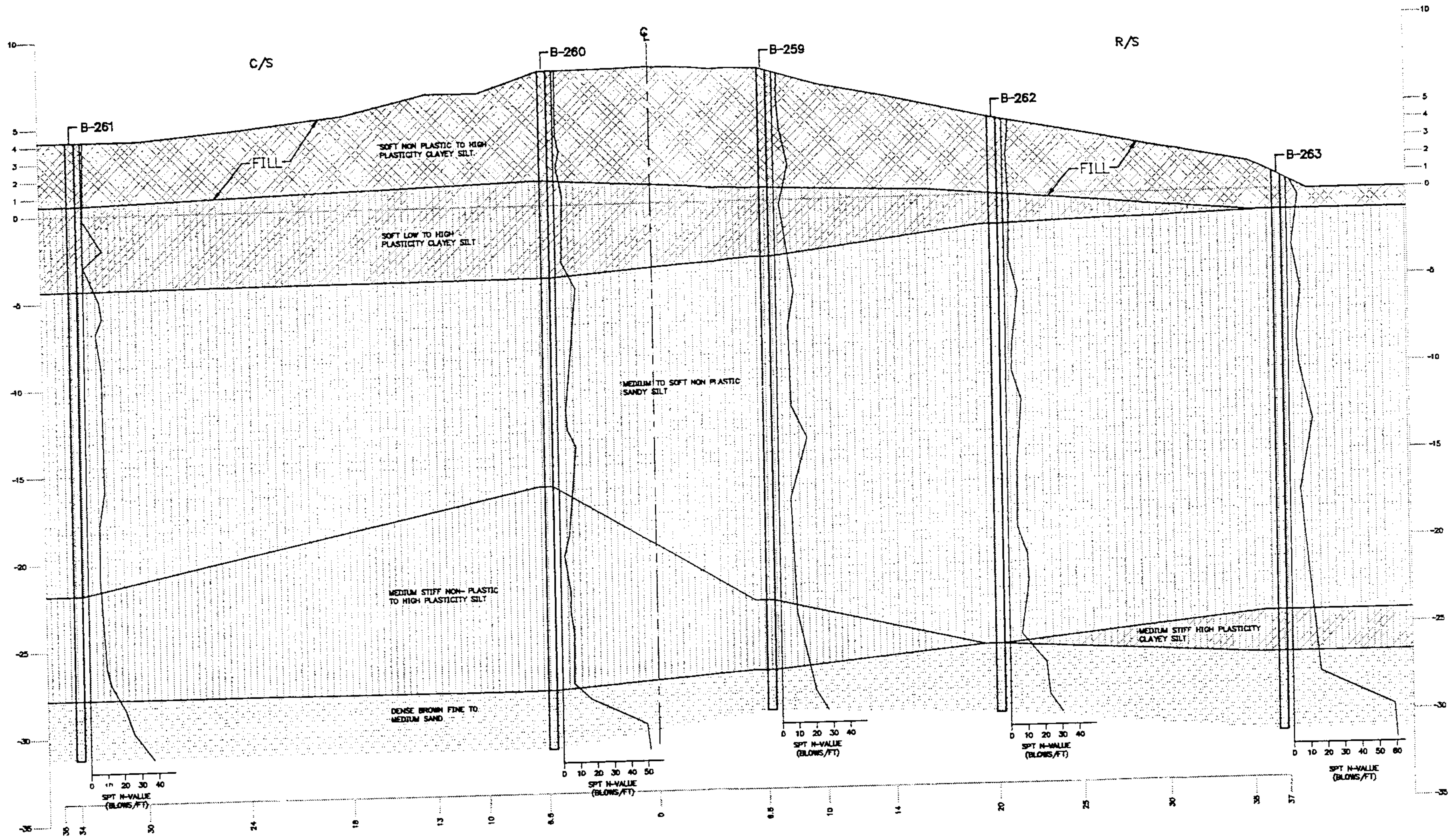
SECTION
CS.NO-27 CH.-11000M

SCALE: HOR.: 1CM=1M
SCALE: VERT.: 1CM=8M

GeoSyntec Consultants

FIGURE NO.	2-4
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	21

DWG: 2043F004 199209101121 SB



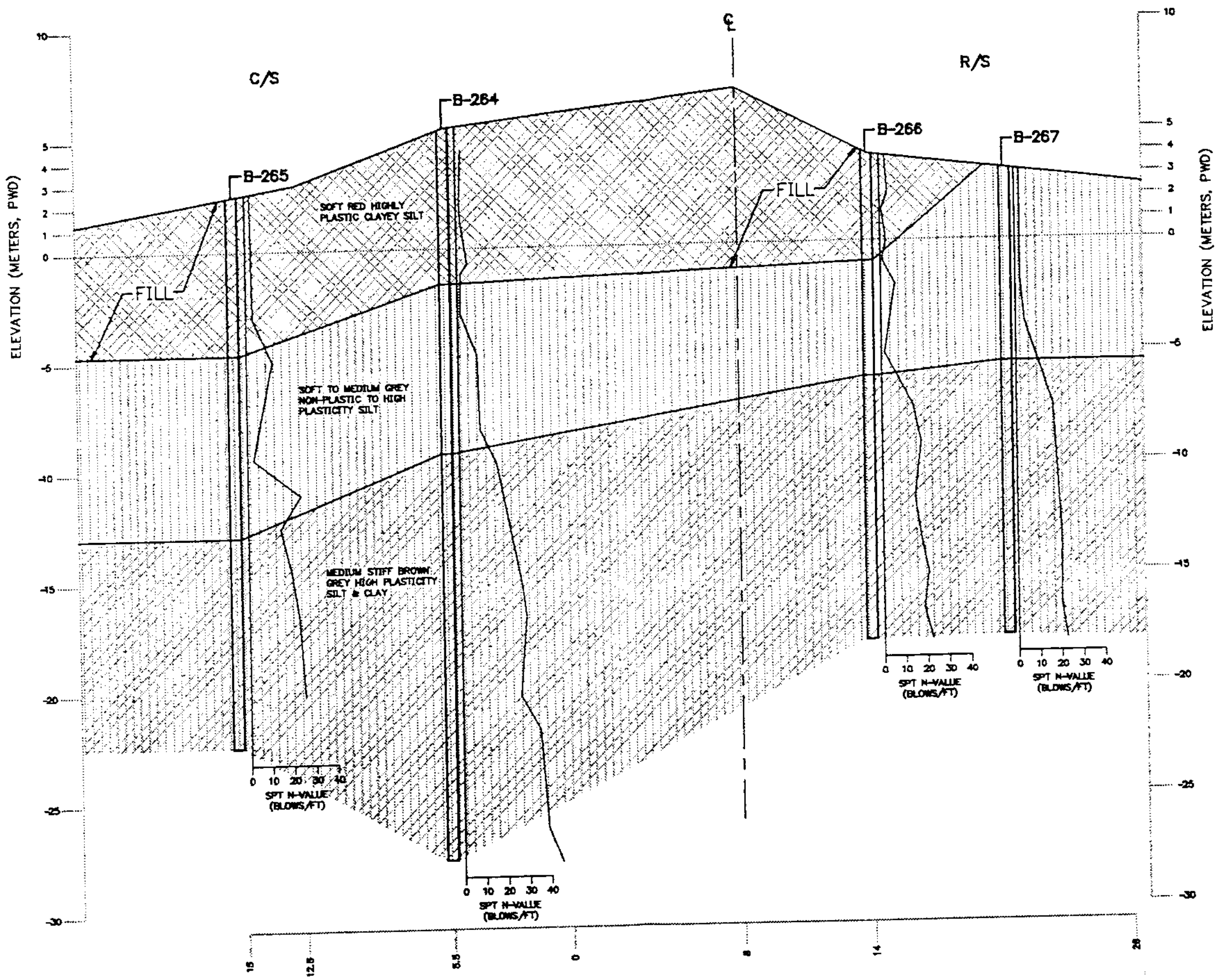
SECTION
CS.NO- CH.-13925M

SCALE: HOR.: 1CM=1M
SCALE: VERT.: 1CM=1M

GeoSyntec Consultants

FIGURE NO.	2-5
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	22

DWG: 2043F005 199209101125 SB



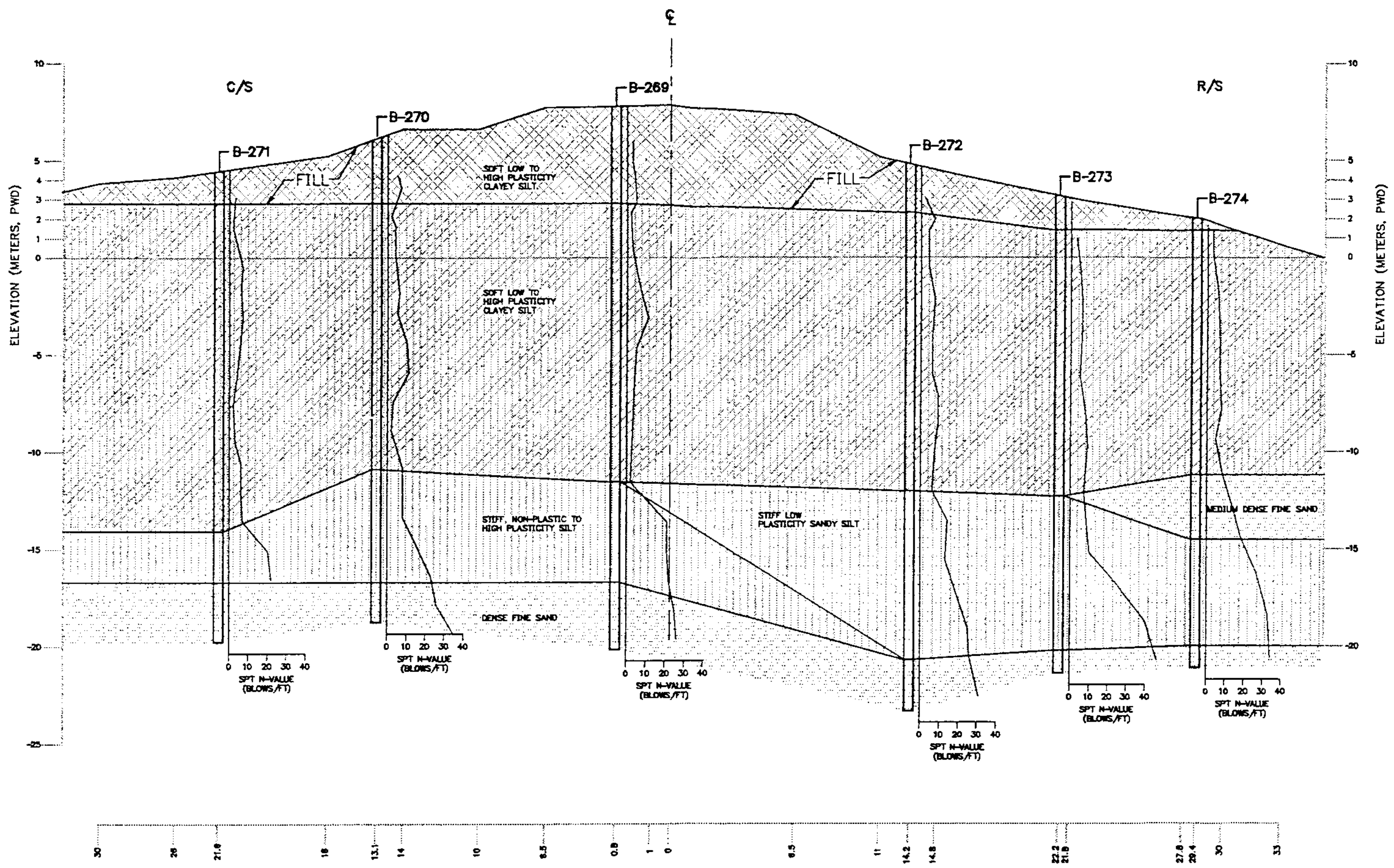
SECTION
C/S-31 CH.-14320M

SCALE: HOR.: 1CM=1M
SCALE: VERT.: 1CM=4M

GeoSyntec Consultants

FIGURE NO.	2-6
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	23

DWG: 2043F006 199209101127 SB



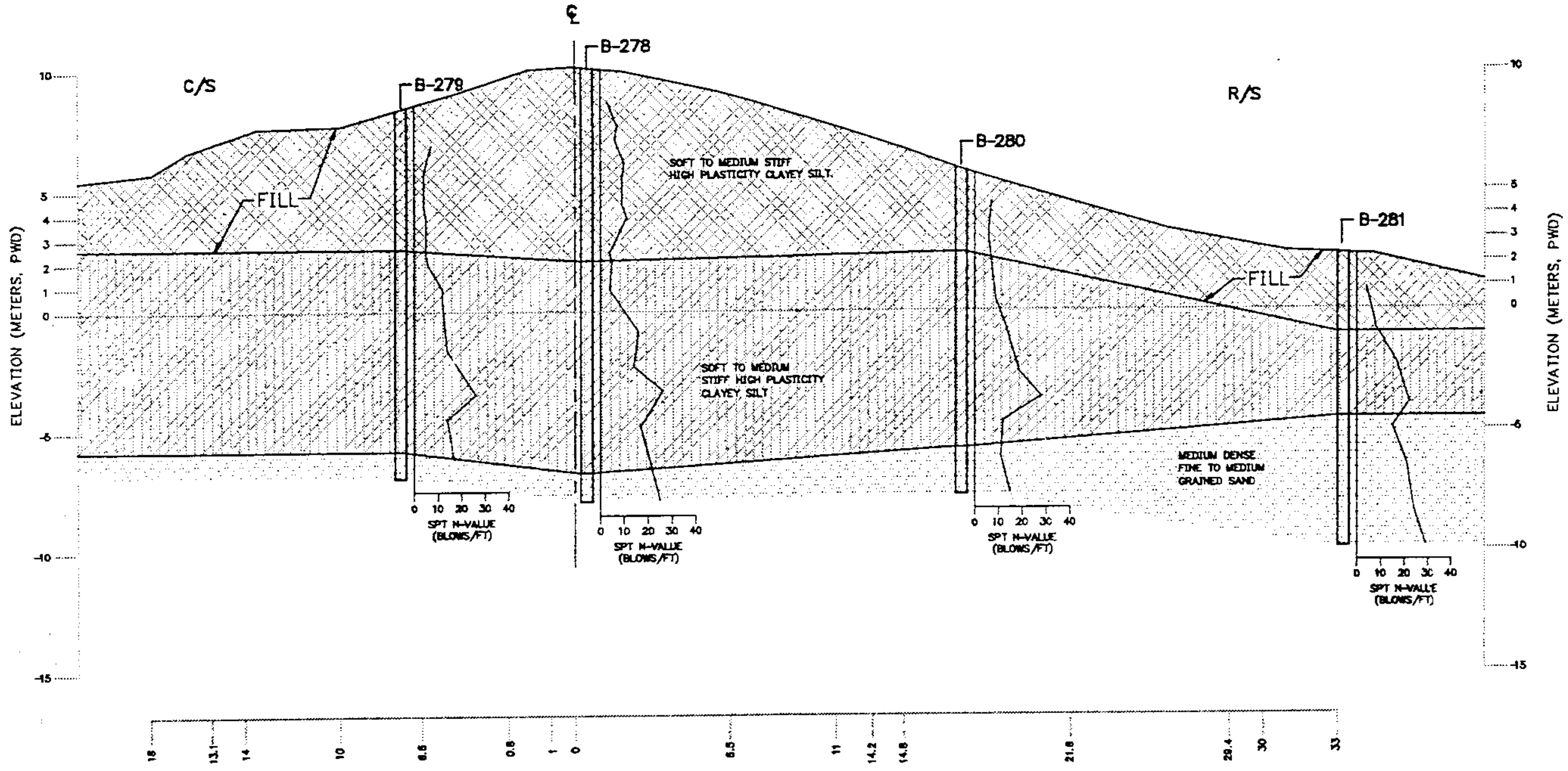
SECTION
C/S-33 CH-15035M
SCALE: HOR.: 1CM=1M
SCALE: VERT.: 1CM=8M

GeoSyntec Consultants

FIGURE NO.	2-7
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	24

DWG: 2043F007 199209101128 SB

90



SECTION
C/S-33 CH-16610

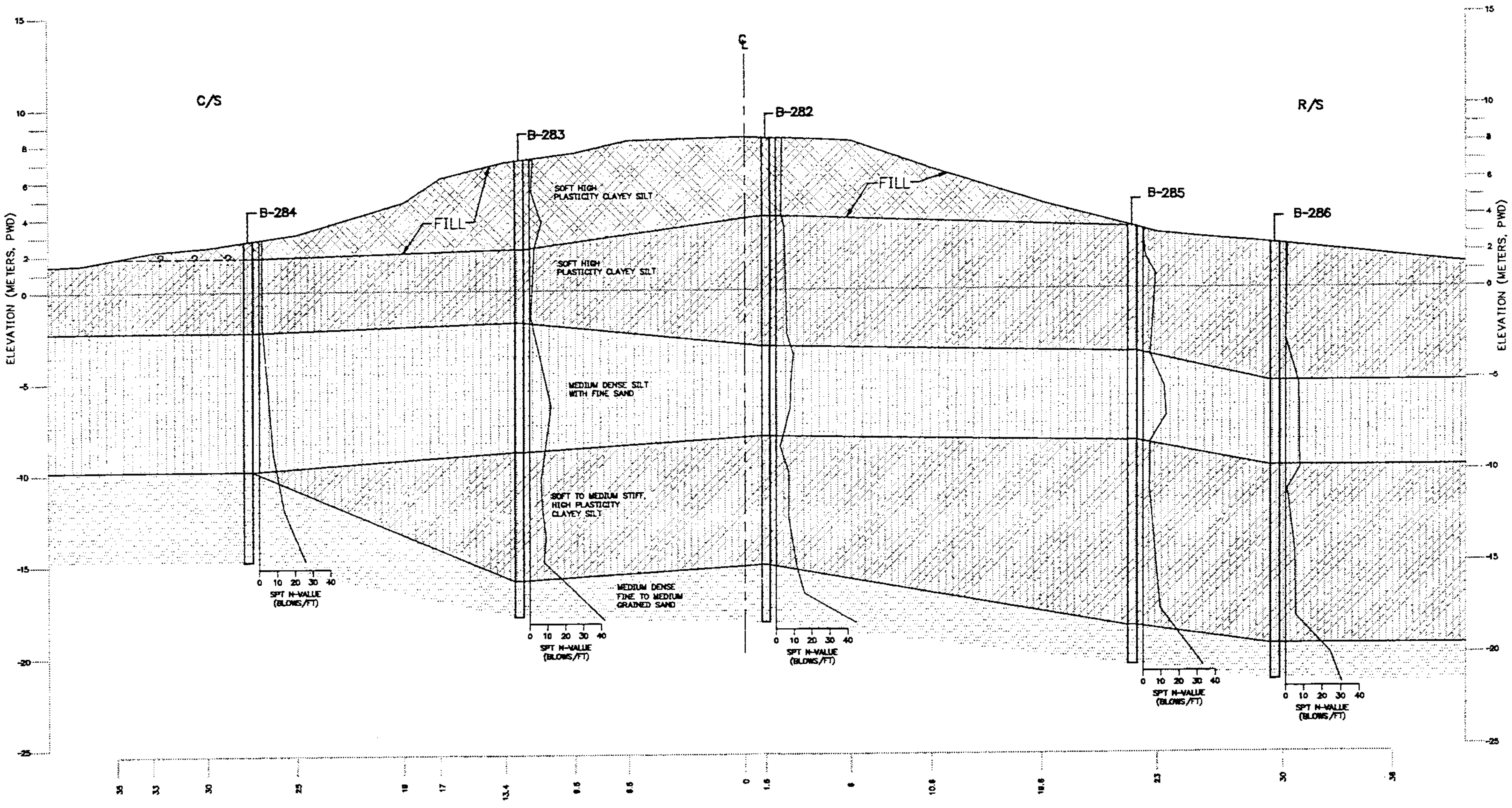
SCALE: HORI.: 1CM=1M
SCALE: VERT.: 1CM=8M

GeoSyntec Consultants

FIGURE NO.	2-8
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	25

DWG: 2043F008 199209101129 SB

92



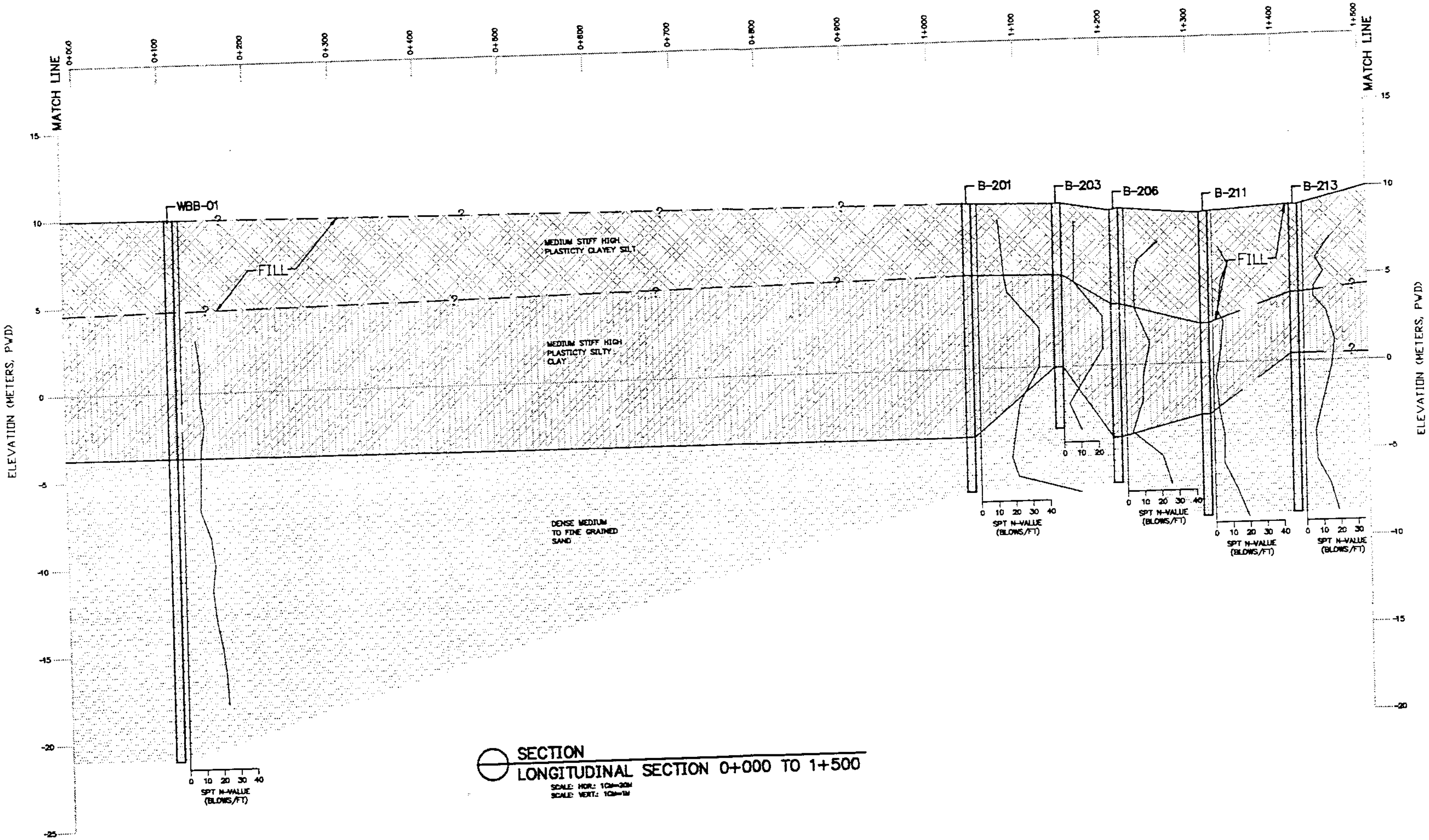
SECTION
C/S- CH-171140

SCALE: HOR.: 1CM=1M
SCALE: VERT.: 1CM=8M

GeoSyntec Consultants

FIGURE NO.	2-9
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	26

DWG: 2043F009 199209101131 SB

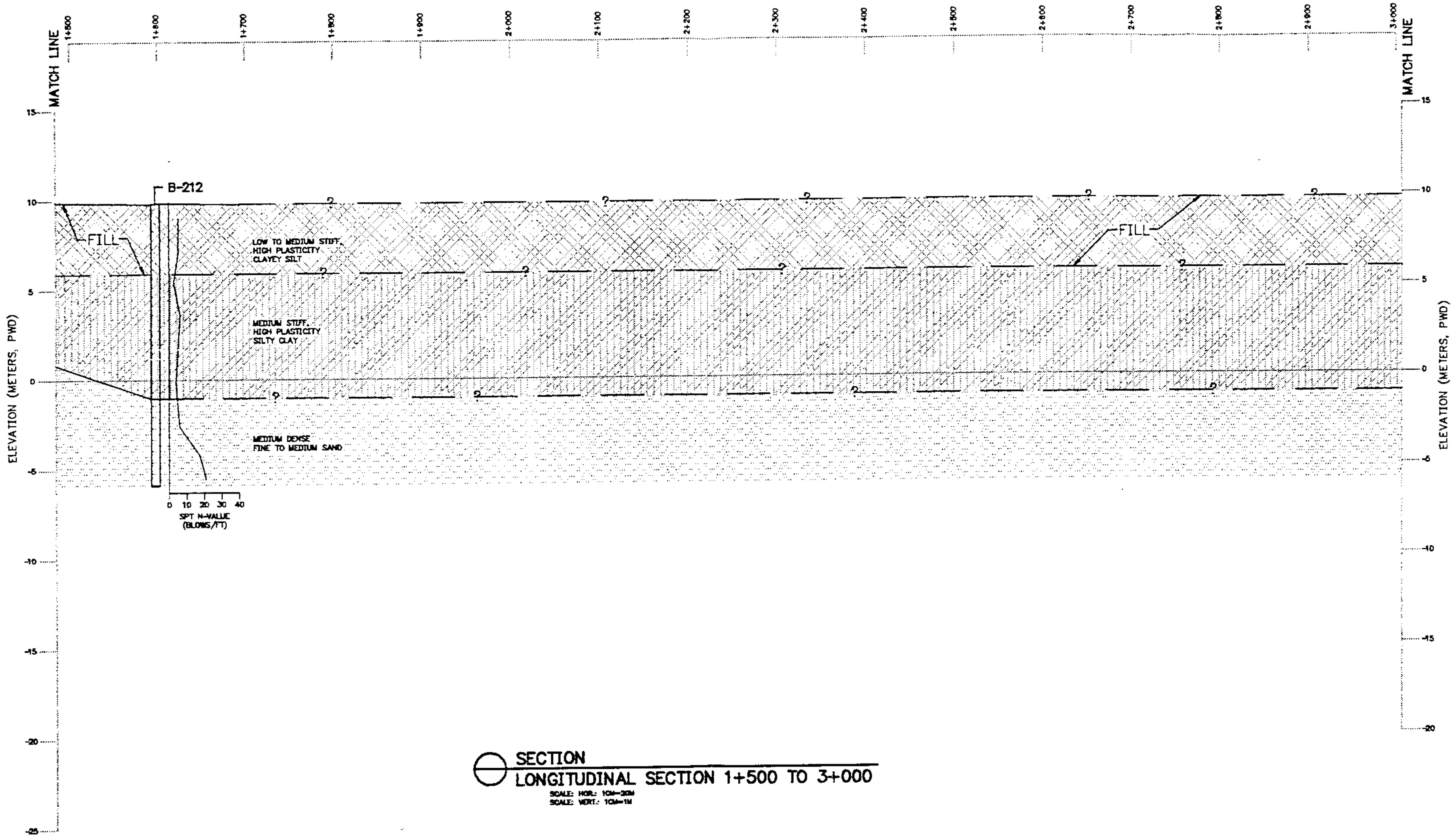


SECTION
LONGITUDINAL SECTION 0+000 TO 1+500

GEOSYNTEC CONSULTANTS

FIGURE NO.	2-10
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	27

DWG: 2043F017 199209101133 SB

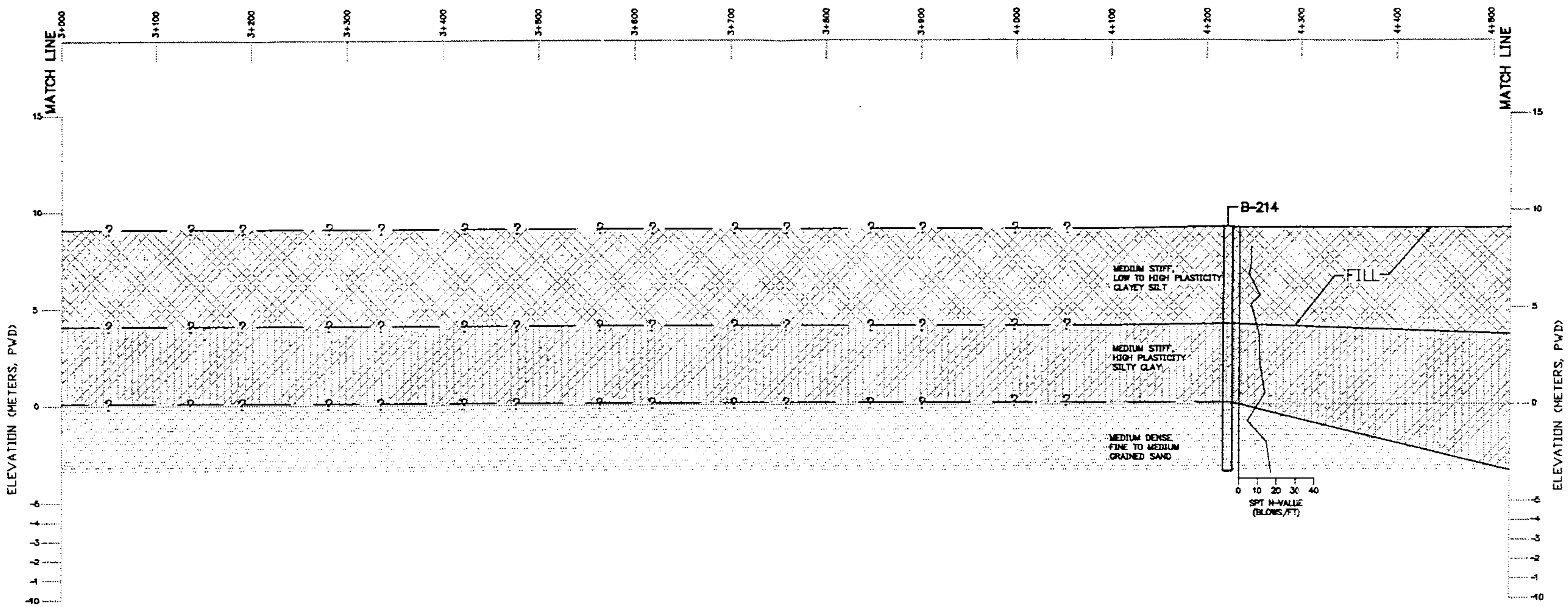


SECTION
LONGITUDINAL SECTION 1+500 TO 3+000
SCALE: HOR.: 1CM=20M
SCALE: VERT.: 1CM=1M

GEOSYNTEC CONSULTANTS

FIGURE NO.	2-11
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	28

DWG: 2043F018 199209101136 SB



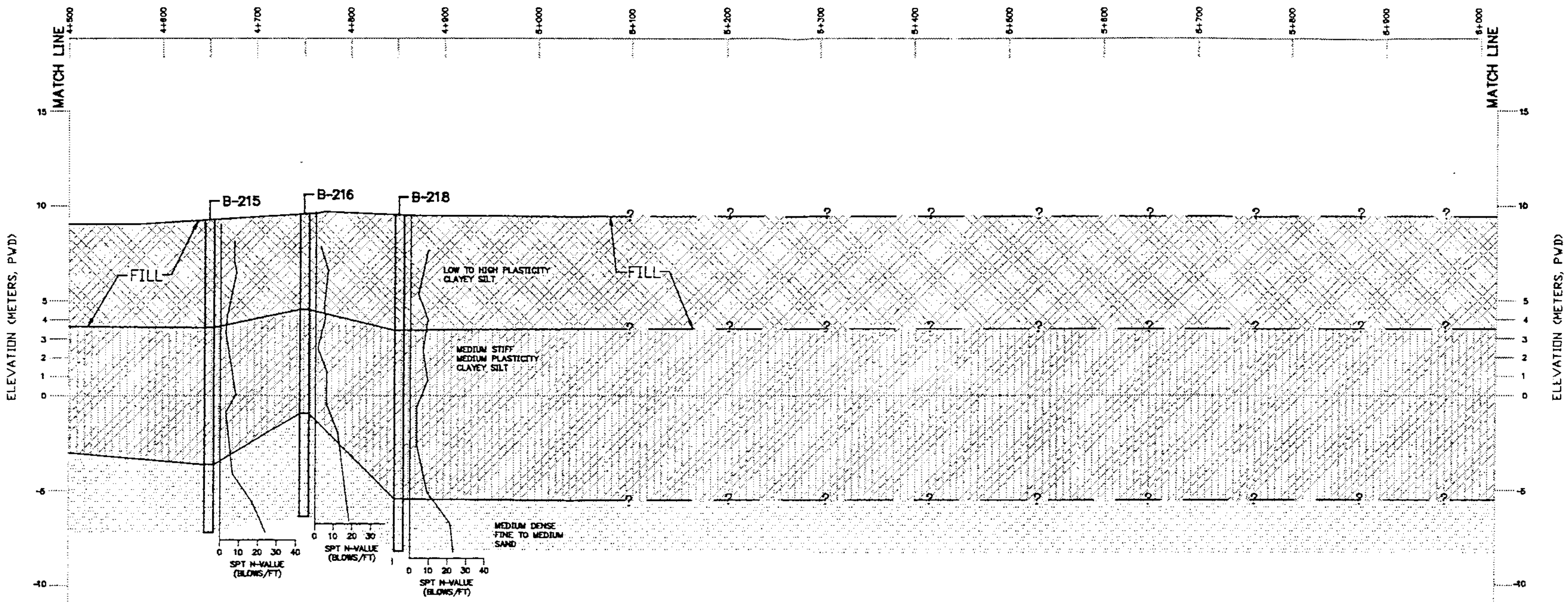
SECTION
LONGITUDINAL SECTION 3+000 TO 4+500

SCALE: HOR.: 1CM=20M
SCALE: VERT.: 1CM=1M

GEOSYNTEC CONSULTANTS

FIGURE NO.	2-12
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	29

DWG: 2043F010 199209101138 SB



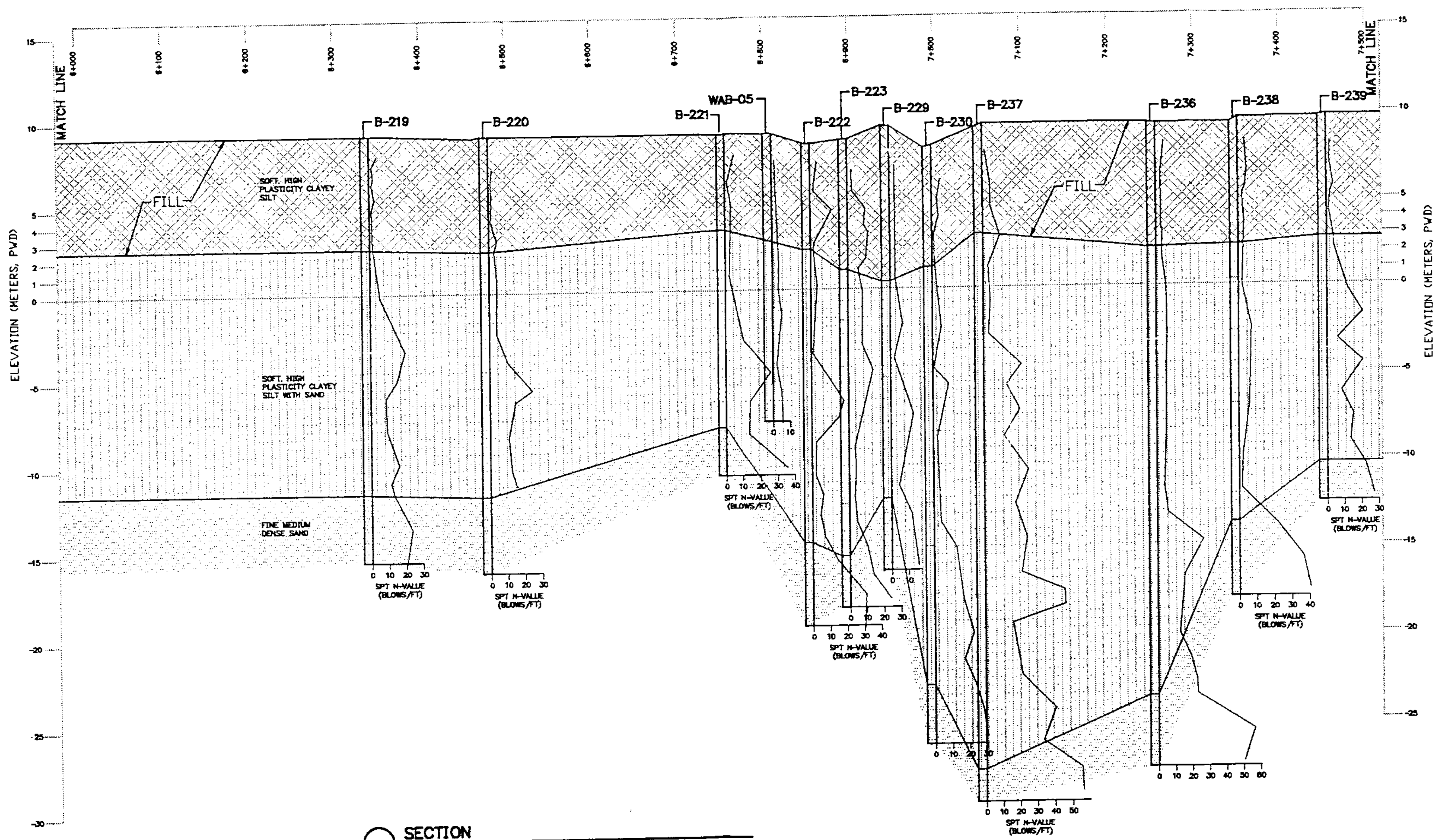
SECTION
LONGITUDINAL SECTION 4+500 TO 6+000

SCALE: HORT. 1CM=20M
SCALE: VERT. 1CM=1M

GeoSyntec Consultants

FIGURE NO.	2-13
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	30

DWG: 2043F016 199209101140 SB

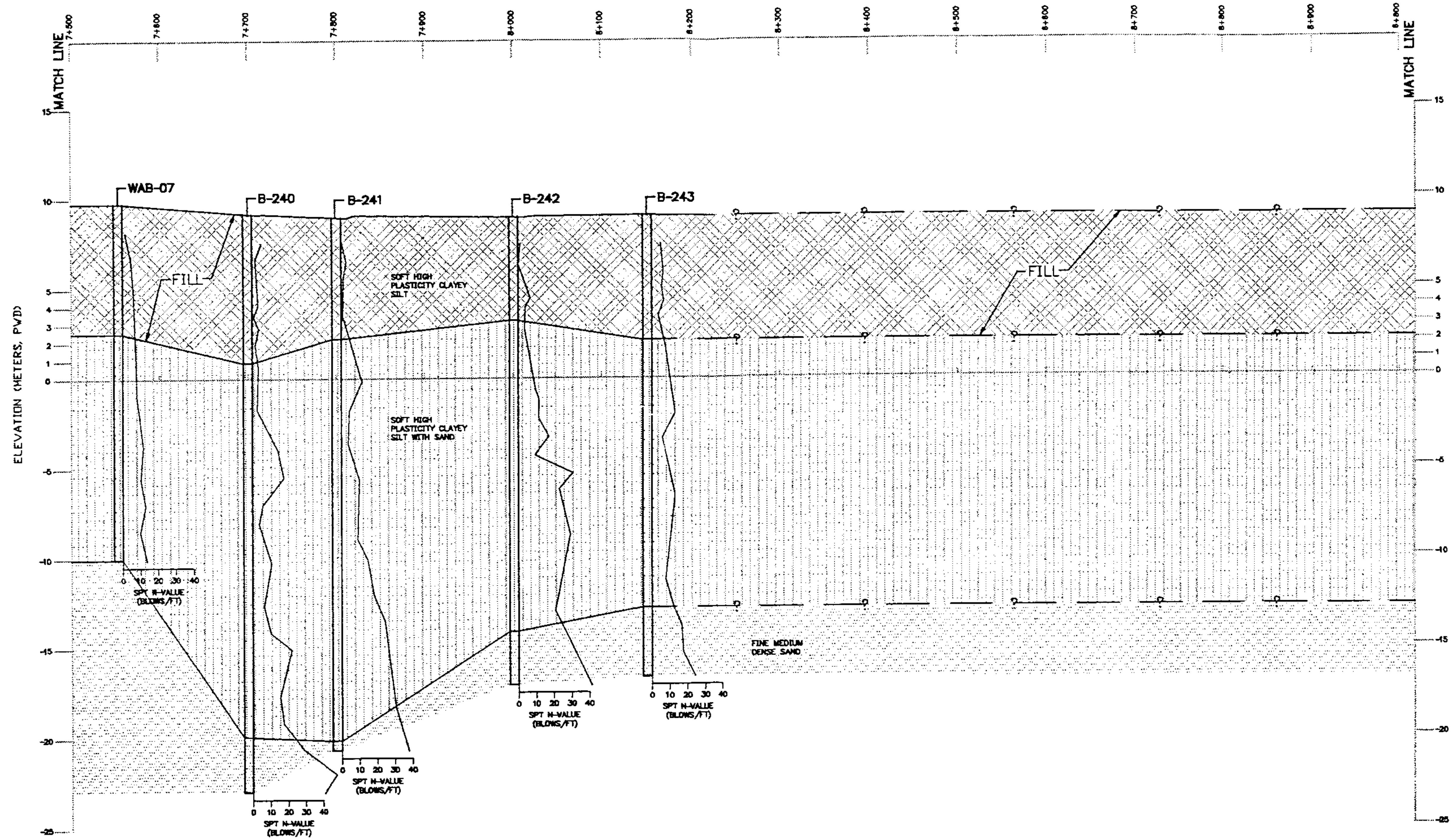


SECTION
LONGITUDINAL SECTION 6+000 TO 7+500
SCALE: HORT. 1CM=20M
SCALE: VERT. 1CM=1M

GeoSyntec Consultants

FIGURE NO.	2-14
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	31

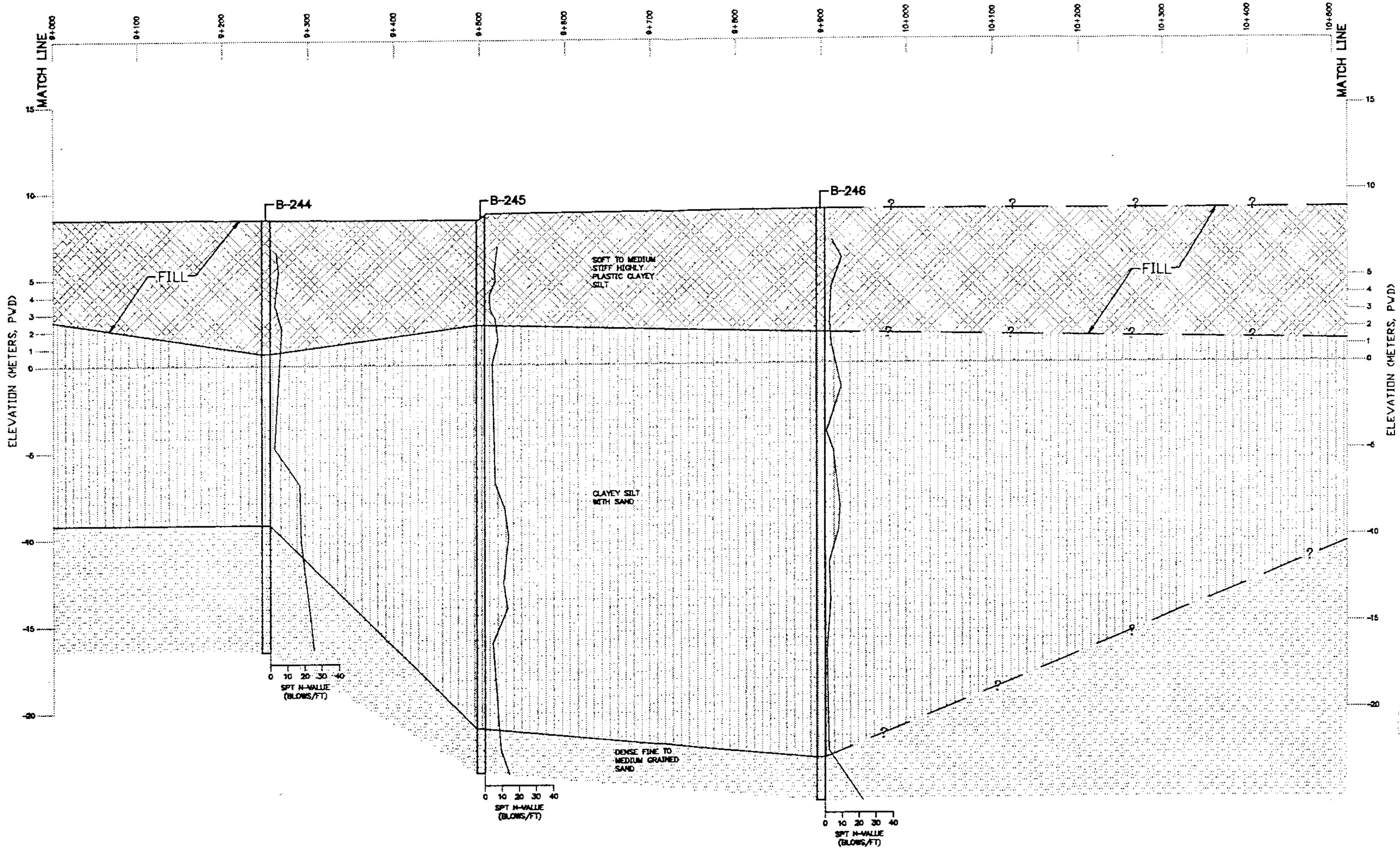
DWG: 2043F011 199209101141 SB



SECTION
LONGITUDINAL SECTION 7+500 TO 9+000

GeoSyntec Consultants	FIGURE NO.	2-15
	PROJECT NO.	FE2043
	DOCUMENT NO.	
	PAGE NO.	32

DWG: 2043F012 199209101143 SB



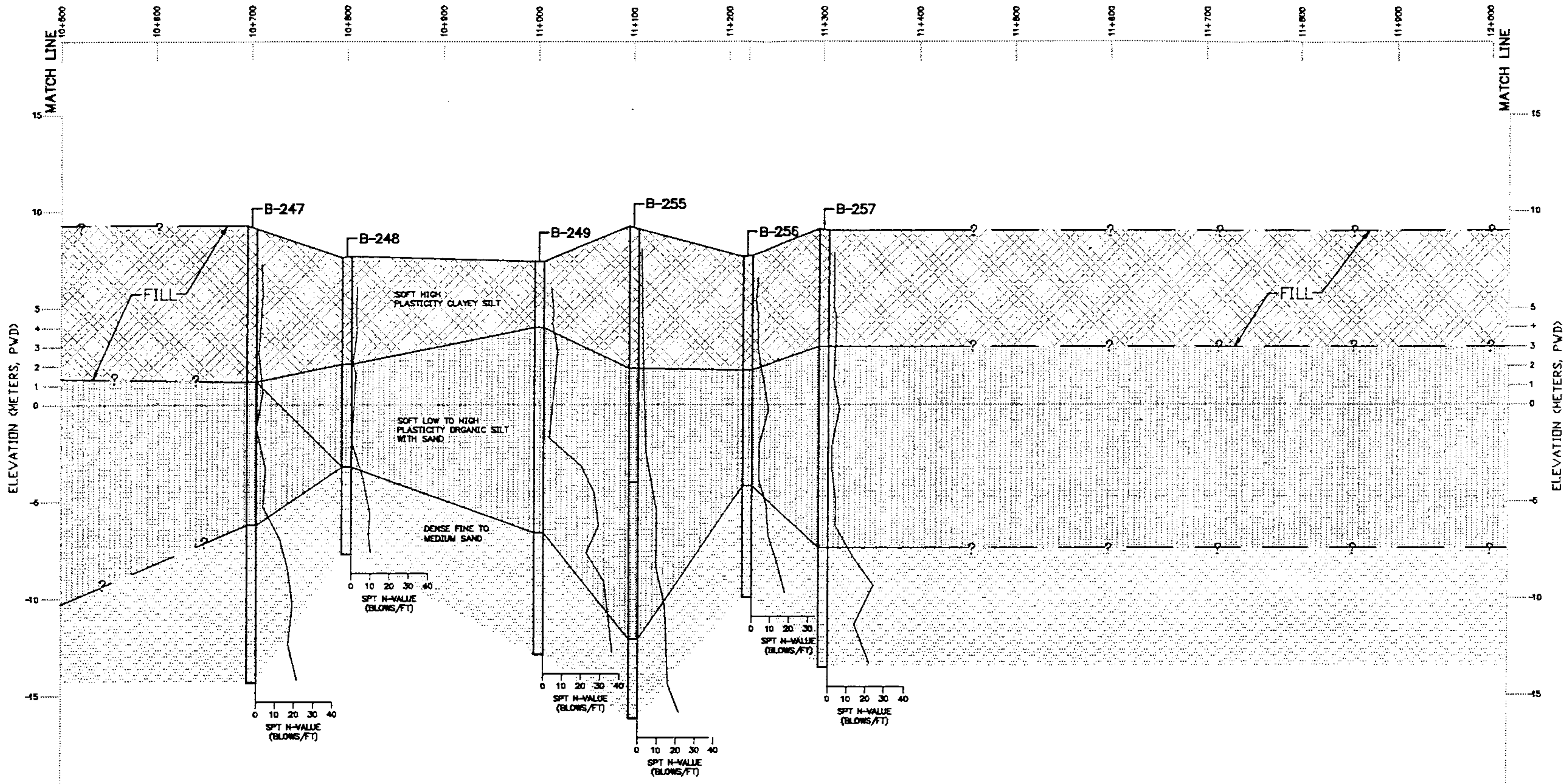
SECTION
LONGITUDINAL SECTION 9+000 TO 10+500
SCALE: HORT. 1CM=20M
SCALE: VERT. 1CM=1M

GeoSyntec Consultants

FIGURE NO.	2-16
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	33

DWG: 2043F013 199209151125 SB

80



SECTION
LONGITUDINAL SECTION 10+500 TO 12+000
SCALE: HOR.: 1CM=20M
SCALE: VERT.: 1CM=1M

GEOSYNTEC CONSULTANTS

FIGURE NO.	2-17
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	34

DWG: 2043F014 199209151128 SB

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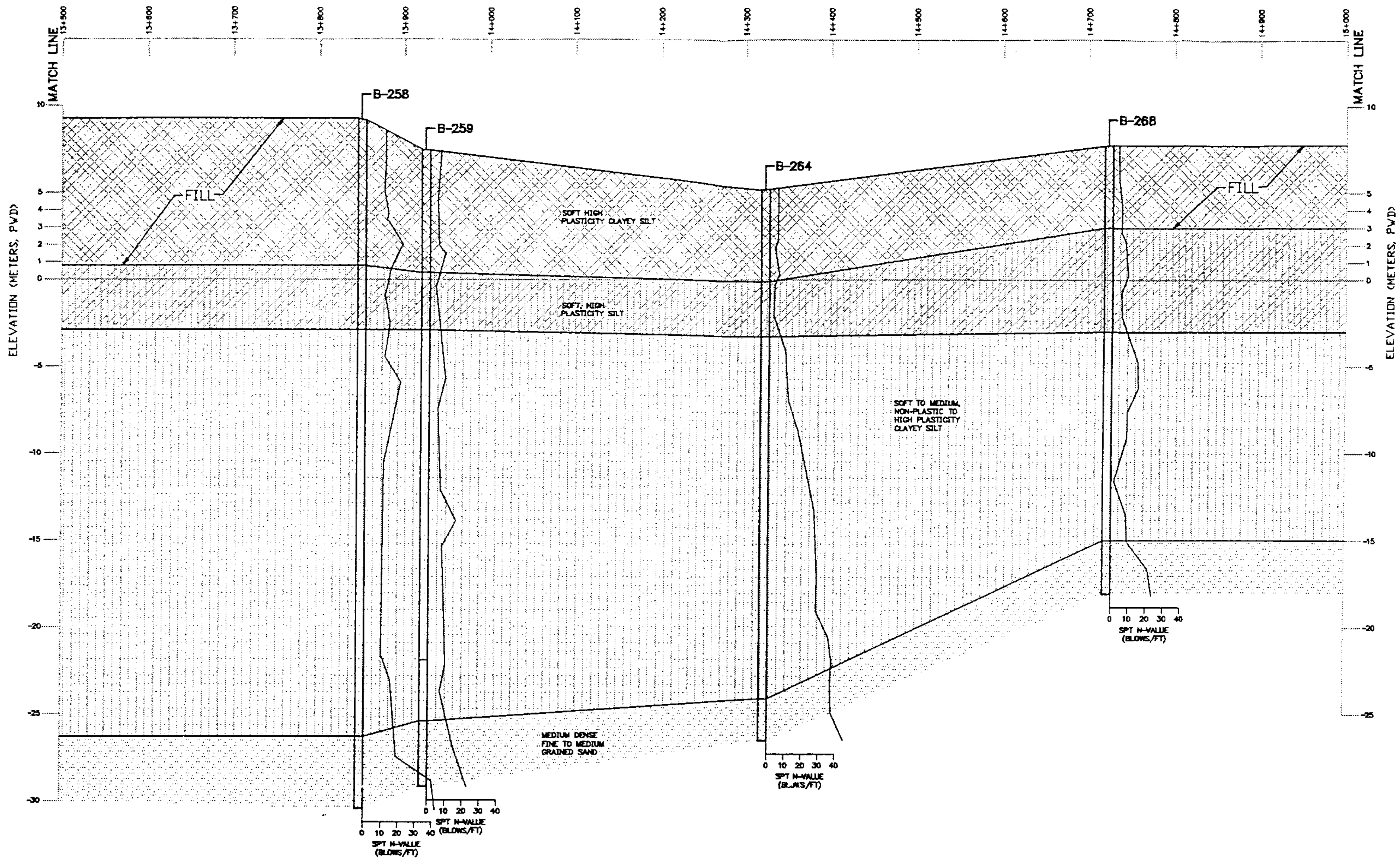

SECTION
LONGITUDINAL SECTION 13+500 TO 15+000

GEOSYNTEC CONSULTANTS

FIGURE NO.	2-18
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	35

DWG: 2043F015 199209101150 SB

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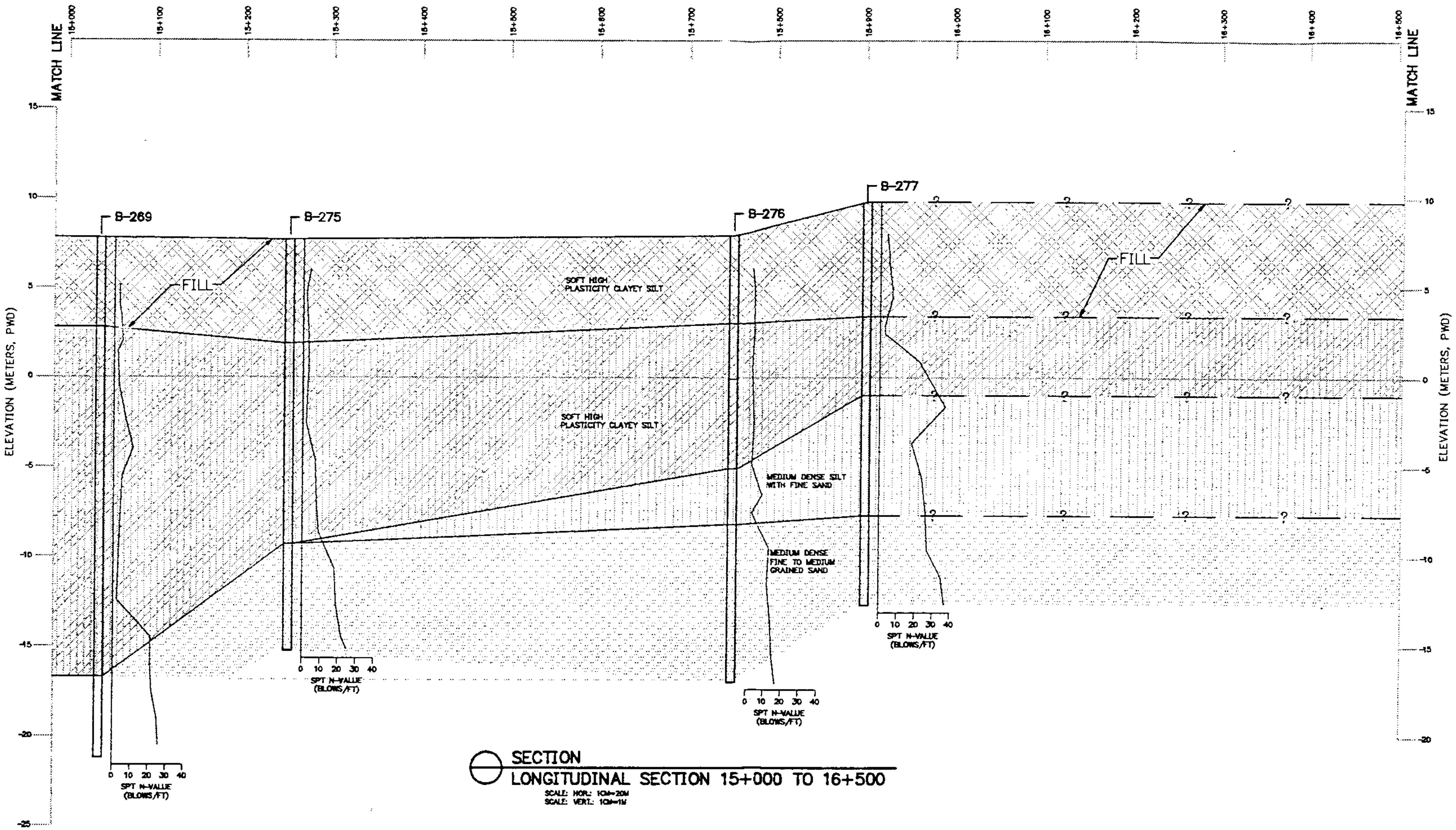
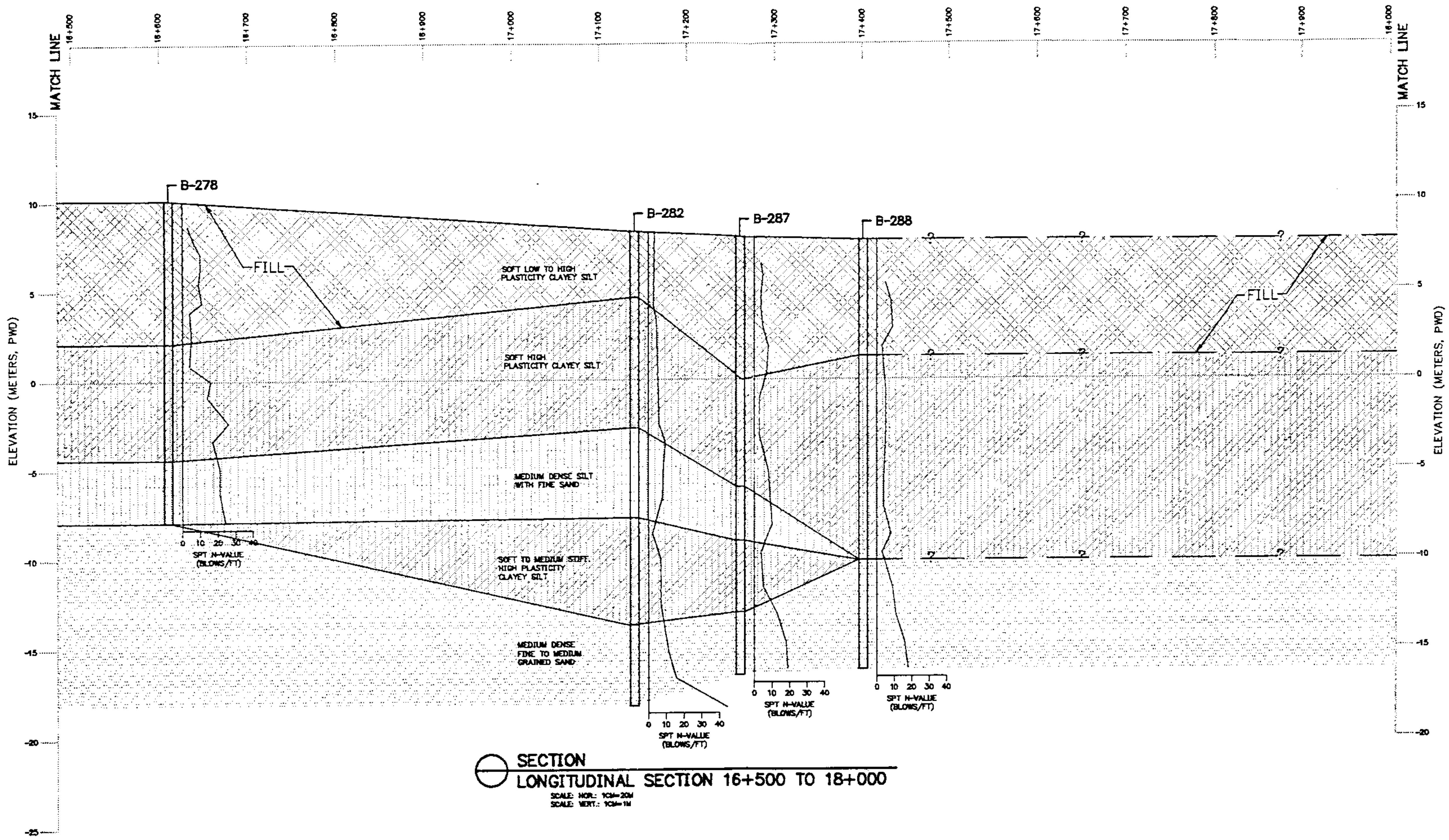


SECTION
LONGITUDINAL SECTION 15+000 TO 16+500
SCALE: HORL: 1CM=20M
SCALE: VERT.: 1CM=1M

GeoSyntec Consultants

FIGURE NO.	2-19
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	36

DWG: 2043F019 199209101152 SB



SECTION

LONGITUDINAL SECTION 16+500 TO 18+000

SCALE: HOR.: 1CM=20M
SCALE: VERT.: 1CM=1M

GEOSYNTEC CONSULTANTS

FIGURE NO.	2-20
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	37

DWG: 2043F020 199209101155 SB

3. LABORATORY TESTING PROGRAM

3.1 Introduction

Laboratory tests were performed on both the undisturbed Shelby-tube and disturbed split-spoon soil samples retrieved from the soil borings to evaluate the geotechnical properties of the embankment and subgrade soils. The laboratory testing program was designed to obtain geotechnical parameters to analyze the stability of the existing embankments and to design appropriate remedial measures. The testing program consisted of the following index (classification), compressibility, and shear strength tests:

- ASTM D 422 "Standard Test Method for Particle-Size Analysis of Soils";
- ASTM D 845 "Standard Test Method for Specific Gravity of Soils";
- ASTM D 2216 "Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures";
- ASTM D 2487-85 "Standard Test Method for Classification of Soils for Engineering Purposes";
- ASTM D 4318 "Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils";
- ASTM D 2974 "Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils";
- ASTM D2435 "Standard Test Method for One-Dimensional Consolidation Properties of Soils;

- ASTM D 2166 "Standard Test Method for Unconfined Compressive Strength of Cohesive Soil";

- ASTM D 4648 "Standard Test Method for Laboratory Miniature Vane Shear Test for Saturated Fine-Grained Clayey Soil;

- ASTM D 4767 "Standard Test Method for Consolidated-Undrained Triaxial Compression Test on Cohesive Soils; and

- ASTM D 3080 "Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions".

The laboratory testing of the soil samples was performed by the following laboratories: (i) M. Ahmed & Associates Ltd., Dhaka; (ii) Foundation Consultants Ltd., Dhaka; (iii) Bangladesh University of Engineering and Technology (BUET), Dhaka; and (iv) River Research Institute (RRI), Faridpur. Each laboratory was assigned a representative number of soil samples and a list of required soil testing programs as detailed in the Field Investigation Plan presented in Attachment A.

Sections 3.2 through 3.6 present a brief description and results of the laboratory testing program.

3.2 Index Properties

3.2.1 Overview

Index property tests performed on the embankment and subgrade soils included particle-size analysis (ASTM D 422), Atterberg Limits (ASTM D 4318), specific gravity (ASTM D 854), and natural moisture content and density determinations (ASTM D 2216). The results of the index property tests are presented in Appendix B.

3.2.2 Grain-Size Distribution

The results of the sieve analyses and hydrometer analyses performed according to ASTM D 422 were used to generate grain-size distributions curves for each soil boring, and are summarized in Appendix B-1. The grain-size distribution curves indicate the embankment and subgrade soils are generally fine-grained soils with 50 percent or more passing through the 0.075 mm sieve size (i.e., U.S. No. 200 sieve), with predominantly silt fractions compared to clay fractions.

3.2.3 Atterberg Limits

Liquid and plastic limits tests were performed on soil samples retrieved from each boring. The results are presented in Appendix B-2, and indicate that the embankment and subgrade soils ranged from non-plastic soils to soils with low to high plasticity. The liquid limit values ranged from as low as 30 percent to as high as 400 percent. For most of these soils, the Atterberg Limits data plotted above the "A" line of the Casagrande plasticity charts as indicated in Appendix B-2.

The results of the plasticity data indicate that the fines fraction of the soils (excluding the non-plastic soils) are predominantly either organic or inorganic clays with low to high plasticity. However, as previously indicated in Section 3.2.2, the fines fractions of the soils were predominantly of silt-size particles. This would make Atterberg Limits data plot below the "A" line of the Casagrande plasticity charts and thereby classify as organic or inorganic silts with low to high plasticity. It is suspected that the discrepancy between the particle-size analysis (ASTM D 422) and Atterberg Limits (ASTM D 4318) test results may be due to whether the exact test protocols as detailed in the ASTM test methods were followed by the laboratories. However, the fact that all four laboratories arrived at similar test values could imply that the deviations might be due to the nature of the embankment and subgrade soils.

3.2.4 Unit Weights

Wet and dry unit weights (or densities) were routinely determined on the undisturbed samples in accordance with ASTM D 2216. The results of the wet and dry unit weights tables for each boring are included in the laboratory data summary presented in Appendix B-3. The wet unit weights of the embankment and subgrade soils typically ranged from 10.2 kN/m^3 (65 lb/ft^3) to about 20 kN/m^3 (127 lb/ft^3). The dry unit weights generally ranged from 9 kN/m^3 (57 lb/ft^3) to about 16 kN/m^3 (102 lb/ft^3).

3.2.5 Moisture Content

Moisture content determinations were routinely performed on both disturbed and undisturbed soil samples from the test borings. The tests were performed in accordance with ASTM D 2216. The results of the moisture content determinations are presented in the laboratory data summary tables in Appendix B-3 and on each boring log in Appendix A. The natural moisture contents ranged from 20 to 40 percent in the embankment soils and from 50 percent to over 100 percent in the subgrade soils.

3.2.6 Specific Gravity

Specific gravity tests of the soil solids were routinely performed in accordance with ASTM D 854. The results of the specific gravity are presented in Appendix B-3 (Laboratory Data Summary) and ranged from 2.0 to about 2.70 for the embankment and subgrade soils.

3.2.7 Organic Content

Organic content determinations were performed in accordance with ASTM D 2974 on soil samples which showed the presence of organics. The percent of organics are summarized on the boring logs in Appendix A, and indicate that for the soil samples tested, the organic content ranged from 2 percent to 64 percent.

3.2.8 Soil Classification

3.2.8.1 Embankment Soils

The results of the soil index analyses presented in Appendix B-1, Appendix B-2, and Appendix B-3 indicate that the embankment soils vary from silty clay to clayey silts interbedded with seams of fine sand and organic silts and clays. The embankment soils ranged from reddish brown to dark gray in color, consisting of very soft to medium stiff, with low to high plasticity. They generally classified as CL and CH according to the Unified Soil Classification System (USCS), as described in ASTM D 2487. These classifications would suggest clay as the predominant fine-size fraction. However, as previously indicated in Sections 3.2.2 and 3.2.3, the hydrometer analyses per ASTM D 422 performed on these soils showed the embankment soils (and also the subgrade soils) to be predominantly of silt-size fine fractions.

3.2.8.2 Subgrade Soils

The subgrade soil profile beneath the western embankment is fairly consistent along the embankment alignment. The subgrade soils consist of an upper 1 m (3 ft) to 30 m (100 ft) layer of soft clayey silt with high plasticity or non-plastic to high plasticity silt, or very soft high plasticity organic clay with silt. This layer is underlain by medium dense silty sand or sand at depth. The average properties of the typical soil types encountered in the borings are discussed below.

The clayey silt or silt layer encountered below the embankment varied in thickness from about 1 m (3 ft) to greater than 30 m (100 ft). These layers tended to be thicker, weaker, and more compressible with seams of organic material near or below areas where the embankment crosses drainage features. The soils ranged from non-plastic silts to high plasticity clayey silts with liquid limits from about 30 to greater than 100. The corresponding plasticity indices ranged from 15 to over 50 for the soils that exhibited some plasticity. The majority of the soils generally classified as CL to CH according to the USCS, even though they were of predominantly silt-size fine fractions.

The upper layer of clayey silt to silt soils were interbedded with very soft, high plasticity organic clays or silts at several locations along the embankment alignment. These soil layers were typically less than 3 m (10 ft) in thickness, and were mostly found in or near drainage features. The organic clay and silt layers were of high plasticity, very weak and highly compressible. The liquid limits ranged from 80 to 400 percent, with corresponding plasticity indices of 60 to 150 percent, respectively. The percent of organic content, as determined by ASTM D 2974 ranged from less than 10 percent to 64 percent.

The upper layer of clayey silt and soils are underlain at depth by silty sand and sand layer. The SPT N-value for these layers ranged from 10 to greater than 30, indicating a relative density of medium dense to dense. The silty sand and sand layers tend to become coarser with depth, and silt content tends to decrease with depth.

4. STABILITY ANALYSES

4.1 Introduction

Stability analyses were performed to evaluate the factor of safety against bearing capacity failure of the embankment. The shear strength parameters for the short-term behavior were calculated from the results of the consolidated undrained triaxial shear test data. For these analyses it was assumed that pore pressure would not dissipate during failure. The long-term behavior was evaluated from the consolidated undrained triaxial shear strength data with pore pressure measurements. Drained shear strength parameters were calculated and used to assess the long-term stability of the embankment.

The analytical approach used to evaluate the stability of the embankment is described in Section 4.2. The methodology used to evaluate the shear strength parameters of the soil is described in Section 4.3. The results of the analyses are summarized and described in Section 4.4.

4.2 Analytical Approach

4.2.1 Conditions Analyzed

The conditions analyzed are illustrated in Figure 4-1. These conditions included:

- shallow stability of the embankment on the city side during a high water period;
- deep stability of the embankment on the city side during a high water period;

- shallow stability of the embankment on the river side during a low water period; and

- deep stability of the embankment on the river side during a low water period.

The stability of these surfaces was evaluated for both short-term undrained, and long-term drained conditions. Short-term undrained behavior is expected until excess pore pressures have dissipated. Long-term drained behavior could occur after excess pore pressures have dissipated.

The city side failure mechanisms are more critical than the river side failure mechanisms because failure could occur during a high water period. If a deep seated failure occurred on the city site during a high water period, the embankment could be breached, resulting in catastrophic failure. For the purpose of this report, catastrophic failure is defined as a failure during a high water period resulting in the embankment being breached. A shallow stability failure on the city side during a period of high water could also result in a breach of the embankment, resulting in catastrophic failure.

Failure of the embankment on the river side during a high water period is unlikely due to the buttressing effect of the water. However, the embankment should also be designed to be stable during low water periods.

It is anticipated that wick drains will be installed in the Class I and Class II areas as described in Sections 5 and 6 of this report. The purpose of the wick drains is to provide vertical drainage, thus reducing the length of the drainage path. Since the undrained shear strength of the soil is closely related to the degree of consolidation of the soil, providing vertical drainage increases the shear strength of the soil with time.

The stability of the embankment is evaluated for various degrees of consolidation. The time required to achieve a given degree of consolidation is a function of the type and spacing of the wick drain and the hydraulic conductivity of the soil. The relationship between the shear strength of the soil and the degree of consolidation is described in Section 4.3.

4.2.2 Method of Analysis

The stability analyses were performed using the computer program PCSTABL6, originally coded at Purdue University. This program uses a limit equilibrium methodology to analyze the shear stresses along an assumed failure surface. The PCSTABL6 program uses a search routine by generating potential failure surfaces and subsequently calculates the factor of safety along these assumed surfaces. The method of limit equilibrium is widely used in geotechnical practice and is a generally accepted method to estimate the factor of safety for stability of an embankment.

The PCSTABL6 computer program uses the Bishop's Modified Method to evaluate the stability. Bishop's Modified Method provides an upper bound solution that is somewhat conservative for very soft subgrades.

A plasticity method of analysis was originally proposed to evaluate a lower bound solution, but was not ultimately included in the scope of work for this project.

4.3 Soil Properties

4.3.1 Undrained Shear Strength

As discussed previously, the undrained shear strength of the soil was measured in consolidated undrained triaxial compression tests. In these tests, undisturbed samples of the soil were placed in a triaxial compression device. The samples were then backpressure saturated to remove air, and prevent overconsolidation of the samples under the applied loads. The B Values of the samples were measured to determine if the samples were fully saturated, where B Value was defined by the following equation:

$$\text{B Value} = \frac{\text{Change in Pore Pressure}}{\text{Change in Confining Stress}} \qquad \text{(Equation 4-1)}$$

If the B Value was greater than 0.9, the sample was assumed to be fully saturated.

Following backpressure saturation and the measurement of a suitable B Value, the sample was consolidated at a specified effective stress. Isotropic consolidation was used in the analyses due to limitations in the capabilities of the testing equipment and due to the very low shear strengths of the soil samples. The samples were consolidated until all excess pore pressures were dissipated. Drainage was then closed and the triaxial shear test was performed at a constant rate of deformation. Pore pressure measurements were obtained during the test to evaluate the drained strength parameters, which are discussed subsequently.

The triaxial shear test was performed by applying vertical loads at a constant vertical strain rate. The vertical loads were measured with a calibrated load cell. The vertical loads and pore pressures were monitored over time until the soil samples failed.

The results of the triaxial shear analyses are presented in Appendix B-6. The triaxial shear data are plotted in p-q diagrams as illustrated in Figure 4-2. As shown in Figure 4-2, the total stress paths are inclined at 45 degrees. The effective stress paths, where effective stress equals the total stress less the pore pressure, curves towards the failure envelope, then follows the failure envelope. Failure of the soil samples occurs when the effective stress path reaches the failure envelope. The shear stress at the point of failure is the undrained shear strength at the prescribed confining stress.

The relationship between the undrained shear strength and the effective stress is illustrated in Figure 4-3. If the Mohr's Circle for the sample is plotted for conditions at failure, the top of the circle is the undrained shear strength of the soil at the specified confining stress, σ_3. A line may then be plotted through the points (σ_{3_1}, S_{u_1}) and (σ_{3_2}, S_{u_2}). The slope of the line is given by the following equation:

$$\psi_u = \tan^{-1}\left[\frac{S_{u_2} - S_{u_1}}{\sigma_{3_2} - \sigma_{3_1}}\right] \qquad \text{(Equation 4-2)}$$

where: ψ_u = the slope of the S_u versus σ_3 line; S_u = the undrained shear strength; and, σ_3 = the total stress. The equation of the line is given by the following equation:

$$S_u = S_{u_o} + \sigma_3 \tan \psi_u \qquad \text{(Equation 4-3)}$$

where: S_u = the undrained shear strength of the soil; S_u = the undrained shear strength of the soil at a total stress of zero (intercept); and σ_3 and ψ_u are as defined previously.

The undrained shear strength of the soil may be related to the consolidated undrained shear strength parameters using the following equation:

$$S_u = C_{cu} \tan (45 + \phi_{cu}/2) + \sigma_{3f} \left[\frac{\tan^2 (45 + \phi_{cu}/2) - 1}{2} \right] \qquad \text{(Equation 4-4)}$$

where: C_{cu} = the consolidated undrained cohesion when the total stress is zero (intercept); σ_{3f} = the total confining stress at failure; ϕ_{cu} = the consolidated undrained friction angle of the soil; and S_u is as defined previously. Combining equations 4-2, 4-3, and 4-4, the undrained shear strength intercept, S_{uo}, and the slope of the S_u versus σ_3 line, ψ_u, are defined by the following equations:

$$S_{u_o} = C_{cu} \tan (45 + \phi_{cu}/2) \qquad \text{(Equation 4-5)}$$

$$\psi_u = \tan^{-1} \left[\frac{\tan^2 (45 + \phi_{cu}/2) - 1}{2} \right] \qquad \text{(Equation 4-6)}$$

where the parameters are as defined previously.

The undrained shear strength may also be related to the effective stress, where effective stress is equal to the total stress minus the pore pressure.

In most cases the effective stress is equal to the overburden pressure exerted by the overlying soil. However, in some areas soil arching may create zones of underconsolidated soil. In these areas, the shear strength is independent of the confining stress under initial conditions. However, construction of the embankment may overcome the effects of arching, resulting in confining stresses equivalent to the height of the embankment plus the depth of the soil.

For initial conditions, the shear strengths used in the analyses were estimated based on the results of the laboratory testing program and field tests, including standard penetration resistances. A summary of the shear strengths from the consolidated undrained triaxial shear tests with pore pressure measurements is presented in Table 4-1. The results of vane shear and cone penetration tests are summarized on the boring logs from previous site investigations in "Preliminary Analysis and Design, Phase I Embankment, Dhaka Integrated Flood Protection Plan", GeoSyntec Consultants, 1991. Using the laboratory and field data and Equations 4-5 and 4-6, the shear strength parameters used in the analyses are summarized in Table 4-2.

4.3.2 Drained Shear Strength

The drained shear strength of a soil is the shear strength after all excess pore pressures have dissipated. In fine-grained soils, such as the soil along much of the embankment, it may take many years to reach drained conditions due to the low hydraulic conductivity of the soil and the length of the drainage path.

The drained strength parameters of the soil were estimate from the data obtained during the consolidated undrained triaxial shear tests with pore pressure measurements. The drained strength parameters estimated from the consolidated undrained triaxial shear tests with pore pressure measurements are summarized in Table 4-1.

4.3.3 Consolidation

The consolidation behavior of the soil was measured in consolidation tests, as summarized in Appendix B-4. The results of the consolidation tests were plotted as shown in Figure 4-4.

The water content may be plotted as a function of log time, as shown in Figure 4-4a. For a given stress increment, the time required to achieve a given degree of consolidation is given by the following equation:

$$U = 100\% \left[1 - \sum_{N=0}^{\infty} \frac{8}{(2N+1)^2 \pi^2} e^{-\left[\frac{(2N+1)^2 \pi^2}{4}\right] \Gamma} \right] \qquad \text{(Equation 4-7)}$$

where: U = the degree of consolidation in percent; and Γ = the time factor. The time factor, Γ, is given by the following equation:

$$\Gamma = \frac{c_v \, t}{H^2} \qquad \text{(Equation 4-8)}$$

where: t = elapsed time; H = the length of the drainage path, and c_v = the coefficient of consolidation. The coefficient of consolidation, c_v, is given by the following equation:

$$c_v = \frac{k\ (1+e_o)}{a_v\ \gamma_w} \qquad \text{(Equation 4-9)}$$

where: k = the hydraulic conductivity of the soil; a_v = the coefficient of compressibility; and, e_o = the initial void ratio of the soil. The coefficient of compressibility, a_v, is given by the following equation:

$$a_v = \frac{de}{d\sigma'} \qquad \text{(Equation 4-10)}$$

where: de = the change in void ratio; and, $d\sigma'$ = the change in effective stress.

Using Equations 4-7 through 4-10, and Equation 4-4, it is possible to derive the undrained shear strength as a function of soil type, effective stress, and degree of consolidation. These data may then be used to evaluate the shear strength as a function of degree of consolidation, which is then related to wick drain spacing and effective stress.

4.4 Stability Analyses

4.4.1 Geometry

The embankment and subgrade soils were divided into horizontal layers corresponding to soil types, and vertical columns corresponding to equivalent increases in vertical effective stress. The shear strengths in each zone were then evaluated using the relationships described in Section 4.3.1 or 4.3.2, and 4.3.3. The geometrical configurations used in the analyses are as shown in Figure 4-5 (Stations 9+750 to 10+100 m), Figure 4-6 (Stations 10+100 to 10+950 m), Figure 4-7 (Stations 13+000 to 13+850 m), and Figure 4-8 (Stations 13+850 to 14+350 m).

4.4.2 Shear Strength

The shear strengths of the soils used in the analyses are summarized in Table 4-3 (Stations 9+750 to 10+100 m). Table 4-4 (Stations 10+100 to 10+950 m), Table 4-5 (Stations 13+000 to 13+850 m), and Table 4-6 (Stations 13+850 m to 14+350 m). As shown in Tables 4-3 through 4-6, the shear strengths of the soil were calculated based on degree of consolidation, confining stress, and position under the embankment. Analyses were performed for existing conditions, varying degrees of consolidation (10, 20, 30, 50, 70, and 90 percent), and for short-term (undrained) and long-term (drained) conditions.

4.4.3 Results of Slope Stability Analyses

The results of the slope stability analyses are summarized in Table 4-7. Four types of analyses were performed in each section: (i) deep river side; (ii) deep city side; (iii) shallow river side; and (iv) shallow city side. Circular failure surfaces were assumed for all four types of analyses.

As shown in Table 4-7, the existing factors of safety vary from about 0.72 to 2.05. The results of the analyses are shown graphically in Appendix C-1. Consolidated analytical results are shown in Figures 4-9 through 4-13.

Between Stations 9+750 and 10+100 m (Figure 4-9), the minimum factor of safety results from a deep river side failure surface when the average degree of consolidation is less than about 33 percent. When the average degree of consolidation is greater than 33 percent, the minimum factor of safety results from shallow failure surfaces on the river side. A minimum factor of safety of 1.2 is achieved when the degree of consolidation reaches 90 percent.

Between Stations 10+100 and 10+950 m (Figure 4-10), the existing factor of safety for a shallow river side failure surface is about 0.84. The factor of safety increases to about 1.2 at an average degree of consolidation of about 90 percent.

The relationships between the average degree of consolidation and the factors of safety for the various surfaces between Stations 13+000 and 13+850 m are shown in Figure 4-11. The minimum existing factor of safety is 0.84 for a shallow river side failure. The factor of safety increases to about 1.2 when the average degree of consolidation for the soils increases to about 90 percent.

The relationships between the average degree of consolidation and the factors of safety for the various surfaces between Stations 13+850 and 14+350 m are shown in Figures 4-12 and 4-13. The minimum initial factors of safety is 0.83 for the shallow river side failure surface. The analyses indicate that the embankment cannot be constructed to the desired final elevations unless the embankment construction is staged. The first stage of construction is the excavation of the embankment to an elevation of 5.0 m (16 ft) PWD. Wick drains would then be installed and the embankment would be constructed to an elevation of about 7.5 m (25 ft) PWD using good soils and compaction techniques. The soil would then be allowed to consolidate to an average of 90 percent consolidation prior to construction of the remaining portions of the embankment. Stage 2 construction would raise the embankment from an elevation of 7.5 m (25 ft) PWD to the final elevations. The minimum factors of safety after

Stage 1 construction and consolidation would be about 1.18. The minimum factors of safety after Stage 2 construction and consolidation would be about 1.24.

The results of the slope stability analyses are summarized in Table 4-8.

TABLE 4-1. SHEAR STRENGTH DATA FROM TRIAXIAL SHEAR TESTS

BORING NO.	SAMPLE NO.	ELEV (m)	c' (kPa)	PHI' (DEG)	C tot (kPa)	PHI tot (DEG)	SPT N VALUE	PLASTICITY RATING	SOIL TYPE
259	UD-9	-4.6	1.52	30.56	-0.32	18.91	6	LOW	CLAYEY SILT
259	UD-17	-13	-1.12	35.92	6.33	23.04	10	LOW	CLAYEY SILT
260	UD-9	-3.8	6.64	19.81	10.85	10.23	2	HIGH	CLAYEY SILT
261	UD-5	-3.1	-7.16	34.34	-2.59	16.55	3	HIGH	SILTY CLAY
261	UD-6	-3.7	23.23	45.58	12.54	14.70	2	HIGH	SILTY CLAY
262	UD-8	-4	0.00	31.46	0.00	25.66	4	HIGH	SILTY CLAY
263	UD-7	-5.8	5.66	33.84	14.82	26.39	4	LOW	CLAYEY SILT W/ SA
267	UD-4	-2.3	-2.76	29.58	2.06	12.63	2	HIGH	SILTY CLAY
267	UD-4	-2.3	0.00	28.14	0.00	13.15	2	HIGH	SILTY CLAY
271	UD-3	0	-1.58	29.36	7.11	11.83	2	HIGH	SILTY CLAY FILL
271	UD-6	-9.5	-1.16	24.91	4.32	10.49	2	HIGH	SILTY CLAY
271	UD-6	-9.5	0.00	24.28	0.00	11.64	2	HIGH	SILTY CLAY
273	UD-5	-5.5	4.45	33.32	-1.15	19.03	4	HIGH	SILTY CLAY
276	UD-6	-1.8	0.61	22.54	5.43	11.31	4	HIGH	CLAYEY SILT
282	UD-5	-1.5	15.07	28.06	14.27	15.77	6	HIGH	SILTY CLAY
284	UD-4	-3.3	0.00	49.92	0.00	23.24	20	HIGH	SILTY CLAY FILL
286	UD-4	-4.5	-0.93	39.71	9.35	14.09	2	LOW	SILTY CLAY
286	UD-6	-10.5	1.66	30.79	24.06	8.17	3	HIGH	SILTY CLAY

Table 4-2. Shear Strength Data Used in the Stability Analyses.

STATIONS (m)	ϕ_{cu} (deg)	C_{cu} (kPa)	ψ_u (deg)	S_{uo} (kPa)
9 + 750 to 10 + 100				
Embankment	11	5.6	13.3	6.8
Very Soft Silt	10	4	--	--
10 + 100 to 10 + 950				
Embankment and Subgrade	11	5.6	13.3	6.8
Dense Sand	34	0	--	--
13 + 000 to 13 + 850				
Embankment and Subgrade	11	5.6	13.3	6.8
Dense Sand	34	0	--	--
13 + 850 to 14 + 350				
Embankment and Subgrade	12	3.0	14.7	3.7

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Table 4-3

EVALUATION OF CONSTRUCTION INDUCED STRESS
SEGMENT 1, STA 9+750 TO 10+100

	SOIL UNIT WEIGHT	17					
UPPER SOIL	COHESION	5.6	5.497112	=a		HEIGHT OF EMBANKMENT	
	TOTAL FRICTION CU	11	0.188543	=PSI (RADIANS)		CAUSING CONSOLIDATION (m)	5.5
SOFT LAYER	COHESION	4	3.939231	=a		ARCH THICKNESS (m)	18
	TOTAL FRICTION CU	10	0.171934	=PSI (RADIANS)			

SOIL UNIT	INFLUENCE FACTOR	SIG'v INIT (KN/m2)	Su (KN/m2)	DEL SIG (KN/m2)	U=10% SIG'v (KN/m2)	Su (KN/m2)	U=20% SIG'v (KN/m2)	Su (KN/m2)	U=30% SIG'v (KN/m2)	Su (KN/m2)	U=50% SIG'v (KN/m2)	Su (KN/m2)	U=70% SIG'v (KN/m2)	Su (KN/m2)	U=90% SIG'v (KN/m2)	Su (KN/m2)
1	NA	NA	20	0	0	20	0	20	0	20	0	20	0	20	0	20
2	NA	NA	20	0	0	20	0	20	0	20	0	20	0	20	0	20
3	0	42.5	16.81494	0	42.5	16.81494	42.5	16.81494	42.5	16.81494	42.5	16.81494	42.5	16.81494	42.5	16.81494
4	0.05	42.5	16.81494	4.675	42.9675	16.92517	43.435	17.03541	43.9025	17.14565	44.8375	17.36612	45.7725	17.5866	46.7075	17.80707
5	0.99	42.5	16.81494	92.565	51.7565	18.99764	61.013	21.18034	70.2695	23.36304	88.7825	27.72845	107.2955	32.09386	125.8085	36.45926
6	0.01	73.5	24.1248	0.935	73.5935	24.14685	73.687	24.1689	73.7805	24.19095	73.9675	24.23504	74.1545	24.27914	74.3415	24.32323
7	0.125	73.5	24.1248	11.6875	74.66875	24.4004	75.8375	24.67599	77.00625	24.95158	79.34375	25.50277	81.68125	26.05396	84.01875	26.60515
8	0.98	73.5	24.1248	91.63	82.663	26.28546	91.826	28.44611	100.989	30.60677	119.315	34.92808	137.641	39.24939	155.967	43.5707
9	0.02	120.3	35.16034	1.87	120.487	35.20444	120.674	35.24853	120.861	35.29263	121.235	35.38082	121.609	35.46901	121.983	35.5572
10	0.215	120.3	35.16034	20.1025	122.3103	35.63437	124.3205	36.10839	126.3308	36.58241	130.3513	37.53045	134.3718	38.47849	138.3923	39.42654
11	0.94	120.3	35.16034	87.89	129.089	37.23281	137.878	39.30527	146.667	41.37774	164.245	45.52267	181.823	49.6676	199.401	53.81253
12	0.03	17.8	8.507476	101.805	27.9805	10.64679	38.161	12.7861	48.3415	14.92542	68.7025	19.20404	89.0635	23.48267	109.4245	27.76129
13	0.275	17.8	8.507476	124.7125	30.27125	11.12816	42.7425	13.74885	55.21375	16.36954	80.15625	21.61091	105.0988	26.85229	130.0413	32.09366
14	0.89	17.8	8.507476	182.215	36.0215	12.33651	54.243	16.16555	72.4645	19.99458	108.9075	27.65265	145.3505	35.31072	181.7935	42.96879
15	0.075	17.8	8.507476	106.0125	28.40125	10.73521	39.0025	12.96293	49.60375	15.19066	70.80625	19.64612	92.00875	24.10158	113.2113	28.55704
16	0.33	17.8	8.507476	129.855	30.7855	11.23623	43.771	13.96498	56.7565	16.69373	82.7275	22.15123	108.6985	27.60873	134.6695	33.06624
17	0.76	17.8	8.507476	170.06	34.806	12.08109	51.812	15.6547	68.818	19.22831	102.83	26.37554	136.842	33.52278	170.854	40.66999

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EVALUATION OF CONSTRUCTION INDUCED STRESS
SEGMENT 2, STA 10+100 TO 10+950 (CLASS II)

SOIL UNIT WEIGHT	17			HEIGHT OF EMBANKMENT	
COHESION	5.6	5.49711	=a	CAUSING CONSOLIDATION (m	5.5
TOTAL FRICTION CU	11	0.18854	=PSI (RADIANS)		

					U=10%		U=20%		U=30%		U=50%		U=70%		U=90%	
SOIL UNI	INFLUENC FACTOR	SIG'v INIT (KN/m2)	Su (KN/m2)	DEL SIG (KN/m2)	SIG'v (KN/m2)	Su (KN/m2)	SIG'v (KN/m2)	Su (KN/m2)	SIG'v (KN/m2)	Su (KN/m2)	SIG'v (KN/m2)	Su (KN/m2)	SIG'v (KN/m2)	Su (KN/m2)	SIG'v (KN/m2)	Su (KN/m2)
1	NA	NA	20	0	0	20	0	20	0	20	0	20	0	20	0	20
2	0	42.5	16.8149	0	42.5	16.8149	42.5	16.8149	42.5	16.8149	42.5	16.8149	42.5	16.8149	42.5	16.8149
3	0.03	17	10.802	2.805	17.2805	10.8681	17.561	10.9343	17.8415	11.0004	18.4025	11.1327	18.9635	11.265	19.5245	11.3973
4	0.97	17	10.802	90.695	26.0695	12.9406	35.139	15.0792	44.2085	17.2178	62.3475	21.495	80.4865	25.7722	98.6255	30.0495
5	1	17	10.802	93.5	26.35	13.0067	35.7	15.2115	45.05	17.4162	63.75	21.8257	82.45	26.2352	101.15	30.6447
6	0.065	44.8	17.3573	6.0775	45.4078	17.5006	46.0155	17.6439	46.6233	17.7872	47.8388	18.0738	49.0543	18.3604	50.2698	18.6471
7	0.93	44.8	17.3573	86.955	53.4955	19.4077	62.191	21.4581	70.8865	23.5085	88.2775	27.6094	105.669	31.7102	123.06	35.811

Table 4-5

EVALUATION OF CONSTRUCTION INDUCED STRESS
SEGMENT 3, 13+000 TO 13+850 (CLASS II)

SOIL UNIT WEIGHT	17			HEIGHT OF EMBANKMENT	
COHESION	5.6	5.49711	=a	CAUSING CONSOLIDATION (m	5.5
TOTAL FRICTION CU	11	0.18854	=PSI (RADIANS)		

					U=10%		U=20%		U=30%		U=50%		U=70%		U=90%	
SOIL UNI	INFLUENC FACTOR	SIG'v INIT (KN/m2)	Su (KN/m2)	DEL SIG (KN/m2)	SIG'v (KN/m2)	Su (KN/m2)	SIG'v (KN/m2)	Su (KN/m2)	SIG'v (KN/m2)	Su (KN/m2)	SIG'v (KN/m2)	Su (KN/m2)	SIG'v (KN/m2)	Su (KN/m2)	SIG'v (KN/m2)	Su (KN/m2)
1	NA	NA	20	0	0	20	0	20	0	20	0	20	0	20	0	20
2	0	42.5	16.8149	0	42.5	16.8149	42.5	16.8149	42.5	16.8149	42.5	16.8149	42.5	16.8149	42.5	16.8149
3	0.05	42.5	16.8149	4.675	42.9675	16.9252	43.435	17.0354	43.9025	17.1456	44.8375	17.3661	45.7725	17.5866	46.7075	17.8071
4	0.95	42.5	16.8149	88.825	51.3825	18.9094	60.265	21.004	69.1475	23.0985	86.9125	27.2875	104.678	31.4765	122.443	35.6656
5	0.99	42.5	16.8149	92.565	51.7565	18.9976	61.013	21.1803	70.2695	23.363	88.7825	27.7284	107.296	32.0939	125.809	36.4593
6	0.01	73.5	24.1248	0.935	73.5935	24.1469	73.687	24.1689	73.7805	24.1909	73.9675	24.235	74.1545	24.2791	74.3415	24.3232
7	0.12	73.5	24.1248	11.22	74.622	24.3894	75.744	24.6539	76.866	24.9185	79.11	25.4477	81.354	25.9768	83.598	26.5059
8	0.87	73.5	24.1248	81.345	81.6345	26.0429	89.769	27.9611	97.9035	29.8792	114.173	33.7155	130.442	37.5517	146.711	41.388
9	0.98	73.5	24.1248	91.63	82.663	26.2855	91.826	28.4461	100.989	30.6068	119.315	34.9281	137.641	39.2494	155.967	43.5707
10	0.02	120.3	35.1603	1.87	120.487	35.2044	120.674	35.2485	120.861	35.2926	121.235	35.3808	121.609	35.469	121.983	35.5572
11	0.215	120.3	35.1603	20.1025	122.31	35.6344	124.321	36.1084	126.331	36.5824	130.351	37.5305	134.372	38.4785	138.392	39.4265
12	0.77	120.3	35.1603	71.995	127.5	36.858	134.699	38.5557	141.899	40.2533	156.298	43.6486	170.697	47.0439	185.096	50.4393
13	0.94	120.3	35.1603	87.89	129.089	37.2328	137.878	39.3053	146.667	41.3777	164.245	45.5227	181.823	49.6676	199.401	53.8125
14	0.03	177.8	48.719	2.805	178.081	48.7851	178.361	48.8513	178.642	48.9174	179.203	49.0497	179.764	49.182	180.325	49.3143
15	0.27	177.8	48.719	25.245	180.325	49.3143	182.849	49.9095	185.374	50.5048	190.423	51.6954	195.472	52.8859	200.521	54.0765
16	0.7	177.8	48.719	65.45	184.345	50.2623	190.89	51.8056	197.435	53.3489	210.525	56.4356	223.615	59.5222	236.705	62.6089
17	0.89	177.8	48.719	83.215	186.122	50.6812	194.443	52.6434	202.765	54.6057	219.408	58.5301	236.051	62.4546	252.694	66.379
18	NA	C=0, PHI=34														

Table 4-6

EVALUATION OF CONSTRUCTION INDUCED STRESS
SEGMENT 4, 13+850 TO 14+350 (CLASS I)
STAGE 1 ELEV 5.5 EMBANKMENT

SOIL UNIT WEIGHT	17			HEIGHT OF EMBANKMENT	
COHESION	3	2.934443	=a	CAUSING CONSOLIDATION (m)	3.5
TOTAL FRICTION CU	12	0.204991	=PSI (RADIANS)		

SOIL UNIT	INFLUENCE FACTOR	SIG'v INIT (KN/m2)	Su (KN/m2)	DEL SIG (KN/m2)	U=10% SIG'v (KN/m2)	Su (KN/m2)	U=20% SIG'v (KN/m2)	Su (KN/m2)	U=30% SIG'v (KN/m2)	Su (KN/m2)	U=50% SIG'v (KN/m2)	Su (KN/m2)	U=70% SIG'v (KN/m2)	Su (KN/m2)	U=90% SIG'v (KN/m2)	Su (KN/m2)
1	NA	NA	20	0	0	20	0	20	0	20	0	20	0	20	0	20
2	0	42.5	14.86032	0	42.5	14.86032	42.5	14.86032	42.5	14.86032	42.5	14.86032	42.5	14.86032	42.5	14.86032
3	0.05	42.5	14.86032	2.975	42.7975	14.93841	43.095	15.0165	43.3925	15.09459	43.9875	15.25077	44.5825	15.40695	45.1775	15.56313
4	0.95	42.5	14.86032	56.525	48.1525	16.34402	53.805	17.82772	59.4575	19.31142	70.7625	22.27882	82.0675	25.24622	93.3725	28.21362
5	0.99	42.5	14.86032	58.905	48.3905	16.4065	54.281	17.95267	60.1715	19.49884	71.9525	22.59118	83.7335	25.68352	95.5145	28.77586
6	0.01	73.5	22.99738	0.595	73.5595	23.01299	73.619	23.02861	73.6785	23.04423	73.7975	23.07546	73.9165	23.1067	74.0355	23.13794
7	0.12	73.5	22.99738	7.14	74.214	23.18479	74.928	23.3722	75.642	23.55962	77.07	23.93445	78.498	24.30928	79.926	24.68411
8	0.87	73.5	22.99738	51.765	78.6765	24.35613	83.853	25.71489	89.0295	27.07364	99.3825	29.79116	109.7355	32.50867	120.0885	35.22618
9	0.98	73.5	22.99738	58.31	79.331	24.52793	85.162	26.05848	90.993	27.58903	102.655	30.65014	114.317	33.71125	125.979	36.77235
10	0.02	120.3	35.2817	1.19	120.419	35.31293	120.538	35.34417	120.657	35.3754	120.895	35.43788	121.133	35.50035	121.371	35.56282
11	0.215	120.3	35.2817	12.7925	121.5793	35.61748	122.8585	35.95327	124.1378	36.28905	126.6963	36.96062	129.2548	37.63219	131.8133	38.30376
12	0.77	120.3	35.2817	45.815	124.8815	36.48427	129.463	37.68685	134.0445	38.88943	143.2075	41.29458	152.3705	43.69974	161.5335	46.10489
13	0.94	120.3	35.2817	55.93	125.893	36.74978	131.486	38.21786	137.079	39.68594	148.265	42.6221	159.451	45.55827	170.637	48.49443
14	0.03	177.8	50.37461	1.785	177.9785	50.42147	178.157	50.46832	178.3355	50.51517	178.6925	50.60888	179.0495	50.70259	179.4065	50.7963
15	0.27	177.8	50.37461	16.065	179.4065	50.7963	181.013	51.21798	182.6195	51.63966	185.8325	52.48303	189.0455	53.32639	192.2585	54.16976
16	0.7	177.8	50.37461	41.65	181.965	51.46786	186.13	52.56112	190.295	53.65437	198.625	55.84087	206.955	58.02738	215.285	60.21388
17	0.89	177.8	50.37461	52.955	183.0955	51.7646	188.391	53.1546	193.6865	54.54459	204.2775	57.32457	214.8685	60.10456	225.4595	62.88454
18	NA	C=0, PHI=34														

STAGE II, AFTER 90% CONSOLIDATION UNDER STAGE 1 LOAD EMBANKMENT RAISED TO 7.5-m
FILL PLACED FULL HEIGHT AFTER CONSOLIDATION UNDER 7.5 m EMBANKMENT

SOIL UNIT	INFLUENCE FACTOR	SIG'v INIT (KN/m2)	Su (KN/m2)	DEL SIG (KN/m2)	U=10% SIG'v (KN/m2)	Su (KN/m2)	U=20% SIG'v (KN/m2)	Su (KN/m2)	U=30% SIG'v (KN/m2)	Su (KN/m2)	U=50% SIG'v (KN/m2)	Su (KN/m2)	U=70% SIG'v (KN/m2)	Su (KN/m2)	U=90% SIG'v (KN/m2)	Su (KN/m2)
1	NA	NA	20	0	0	20	0	20	0	20	0	20	0	20	0	20
2	0	42.5	14.86032	0	42.5	14.86032	42.5	14.86032	42.5	14.86032	42.5	14.86032	42.5	14.86032	42.5	14.86032
3	0.05	45.1775	15.56313	1.9975	45.37725	15.61556	45.577	15.66799	45.77675	15.72042	46.17625	15.82529	46.57575	15.93015	46.97525	16.03501
4	0.95	93.3725	28.21362	37.9525	97.16775	29.20982	100.963	30.20601	104.7583	31.20221	112.3488	33.19461	119.9393	35.187	127.5298	37.1794
5	0.99	95.5145	28.77586	39.5505	99.46955	29.81401	103.4246	30.85215	107.3797	31.89029	115.2898	33.96658	123.1999	36.04286	131.11	38.11915
6	0.01	74.0355	23.13794	0.3995	74.07545	23.14842	74.1154	23.15891	74.15535	23.1694	74.23525	23.19037	74.31515	23.21134	74.39505	23.23231
7	0.12	79.926	24.68411	4.794	80.4054	24.80994	80.8848	24.93578	81.3642	25.06161	82.323	25.31328	83.2818	25.56496	84.2406	25.81663
8	0.87	120.0885	35.22618	34.7565	123.5642	36.13849	127.0398	37.0508	130.5155	37.9631	137.4668	39.78772	144.4181	41.61233	151.3694	43.43695
9	0.98	125.979	36.77235	39.151	129.8941	37.80001	133.8092	38.82767	137.7243	39.85532	145.5545	41.91064	153.3847	43.96595	161.2149	46.02126
10	0.02	121.371	35.56282	0.799	121.4509	35.58379	121.5308	35.60476	121.6107	35.62574	121.7705	35.66768	121.9303	35.70963	122.0901	35.75157
11	0.215	131.8133	38.30376	8.58925	132.6722	38.52921	133.5311	38.75467	134.39	38.98012	136.1079	39.43103	137.8257	39.88194	139.5436	40.33286
12	0.77	161.5335	46.10489	30.7615	164.6097	46.91234	167.6858	47.71978	170.762	48.52723	176.9143	50.14212	183.0666	51.75701	189.2189	53.37189
13	0.94	170.637	48.49443	37.553	174.3923	49.48014	178.1476	50.46585	181.9029	51.45156	189.4135	53.42299	196.9241	55.39441	204.4347	57.36583
14	0.03	179.4065	50.7963	1.1985	179.5264	50.82775	179.6462	50.85921	179.7661	50.89067	180.0058	50.95359	180.2455	51.01651	180.4852	51.07943
15	0.27	192.2585	54.16976	10.7865	193.3372	54.45289	194.4158	54.73602	195.4945	55.01915	197.6518	55.58541	199.8091	56.15167	201.9664	56.71793
16	0.7	215.285	60.21388	27.965	218.0815	60.94792	220.878	61.68196	223.6745	62.416	229.2675	63.88408	234.8605	65.35217	240.4535	66.82025
17	0.89	225.4595	62.88454	35.5555	229.0151	63.81782	232.5706	64.7511	236.1262	65.68438	243.2373	67.55094	250.3484	69.4175	257.4595	71.28406
18	NA															

Table 4-7

SUMMARY OF STABILITY ANALYSIS RESULTS

MINIMUM BISHOP FACTOR OF SAFETY FOR SPECIFIED FAILURE TYPE

DEGREE OF CONSOLIDATION U (%)	SEGMENT 1, STA 9+750 TO 10+100				SEGMENT 2, STA 10+100 TO 10+950				SEGMENT 3, STA 13+000 TO 13+850			
	DEEP RIVER	DEEP CITY	SHALLOW RIVER	SHALLOW CITY	DEEP RIVER	DEEP CITY	SHALLOW RIVER	SHALLOW CITY	DEEP RIVER	DEEP CITY	SHALLOW RIVER	SHALLOW CITY
0	0.717	0.813	0.84	1.081	1.446	1.893	0.84	1.081	1.253	1.656	0.84	1.081
20	0.879	1.034	0.926	1.235	1.523	2.027	0.926	1.235	1.352	1.86	0.926	1.235
50	1.109	1.362	1.047	1.435	1.632	2.221	1.047	1.435	1.459	2.04	1.047	1.435
90	1.386	1.739	1.206	1.621	1.78	2.482	1.206	1.621	1.604	2.279	1.206	1.621

DEGREE OF CONSOLIDATION U (%)	SEG. 4 STG. 1, STA 13+850 TO 14+350				SEG. 4 STG. 2, STA 13+850 TO 14+350			
	DEEP RIVER	DEEP CITY	SHALLOW RIVER	SHALLOW CITY	DEEP RIVER	DEEP CITY	SHALLOW RIVER	SHALLOW CITY
0	1.478	2.027	0.877	1.014	1.491	2.048	1.068	1.435
20	1.547	2.114	0.948	1.113	1.526	2.103	1.107	1.495
50	1.64	2.235	1.052	1.268	1.577	2.185	1.165	1.583
90	1.76	2.396	1.175	1.461	1.645	2.295	1.241	1.702

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Table 4-8. *Summary of the Results of Stability Analysis.*

EMBANKMENT SEGMENT NO., CLASSIFICATION, AND STATIONING	SUBSURFACE CONDITIONS RELATED TO STABILITY ANALYSIS	STRENGTH PARAMETERS USED TO CALCULATE UNDRAINED SHEAR STRENGTH (C_{ou} (kPa) AND ϕ(°))	DISCUSSION OF MOST CRITICAL FAILURE SURFACES AND ANALYSIS RESULTS	REQUIRED CONSOLIDATION (U%) TO PROVIDE DESIGN FOS
Segment 1 Class I Sta 9 + 750 to Sta 10 + 100	An apparent deep-seated failure of existing embankment was observed. Very soft layer of silt (N = 1) encountered from elevation -16 to -24 m. This soil is potentially underconsolidated due to arching of the stronger surface soil across an ancient channel filled with the very soft silt.	Surface soil (elevation 2 to -16 m): C_{cu} 5.6 kPa, ϕ_{cu} = 11° Very soft silt: C_{cu} = 4 kPa, ϕ_{cu} = 10° Note: Surface soil was consolidated beneath embankment to 7.5 m elevation while very soft layer was consolidated beneath embankment plus 18 m thick disturbed arch.	Critical case exists on river side during periods of low water and on city side during periods of high water. Deep-seated failures through underconsolidated layer are most critical at consolidation below 33%. At consolidation levels above 33%, the most critical failure surfaces do not pass through the under-consolidated layer and are similar in geometry and FOS to those of segments 2 and 3.	U = 33% required to prevent deep-seated failure. U = 85 to 90% required to meet design FOS for shallow failure surfaces.
Segment 2 Class II Sta 10 + 100 to Sta 10 + 950	Soft to very soft clayey silt above elevation -6 m overlying a sandy silt layer. The sandy silt layer was assumed to fail in drained shear with ϕ' = 34°	Soft clayey silt: C_{cu} = 5.6 kPa, ϕ_{cu} = 11°	Critical case exists on river side during periods of low water and on city side during periods of high water. The most critical failure surfaces are shallow surfaces and do not pass through the sandy silt layer.	U = 83% required to meet design FOS for shallow failure surfaces.
Segment 3 Class II Sta 13 + 000 to Sta 13 + 85-	Conditions very similar to segment 2 except sandy silt layer was encountered much deeper. For the analysis, the sandy silt layer was placed below elevation -16 m.	Soft clayey silt: C_{cu} = 5.6 kPa, ϕ_{cu} = 11°	Critical case exists on river side during periods of low water and on city side during periods of high water. Analysis shows that the most critical failure surfaces are shallow, similar to those of segment 2.	U = 83% based on results of segment 2 analysis.

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Table 4-8. *Summary of the Results of Stability Analysis (continued).*

EMBANKMENT SEGMENT NO., CLASSIFICATION, AND STATIONING	SUBSURFACE CONDITIONS RELATED TO STABILITY ANALYSIS	STRENGTH PARAMETERS USED TO CALCULATE UNDRAINED SHEAR STRENGTH (C_{ou} (kPa) AND ϕ(°))	DISCUSSION OF MOST CRITICAL FAILURE SURFACES AND ANALYSIS RESULTS	REQUIRED CONSOLIDATION (U%) TO PROVIDE DESIGN FOS
Segment 4 Class I Sta 13 + 850 to Sta 14 + 350	Very soft medium to low plasticity silt was encountered to elevation -5 m. Soft to medium stiff silt extended from elevation -5 to -16 m. Numerous landslides were observed in existing embankment.	C_{cu} 3 kPa, ϕ_{cu} = 12° Note: Very low initial shear strengths lead to instability. Embankment must be constructed in two stages. Stage 1 consists of cutting the embankment to a top elevation of 5 m. Stage 2 consists of constructing the embankment to the final elevations.	Critical case exists on river side during periods of low water and on city side during periods of high water. For this analysis, the stage II consolidation load was applied after 90% consolidation under the Stage I consolidation load.	U = 90% under embankment at elevation 5.5 m prior to applying fill to elevation 7.5 m. U = 68% under embankment at elevation 7.5 m prior to applying fill to final grade.

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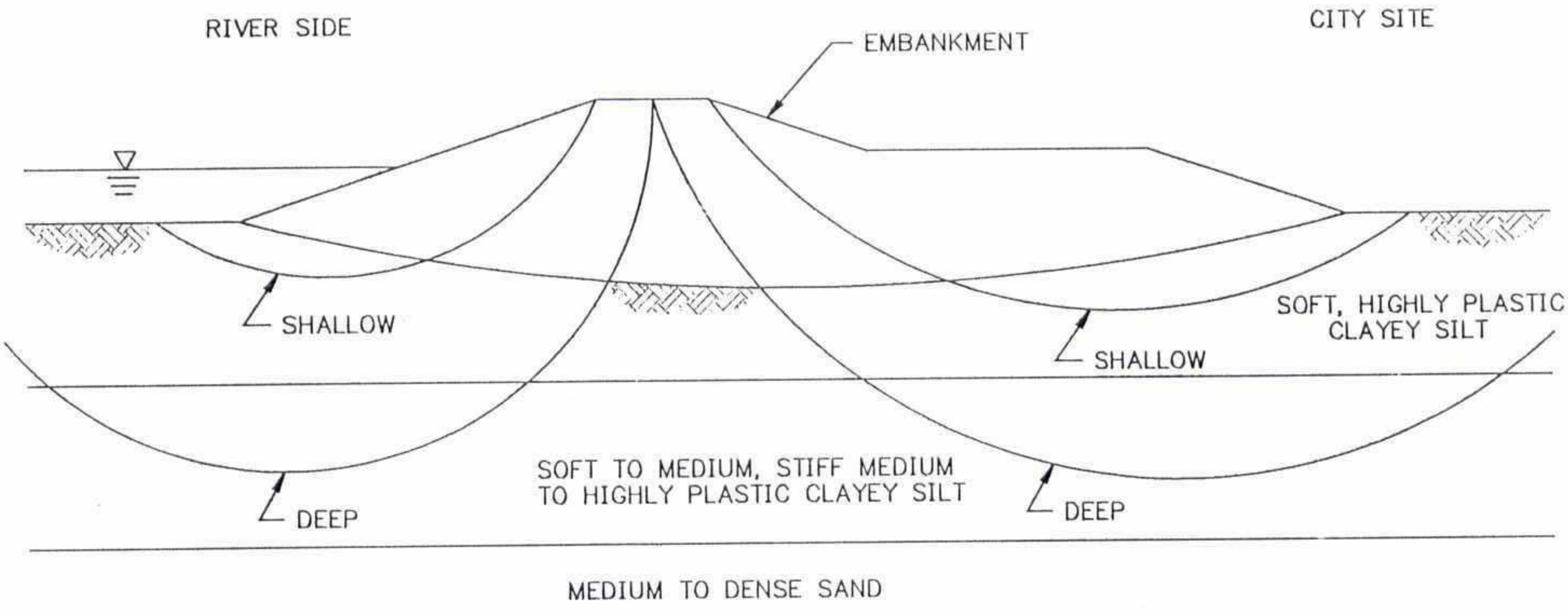

CONDITIONS ANALYZED

GEOSYNTEC CONSULTANTS	FIGURE NO.	4-1
	PROJECT NO.	FE2043
	DOCUMENT NO.	
	PAGE NO.	65

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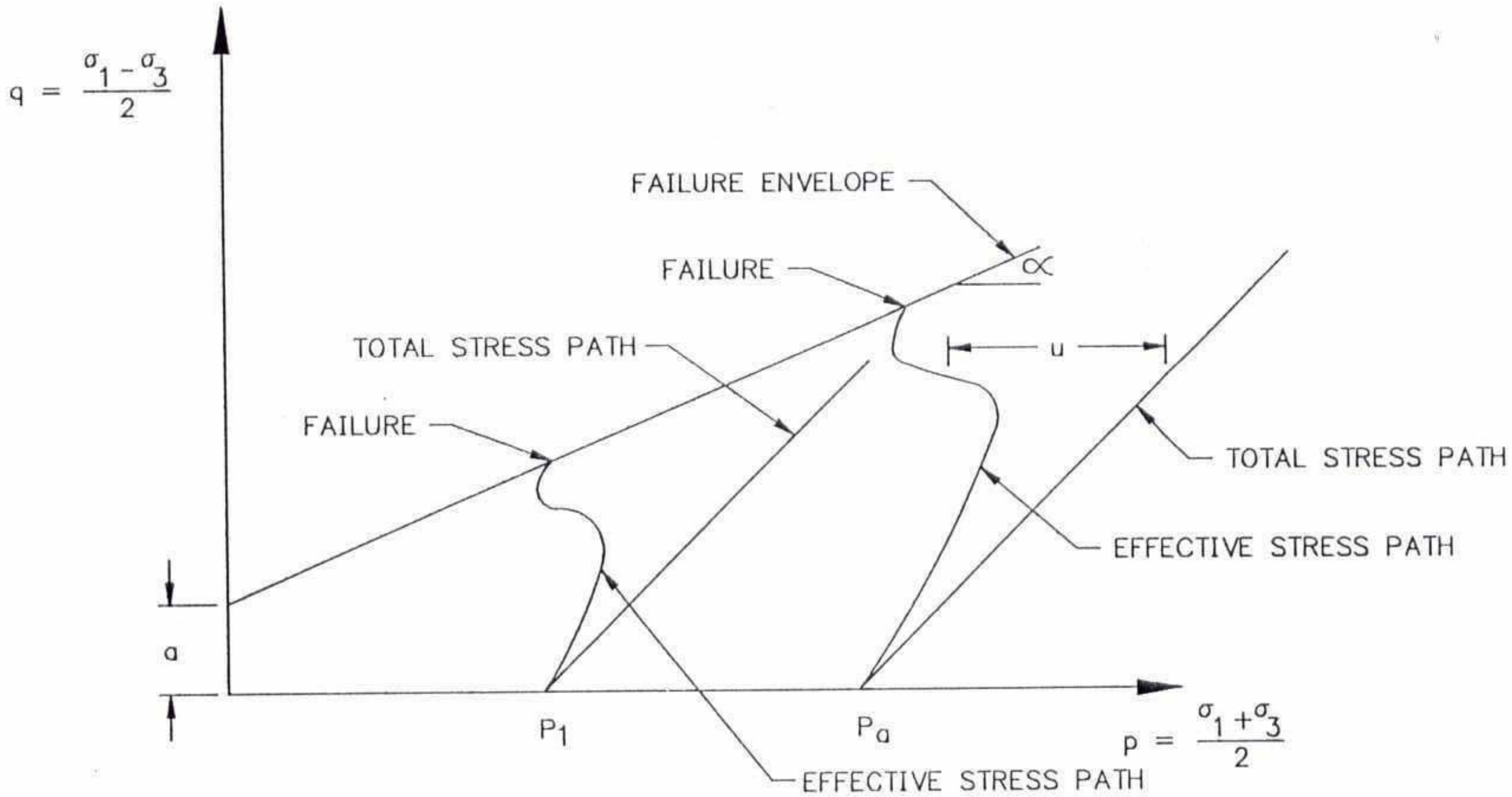

p-q DIAGRAMS OF TRIAXIAL STRESS DATA

GEOSYNTEC CONSULTANTS	FIGURE NO. 4-2
	PROJECT NO. FE2043
	DOCUMENT NO.
	PAGE NO. 66

DWG: 2043F022 199209101101 SB

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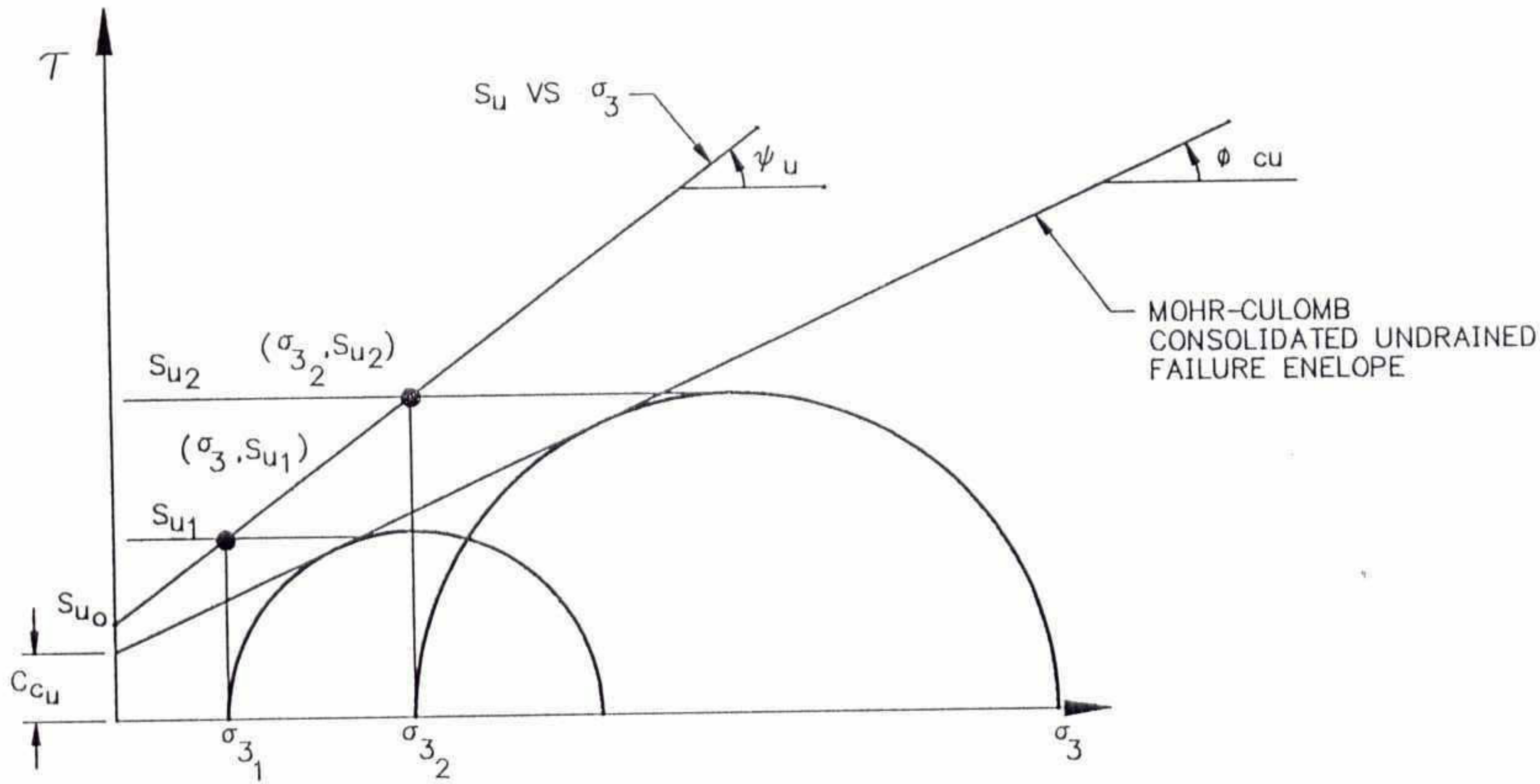

$$\psi_u = \tan^{-1}\left(\frac{S_{u_2} - S_{u_1}}{\sigma_{3_2} - \sigma_{3_1}}\right)$$

$$S_u = S_{u_o} + \sigma_3 \tan \psi_u$$

UNDRAINED SHEAR STRENGTH VERSUS TOTAL STRESS RELATIONSHIP

GeoSyntec Consultants

FIGURE NO.	4-3
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	67

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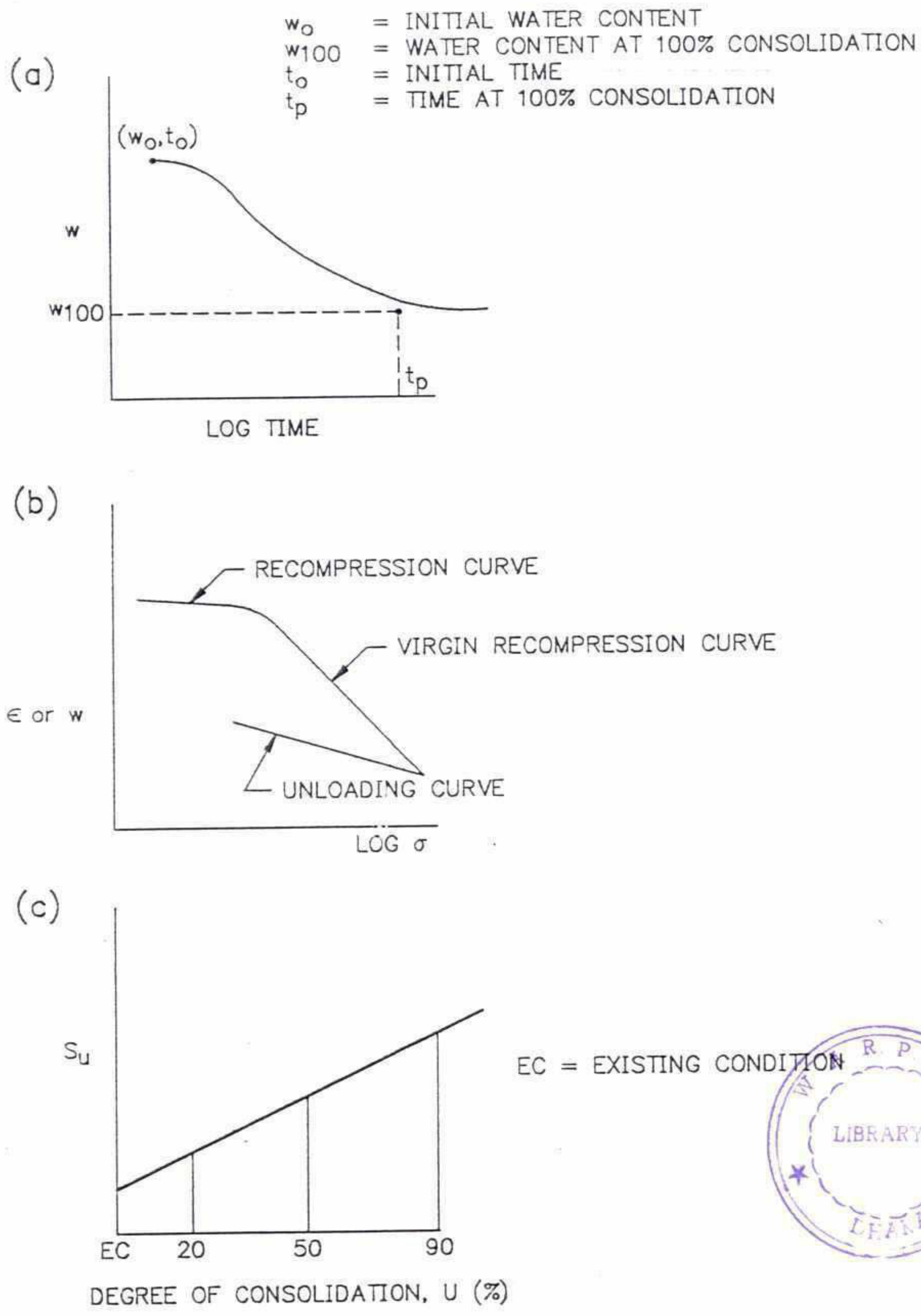
9a

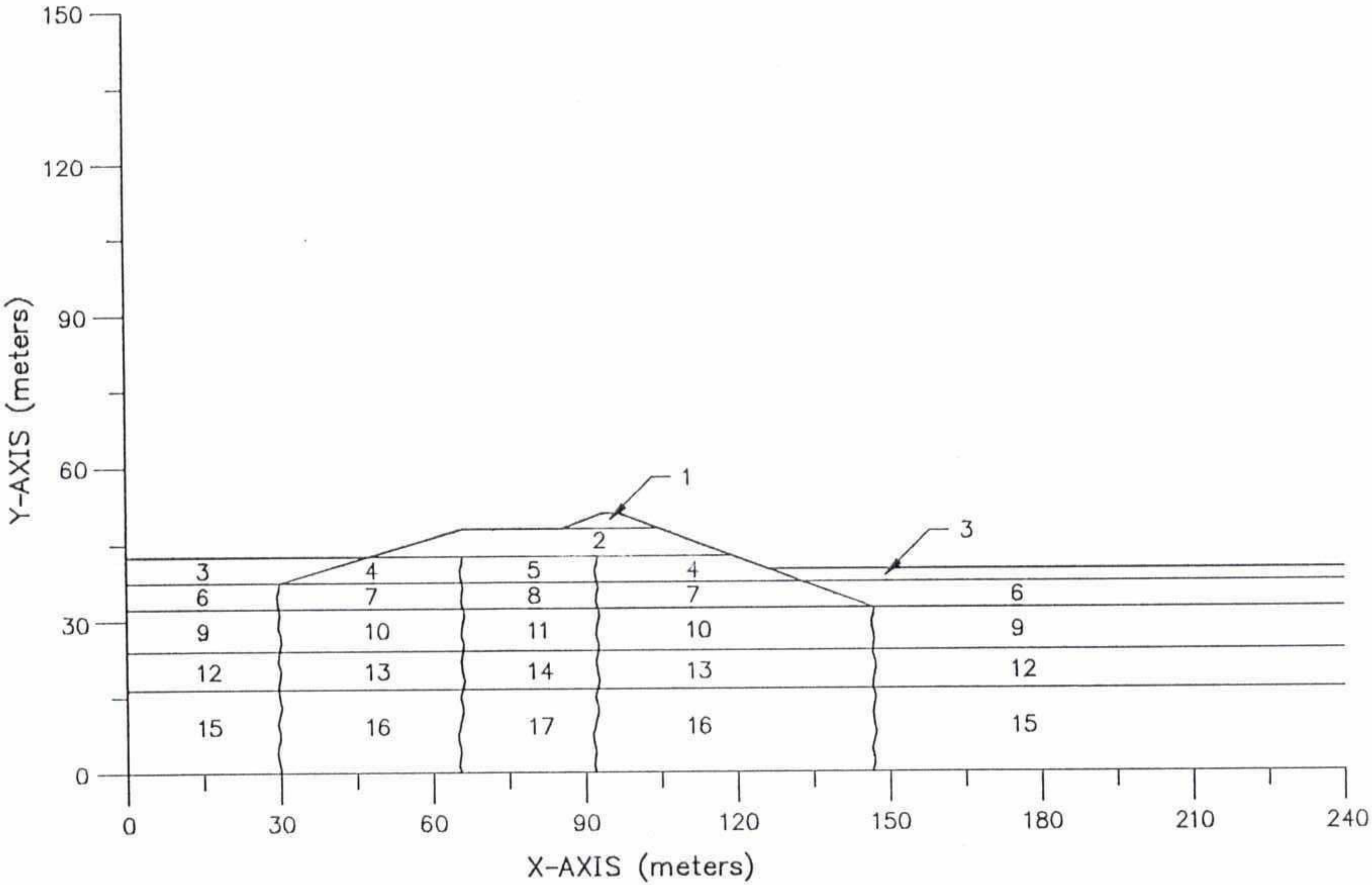


CONSOLIDATION RELATIONSHIP

GEOSYNTEC CONSULTANTS

FIGURE NO.	4-4
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	68

DWG: 2043F022 199209101101 SB



SOIL TYPES FOR SECTION BETWEEN STATIONS 9+750 AND 10+100m.

GEOSYNTEC CONSULTANTS

FIGURE NO.	4-5
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	69

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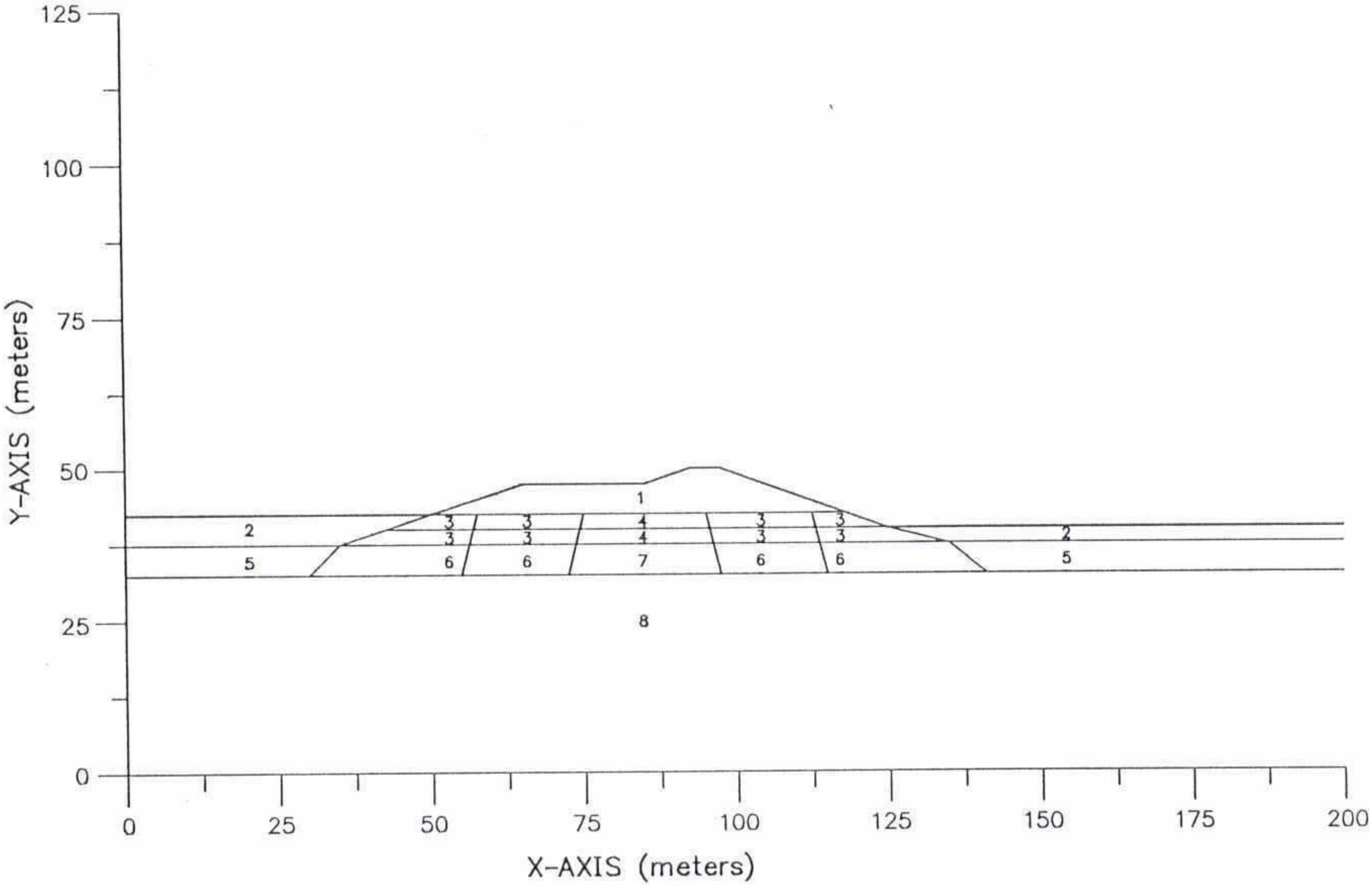

SOIL TYPES FOR SECTION BETWEEN STATIONS 10+100 AND 10+950m.

GEOSYNTEC CONSULTANTS

FIGURE NO.	4-6
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	70

DWG: 2043F022 199209101100 SB

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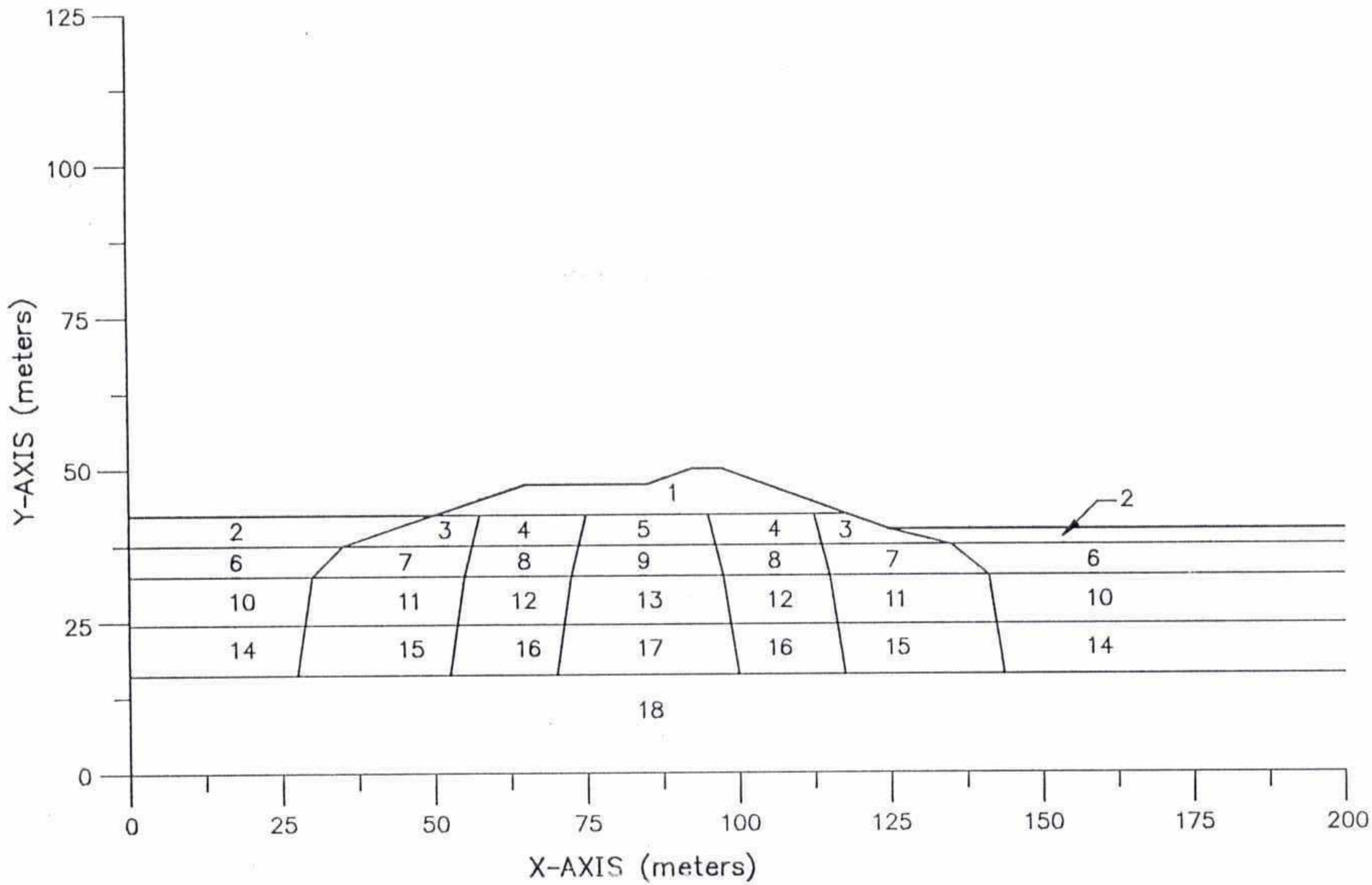

SOIL TYPES FOR SECTION BETWEEN STATIONS 13+000 AND 13+850m.

GEOSYNTEC CONSULTANTS

FIGURE NO.	4-7
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	71

DWG: 2043F022 199209101100 SB

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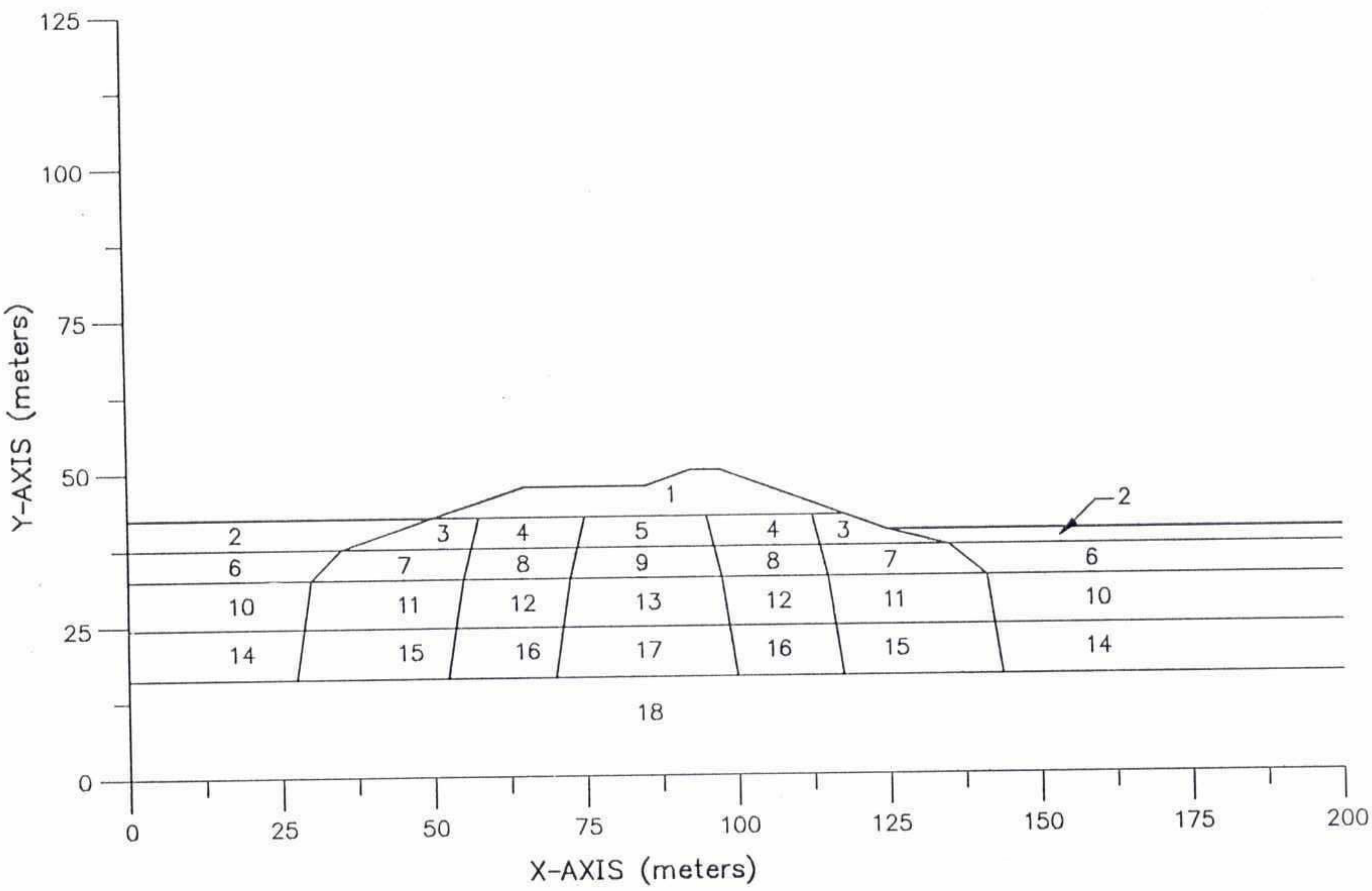


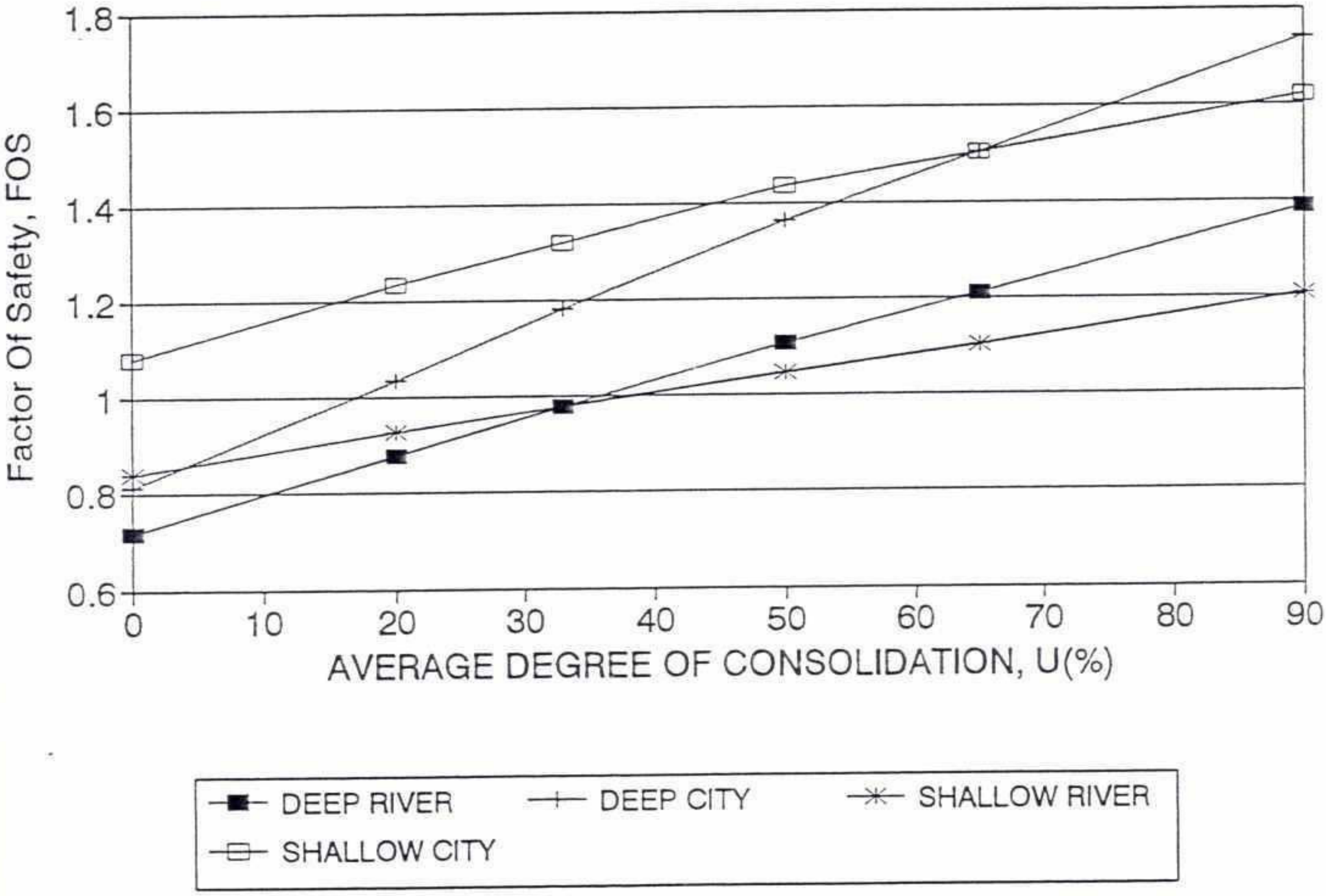
SOIL TYPES FOR SECTION BETWEEN STATIONS 13+850 AND 14+350m.

GEOSYNTEC CONSULTANTS

FIGURE NO.	4-8
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	72

DWG: 2043F022 199209101100 SB

STABILITY INCREASE WITH CONSOLIDATION
STATION 9+750 TO 10+100



GEOSYNTEC CONSULTANTS

FIGURE NO.	4-9
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	73

DWG: 2043F023 199209101002 SB

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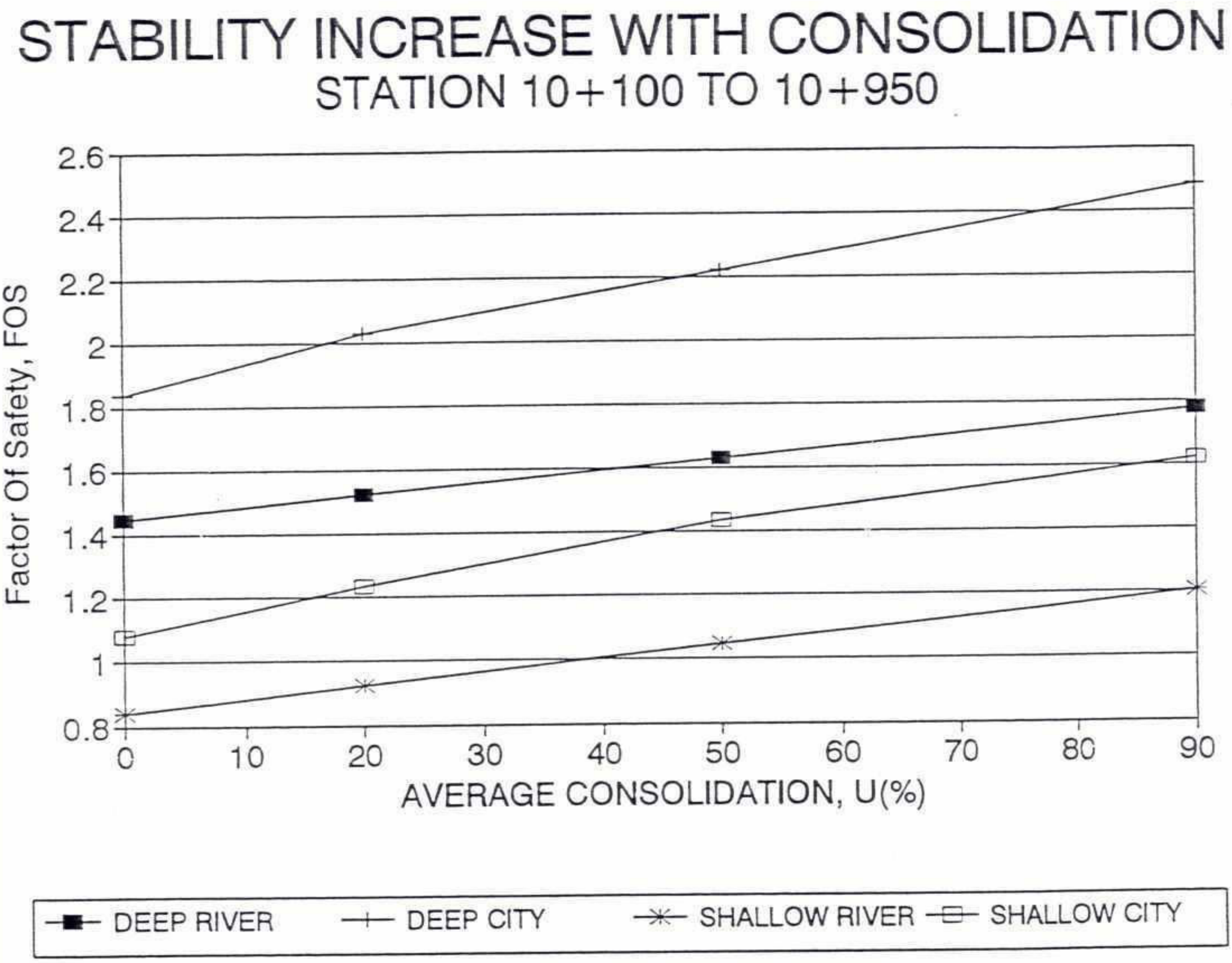

GeoSyntec Consultants

FIGURE NO.	4-10
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	74

DWG: 2043F023 199209101002 SB

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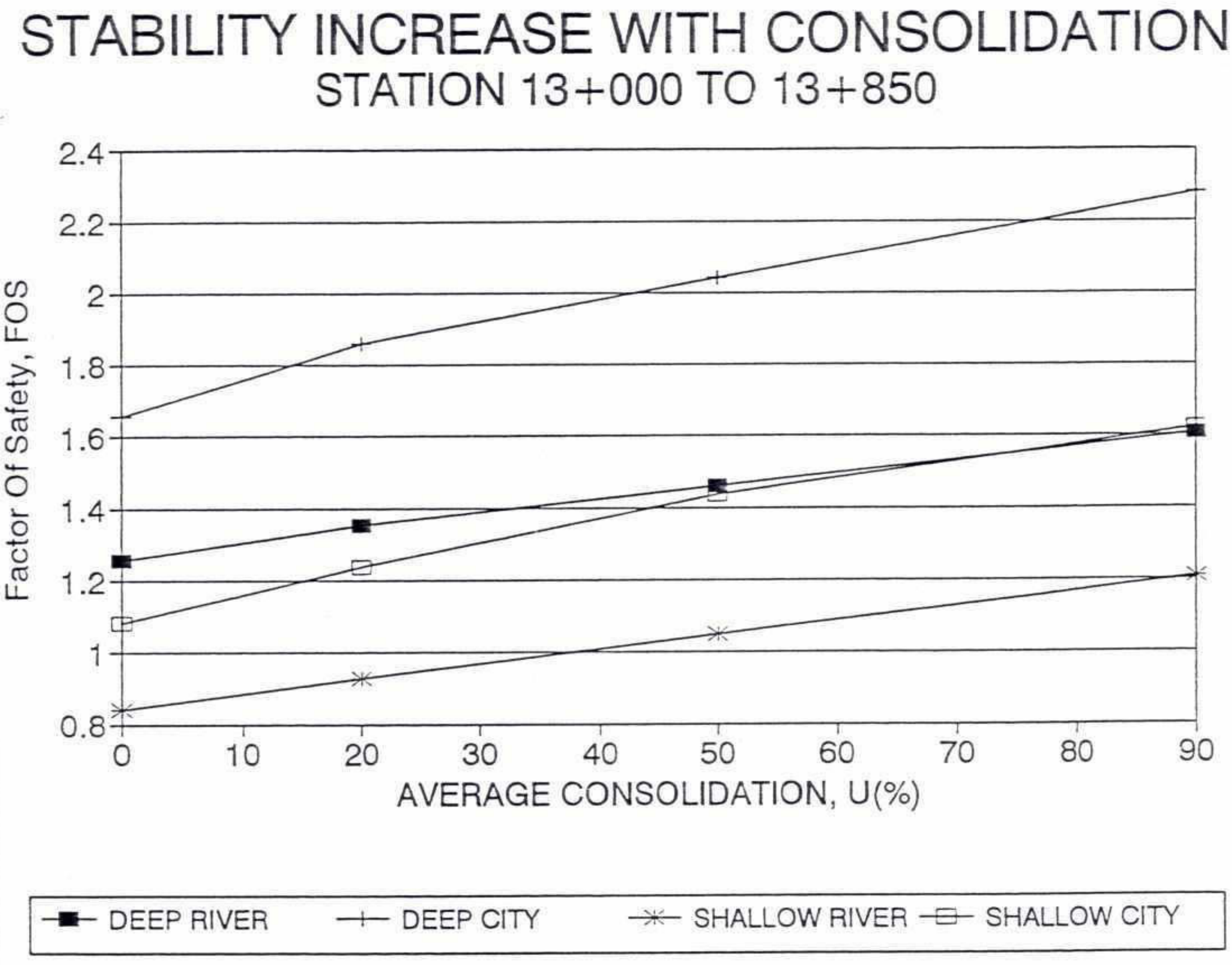

GEOSYNTEC CONSULTANTS

FIGURE NO.	4-11
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	75

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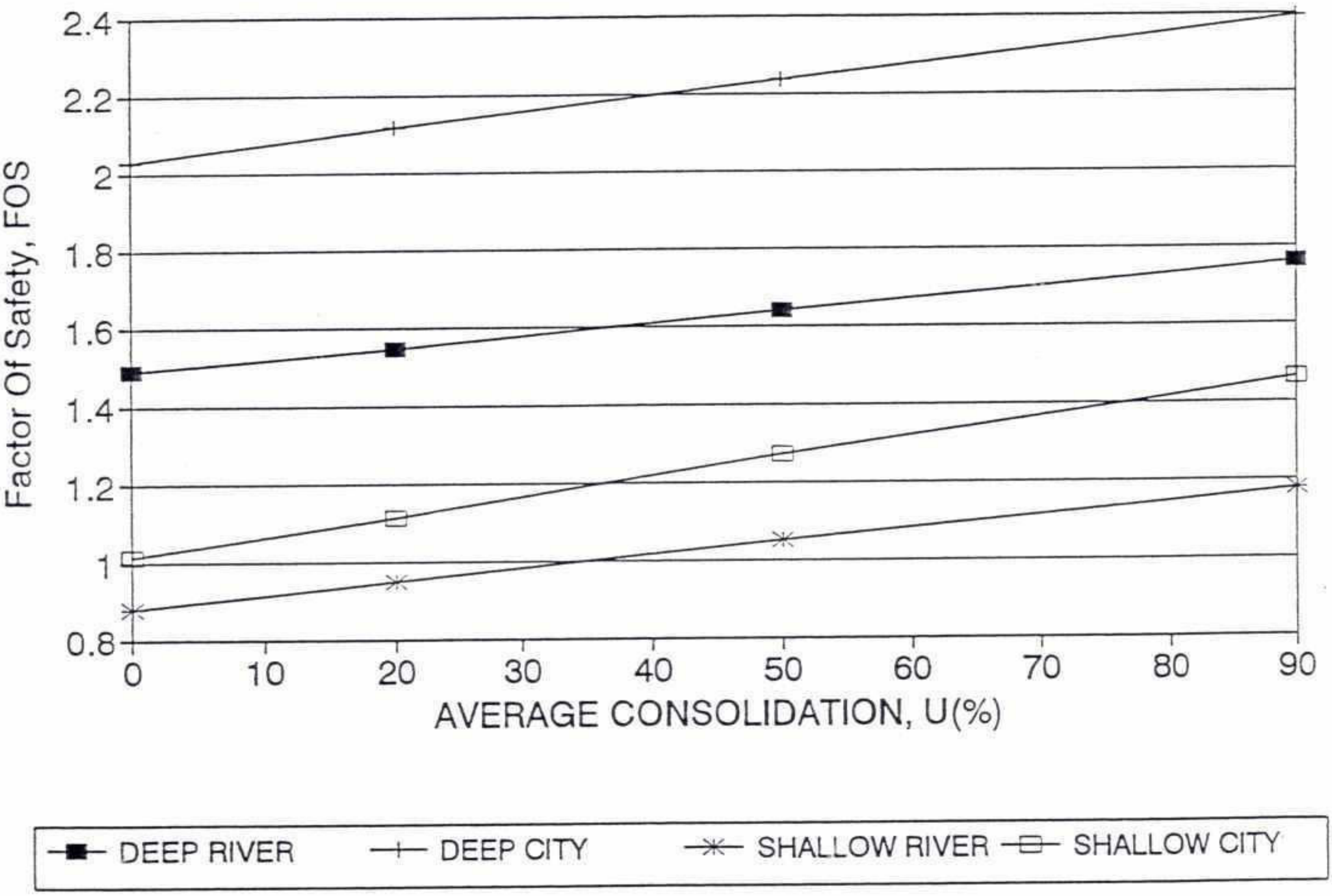
STABILITY INCREASE WITH CONSOLIDATION
STATION 13+850 TO 14+350 STAGE 1



GeoSyntec Consultants

FIGURE NO.	4-12
PROJECT NO.	FE2043
DOCUMENT NO.	
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DWG: 2043F023 199209101002 SB

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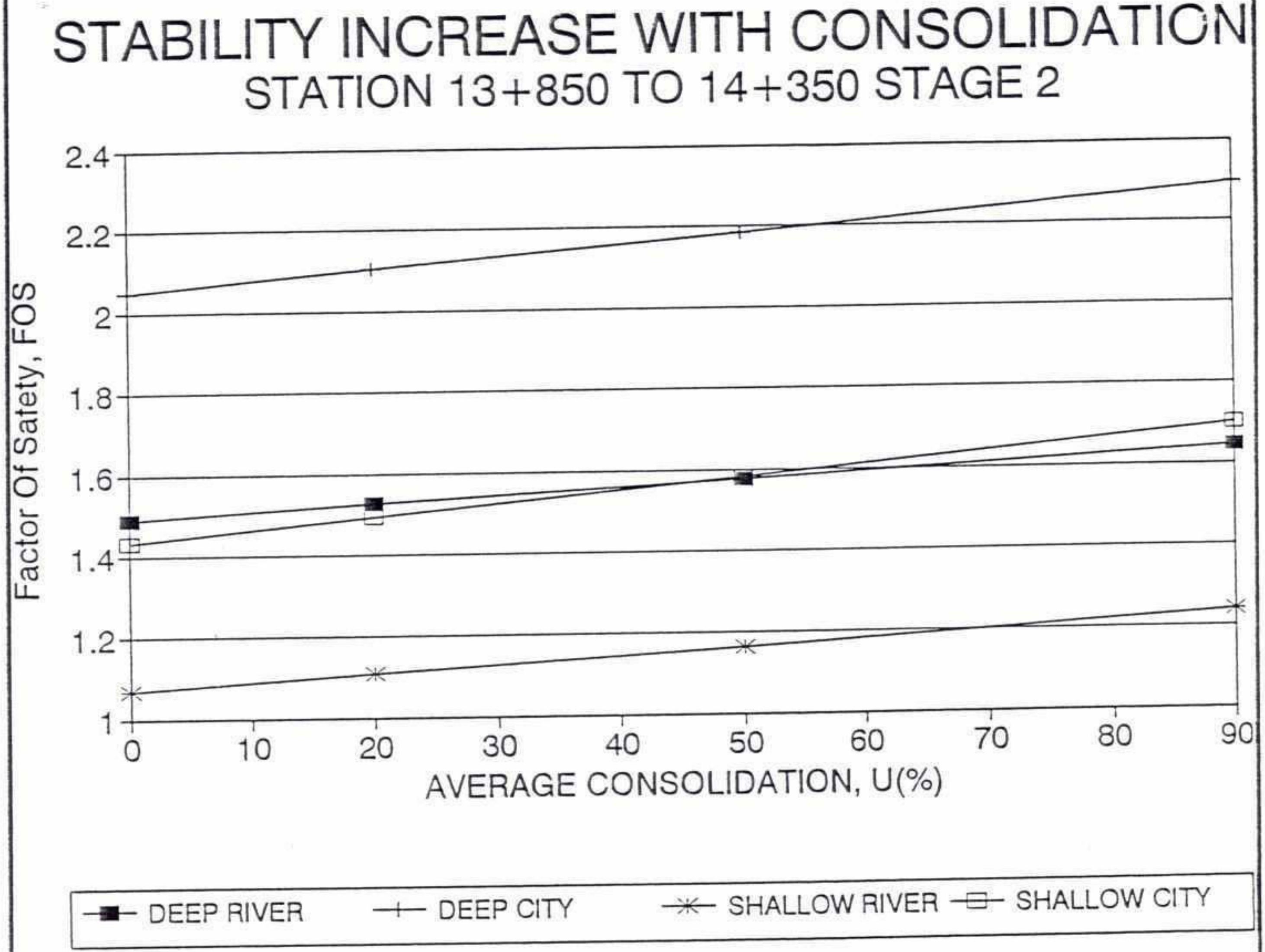

GeoSyntec Consultants

FIGURE NO.	4-13
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	77

DWG: 2043F023 199209101002 SB

5. REMEDIAL DESIGN

5.1 Introduction

The remedial design follows a feasibility study and preliminary remedial design performed by LBII and GeoSyntec Consultants. A preliminary damage survey performed during these initial investigations identified:

- 4,100 m of embankment potentially subject to catastrophic failure (Class I);
- 3,050 m of embankment which could fail, but not catastrophically (Class II); and
- 3,150 m of embankment damaged by erosion, vectors, and desiccation.

The cause of failure of the embankment was also investigated. Based on field observations, laboratory data, and the site investigations, the probable causes of the embankment failure were identified as:

- poor compaction of embankment soils;
- inadequate subgrade improvement;
- lack of erosion protection; and
- use of inappropriate soils.

Based on the damage survey and an assessment of available data, LBII and GeoSyntec Consultants made the following recommendations:

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- perform additional field investigations and laboratory analyses to refine estimates of Class I and Class II areas;

- use vertical drains and high strength geosynthetics to remediate Class I areas;

- use vertical drains to remediate Class II areas;

- investigate the use of berms for remediation of Class II areas; and

- complete the remedial design.

The WDB, LBII, and GeoSyntec Consultants worked closely to develop an appropriate scope of work for the remedial investigation. The WDB directed LBII and GeoSyntec Consultants to find potential remedial actions which achieved the required factor of safety (1.2) at the lowest cost. The WDB stated that it would be acceptable to design remedial actions which achieved the factor of safety over a period of time, possibly up to several years.

Based on the input from the WDB and the results of the remedial investigations, the remedial design was changed to allow remedial actions which had low initial factors of safety, but which achieved the desired factor of safety within several years. This approach eliminated the need for a high strength geotextile, which provided a short-term increase in the factor of safety. Slope stability and bearing capacity analyses were performed to evaluate the existing factors of safety at several points along the embankment, as discussed in Section 4 of this report.

The remedial designs of the existing embankment are divided into three categories:

- Class I Areas (850 m or 2,800 ft). These are the areas which have very low factors of safety and could fail catastrophically. The remedial design of the Class I areas is discussed in Section 5.2 of this report.

- Class II Areas Requiring Subgrade Improvement (1700 m or 5,600 ft). These are the areas which are underlain by more than about 10 m (33 ft) of very soft soils. These areas have low factors of safety and could fail, but would likely not fail in a way that would result in a breach of the embankment. The remedial design of the Class II areas requiring additional subgrade improvement is discussed in Section 5.3.2 of this report.

- Class III Areas Requiring Monitoring and Inspection (4500 m or 14,800 ft). The remaining Class II areas have been remediated by constructing toe berms, flattening slopes, and reducing the height of the embankment. While these areas may have relatively low factors of safety, it is believed that the embankment would not fail in a way that would breach the embankment, and the previous remedial actions have increased the factor of safety to the extent that monitoring and inspection can be used to assess if failure is imminent. The remedial design of the Class II areas requiring monitoring and inspection is discussed in Section 5.3.3 of this report.

As a result of the site investigations, laboratory and field testing programs, and the policy of the WDB to allow the minimum factor of safety (1.2) to be achieved over time, the length of the Class I area requiring remediation was decreased from 4100 m (13,500 ft) to 850 m (2,800 ft). The length of the Class II areas requiring additional remediation was decreased from 3050 m (10,000 ft) to about 1700 m (5,600 ft). Monitoring and inspection will be required for an additional 4500 m (14,800 ft).

5.2 Remedial Design of Class I Areas

5.2.1 Overview

As discussed previously, the Class I areas are defined as those areas which have low factors of safety and could fail catastrophically. As stated previously, catastrophic failure is defined as a failure which could breach the embankment during a period of high water on the embankment. The embankment in the Class I areas was constructed on very weak, soft subgrade soils to depths greater than 20 m (66 ft), that cannot be effectively remediated with toe berms or by flattening the slopes of the embankment.

As described in Section 4 of this report, the existing factor of safety of the embankment in the Class I areas is on the order of 0.8. A factor of safety less than 1.0 implies that the embankment is unstable and could fail at any time. These Class I areas have failed and have been reconstructed as many as 20 times.

5.2.2 Extent of Class I Areas

The Class I areas of the embankment include:

- Stations 9+750 to 10+100 m. In this area the soft clayey silt subgrade soil extends from the ground surface at an elevation of about 2 m (6 ft) PWD to about -23 m (-75 ft) PWD. The soft clayey silt layer is underlain by dense fine to medium grained sand at an elevation of about -23 m (-75 ft) PWD.

- Stations 13+850 to 14+350 m. The soil profile beneath the embankment consists of soft, high plasticity silt from an elevation of 1 m (3 ft) PWD to an elevation of about -3 m (10 ft) PWD. The soft high plasticity silt is underlain by soft non-

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plastic to high plastic clayey silt to an elevation of about -26 m (-85 ft) PWD. The soft non-plastic to high plastic clayey silt is underlain by medium dense, fine to medium grained sand.

The remedial designs for the two Class I areas are described below.

5.2.3 Stations 9+750 to 10+100 m

The remedial design of the embankment between Stations 9+750 and 10+100 m will consist of installing wick drains through the existing embankment into the subgrade soils. Temporary benches will be required in order to install the wick drains.

The wick drains will consist of a continuous polypropylene drainage core wrapped in a needlepunched or heatbonded nonwoven geotextile. The minimum discharge capacity of the wick drain will be 1.2×10^{-5} m^3/s (4.2×10^{-4} ft^3/s), and the minimum hydraulic conductivity will be 6.5×10^{-2} cm/s. The minimum width and thickness of the wick drain, including the geotextile overwrap, will be about 100 mm (4 in.) and 3 mm (0.125 in.), respectively.

The wick drains in the Class I area between Stations 9+750 and 10+100 m will be installed to an average bottom elevation about -23 m (-75 ft) PWD. The wick drain will be installed vertically within a tolerance of 21 mm/m (0.25 in./ft). The wick drain will be extended a minimum of 30 cm (12 in.) above the embankment.

The spacing of the wick drains required to achieve various degrees of consolidation was evaluated using the equation developed by Hansbo (1979) in, "Consolidation of Clay by Band-Shaped Prefabricated Drains," Ground Engineering, Vol. 12, No. 5, pp. 16-25:

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$$t = \frac{D^2}{8C_r}\left[\ln\left(\frac{D}{d_d}\right) - 0.75\right]\ln\left[\frac{1}{(1-\overline{U}_r)}\right] \qquad \text{(Equation 5-1)}$$

where: t = time; D = center to center spacing between the drains; d_d = equivalent diameter of the drain; $\overline{U}_r$ = average degree of consolidation due to radial drainage; and C_r = coefficient of radial consolidation. The equivalent diameter of the drain is calculated as follows:

$$d_d = \frac{2(a+b)}{\pi} \qquad \text{(Equation 5-2)}$$

where: a = width of drain; and b = thickness of drain. The following parameters were used in the analyses of the Class I area between Stations 9+750 and 10+100 m:

- C_r = 6 x 10^{-4} cm^2/s;
- a = 10 cm;
- b = 0.3 cm;
- D = 2 m: and
- $\overline{U}_r$ = varied.

Based on these analyses, the approximate relationships between time, t, and degree of consolidation, U, is shown in Figure 5-1.

As shown in Figure 5-1, if the vertical drains are spaced at about 2 m (6.6 ft) on center, 90 percent consolidation is achieved in about 10 months. The critical mode of failure changes from a deep circle to a shallow circle at an average degree of consolidation of about 33 percent, which would be achieved in about 3.5 months.

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5.2.4 Stations 13+850 to 14+350 m

The embankment between Stations 13+850 and 14+350 m has failed and been reconstructed on numerous occasions. Borings in this area of the embankment show that the subgrade soil is highly variable, but consists of, from top to bottom:

- a 0- to 4-m (0- to 13-ft) thick very soft, highly plastic clayey silt layer;

- a 14- to 23-m (46- to 75-ft) thick soft to medium, non-plastic to highly plastic silt;

- a 0- to 10-m (0- to 33-ft) thick medium stiff, medium plastic silt; and

- dense, fine to medium grained sand.

The slope stability analyses were performed along a cross section at Station 13+925 m, through borings B-259 through B-263. A sloping boundary between the non-plastic silt and the underlying medium stiff highly plastic silt may have contributed to failures at this location.

The remedial design at this location will consist of installing wick drains on 2 m (6.6 ft) centers from Stations 13+850 to 14+350 m. The wick drains will be installed to an average bottom elevation of -26 m (-85 ft) PWD. The wick drains will be installed vertically within a tolerance of 21 mm/m (0.25 in./ft). The wick drains will be extended a minimum of 30 cm (12 in.) above the embankment. Due to the low subgrade shear strength, the construction of the embankment in this area must be staged in order to prevent failure during construction. The embankment will be cut to an elevation of 5.0 m (16 ft) PWD. Wick drains will then be installed in 2 m (6.6 ft) centers as shown in Drawing WK-01. The wick drains will be wrapped around perforated polyvinyl chloride (PVC) header pipes as shown in Drawing WK-01. Uniform pea gravel or clean coarse sand will then be placed over the header pipes as shown in Drawing WK-01. The sand or gravel will be wrapped in a needlepunched nonwoven geotextile filter fabric.

Clay soil will be compacted around and over the geotextiles as shown in Drawing WK-01. The embankment will then be extended to an elevation of 7.0 to 7.5 m (23 to 25 ft) PWD. The embankment will remain at this elevation until the subgrade soils achieve an average degree of consolidation of 90 percent.

Phase II of the embankment construction between Stations 13+850 and 14+350 m will consist of constructing the embankment to the final elevations. The Phase II construction will occur after the subgrade and embankment soils have achieved an average degree of consolidation of about 90 percent.

Excavation of the embankment between Stations 13+850 and 14+350 m is also needed because substandard soils were used in embankment construction. The embankment in these areas is predominantly non-plastic silt or organic clay. These soils should not have been used in embankment construction because they are difficult to compact, have low shear strength, and erode easily. These soils may be used for construction of fill for Pump Station No. 2.

5.3 Remediation of the Class II Areas

5.3.1 Introduction

As discussed previously, the Class II areas are divided into two groups: (i) areas requiring additional remediation; and (ii) areas where previous remedial efforts have been successful, and monitoring and

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inspection are needed. The remedial designs for those areas are described below.

5.3.2 Class II Areas Requiring Additional Remediation

There are two Class II areas requiring additional remediation: (i) Stations 10+100 to 10+950 m; and (ii) Stations 13+000 to 13+850 m. The idealized soil profile between Stations 10+100 and 10+950 m consists of, from top to bottom:

- 3 to 25 m (10 to 82 ft) of soft to medium stiff, medium to highly plastic clayey silt; and
- dense, fine to medium grained sand beneath a depth of about -3 to -23 m (-10 to -75 ft) PWD.

The idealized soil profile between Stations 13+000 and 13+850 m consists of, from top to bottom:

- 3- to 4-m (10- to 13-ft) thick soft, highly plastic silt layer;
- 24 m soft to medium stiff, non-plastic to highly plastic clayey silt layer; and
- medium dense, fine to medium grained sand below an elevation of about -27 m (-89 ft) PWD.

It is anticipated that vertical drains will be installed through the existing embankment in these two areas. The vertical drains will be installed on 2-m (6.6-ft) centers, as described previously. The wick drains between Stations 10+100 and 10+950 m will be extended to the top of the sand layer, which varies from elevation of -3 m (-10 ft) PWD at Station 10+**950** m, to -23 m (-75 ft) PWD at Station 10+100 m. The wick drains between Stations 13+100 and 13+850 m will be installed to the top of the medium stiff clayey silt layer, at an average depth of about 15 m (49 ft) below the original grade. The wick drains will be installed vertically to a tolerance of 21 mm/m (0.25 in./ft). The wick drains will extend a minimum of 30 cm (12 in.) above the existing grade. After the embankment reaches about 90 percent consolidation under the existing load, the embankment will be extended to the final elevations.

5.3.3 Class II Areas Requiring Monitoring and Inspection

The following Class II areas have been remediated by constructing toe berms, flattening the slopes, and reconstructing the embankment:

- Stations 6+750 to 7+450 m;
- Stations 7+550 to 8+000 m;
- Stations 9+250 to 9+750 m;
- Stations 11+200 to 12+500 m; and
- Stations 14+350 to 15+900 m.

The previous remedial actions in these areas have increased the factors of safety and reduced the probability of failure. While the existing factor of safety may still be low, it is anticipated that an acceptable factor of safety (1.2) will be achieved in these areas over time. In addition, failure of these sections of the embankment, if it did occur, would not likely be catastrophic. Therefore, it is recommended that a monitoring and inspection program be developed.

The monitoring and inspection program will consist, at a minimum, of the following components:

- monthly inspection of all Class II areas;
- visual inspection for cracking, bulging, deformations, settlements, and vandalism;
- installation of settlement plates and quarterly survey to assess movement; and
- preparation of quarterly monitoring and inspection reports.

The quarterly monitoring and inspection reports will summarize the above activities. If remedial actions are required, the areas will be identified and specific recommendations will be made.

5.4 Material Quantities

The material quantities required to implement the remedial designs described in Sections 5.1 through 5.3 are summarized in Table 5-1.

5.5 Engineering Drawings

The engineering drawings are included as Appendix D-1.

Table 5-1. Material Quantities

STATIONS		9+750 to 10+100 m	10+100 to 10+950 m	13+000 to 13+850 m	13+850 to 14+350 m
Description	Units				
Excavation	m^3	0	0	0	72,550
Construction of Temporary Benches	m^3	28,000	68,000	68,000	0
Fill	m^3	7018	17,043	17,043	72,550
Wick Drain Placement	m	217,525	341,275	358,275	300,000
Perforated PVC Pipe	m	0	0	0	20,000
Sand/Gravel Drainage Materials	m^3	0	0	0	1950
Geotextile Filter	m^2	0	0	0	30,000

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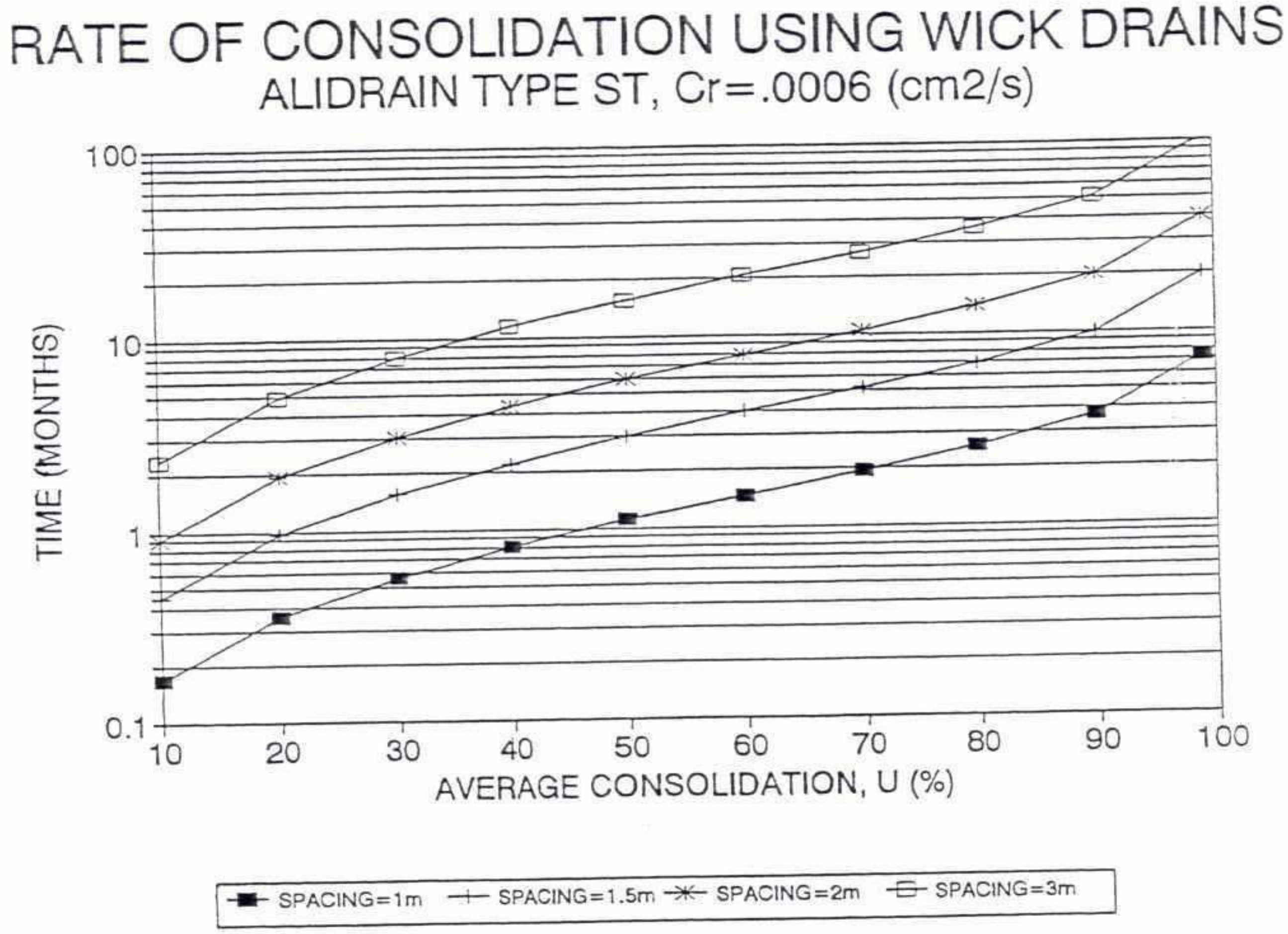

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FIGURE NO.	5-1
PROJECT NO.	FE2043
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6. EMBANKMENT EXTENSION

6.1 Introduction

The existing embankment was terminated on a peninsula in Kellar Mohr, near Station 29+950 m. Embankment construction was stopped in this area due to: (i) the high population density of the area; (ii) difficulties in selecting an appropriate embankment alignment; and (iii) difficulties associated with crossing Buriganga Jote Nadi.

Due to the high population density and building along the banks of the Burni Ganga River, construction of the embankment extension on the original alignment along the north bank of the Burni Ganga River is not feasible. An alternate alignment in the Burni Ganga River, near the north bank is proposed as shown in Figure 6-1. However, due to the depth of water along the alignment, soft subgrade soils, and physical constraints on the width of the embankment, an alternative profile is proposed for the embankment extension from Keller Mohr to Mittford Hospital.

The site investigation program performed along the proposed embankment alignment is discussed in Section 6.2. The conceptual design of the embankment extension is presented in Section 6.3.

6.2 Site Characterization

A total of 34 soil borings were advanced along the proposed alignment of the embankment extension. The boring locations are shown in Figure 6-1. The boring logs are presented in Appendix A.

The borings were advanced using the same approach described previously. Both split spoon and Shelby tube samples were obtained from the subgrade soils. Laboratory tests on the soil samples included

Atterberg Limits, densities, water contents, and grain size distribution analyses. The Shelby tube samples were extruded in the field, where a visual classification was performed by a project engineer.

Cross sections were measured at 27 locations along the proposed alignment of the embankment extension.

Based on the results of the soil borings, laboratory analyses, and site surveys, the idealized soil profile in the Kellar Mohr area consists of, from top to bottom:

- Soft, medium plasticity clayey silt from the ground surface at an elevation of about 2 m (6.6 ft) PWD, to a depth of 5 to 10 m (16 to 33 ft);

- Medium stiff, non-plastic to medium plastic silt from 5 to 10 m (16 to 33 ft) in thickness; and

- Medium to dense, fine to medium grained sand at an average elevation of 9 to 13 m (30 to 43 ft) PWD.

The soil profile in the river from Kellar Mohr to the Mittford Hospital typically consists of, from top to bottom:

- soft to medium stiff, medium to high plasticity clayey silt from the surface to depths of up to 20 m (66 ft);

- medium stiff, medium plasticity clayey silt from 0- to 13-m (0.- to 43-ft) thick; and

- dense, fine to medium grained sand at an elevation ranging from -5 to -25 m (-16 to -82 ft).

A longitudinal profile along the proposed embankment alignment is presented in Figure 6-2. The cross sections are shown in Figures 6-3 through 6-8.

6.3 Conceptual Design

The soil borings along the proposed embankment alignment were advanced from April 1992 to July 1992. The laboratory testing program is ongoing and will be completed during a subsequent phase of the project.

Due to the fact that the data from the investigation of the embankment extension were not available, it was not possible to perform the final design of the embankment extension. However, several design alternatives were investigated. Based on available data, the following conceptual designs are recommended for the embankment extension.

6.3.1 Option 1: Conventional Construction

Option 1, the conventional embankment, will be constructed in areas where the subgrade soil has sufficient bearing capacity to support the overlying embankment with a factor of safety greater than or equal to 1.2. The configuration of the embankment in these areas is shown in Figure 6-9.

The conventional embankment is about 80-m (262-ft) wide. In areas where there is insufficient space to construct the conventional embankment, either the reinforced embankment (Option 3) or the Sheet Pile Wall (Option 4) will be constructed.

6.3.2 Option 2: Vertical Drains with Base Reinforcement

In areas where the embankment will be underlain by relatively deep, soft subgrade soils, and where there is adequate space for an 80-m (262-ft) wide embankment, Option 2 will be installed.

Option 2 consists of a base reinforced embankment with vertical drains, as shown in Figure 6-10. In these areas, a woven geotextile will be placed over the subgrade. Fill will then be compacted in horizontal lifts to an elevation of approximately 4.0 to 5.0 m (13 to 16 ft) PWD. Vertical drains will be installed, through the first stage of the embankment, to the desired depth. The subgrade will continue to consolidate until an average consolidation of 90 percent is reached. The embankment will then be constructed to the desired final elevations.

6.3.3 Option 3: Reinforced Embankment with or without Vertical Drains

Option 3 consists of a reinforced embankment with (or without) vertical drains, as shown in Figure 6-11. Option 3 would be installed in areas where the subgrade is above water and where there are space limitations that would preclude construction of the conventional embankment.

Facing panels are recommended on the river side face and between the two terraces to minimize erosion and prevent vandalism. Facing panels are optional on the city side of the embankment, but are recommended to minimize damage from vandalism.

It is anticipated that the reinforcement would be placed in horizontal layers as shown in Figure 6-11. The number of layers of reinforcement required to construct the embankment depends on the long-term tensile strength of the reinforcement, the shear strength of the embankment soils, and the shear strength of the subgrade soil. Good compaction of the embankment will increase the strength of the embankment soil, decrease the required strength of the reinforcement, thus decreasing the cost of the embankment.

If vertical drains are needed to stabilize the subgrade, the reinforced embankment will be constructed to an elevation of approximately 4.0 to 5.0 m (13 to 16 ft) PWD. The vertical drains will then be installed through the reinforced embankment into the subgrade soil.

Following installation of the vertical drains, the subgrade soils will consolidate until an average degree of consolidation of about 90 percent is achieved. The embankment will then be constructed to the full height as shown in Figure 6-11.

6.3.4 Option 4: Sheet Pile Wall

In areas where the subgrade soils are under water, the sheet pile wall option will be installed. The sheet pile wall will consist of interlocking steel piles driven in two rows, 8 to 10 m (26 to 33 ft) apart, along the embankment alignment, as shown in Figure 6-12. The space between the sheet piles will be backfilled with hydraulic fill (sand) to the water table, then with compacted low permeability soil.

The exterior, or river side, sheet pile wall will be driven into the dense sand subgrade soil. The interior, or city side, sheet pile wall will be driven to sufficient depth to provide anchorage for the exterior wall. Tie rods and anchor plates will be installed between the interior and exterior walls to provide stability. The sheet pile walls will be designed under a subsequent contract, once the field investigation and laboratory testing programs have been completed.

The sheet pile wall alternative offers tremendous flexibility, particularly in the commercial areas of Dhaka where floating docks could be installed. These floating docks could be accessed by self-compensating ramps, as shown in Figure 6-13, which would allow restricted access to the river regardless of the river stage.

The floating dock could be configured in a way that provided tremendous flexibility to the Port Authority, as shown in Figure 6-14. In addition, access to the docks could be controlled. Tariffs or port fees could be administered in a controlled way at the access points to the docks.

The fill on the city side could be brought level with the top of the interior sheet pile wall, which would create additional land for development. In areas where existing development precludes placement of fill, limited access could be provided using access ramps. A typical profile of an access ramp is shown in Figure 6-15.

6.4 Summary

The selection of the remedial design options along the proposed embankment extension will depend on:

- shear strength of the subgrade soil;
- space for the embankment; and
- proximity to the water table or to the river.

Four options have been identified for construction of the embankment extension. These options will be installed as follows:

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- Option 1: Conventional Embankment - installed on firm subgrade soils where space allows;

- Option 2: Base Reinforced Embankment with Vertical Drains - installed on soft subgrade soils where space allows;

- Option 3: Reinforced Embankment with or without Vertical Drains - installed in areas where there is insufficient space to install a conventional embankment; and

- Option 4: Sheet Pile Wall - installed in areas where the subgrade is submerged during construction.

The locations where each option will be installed will be determined in the field by the Geotechnical Expert. The transitions between each section will be designed during a subsequent phase of the contract.

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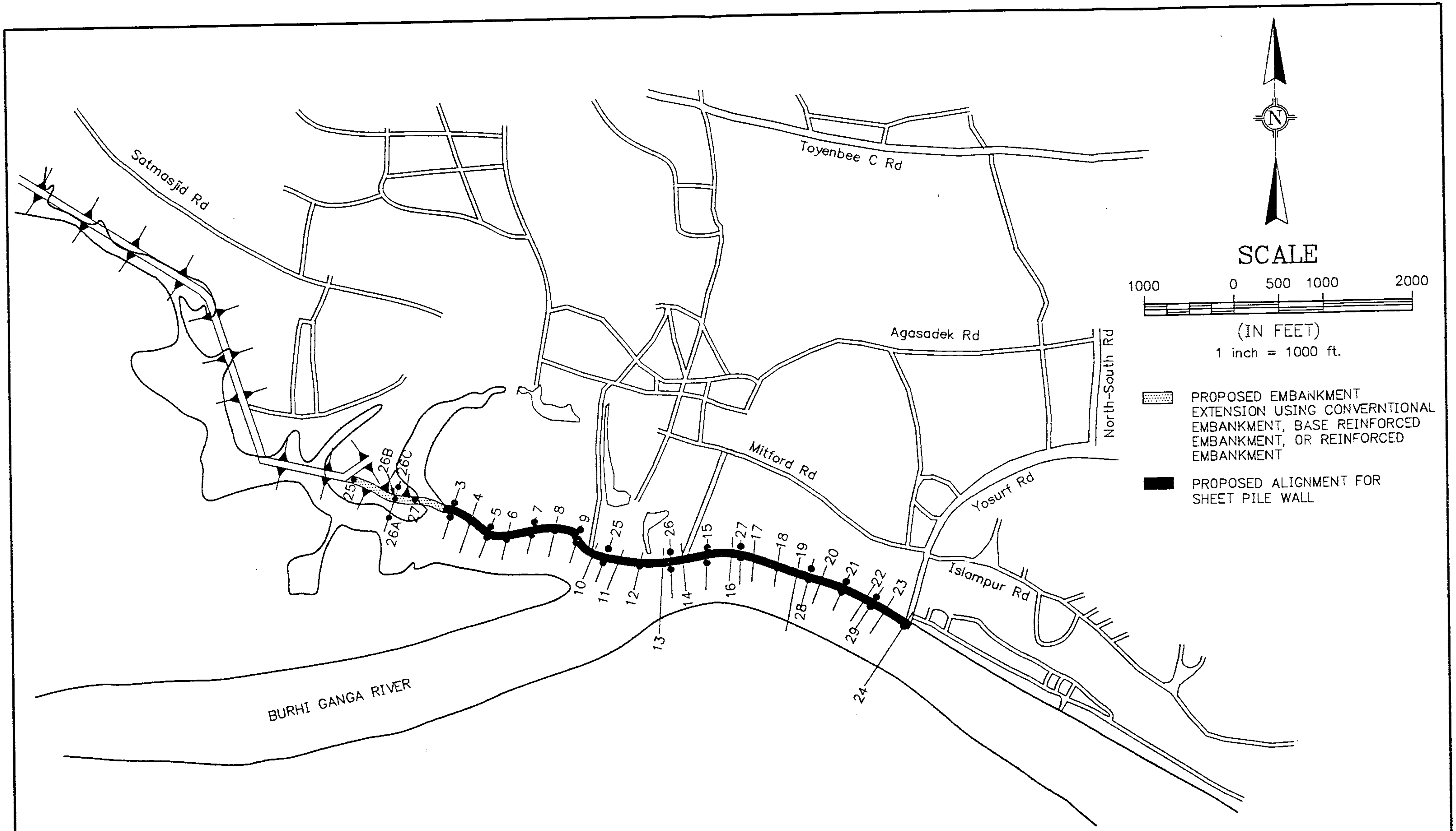

EMBANKMENT EXTENSION SITE PLAN

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FIGURE NO.	6-1
PROJECT NO.	FE2043
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PAGE NO.	98

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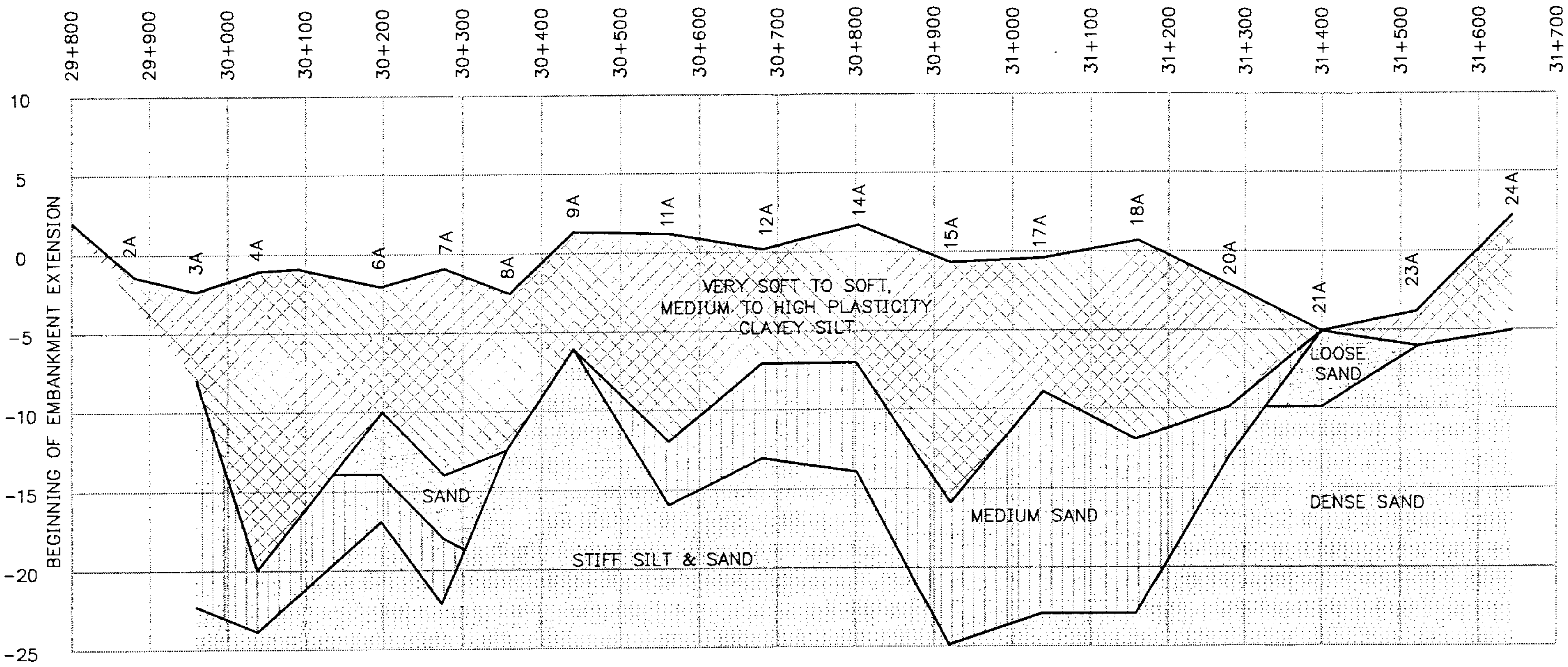

EMBANKMENT EXTENSION LONGITUDINAL SECTION

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FIGURE NO.	6-2
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	99

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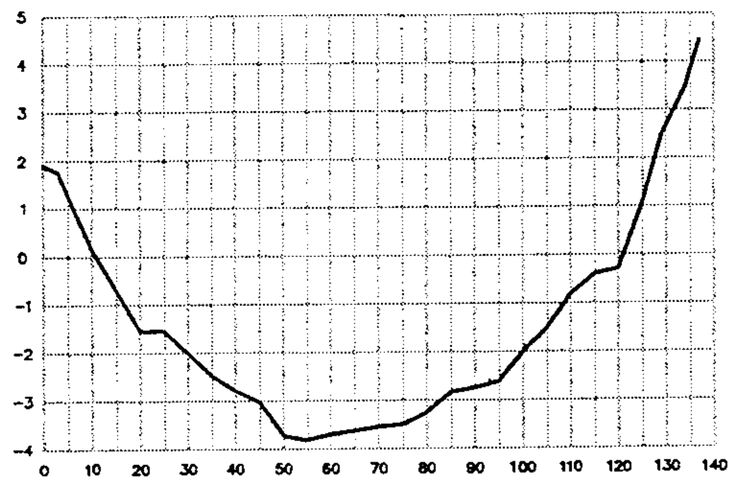

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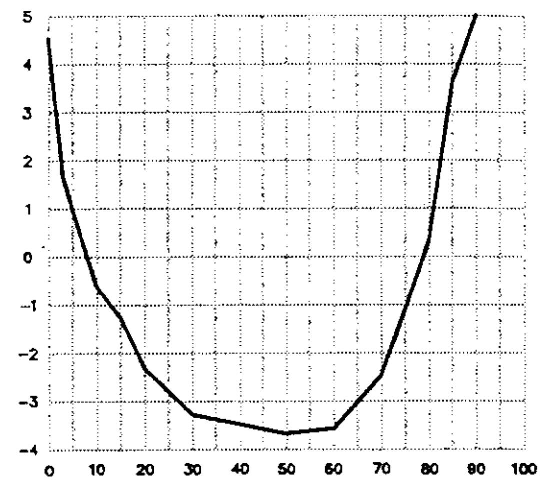
OF EMBANKMENT EXTENSION AREA



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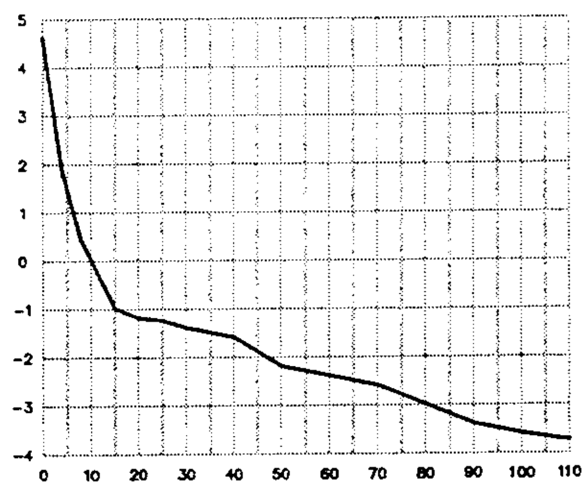

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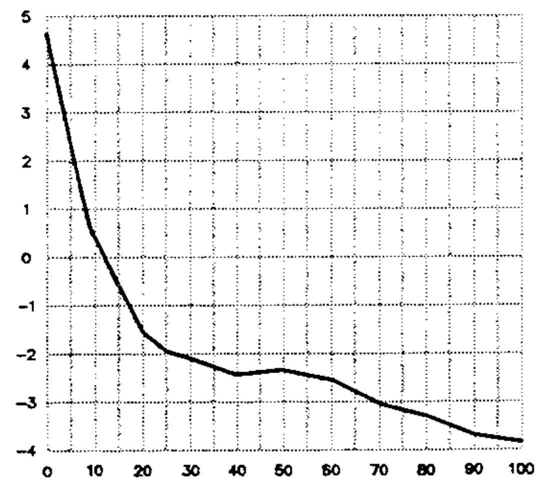

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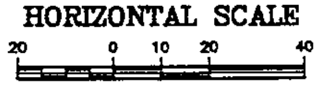
OF EMBANKMENT EXTENSION AREA

HORIZONTAL SCALE



(IN FEET)
1 inch = 20 ft.

VERTICAL SCALE

(IN FEET)
1 inch = 2 ft.

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FIGURE NO.	6-3
PROJECT NO.	FE2043
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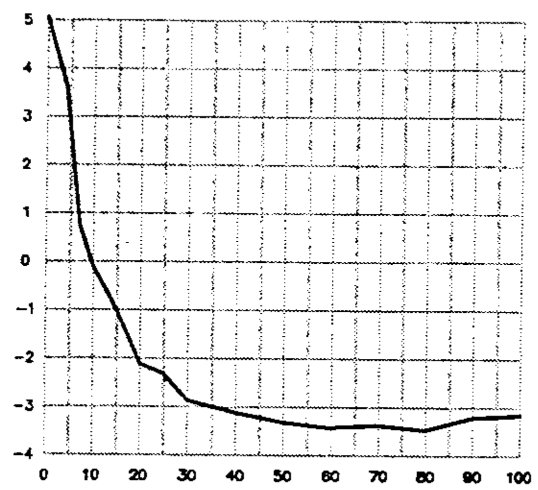

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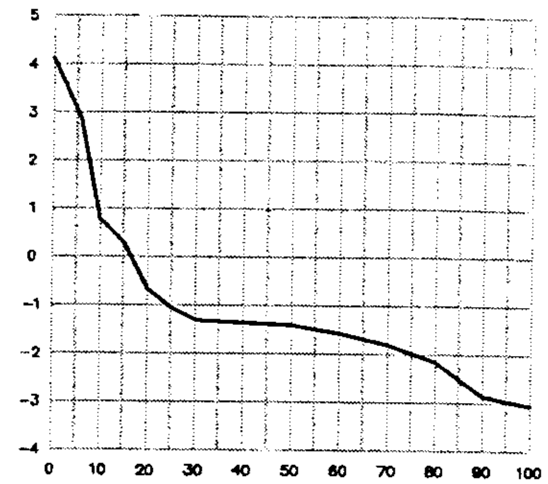

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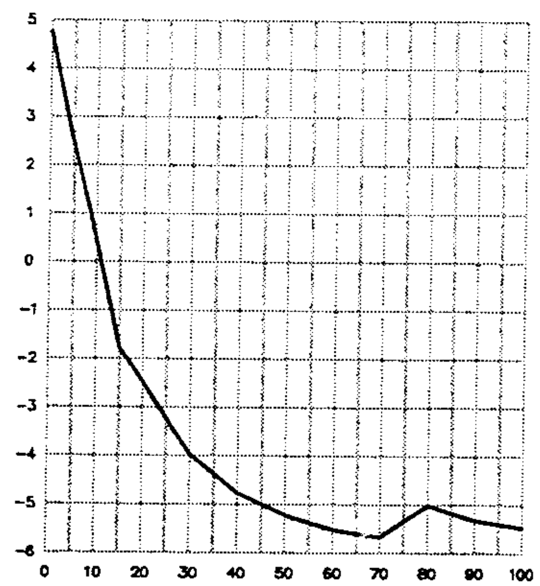

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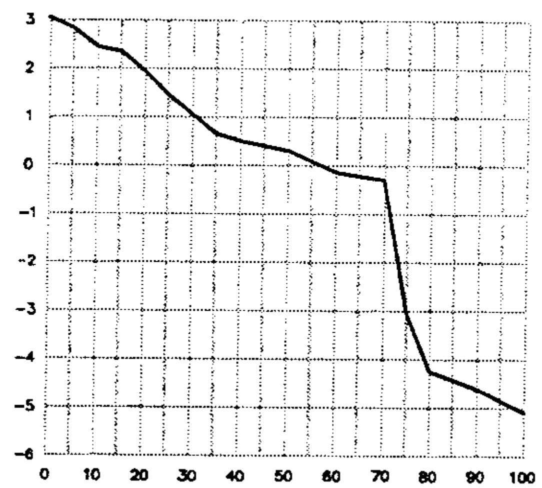

SECTION NO. 9
OF EMBANKMENT EXTENSION AREA

HORIZONTAL SCALE
20 0 10 20 40
(IN FEET)
1 inch = 20 ft.

VERTICAL SCALE
2 0 1 2 4
(IN FEET)
1 inch = 2 ft.

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FIGURE NO.	6-4
PROJECT NO.	FE2043
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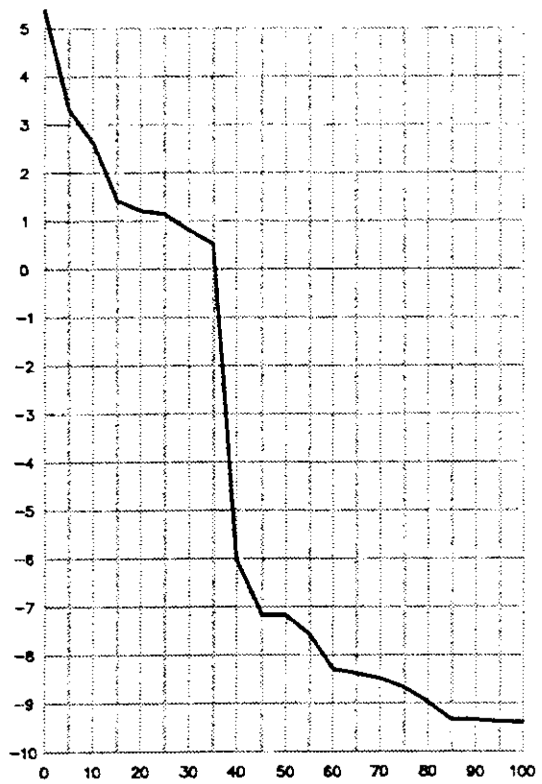
PAGE NO.	101



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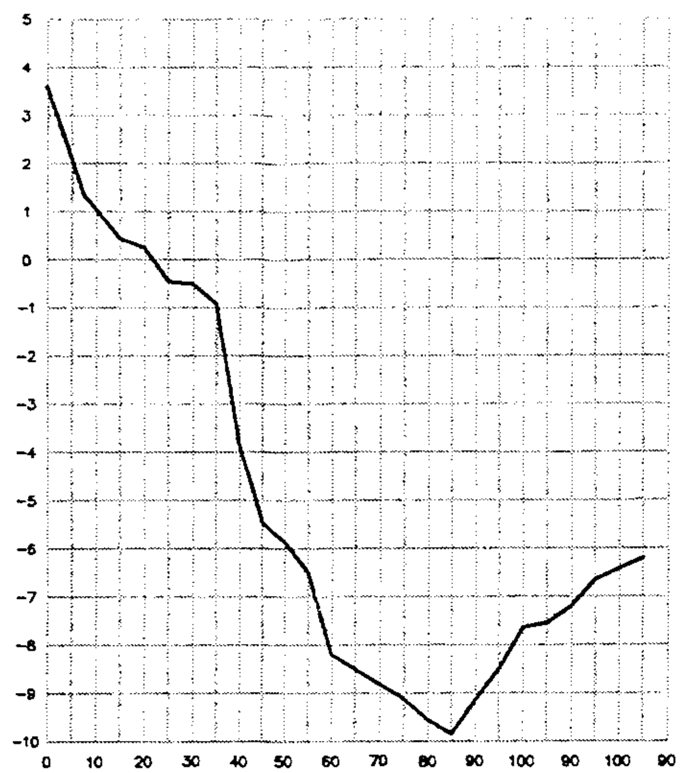
OF EMBANKMENT EXTENSION AREA



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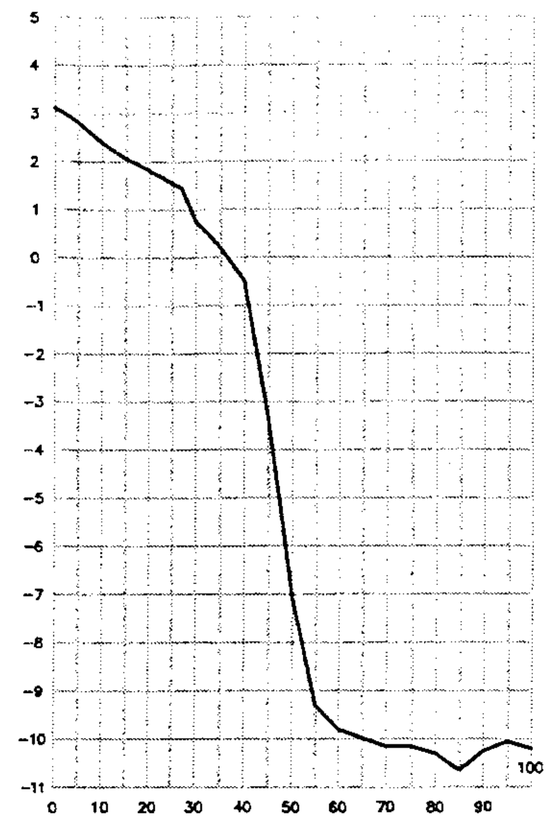

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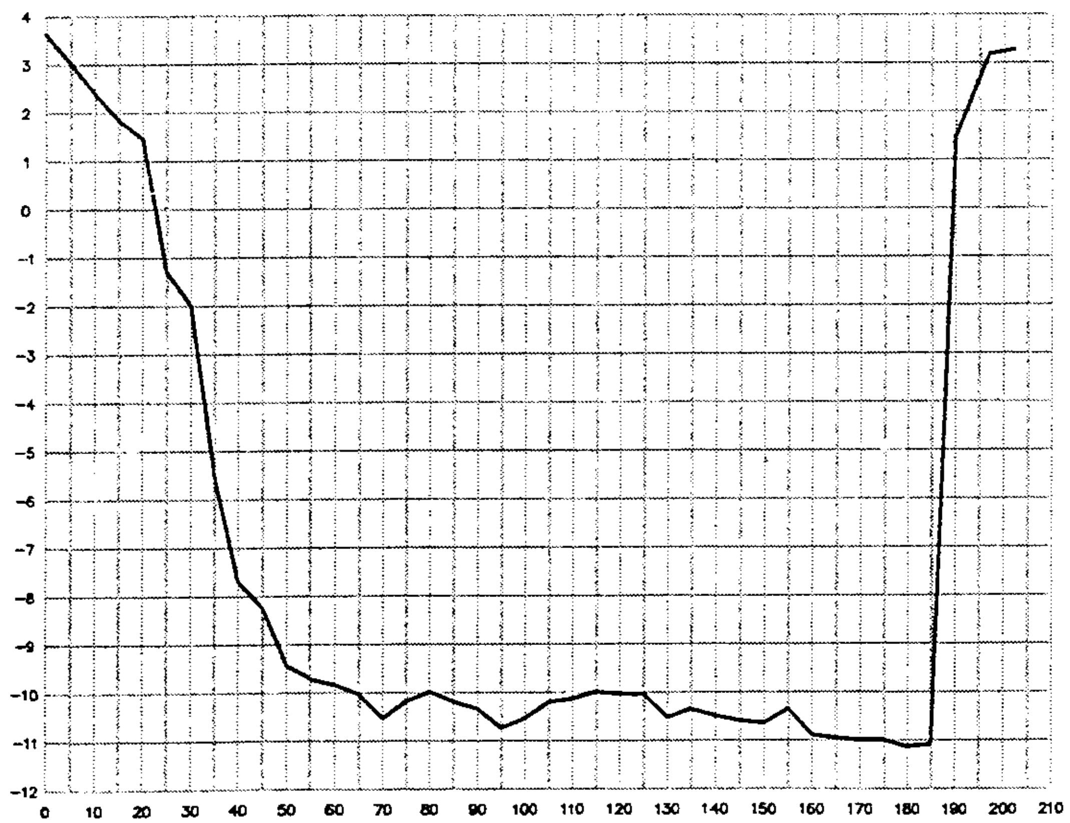

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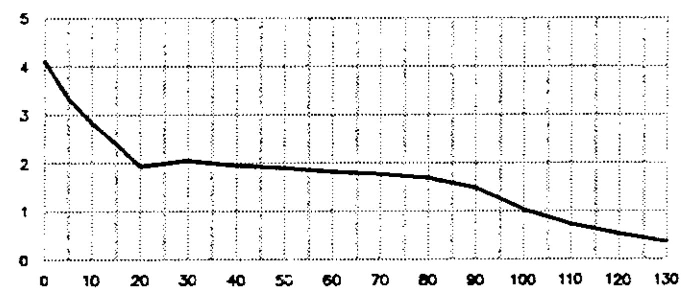
OF EMBANKMENT EXTENSION AREA



SECTION NO. 10
OF EMBANKMENT EXTENSION AREA

HORIZONTAL SCALE

20 0 10 20 40

(IN FEET)
1 inch = 20 ft.

VERTICAL SCALE

2 0 1 2 4

(IN FEET)
1 inch = 2 ft.

GEOSYNTEC CONSULTANTS

FIGURE NO.	6-5
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	102

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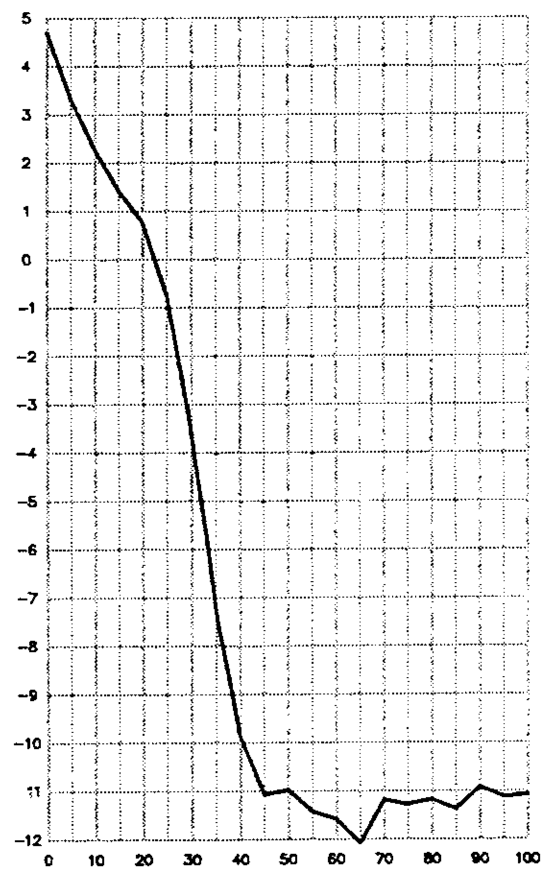
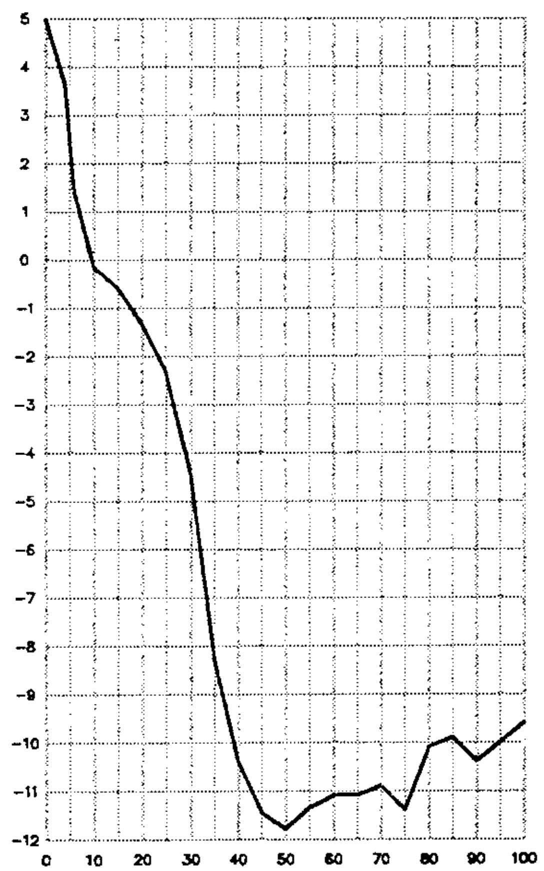
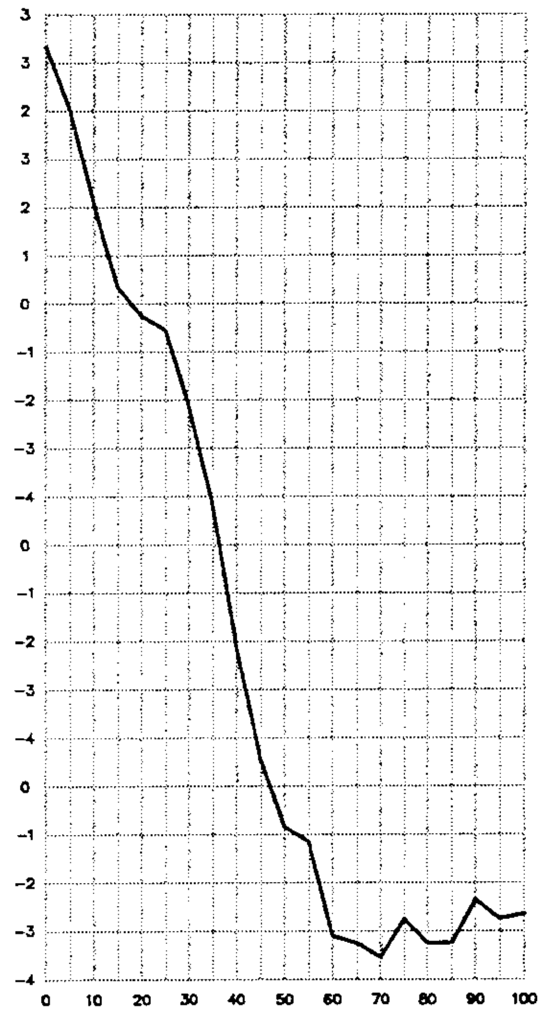


SECTION NO. 15
OF EMBANKMENT EXTENSION AREA



SECTION NO. 16
OF EMBANKMENT EXTENSION AREA



SECTION NO. 17
OF EMBANKMENT EXTENSION AREA

SECTION NO. 18
OF EMBANKMENT EXTENSION AREA

HORIZONTAL SCALE

(IN FEET)
1 inch = 20 ft.

VERTICAL SCALE

(IN FEET)
1 inch = 2 ft.

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FIGURE NO.	6-6
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	103

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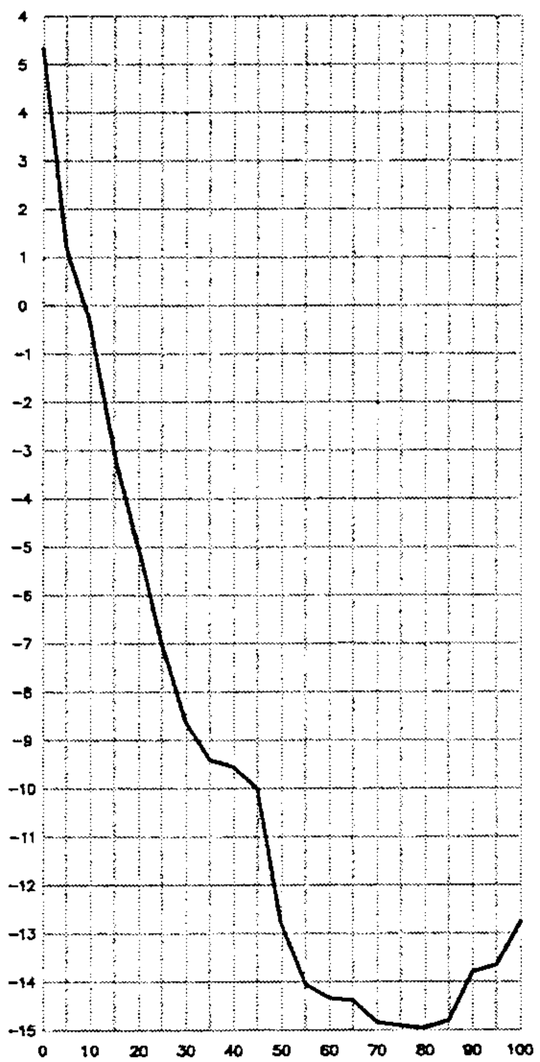

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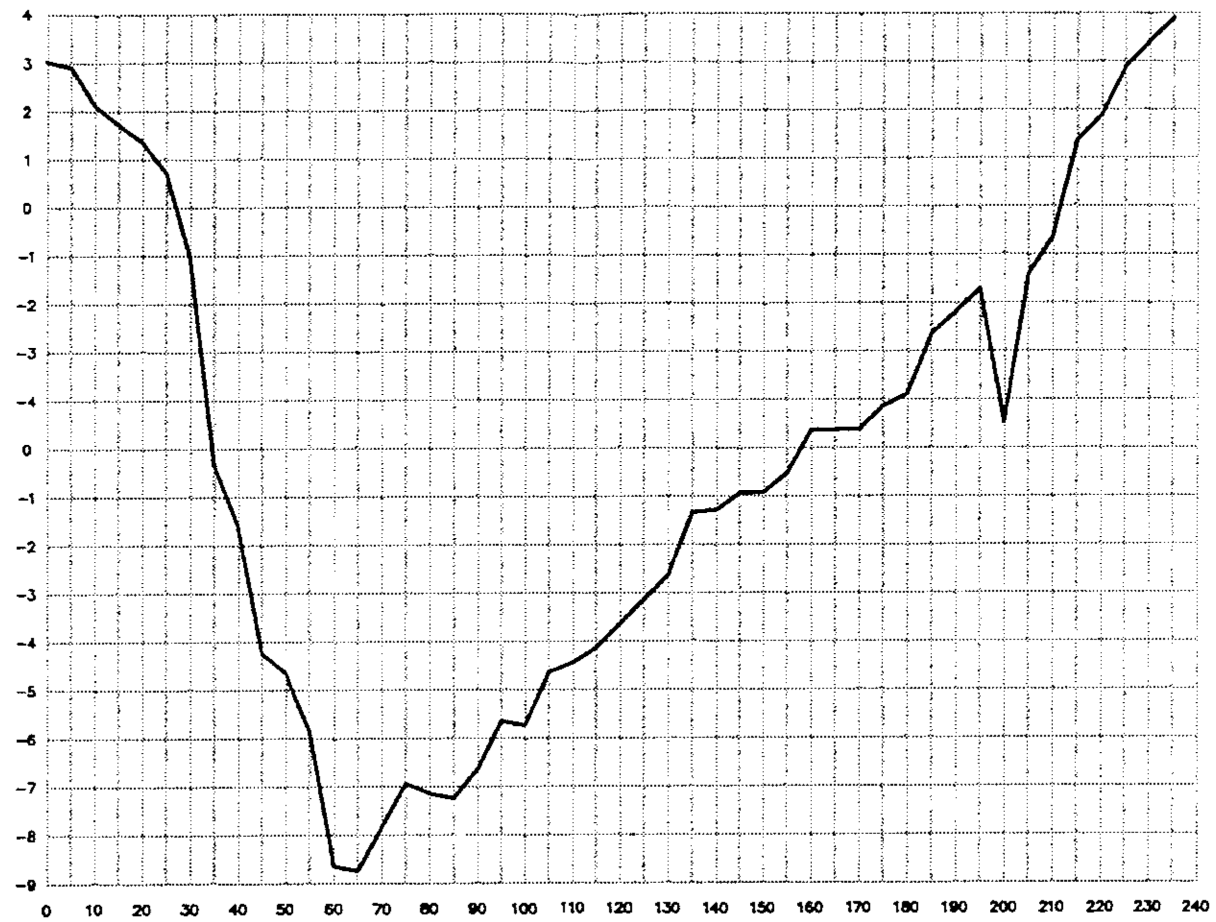
OF EMBANKMENT EXTENSION AREA



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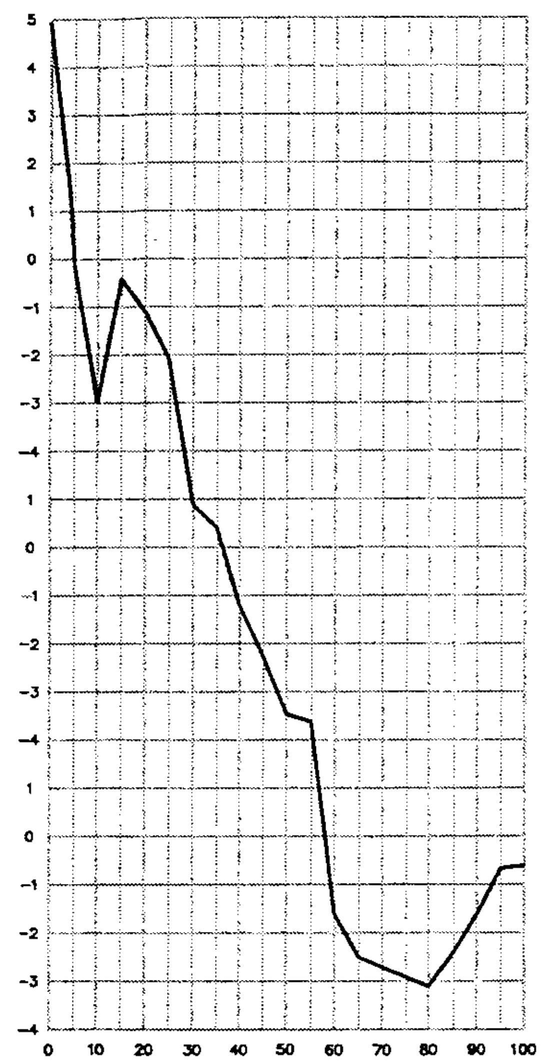
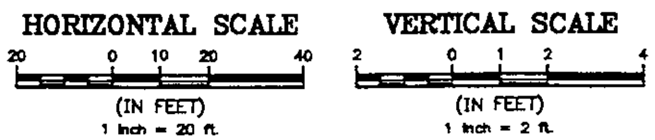
OF EMBANKMENT EXTENSION AREA

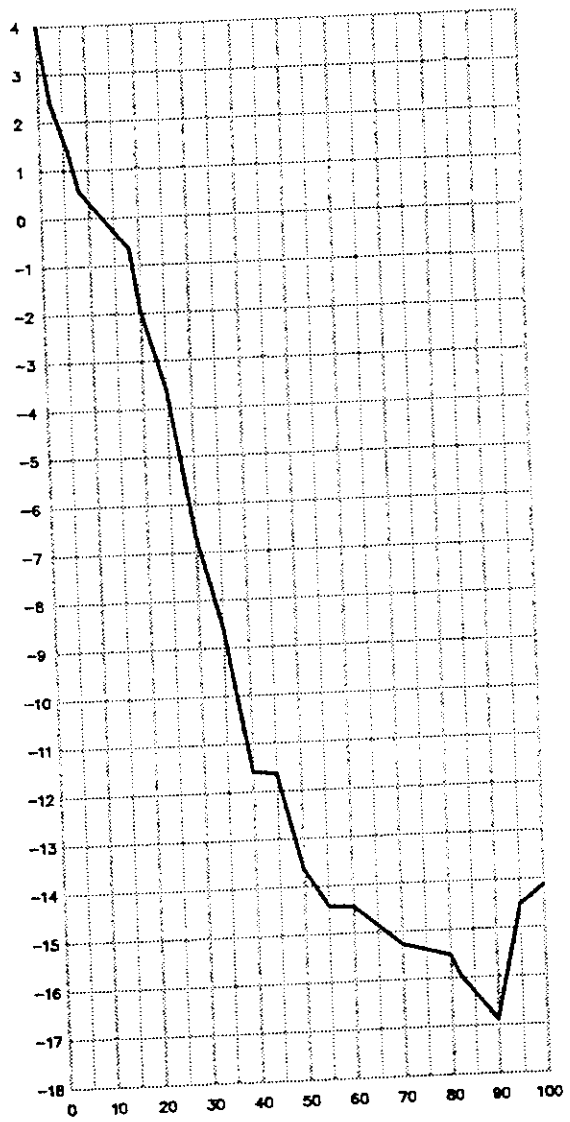


SECTION NO. 20
OF EMBANKMENT EXTENSION AREA



GEOSYNTEC CONSULTANTS

FIGURE NO.	6-7
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	104



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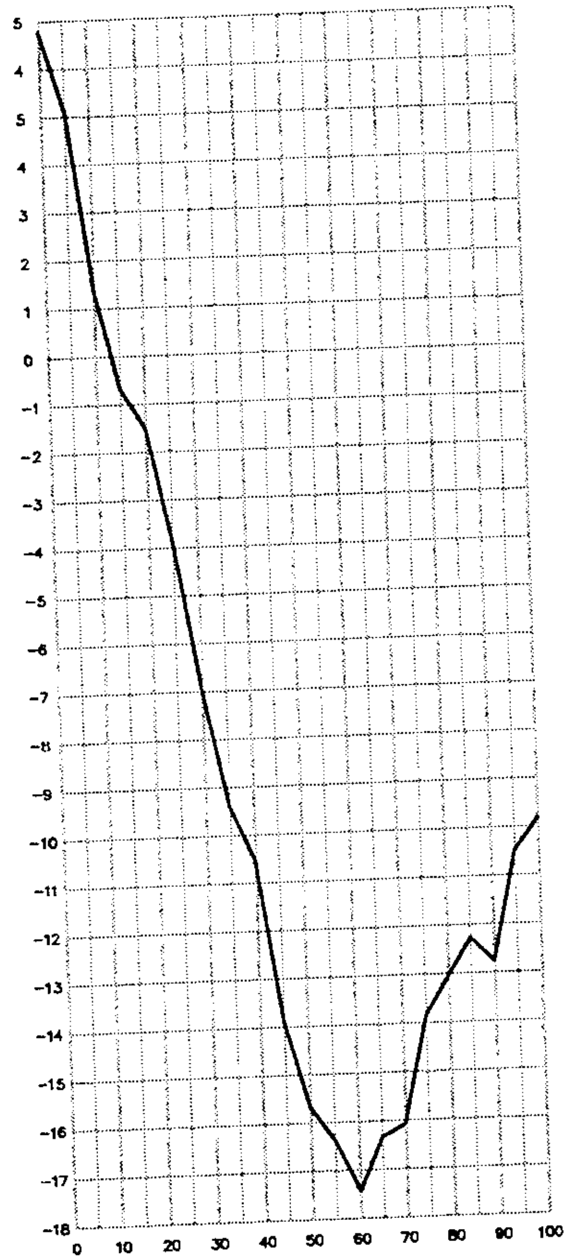
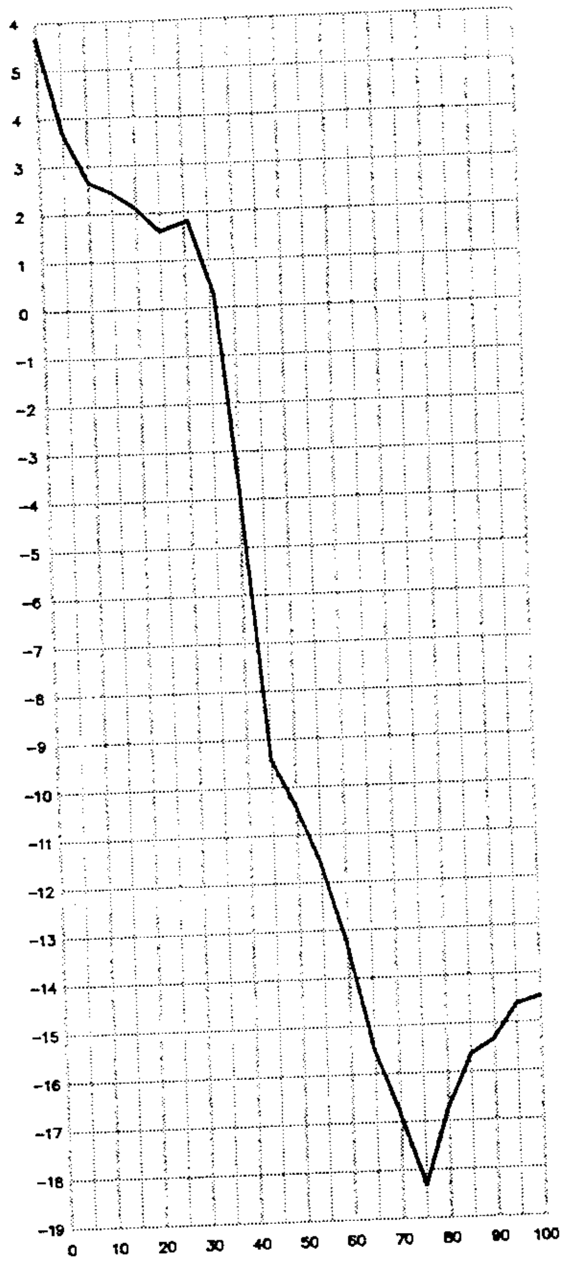
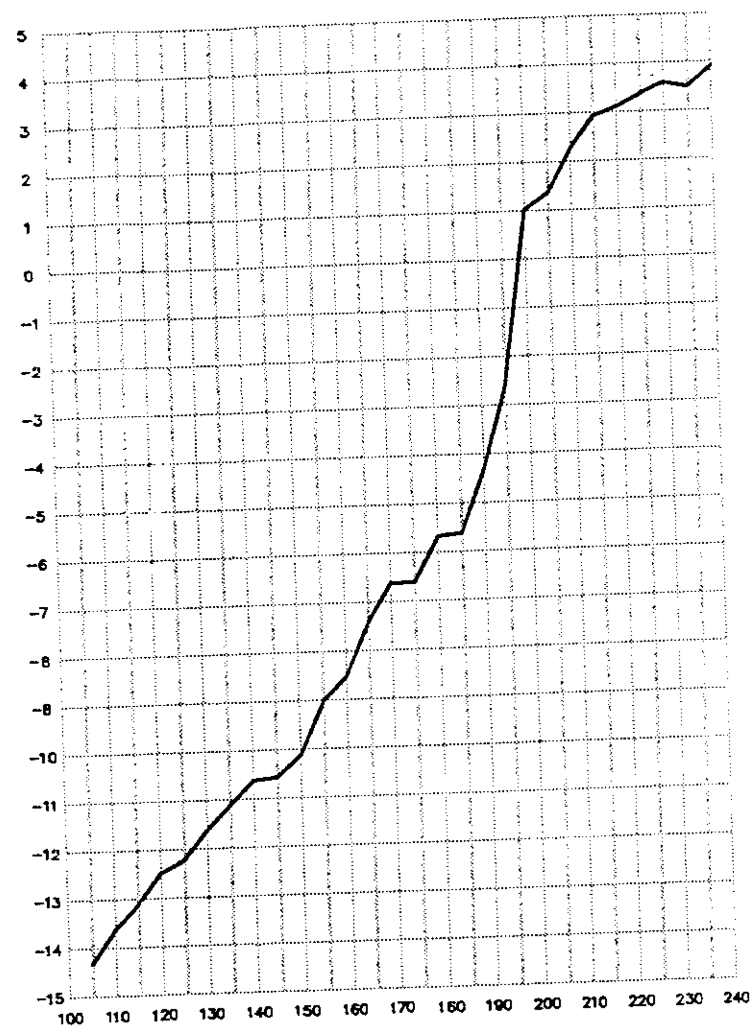
OF EMBANKMENT EXTENSION AREA



SECTION NO. 23
OF EMBANKMENT EXTENSION AREA



SECTION NO. 24
OF EMBANKMENT EXTENSION AREA



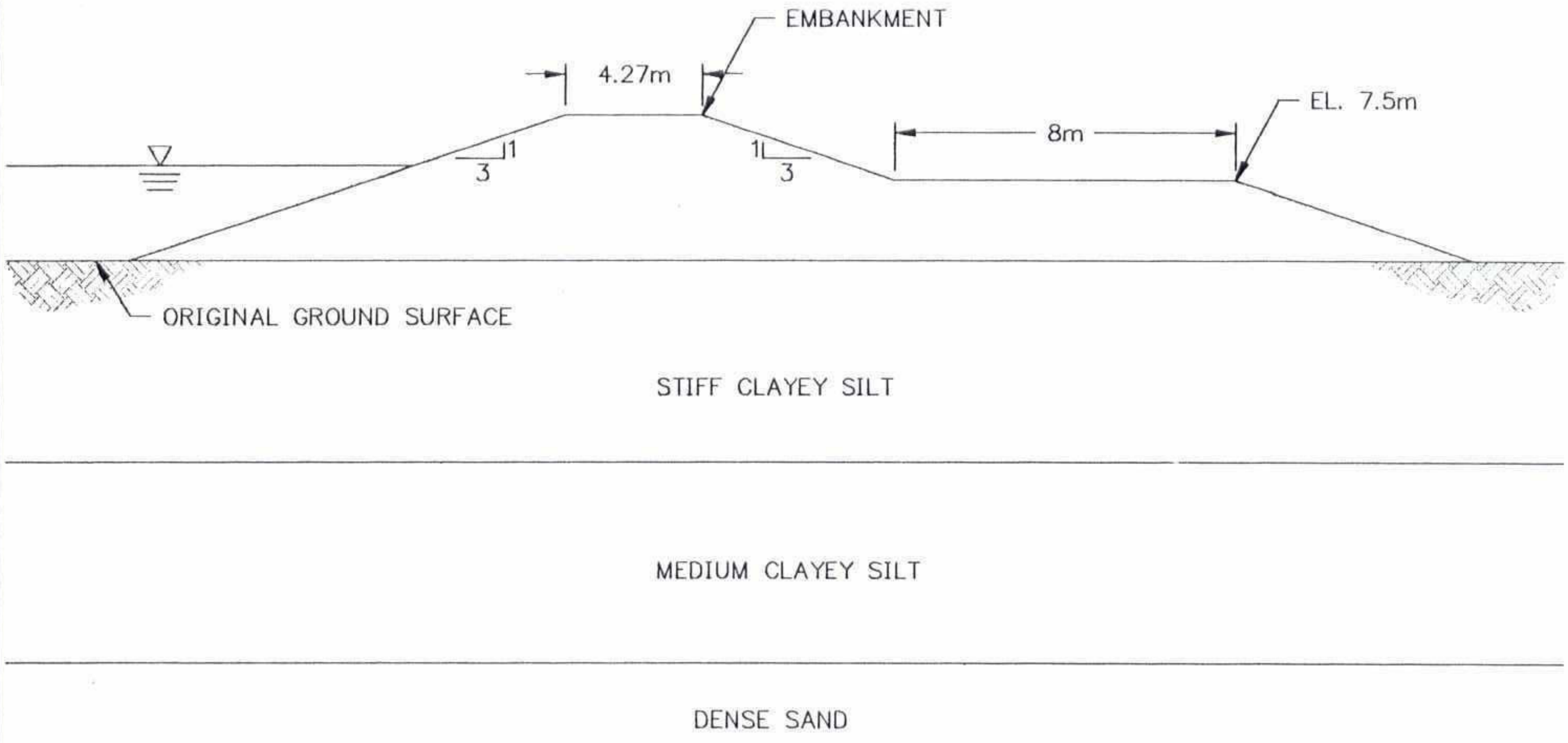
SECTION NO. 25
OF EMBANKMENT EXTENSION AREA

HORIZONTAL SCALE
20 0 10 20 40
(IN FEET)
1 inch = 20 ft.

VERTICAL SCALE
2 0 1 2 4
(IN FEET)
1 inch = 2 ft.

GEOSYNTEC CONSULTANTS

FIGURE NO.	6-8
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	105

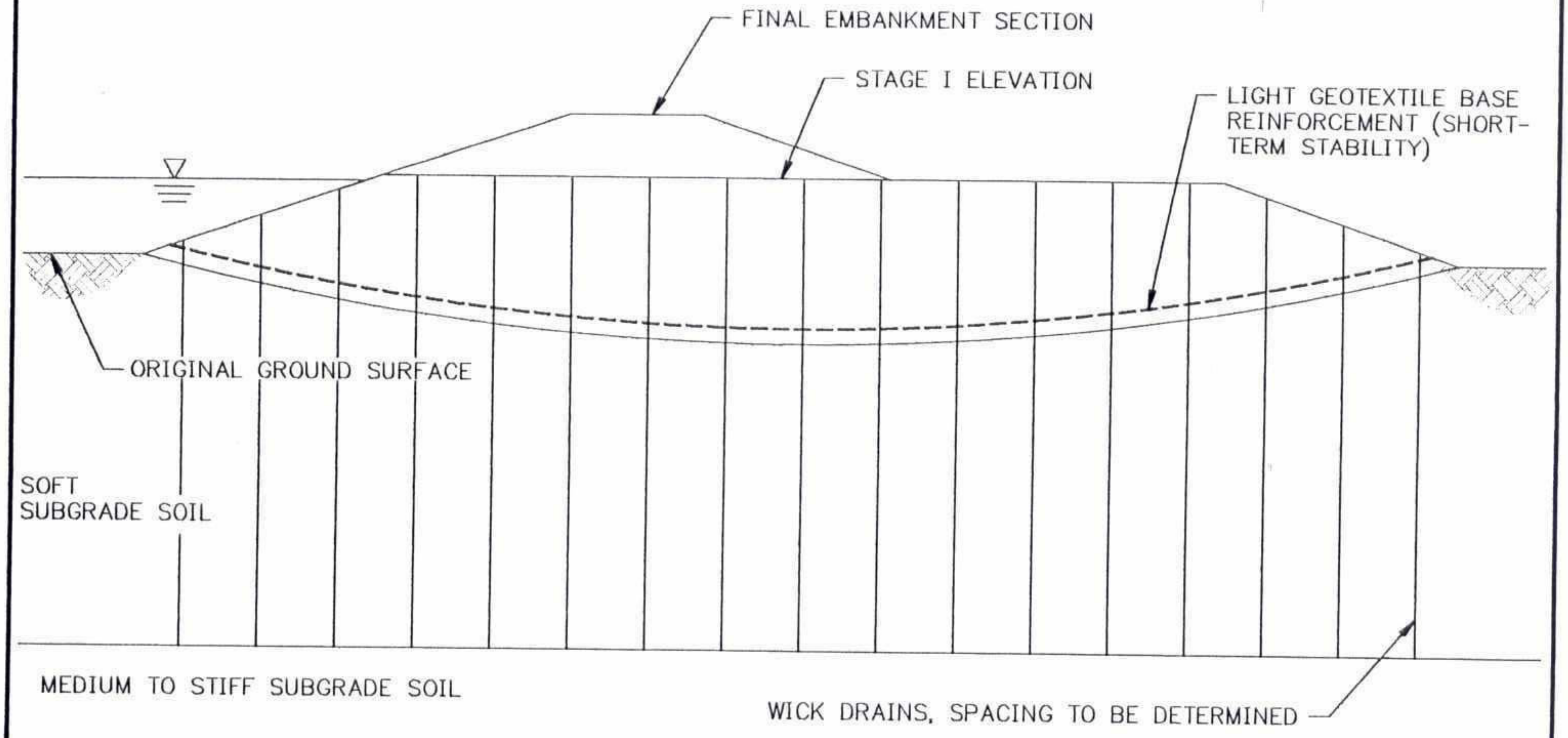


EMBANKMENT EXTENSION OPTION 1 CONVENTIONAL CONSTRUCTION

GeoSyntec Consultants

FIGURE NO.	6-9
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	106

DWG: 2043F021 199209100956 SB



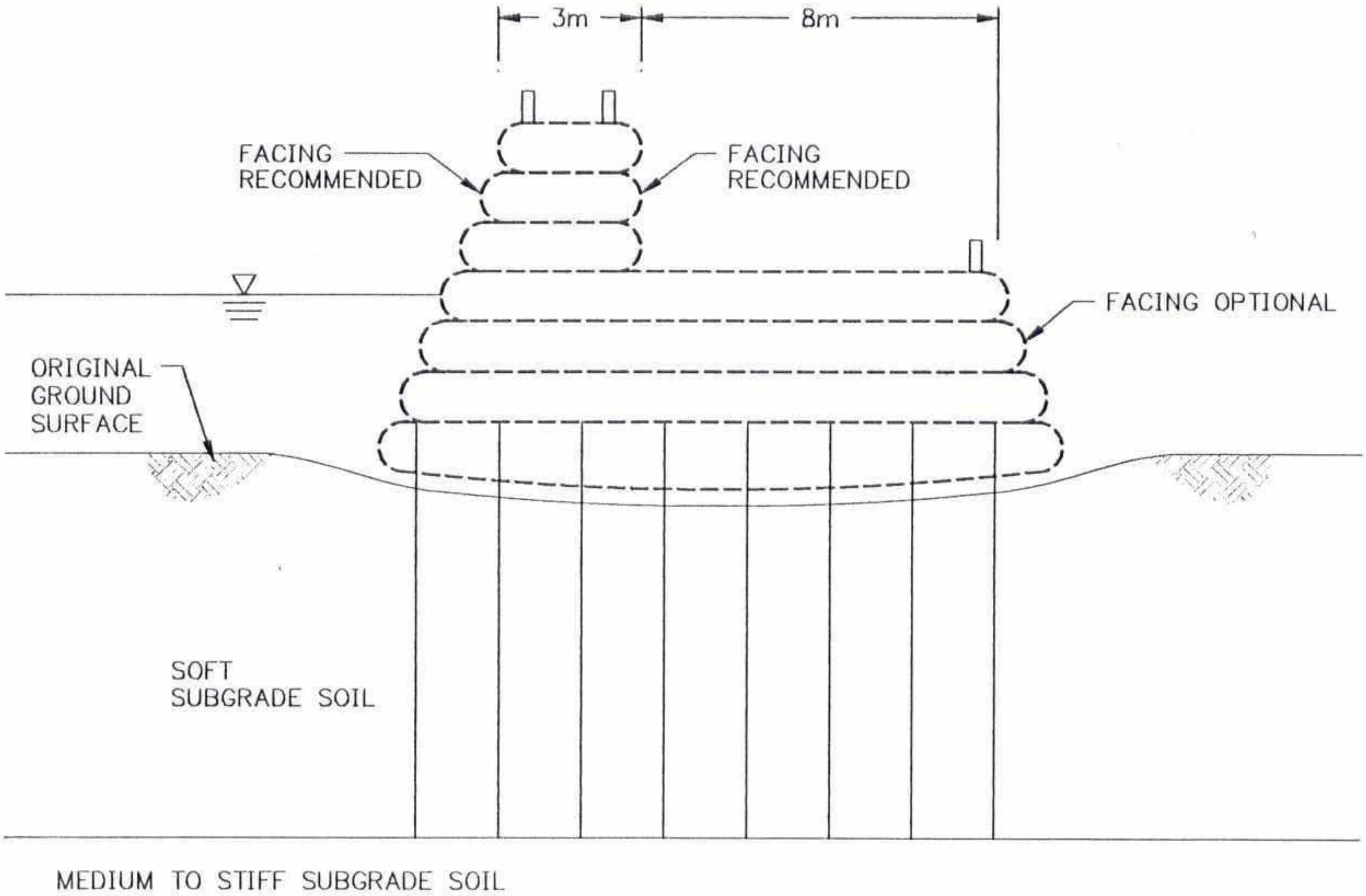
EMBANKMENT EXTENSION OPTION 2 VERTICAL
DRAINS WITH BASE REINFORCEMENT

GEOSYNTEC CONSULTANTS

FIGURE NO.	6-10
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	107

DWG: 2043F021 199209100956 SB

238

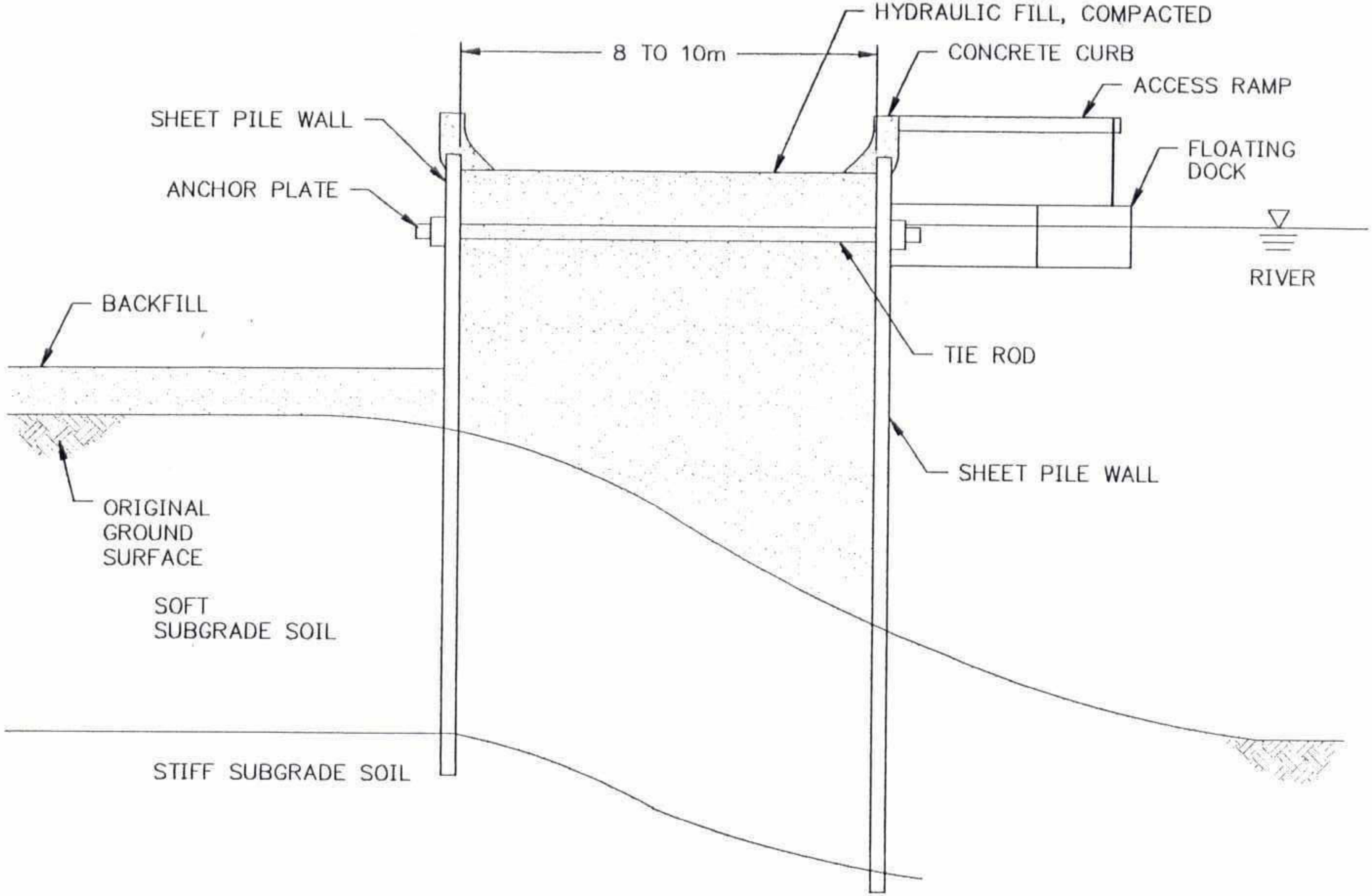


EMBANKMENT EXTENSION OPTION 3 WICK
DRAINS WITH EMBANKMENT REINFORCEMENT

GEOSYNTEC CONSULTANTS

FIGURE NO.	6-11
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	108

DWG: 2043F021 199209100956 SB



EMBANKMENT EXTENSION OPTION 4, SHEET PILE WALL

GeoSyntec Consultants		
	FIGURE NO.	6-12
	PROJECT NO.	FE2043
	DOCUMENT NO.	
	PAGE NO.	109

DWG: 2043F021 199209100956 SB

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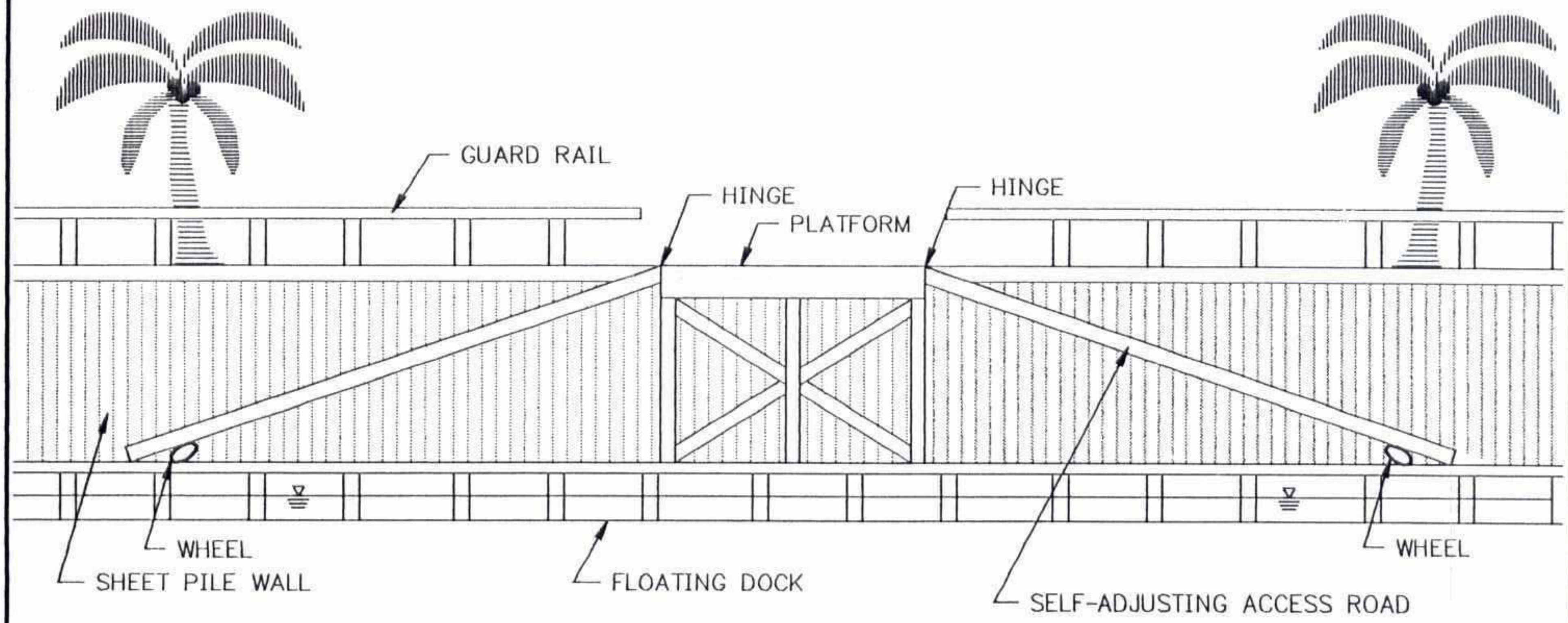
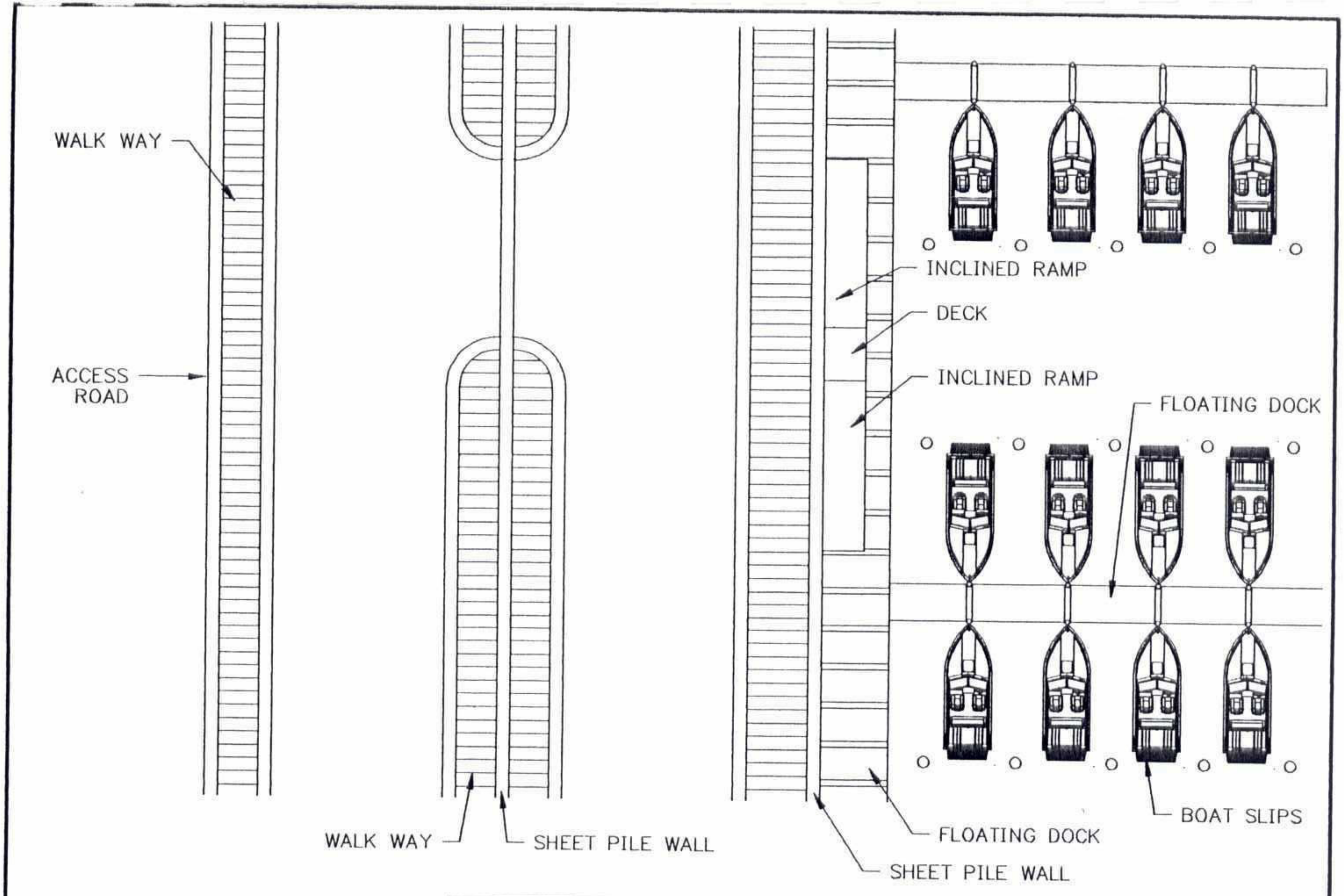


RIVER SIDE ACCESS RAMP

GEOSYNTEC CONSULTANTS

FIGURE NO.	6-13
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	110

DWG: 2043F021 199209100956 SB



PLAN VIEW OF ACCESS AREA ON SHEET PILE WALL SECTION

GeoSyntec Consultants

FIGURE NO.	6-14
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	111

DWG: 2043F021 199209100956 SB

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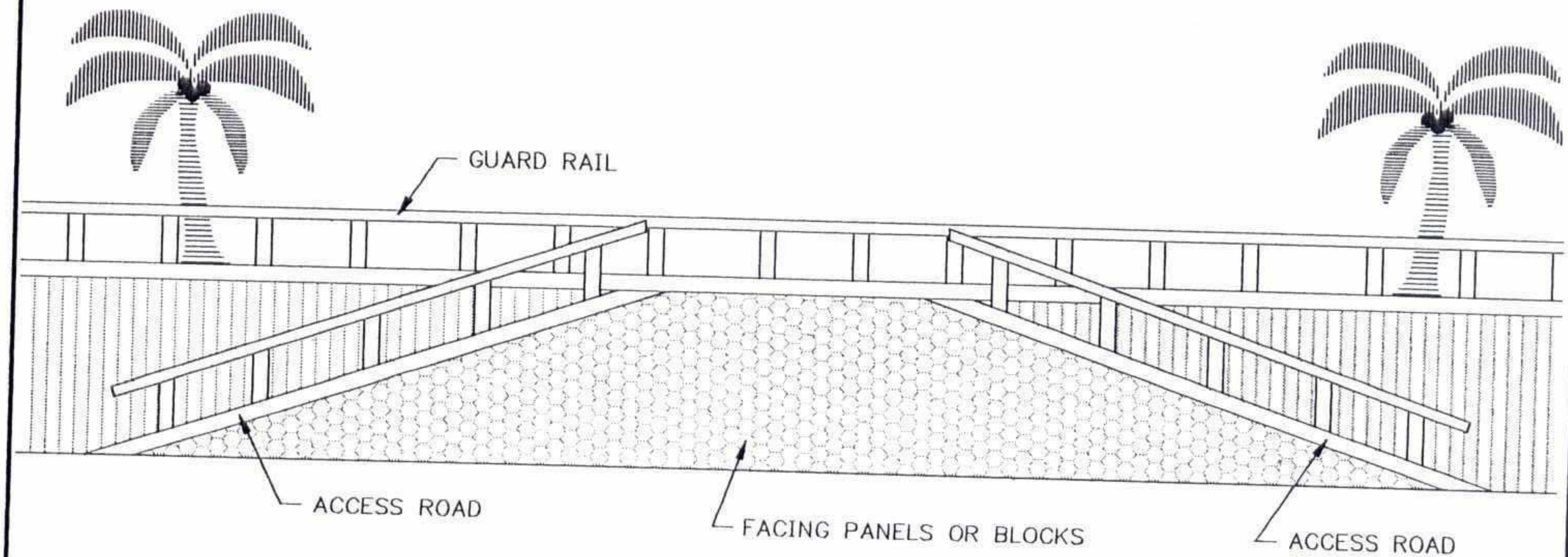

CITY SIDE ACCESS ROAD
SHEET PILE WALL SECTION

GEOSYNTEC CONSULTANTS

FIGURE NO.	6-15
PROJECT NO.	FE2043
DOCUMENT NO.	
PAGE NO.	112

DWG: 2043F021 199209100956 SB

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7. CONSTRUCTION QUALITY ASSURANCE

7.1 Introduction

Construction Quality Assurance (CQA) is defined as the methodology used to assure or certify that the structures or facilities are constructed as designed. CQA is implemented through the design report, CQA plan, engineering drawings, and construction specifications.

7.2 CQA Plan

A CQA plan will be developed that outlines and describes the responsibilities of the Owner's Representative (WDB), CQA Project Manager, and the Contractors. The CQA plan will describe the meetings, interaction between parties, responsibilities before, during and after construction, monitoring activities, and dispute resolution procedures.

The CQA plan will be prepared during a subsequent phase of the Contract.

7.3 Construction Specifications

Construction specifications have been prepared for Earthwork (Section 02200), and Vertical Strip Drain Installation (Section 02300). These specifications are presented in Appendix D-2.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 Introduction

Conclusions and recommendations have been prepared based on the previous site investigations and studies, the field investigation described in Section 2, the laboratory testing program described in Section 3, and the stability analyses described in Section 4. The conclusions and recommendations are presented in Sections 8.2 and 8.3, respectively.

8.2 Conclusions

The following conclusions have been drawn based on the work performed to date:

- Approximately 850 m (2800 ft) of the existing embankment is unstable and could fail catastrophically. These sections are classified as Class I areas and include:

  - Stations 9+750 to 10+100 m, and

  - Stations 13+850 to 14+350 m.

- Approximately 1700 m (5600 ft) of the embankment is unstable and will fail unless remediated. These areas will not likely fail catastrophically, and are classified as Class II areas requiring additional remediation.

- Approximately 4500 m (15,000 ft) of the existing embankment was previously classified as a Class I or Class II area, but has been satisfactorily remediated. Some of these areas have low existing factors of safety (less than 1.2), but stability will improve as

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the subgrade and embankment consolidate. These areas are classified as Class II areas requiring monitoring and inspection.

- Failure of the existing embankment resulted from:

  - poor compaction of the embankment soils;

  - inadequate subgrade improvement;

  - lack of erosion protection; and

  - use of inappropriate soils in the embankment.

- Construction of toe berms or flattening the slopes in the Class I and Class II areas requiring remediation has not been effective due to the depth of soft subgrade soils and the low shear strength of the embankment.

- Installation of vertical drains, staged construction, and good construction procedures will increase the factor of safety to more than or equal to 1.2 in the Class I and Class II areas needing remediation.

- The factor of safety will increase over time until a factor of safety of 1.2 is achieved. Failure of the embankment is possible until the desired factor of safety is achieved, but the probability of failure decreases with time as the subgrade soils and embankment consolidate.

- Soft subgrade soils exist along the proposed alignment of the embankment extension between Kellar Mohr and the Mittford Hospital. These subgrade soils must be stabilized or the embankment extension will fail.

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- In areas where the proposed embankment extension is under water, the clayey silt should not be placed under water. The sheet pile wall option is recommended in these areas.

8.3 Recommendations

The following recommendations were developed based on the work performed to date:

- The installation of vertical drains in the Class I areas and Class II areas requiring remediation.

- The embankment between Stations 13+850 and 14+350 m should be excavated to an elevation of 5.0 m (16 ft) PWD. Wick drains should then be installed and the embankment allowed to consolidate prior to final construction. Staged construction is required in this area because the embankment and subgrade shear strength is very low. In addition, significant portions of the embankment in this area were constructed with inappropriate soils. These soils must be removed and replaced.

- A CQA plan should be developed, agreed to, and signed by the WDB, CQA engineer, and contractors. Remedial construction and construction of the embankment extension should not be initiated until the CQA plan has been signed and all parties agree to its implementation.

- A Geotechnical Expert should be used to complete the design of the embankment extension and oversee construction.

02 &

APPENDIX A

BORING LOGS FOR WESTERN EMBANKMENT

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APPENDIX B

RESULTS OF LABORATORY TESTS

APPENDIX B-1

GRAIN-SIZE DISTRIBUTION CURVES

APPENDIX B-2

ATTERBERG LIMITS

RJ

APPENDIX B-3

LABORATORY DATA SUMMARY TABLES

APPENDIX B-4

CONSOLIDATION TEST RESULTS

APPENDIX B-5

UNCONFINED COMPRESSION

TEST RESULTS

APPENDIX B-6

CONSOLIDATED UNDRAINED

($\overline{\text{CU}}$) TRIAXIAL SHEAR

STRENGTH TEST RESULTS

202

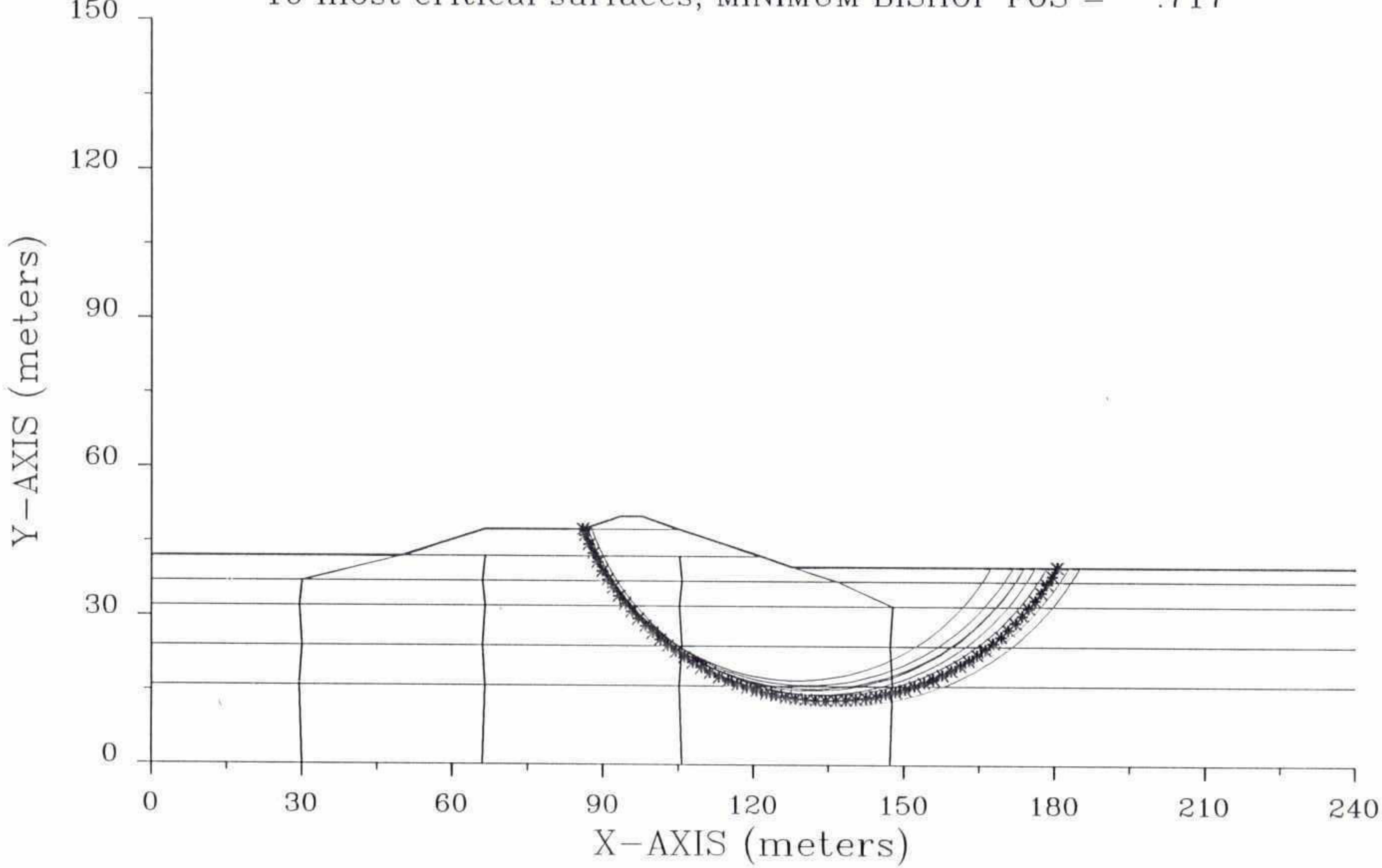
APPENDIX C-1

RESULTS OF STABILITY ANALYSES

DRSX1U0 17:28

SEGMENT 1 RIVER SIDE U=0

10 most critical surfaces, MINIMUM BISHOP FOS = .717



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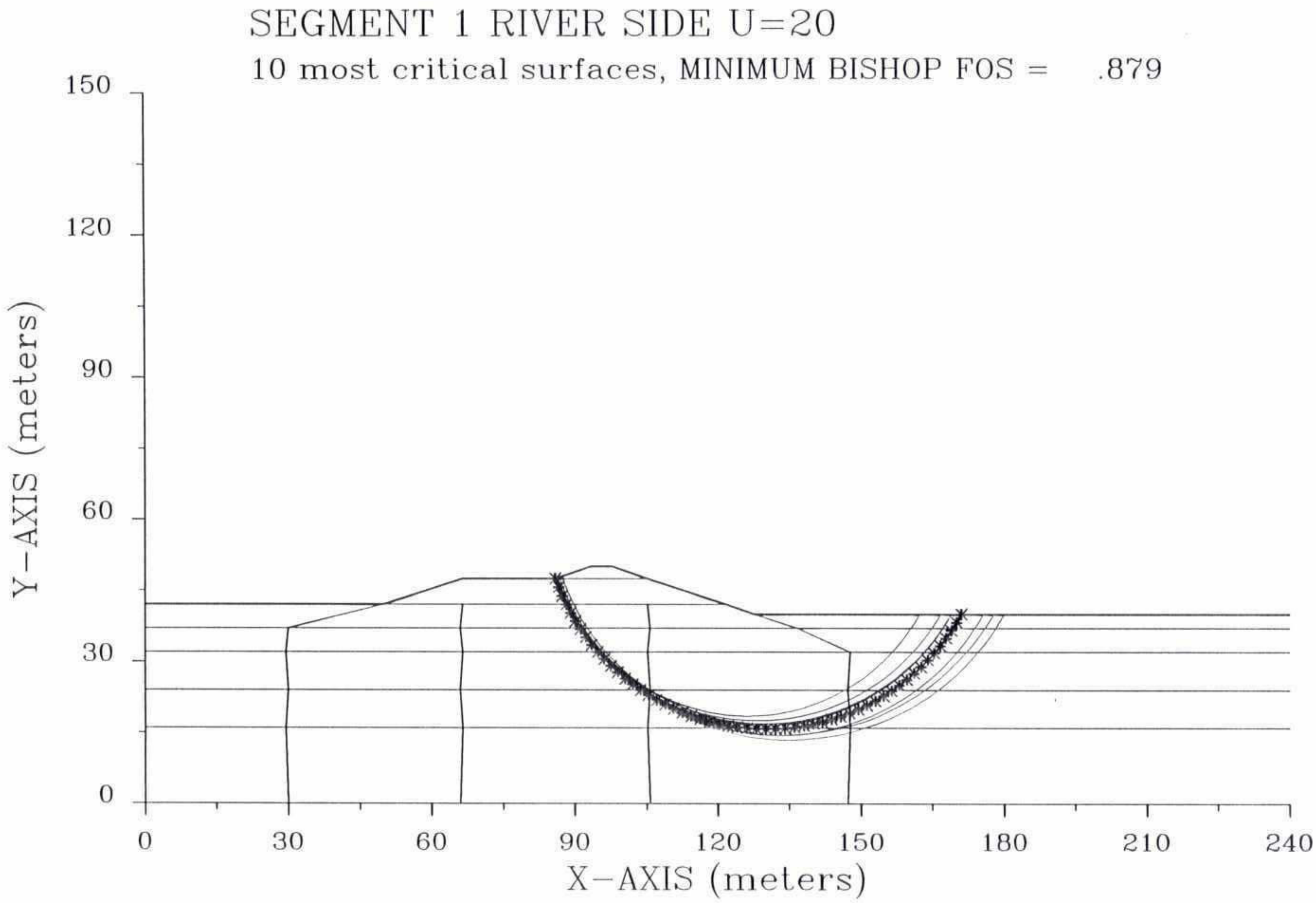
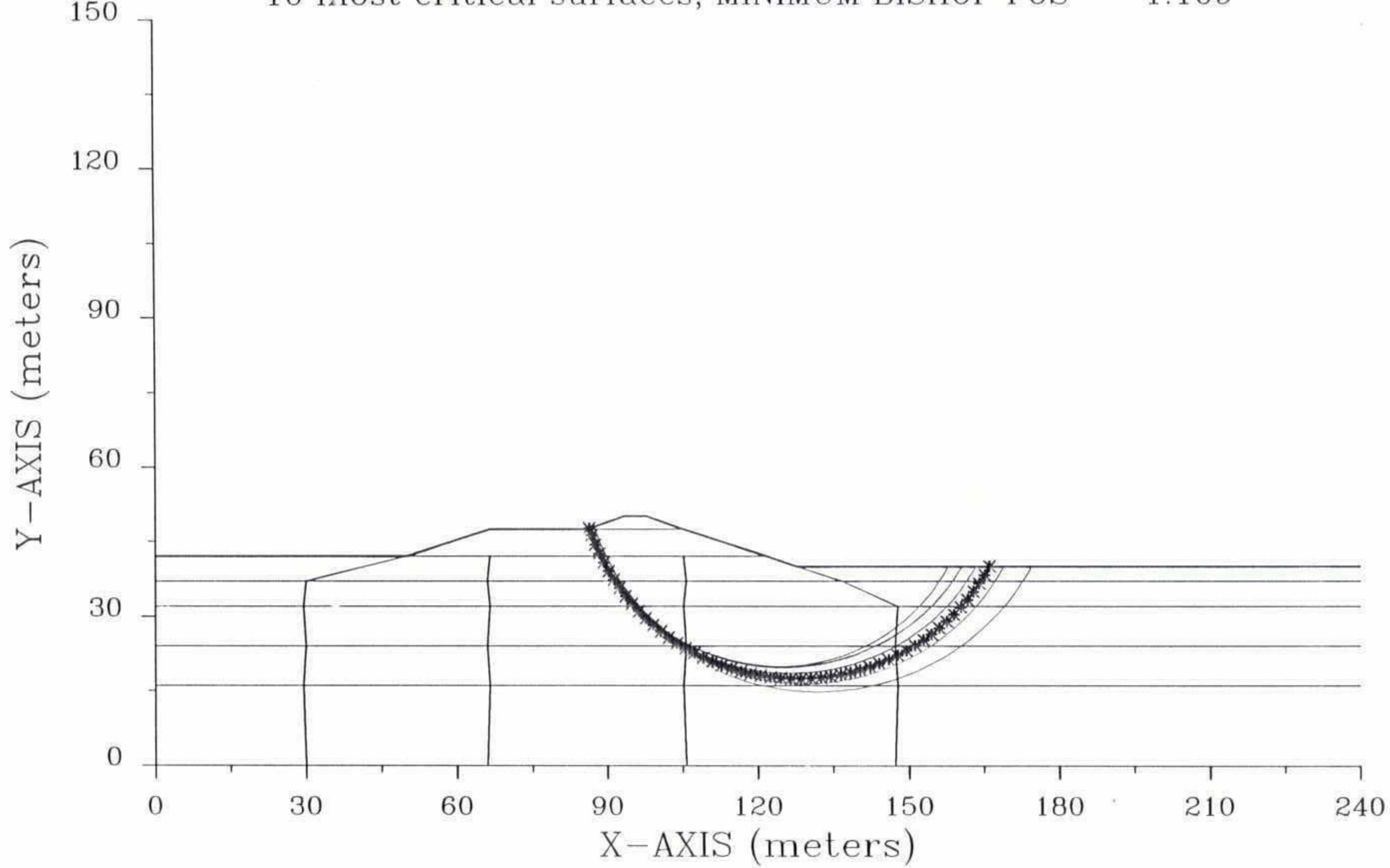


208

DRSX1U50 17:05

SEGMENT 1 RIVER SIDE U=50

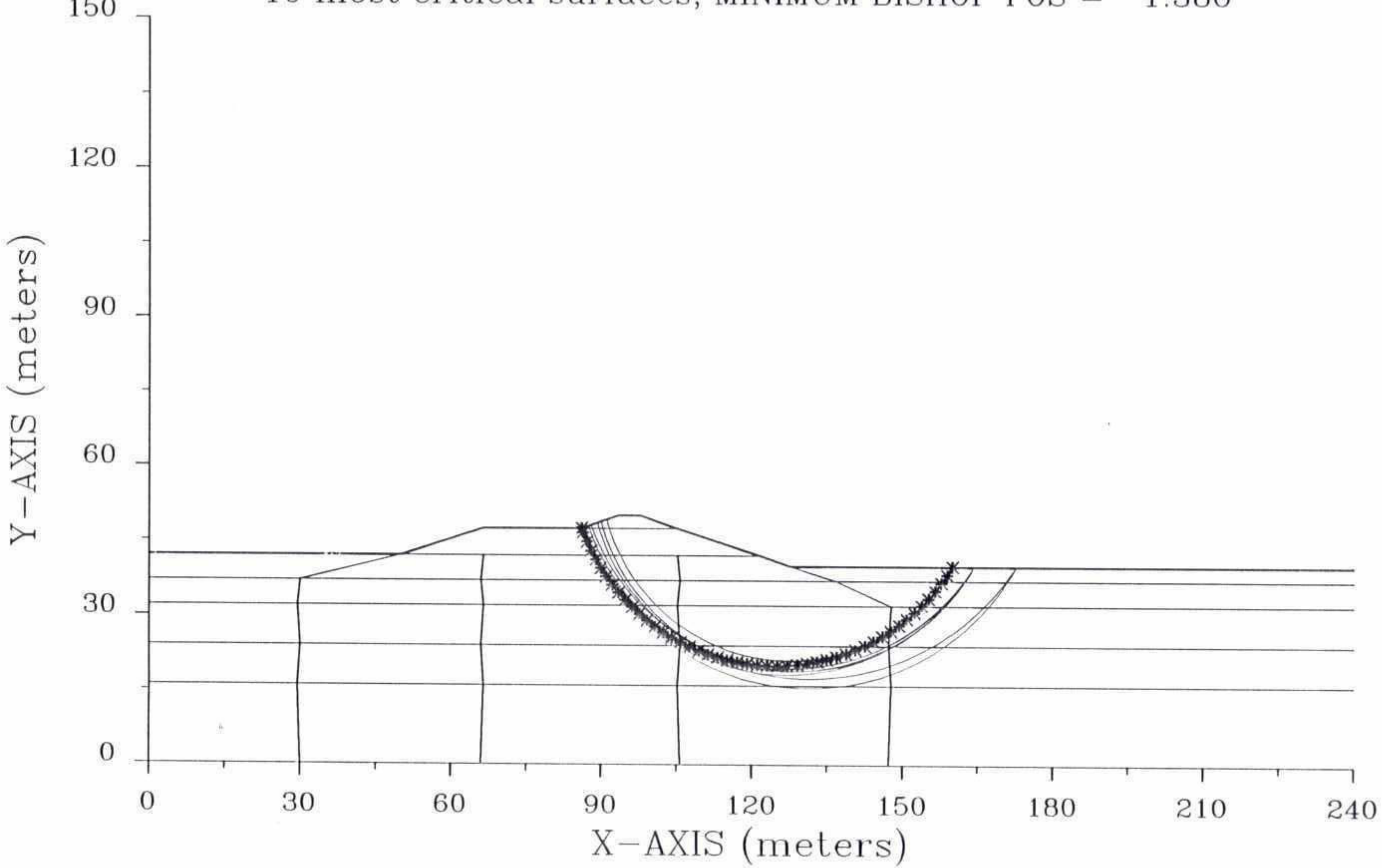
10 most critical surfaces, MINIMUM BISHOP FOS = 1.109



DRSX1U90 16:47

SEGMENT 1 RIVER SIDE U=90

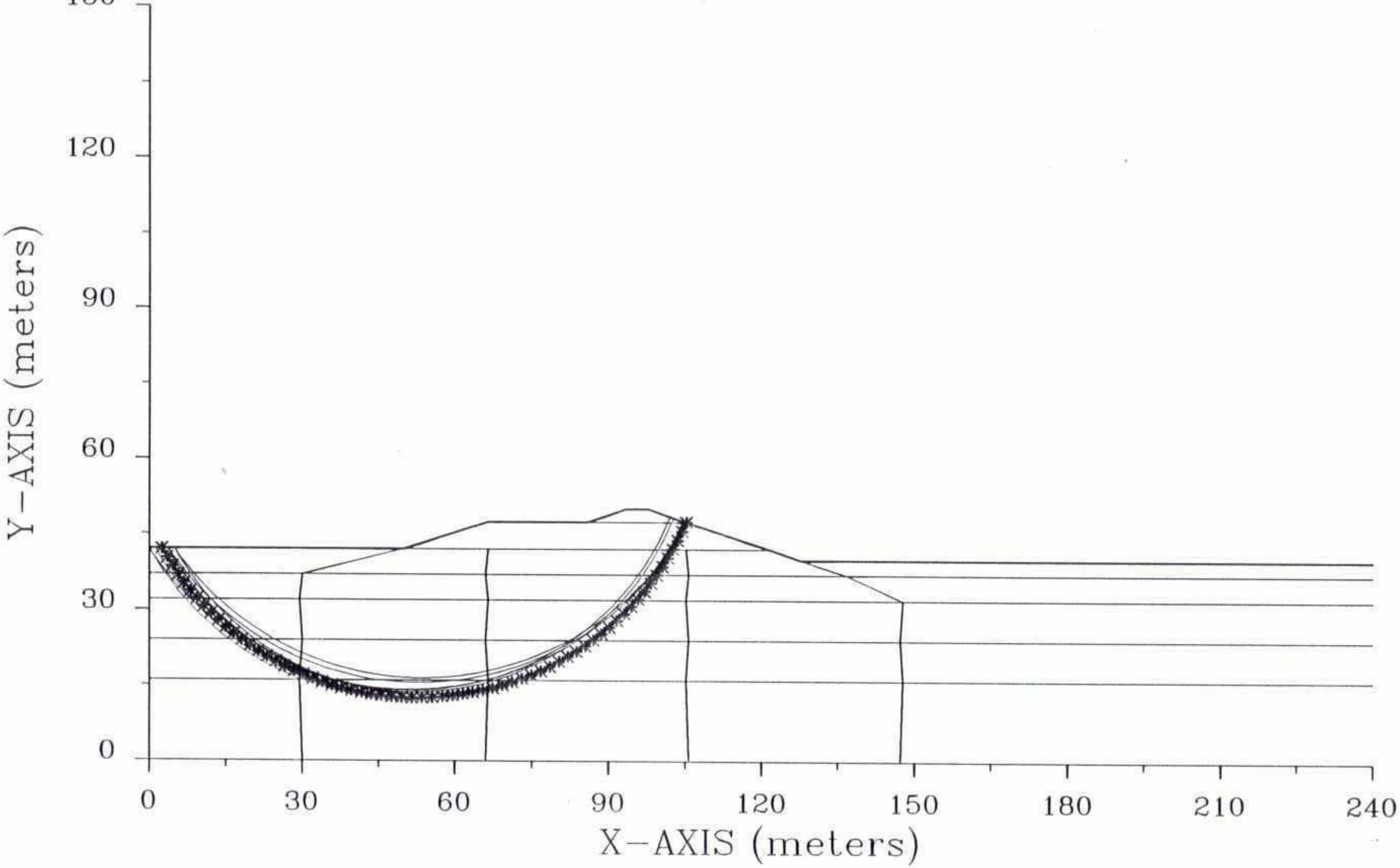
10 most critical surfaces, MINIMUM BISHOP FOS = 1.386



DCSX1U5D 17:43

SEGMENT 1 CITY SIDE, U=50

10 most critical surfaces, MINIMUM BISHOP FOS = 1.362

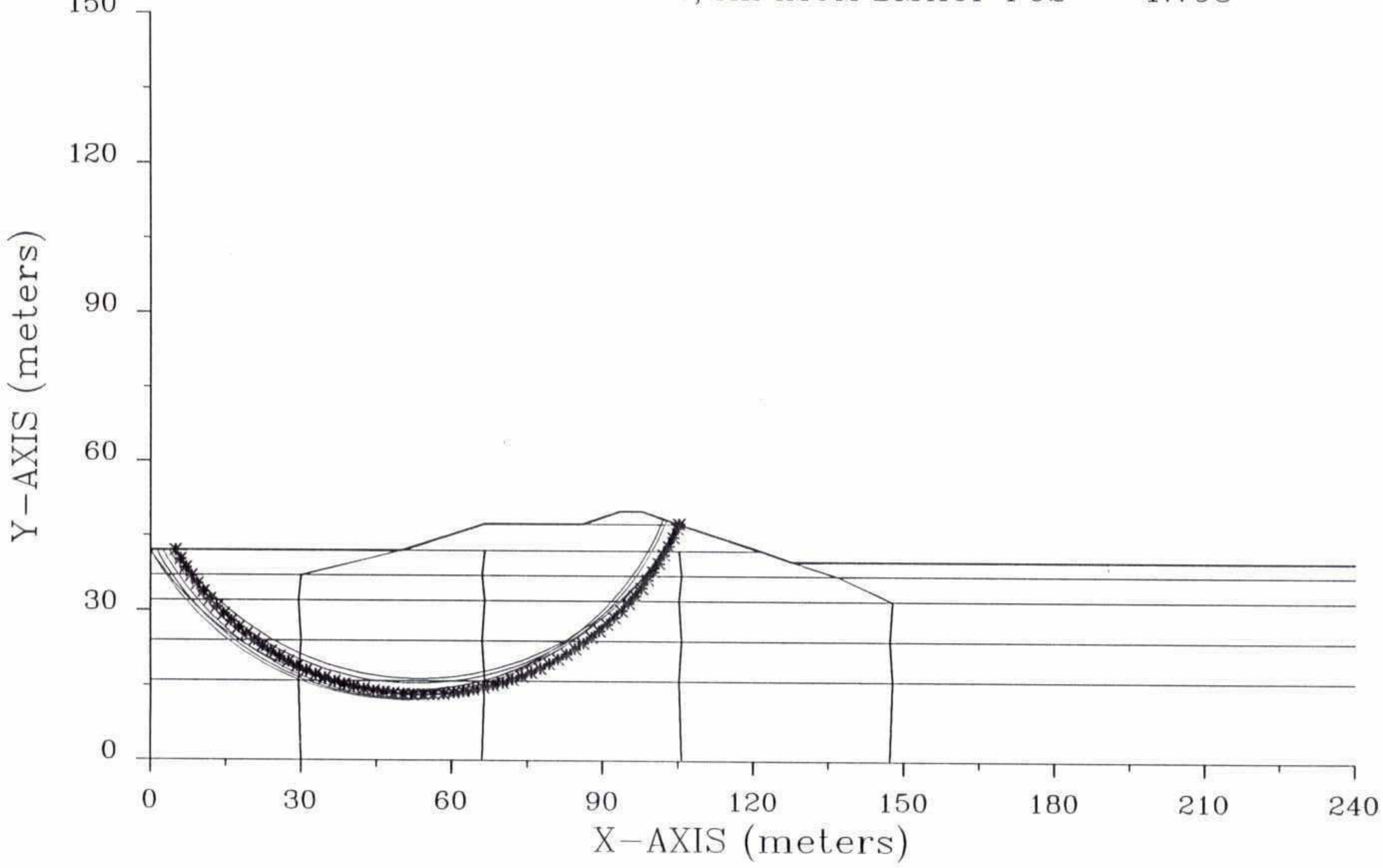


209

DCSX1U9D 17:46

SEGMENT 1 CITY SIDE, U=90

10 most critical surfaces, MINIMUM BISHOP FOS = 1.793

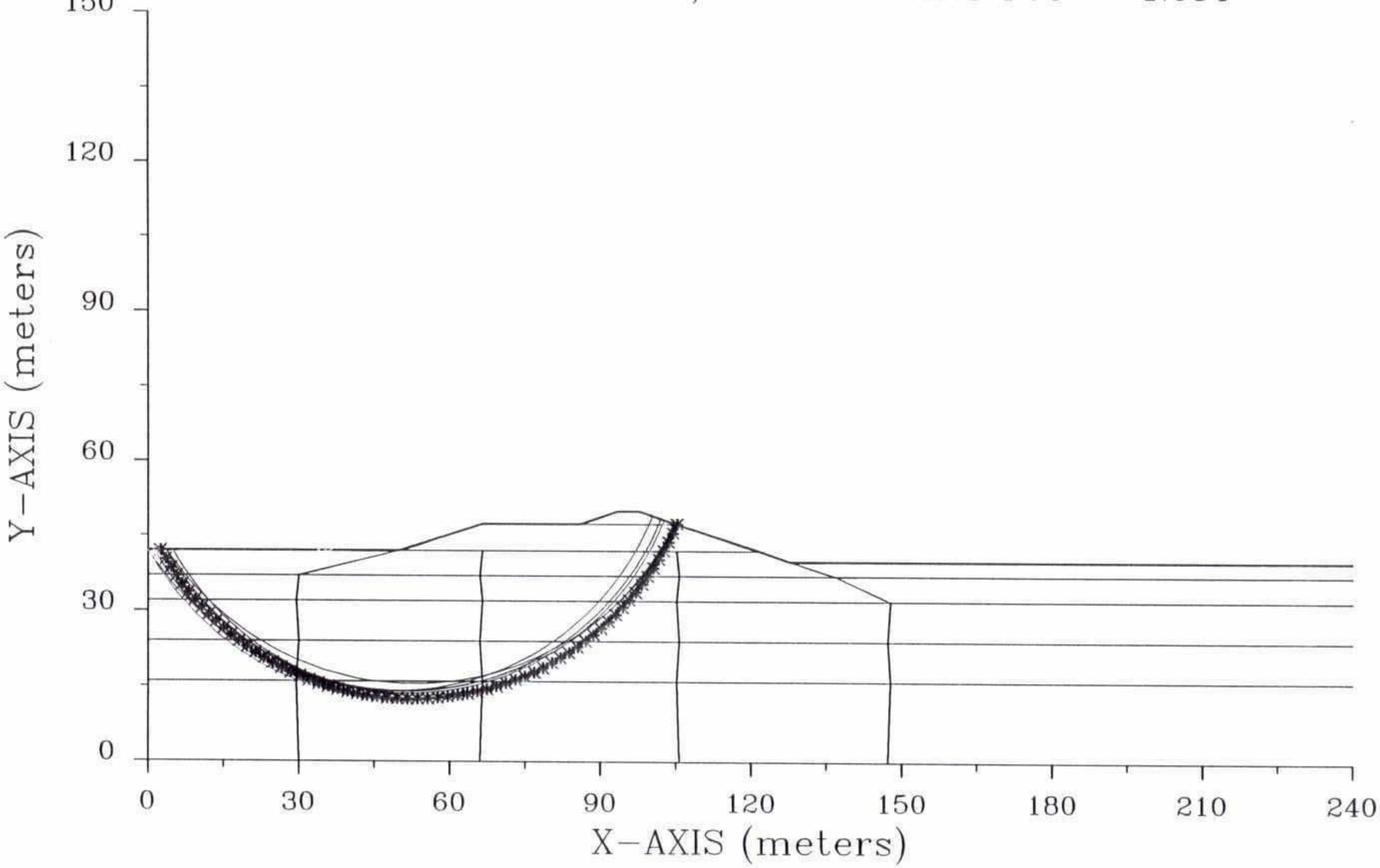


206

DCSX1USD 17:40

SEGMENT 1 CITY SIDE, U=20

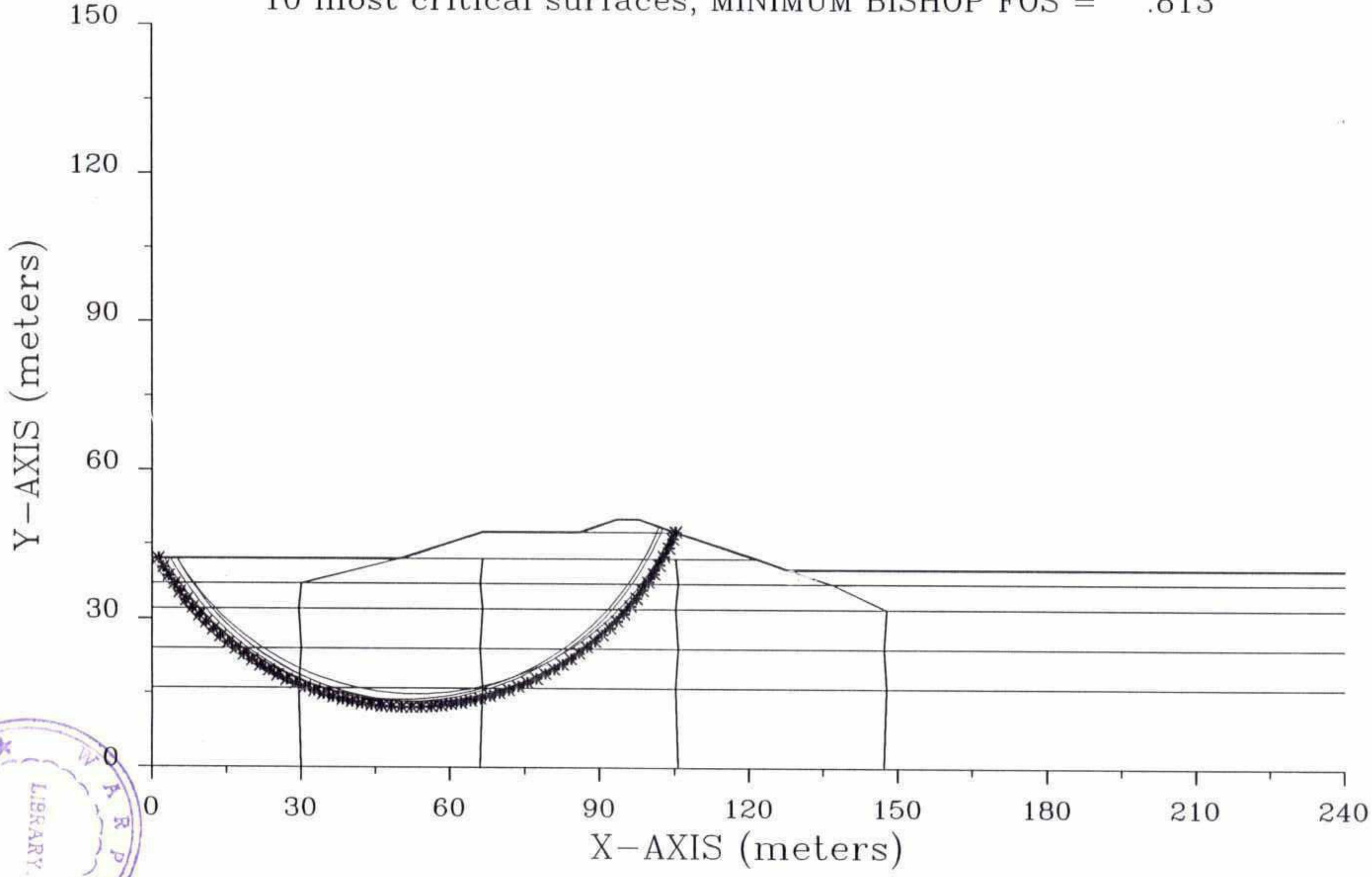
10 most critical surfaces, MINIMUM BISHOP FOS = 1.034



DCSX1U0D 17:22

SEGMENT 1 CITY SIDE, U=0

10 most critical surfaces, MINIMUM BISHOP FOS = .813

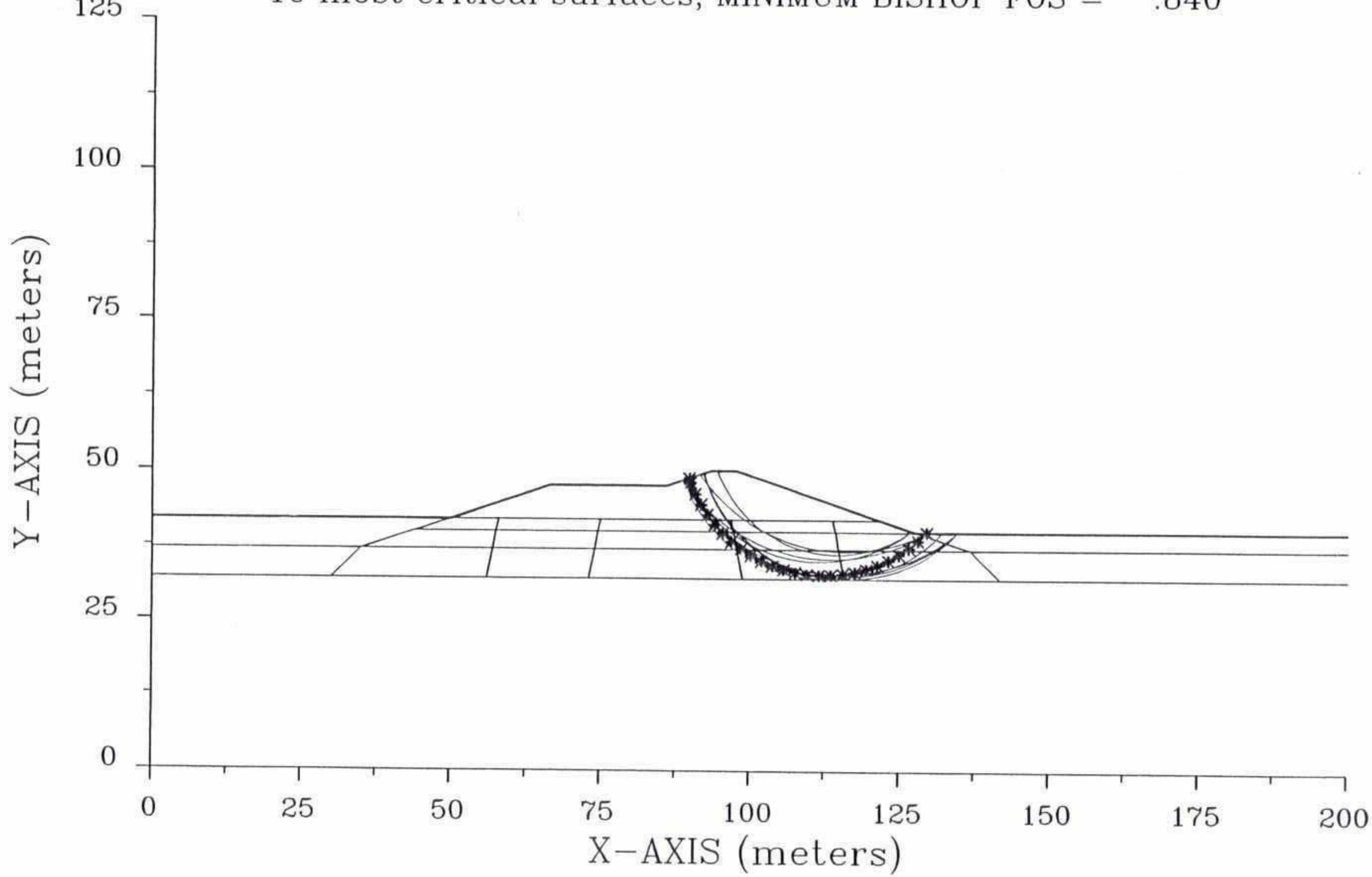


280

DRSX2U0 11:42

SEGMENT 2 RIVER SIDE, U=0

10 most critical surfaces, MINIMUM BISHOP FOS = .840

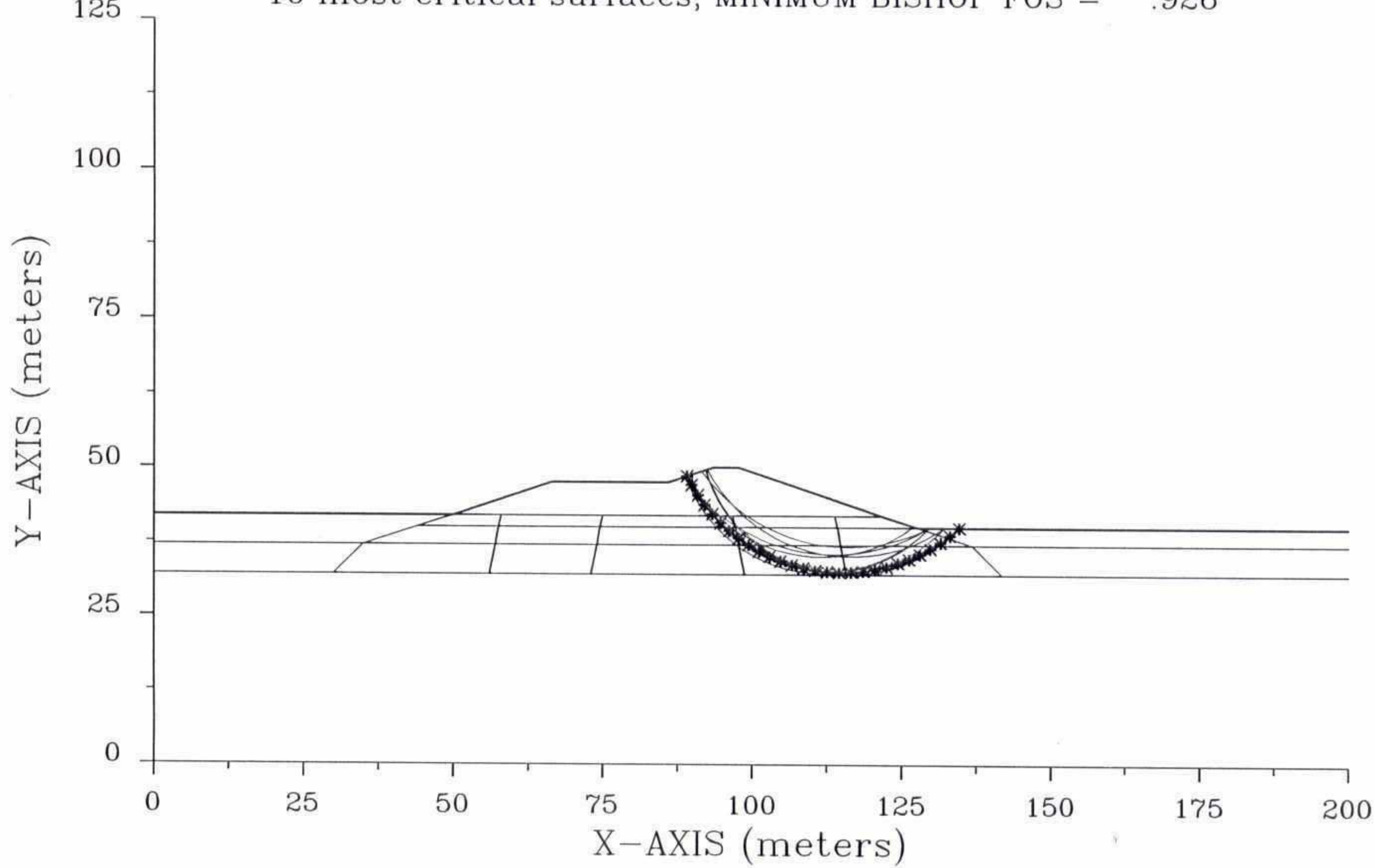


282

DRSX2U20 11:49

SEGMENT 2 RIVER SIDE, U=20

10 most critical surfaces, MINIMUM BISHOP FOS = .926

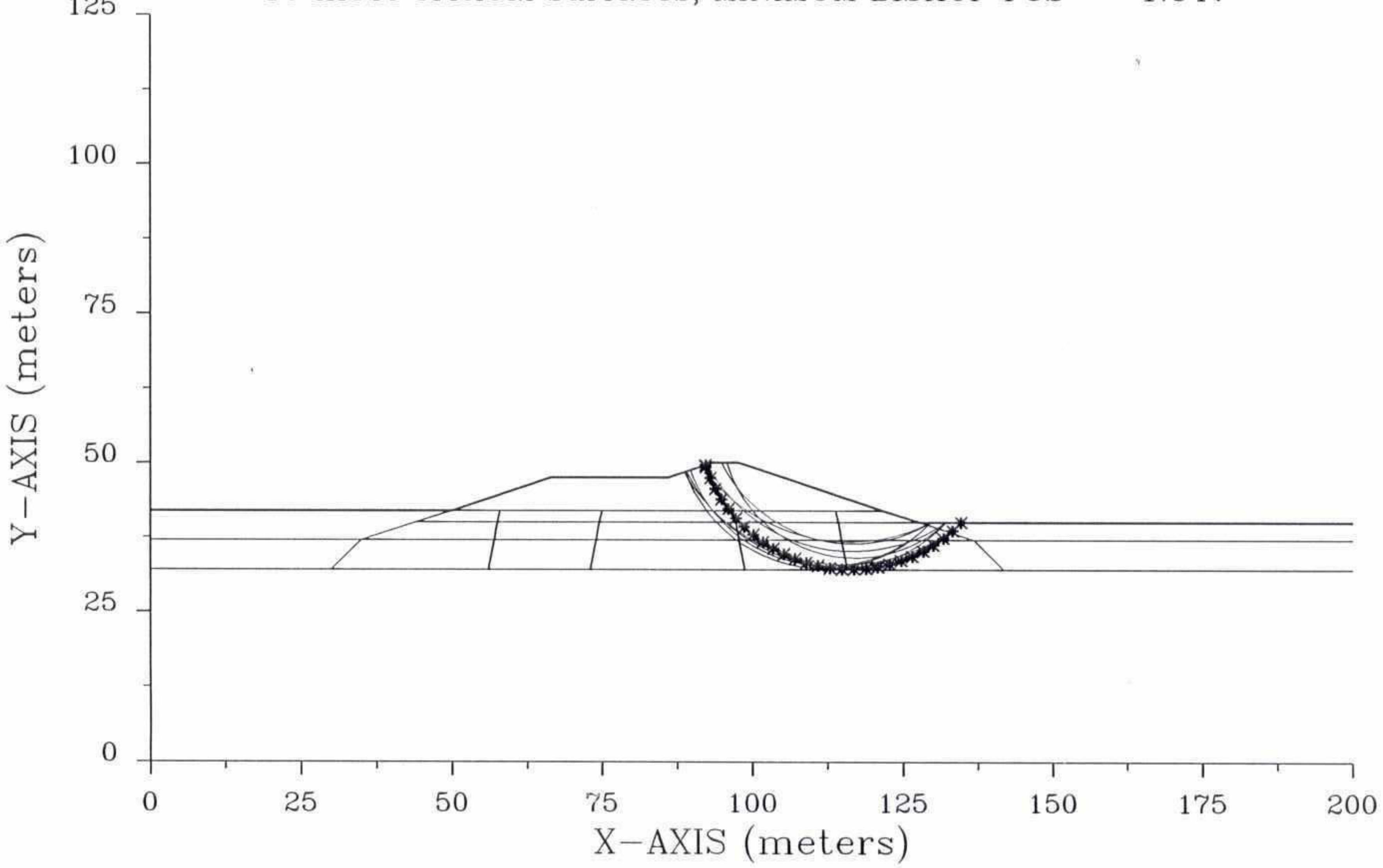


282

DRSX2U50 11:52

SEGMENT 2 RIVER SIDE, U=50

10 most critical surfaces, MINIMUM BISHOP FOS = 1.047



286

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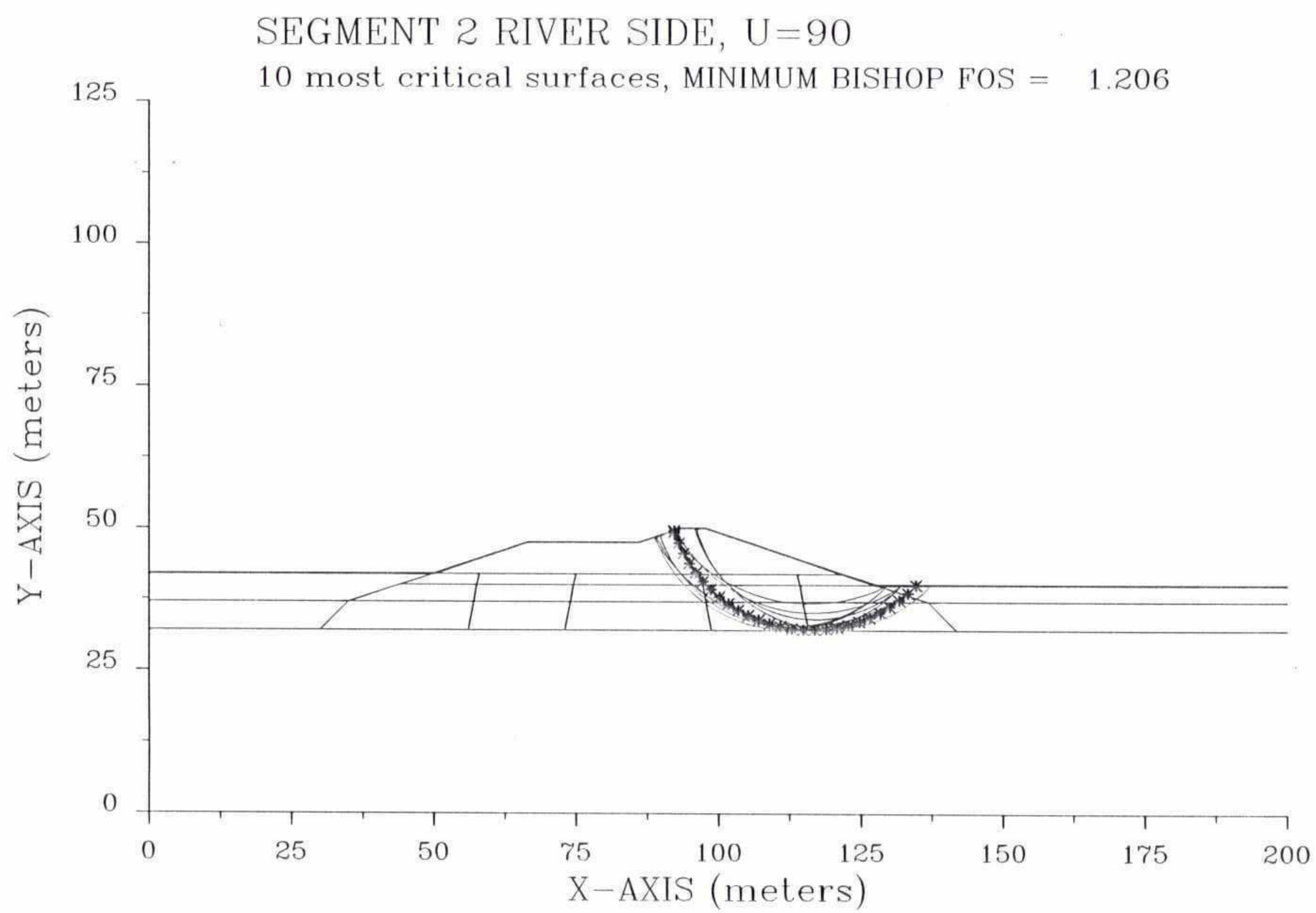
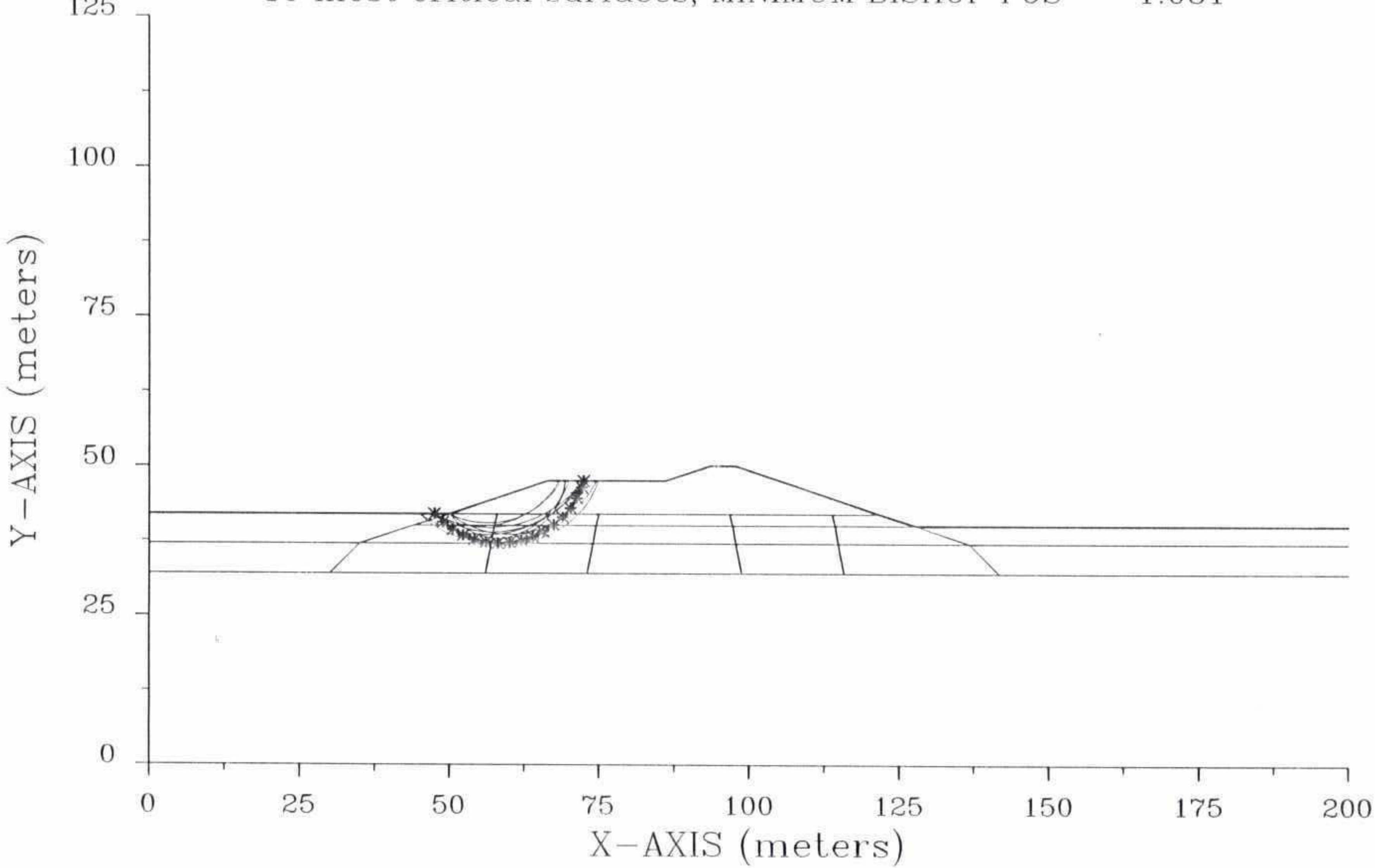


288

DCSX2U0 9:19

SEGMENT 2 CITY SIDE, U=0

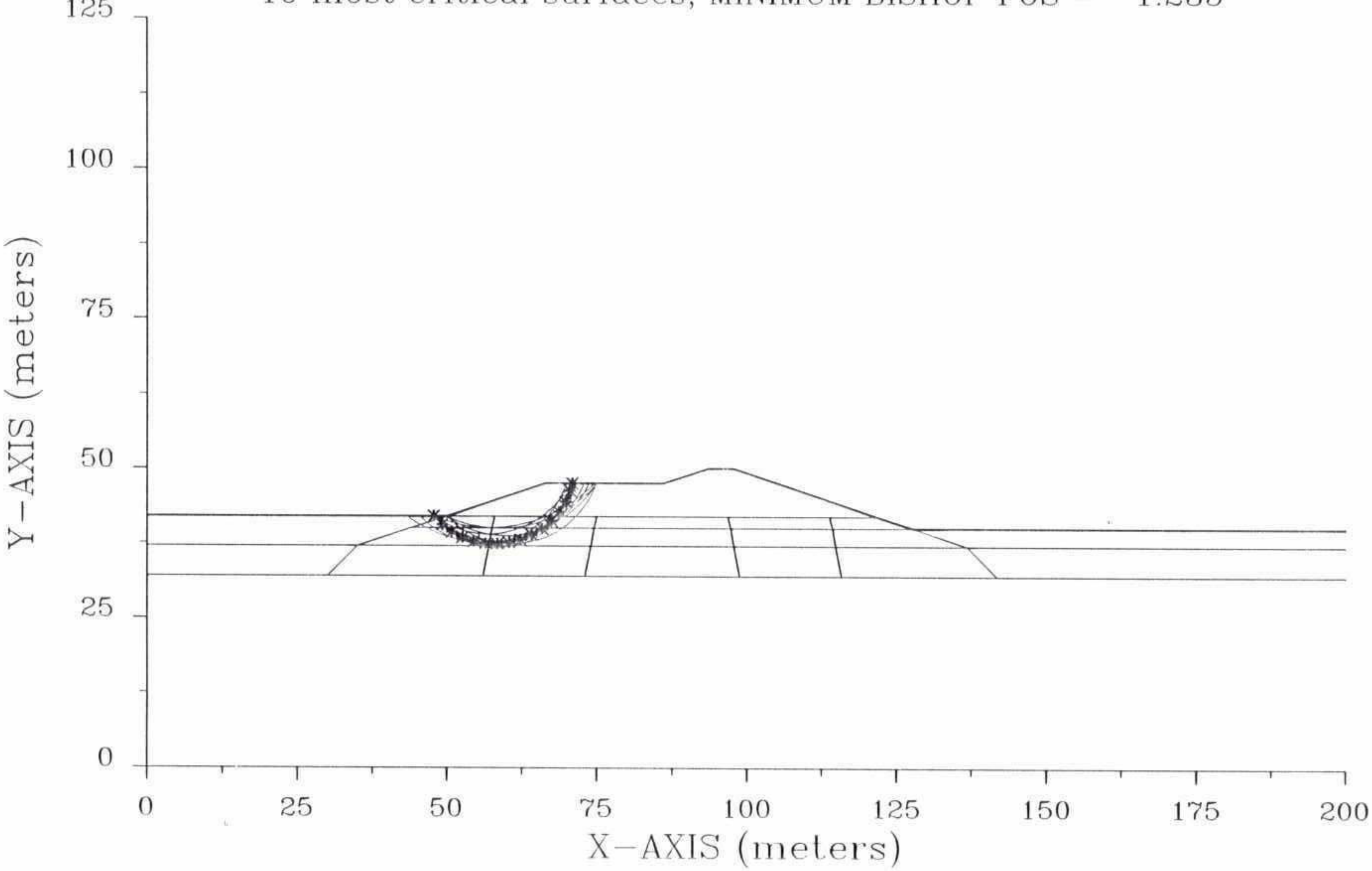
10 most critical surfaces, MINIMUM BISHOP FOS = 1.081



DCSX2U20 9:30

SEGMENT 2 CITY SIDE, U=20

10 most critical surfaces, MINIMUM BISHOP FOS = 1.235

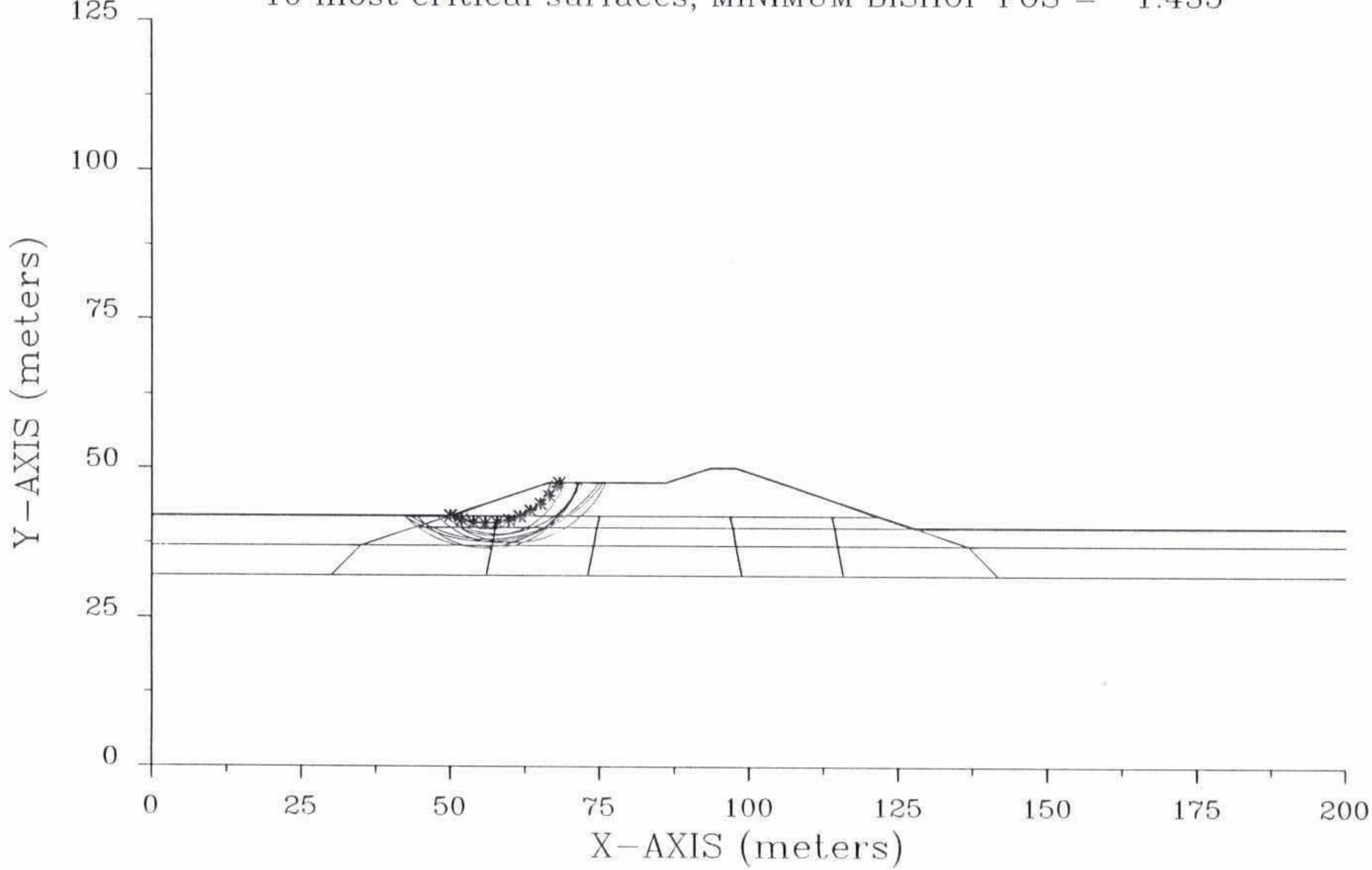


283

DCSX2U50 9:33

SEGMENT 2 CITY SIDE, U=50

10 most critical surfaces, MINIMUM BISHOP FOS = 1.435

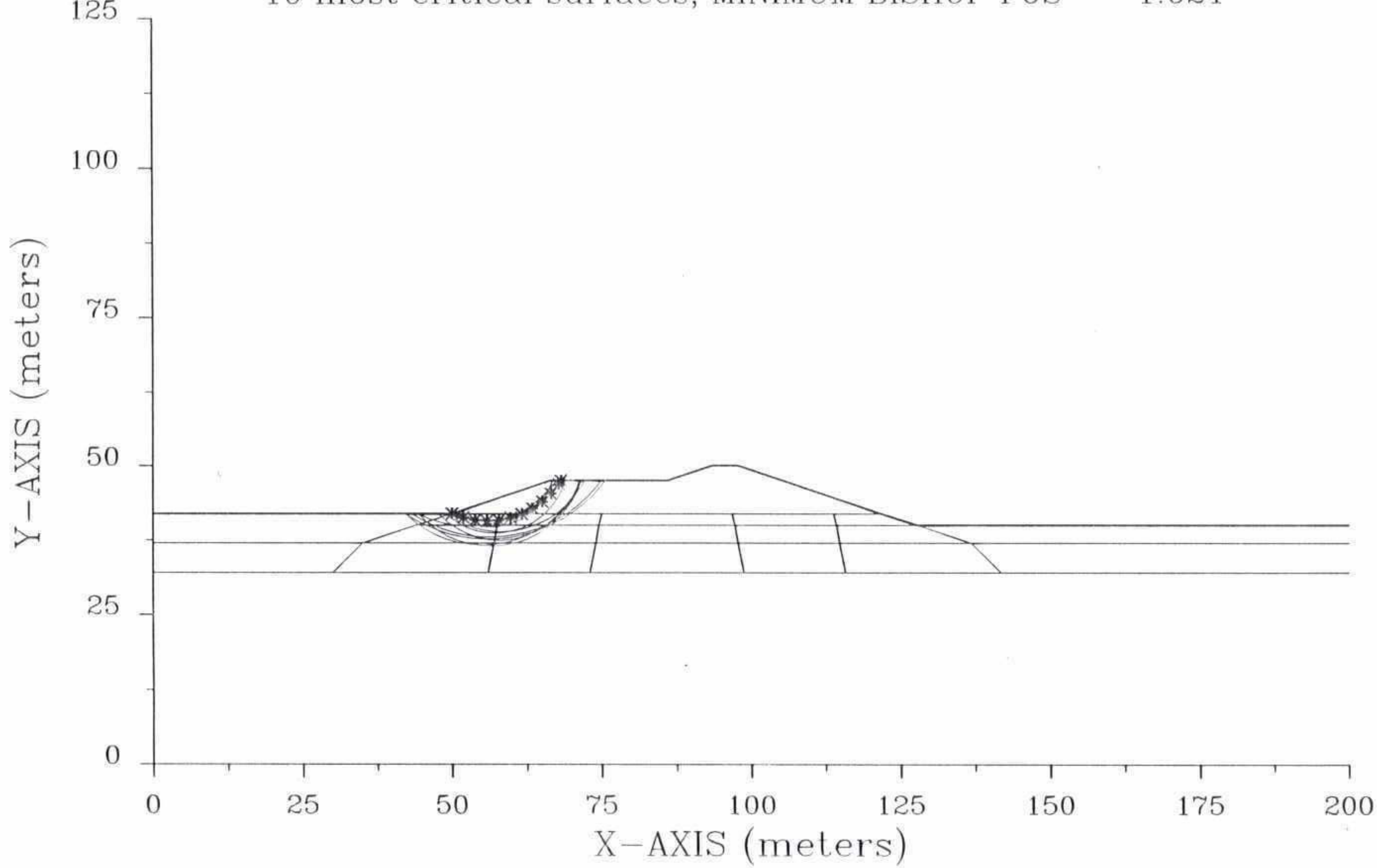


289

DCSX2U90 9:37

SEGMENT 2 CITY SIDE, U=90

10 most critical surfaces, MINIMUM BISHOP FOS = 1.621

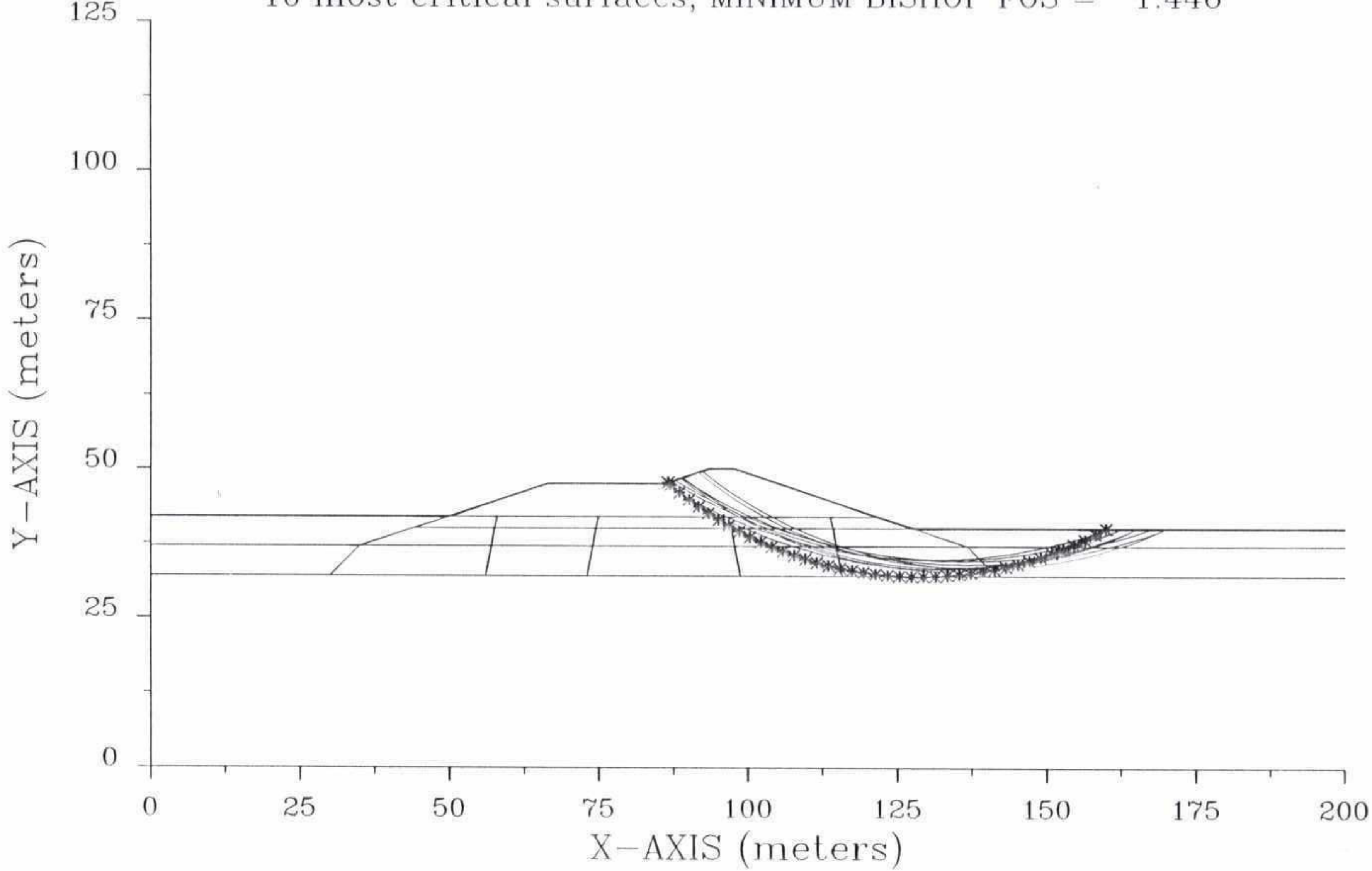


28

DRSX2U0D 14:38

SEGMENT 2 RIVER SIDE, U=0

10 most critical surfaces, MINIMUM BISHOP FOS = 1.446



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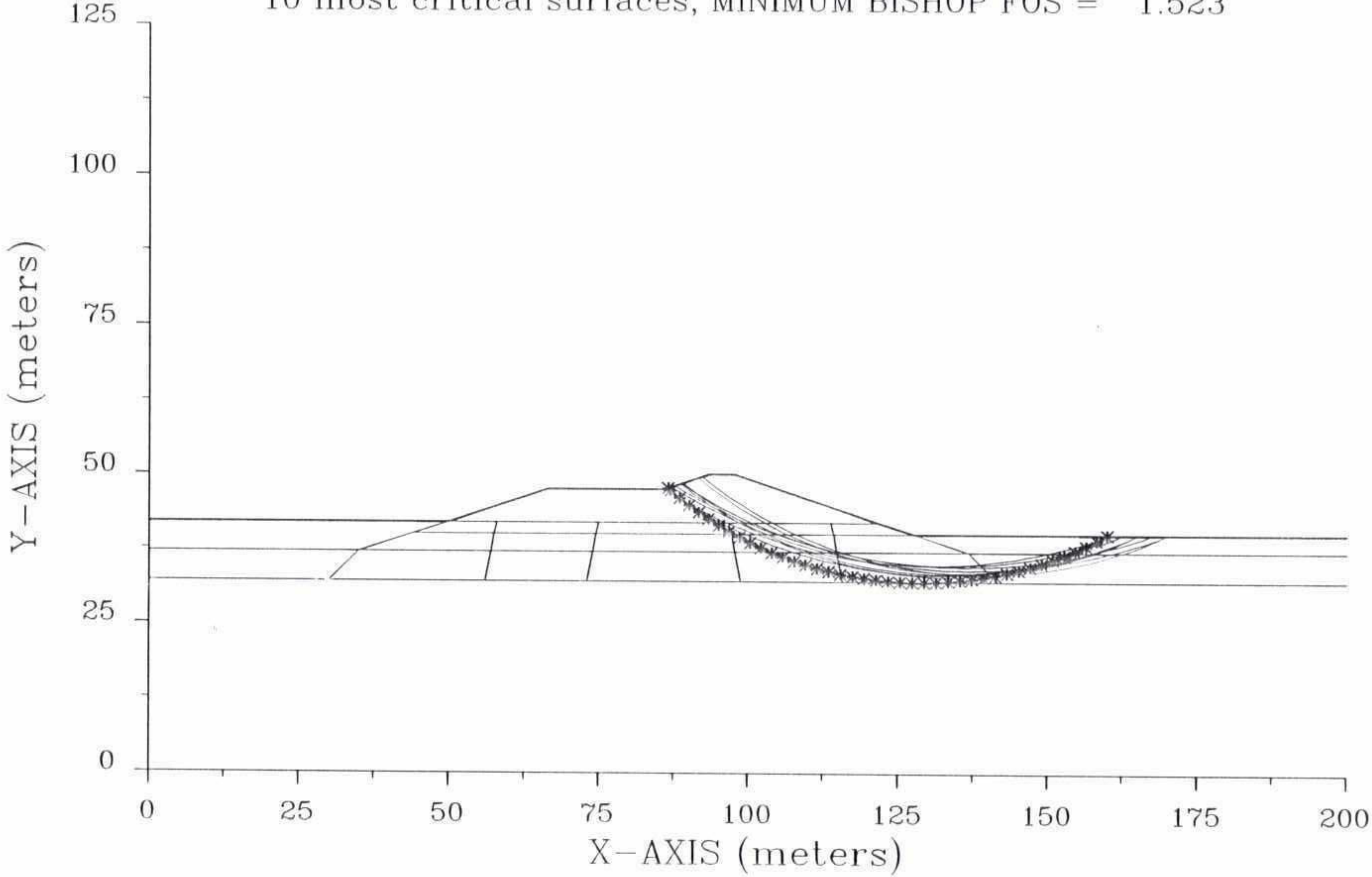
DRSX2U2D 14:41

SEGMENT 2 RIVER SIDE, U=20

10 most critical surfaces, MINIMUM BISHOP FOS = 1.523



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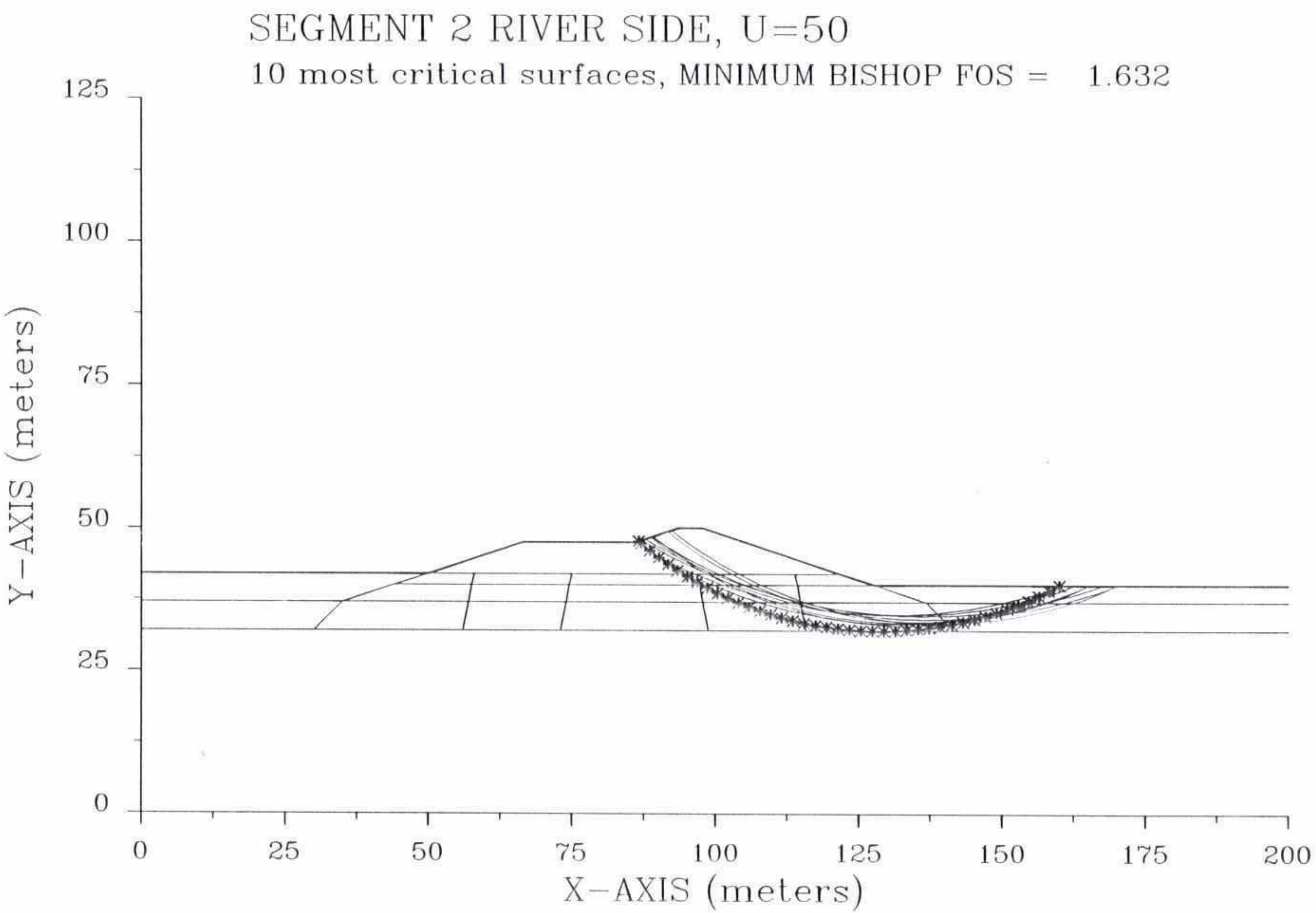

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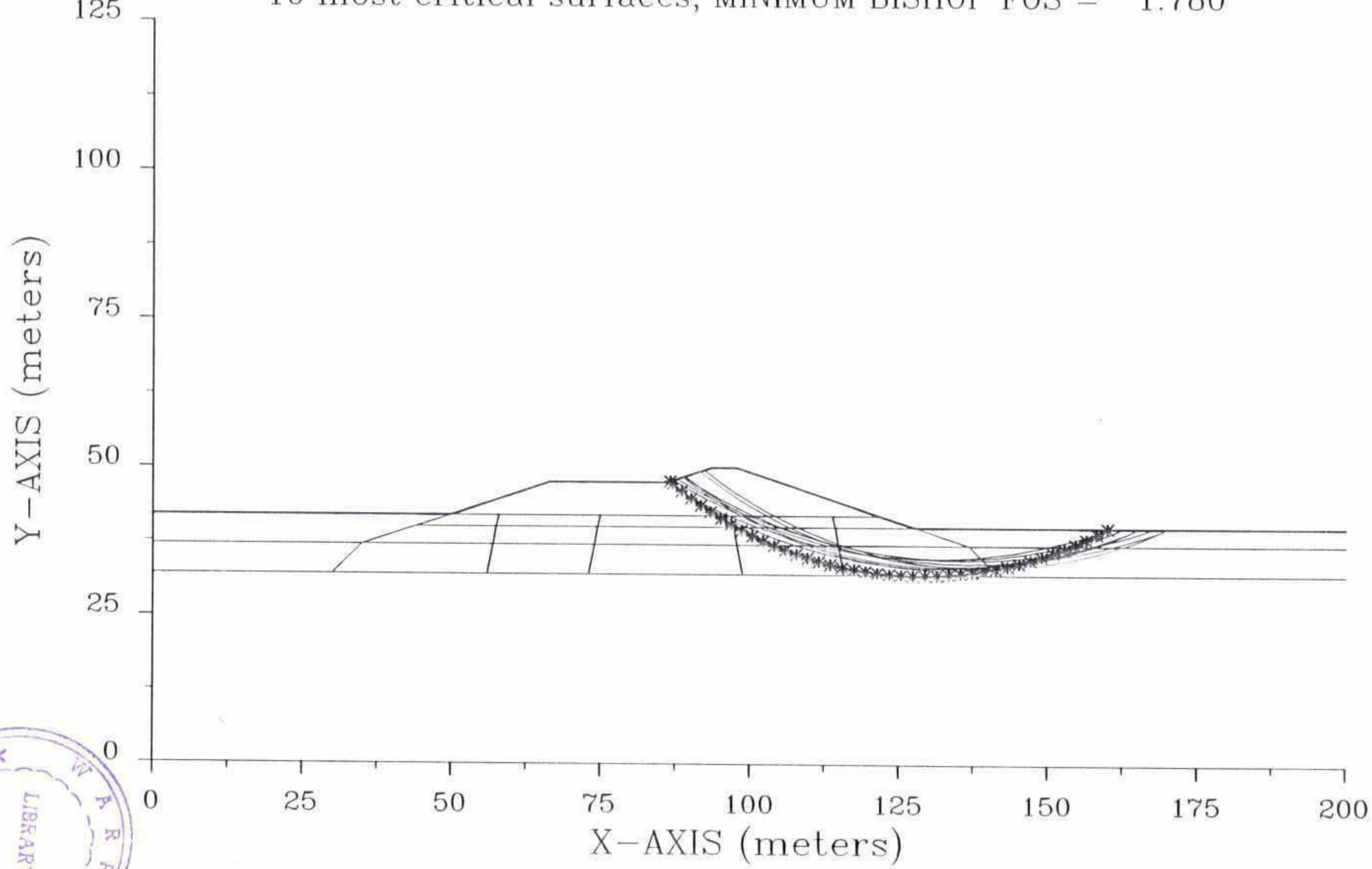
SEGMENT 2 RIVER SIDE, U=90

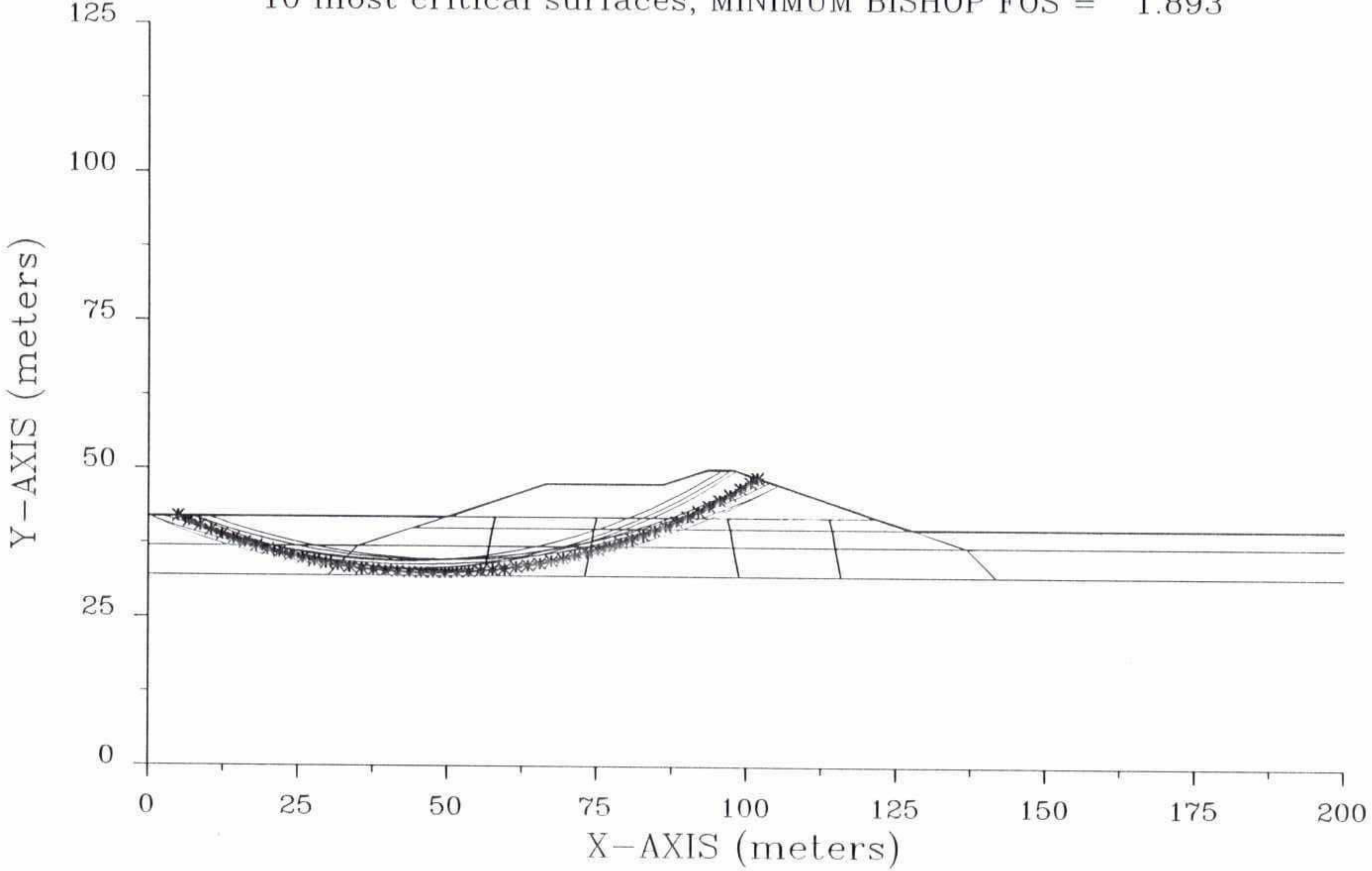
10 most critical surfaces, MINIMUM BISHOP FOS = 1.780



DCSX2U0D 18:03

SEGMENT 2 CITY SIDE, U=0

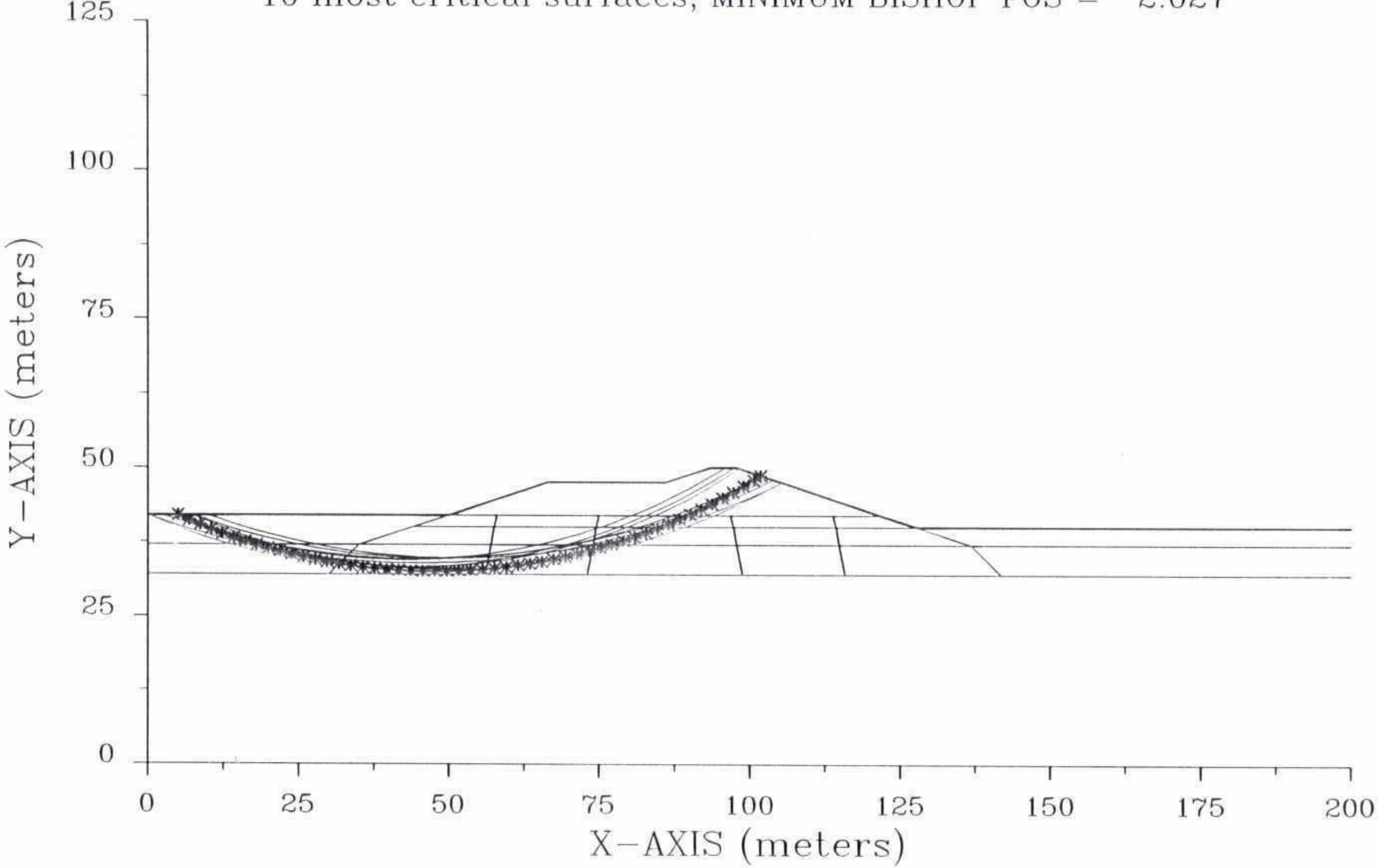
10 most critical surfaces, MINIMUM BISHOP FOS = 1.893



DCSX2U2D 18:06

SEGMENT 2 CITY SIDE, U=20

10 most critical surfaces, MINIMUM BISHOP FOS = 2.027

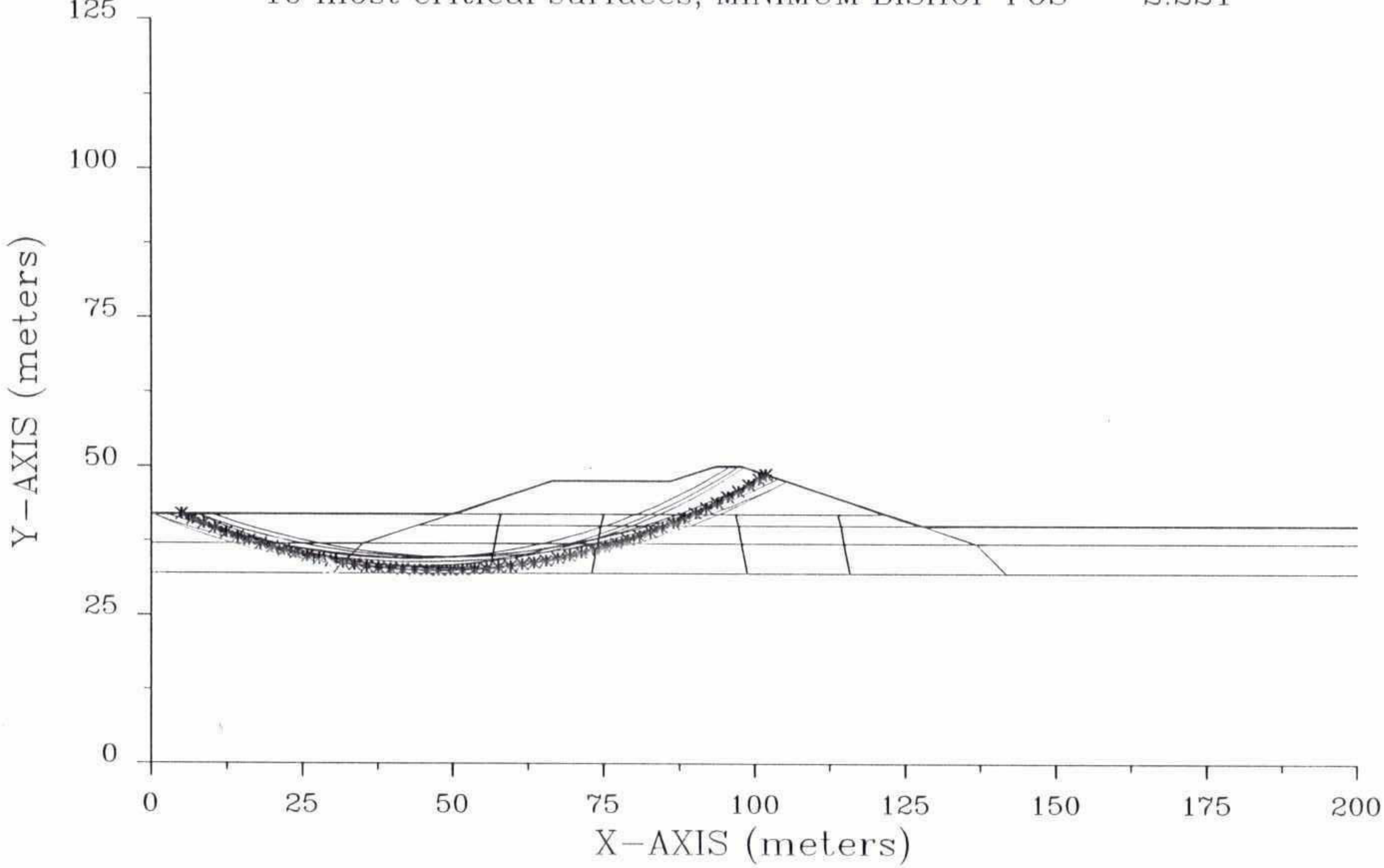


208

DCSX2U5D 18:10

SEGMENT 2 CITY SIDE, U=50

10 most critical surfaces, MINIMUM BISHOP FOS = 2.221



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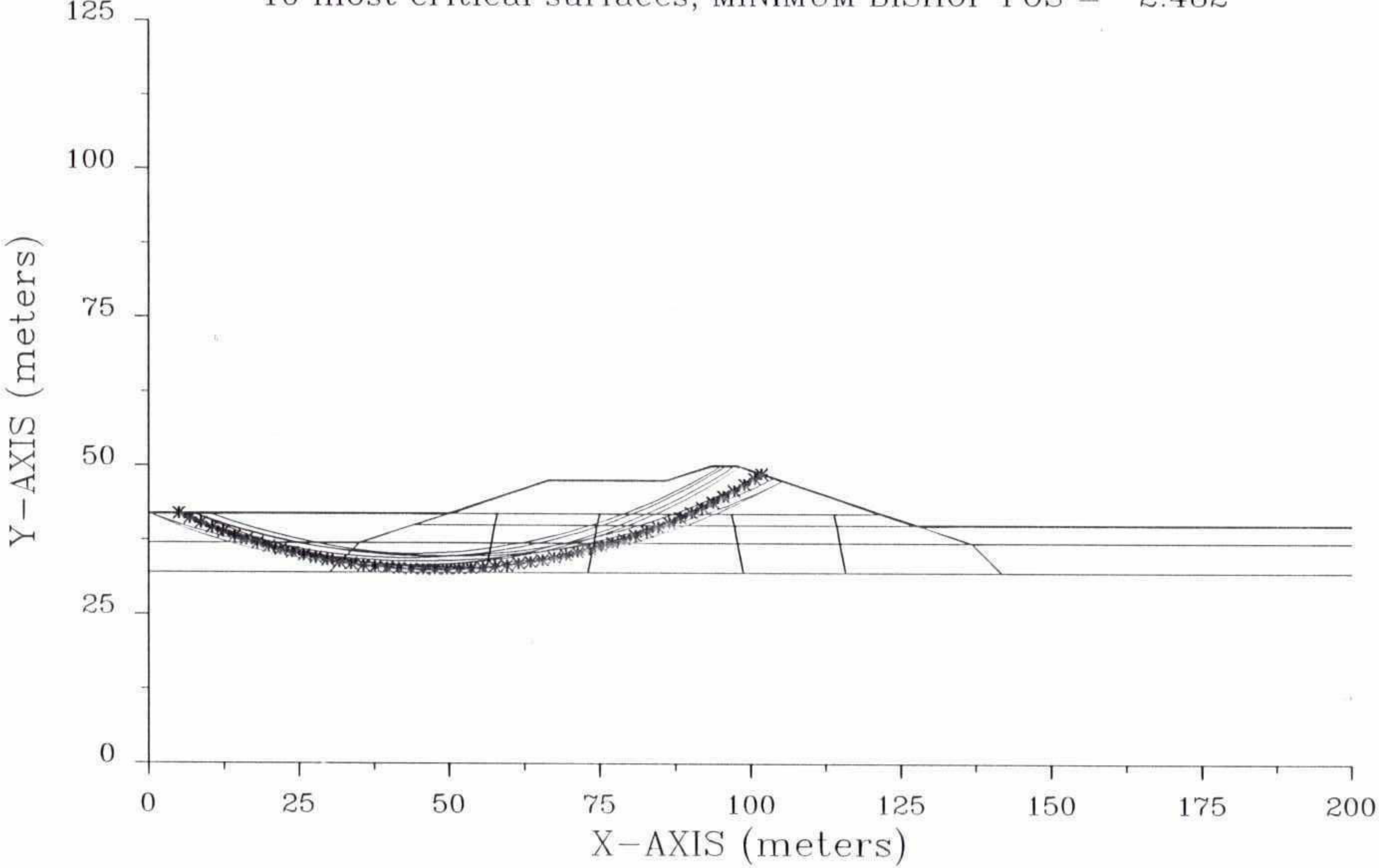
DCSX2U9D 18:12

SEGMENT 2 CITY SIDE, U=90

10 most critical surfaces, MINIMUM BISHOP FOS = 2.482

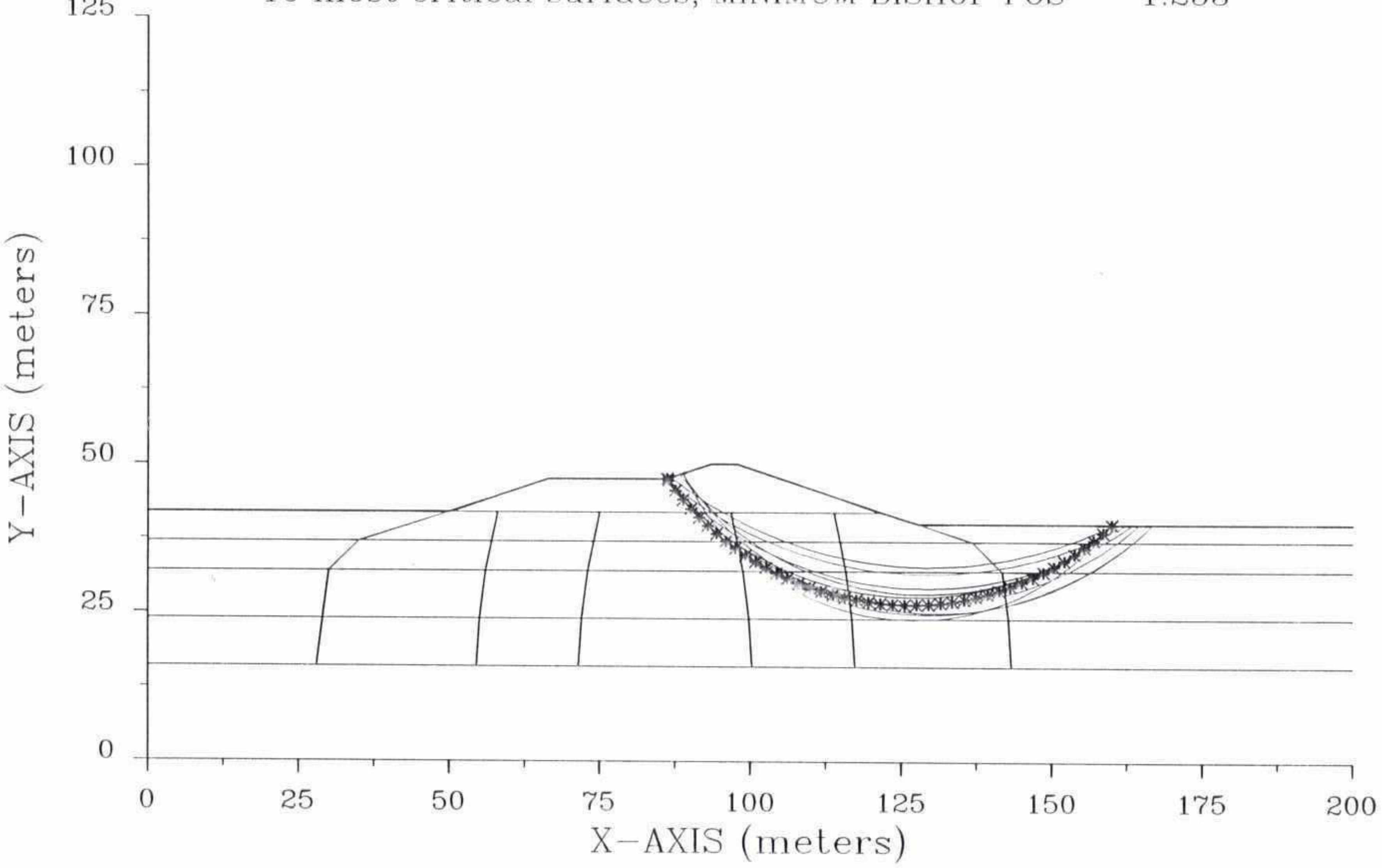


6DC

DRSX3U0D 14:24

SEGMENT 3 RIVER SIDE, U=0

10 most critical surfaces, MINIMUM BISHOP FOS = 1.253



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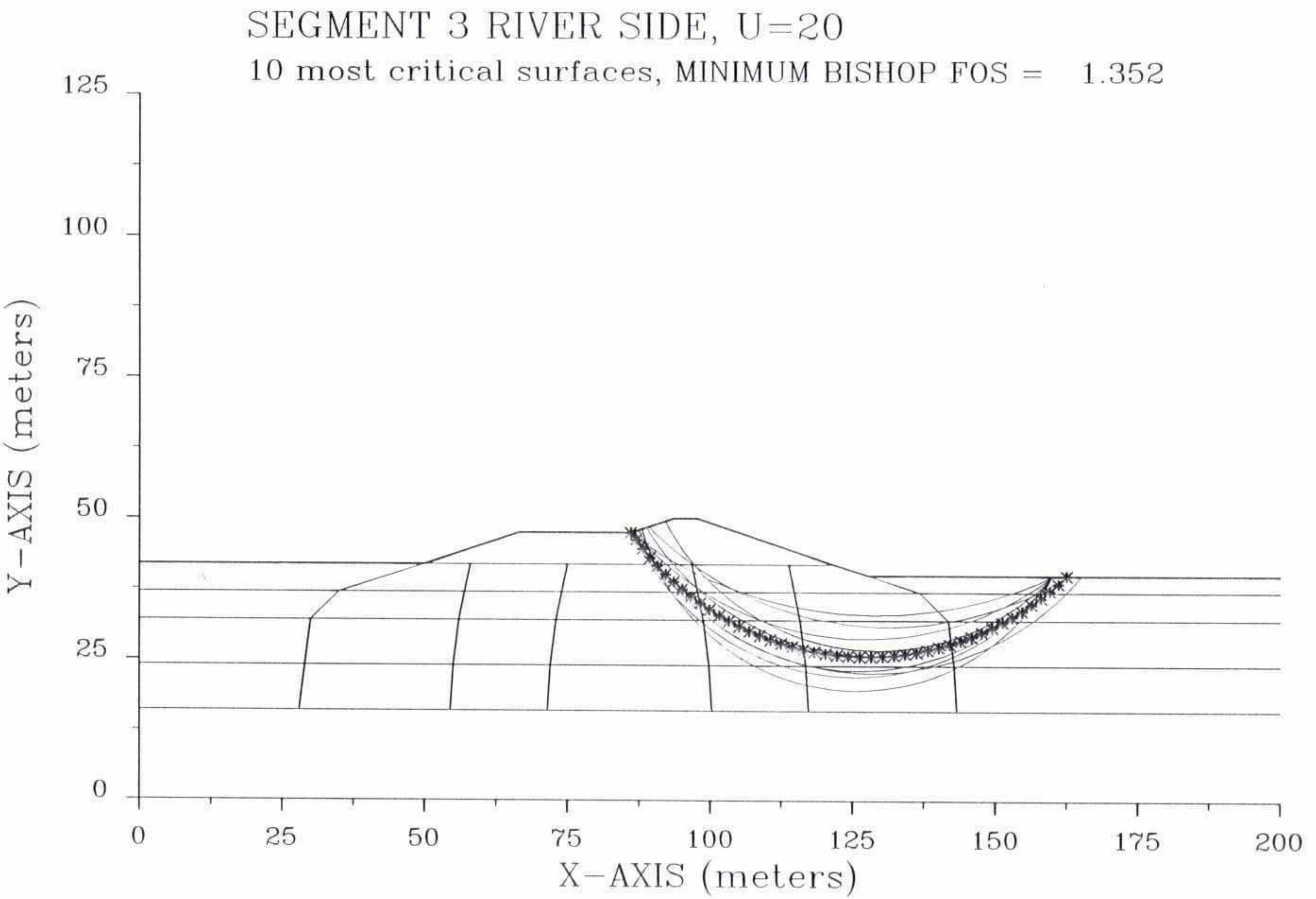
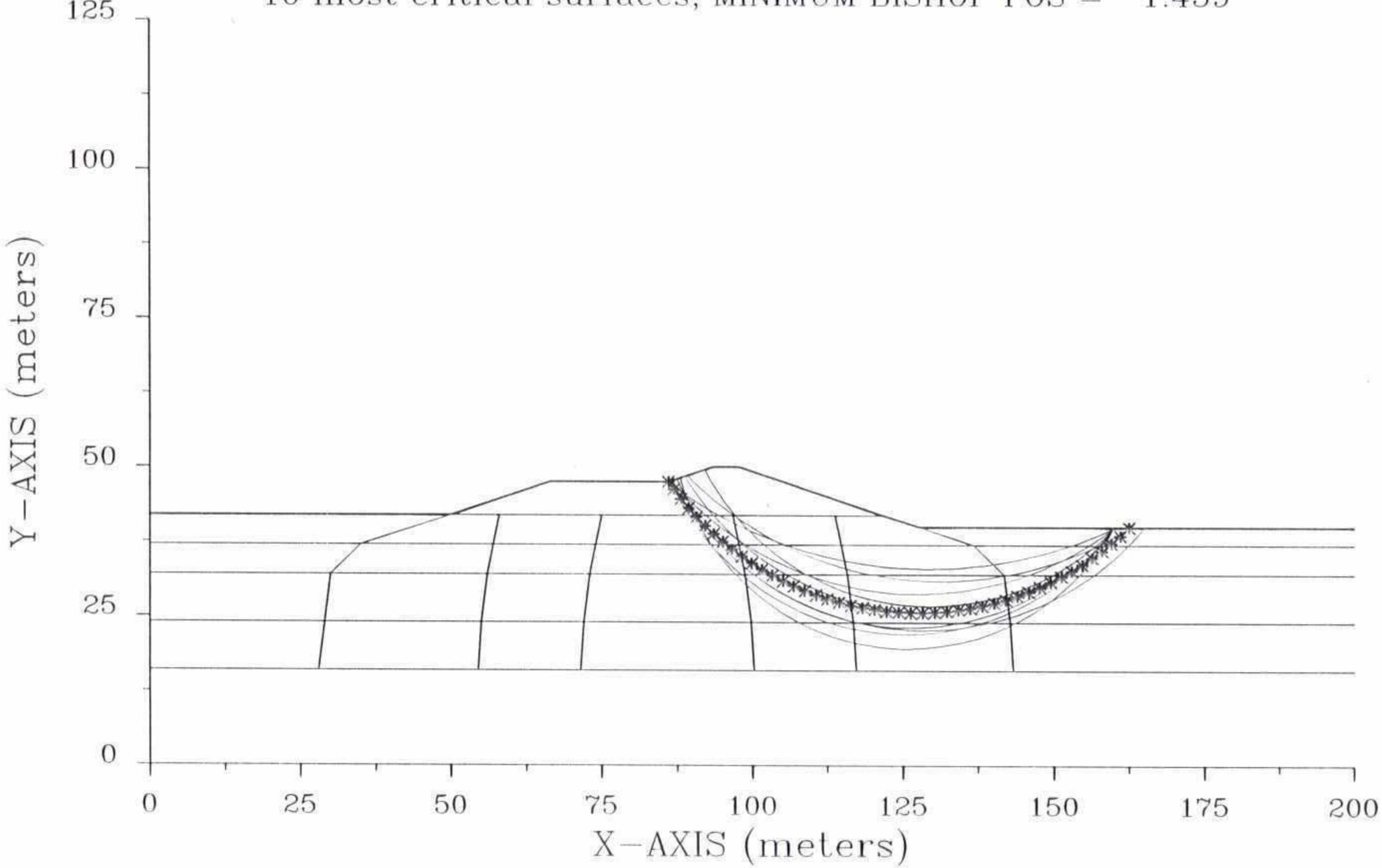


DRSX3U5D 14:32

SEGMENT 3 RIVER SIDE, U=50

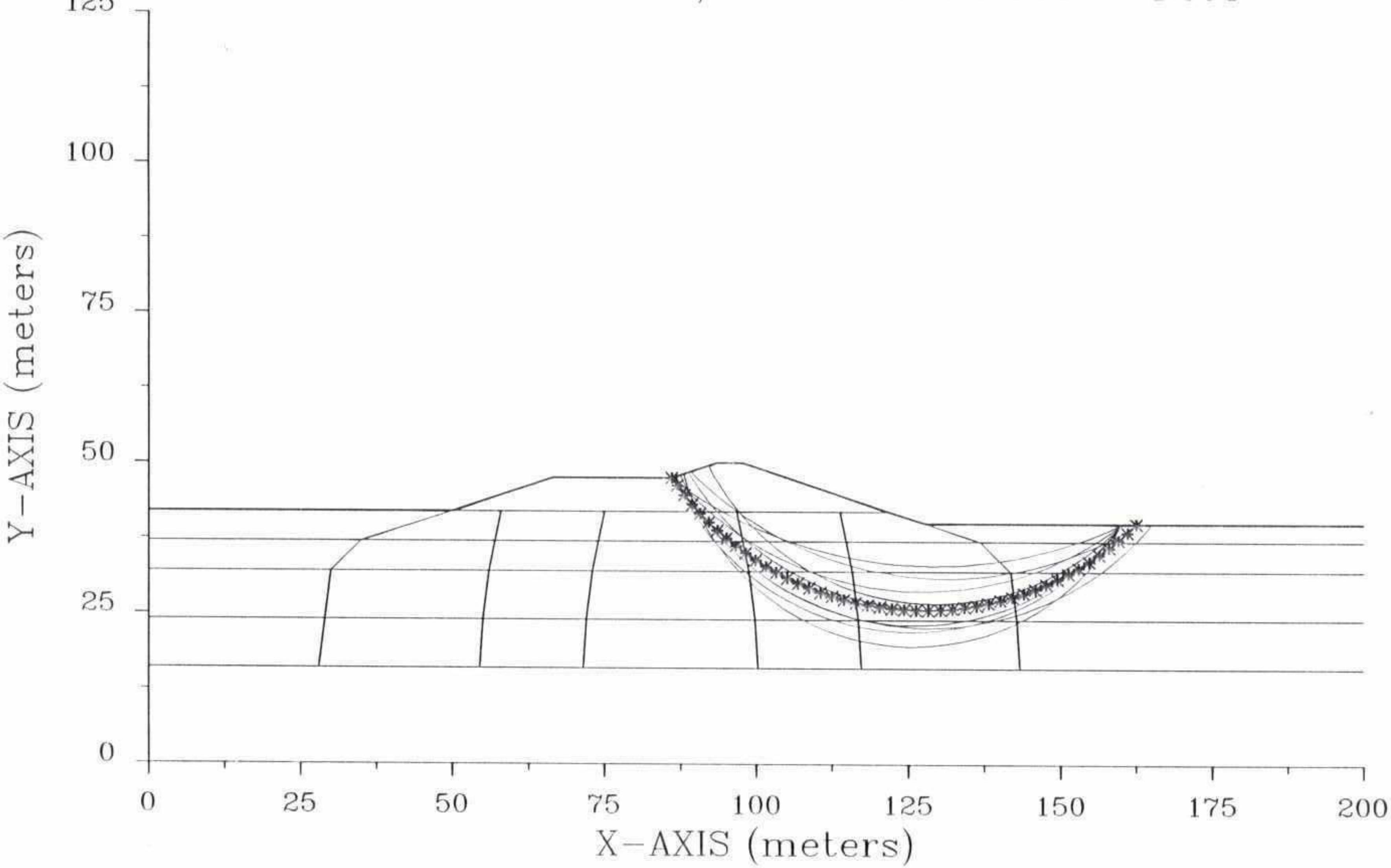
10 most critical surfaces, MINIMUM BISHOP FOS = 1.459



DRSX3U9D 14:34

SEGMENT 3 RIVER SIDE, U=90

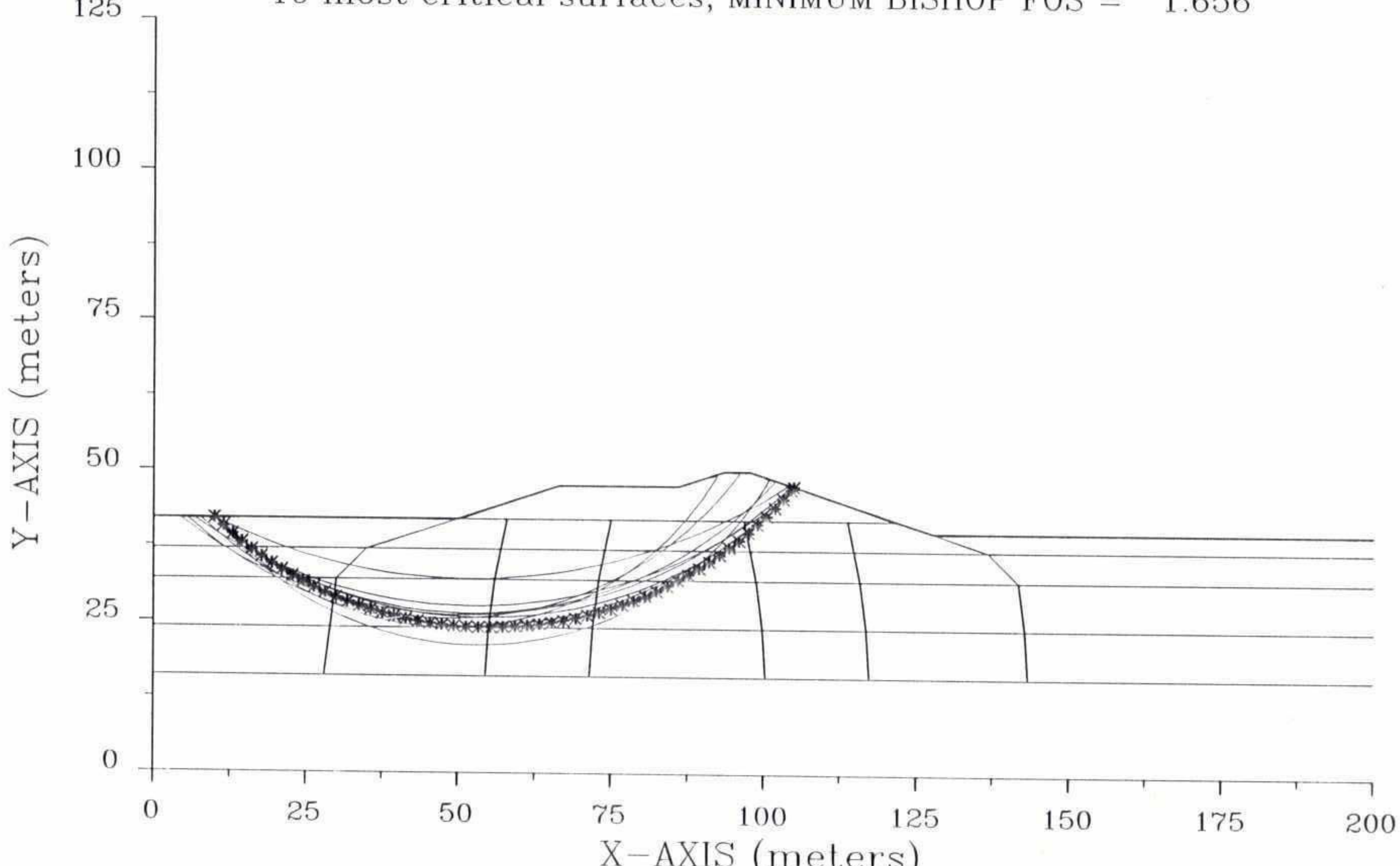
10 most critical surfaces, MINIMUM BISHOP FOS = 1.604



DCSX3U0D 18:15

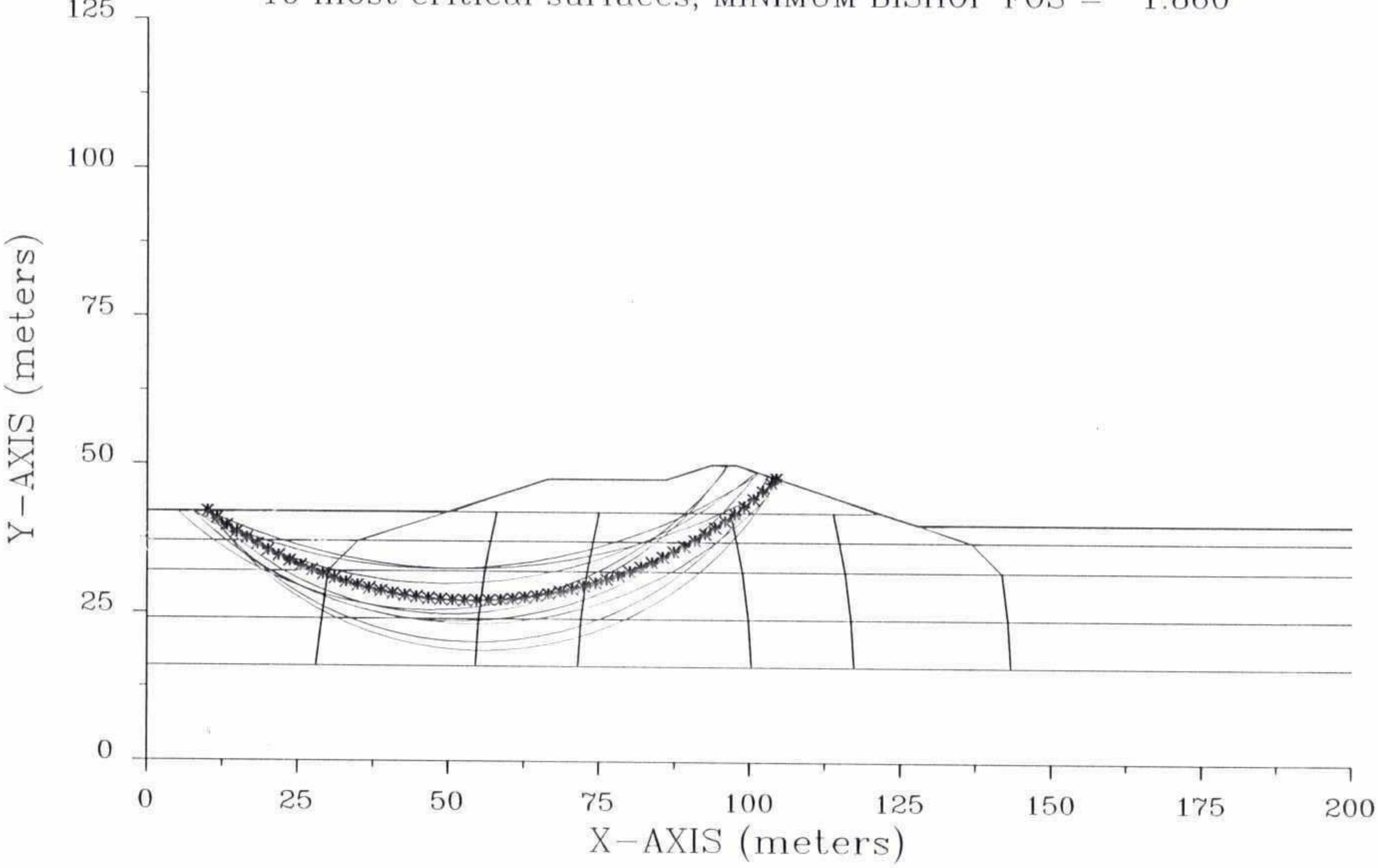
SEGMENT 3 CITY SIDE,U=0

10 most critical surfaces, MINIMUM BISHOP FOS = 1.656



SEGMENT 3 CITY SIDE, U=20

10 most critical surfaces, MINIMUM BISHOP FOS = 1.860

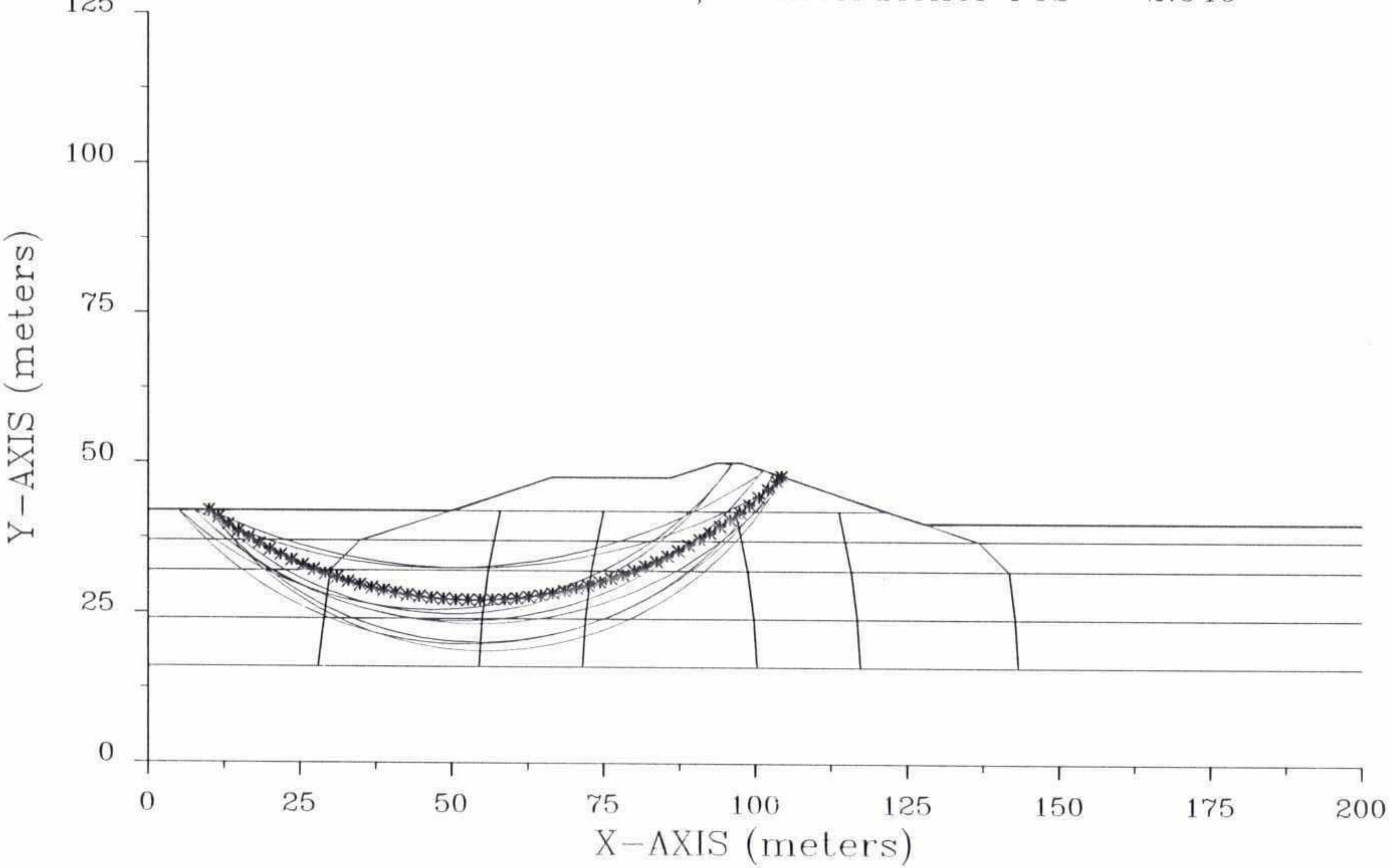


232

DCSX3U5D 18:20

SEGMENT 3 CITY SIDE, U=50

10 most critical surfaces, MINIMUM BISHOP FOS = 2.040

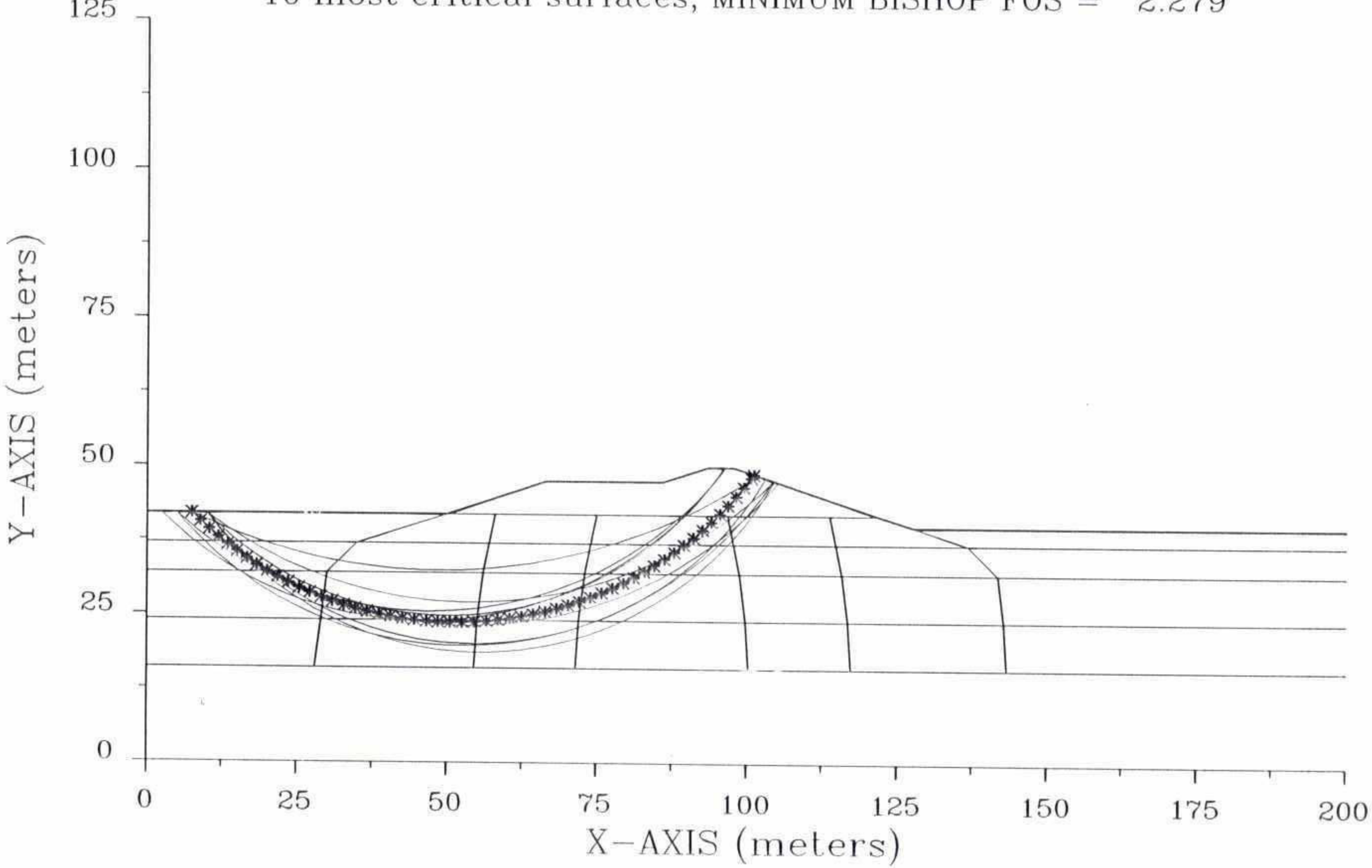


236

DCSX3U9D 18:22

SEGMENT 3 CITY SIDE, U=90

10 most critical surfaces, MINIMUM BISHOP FOS = 2.279

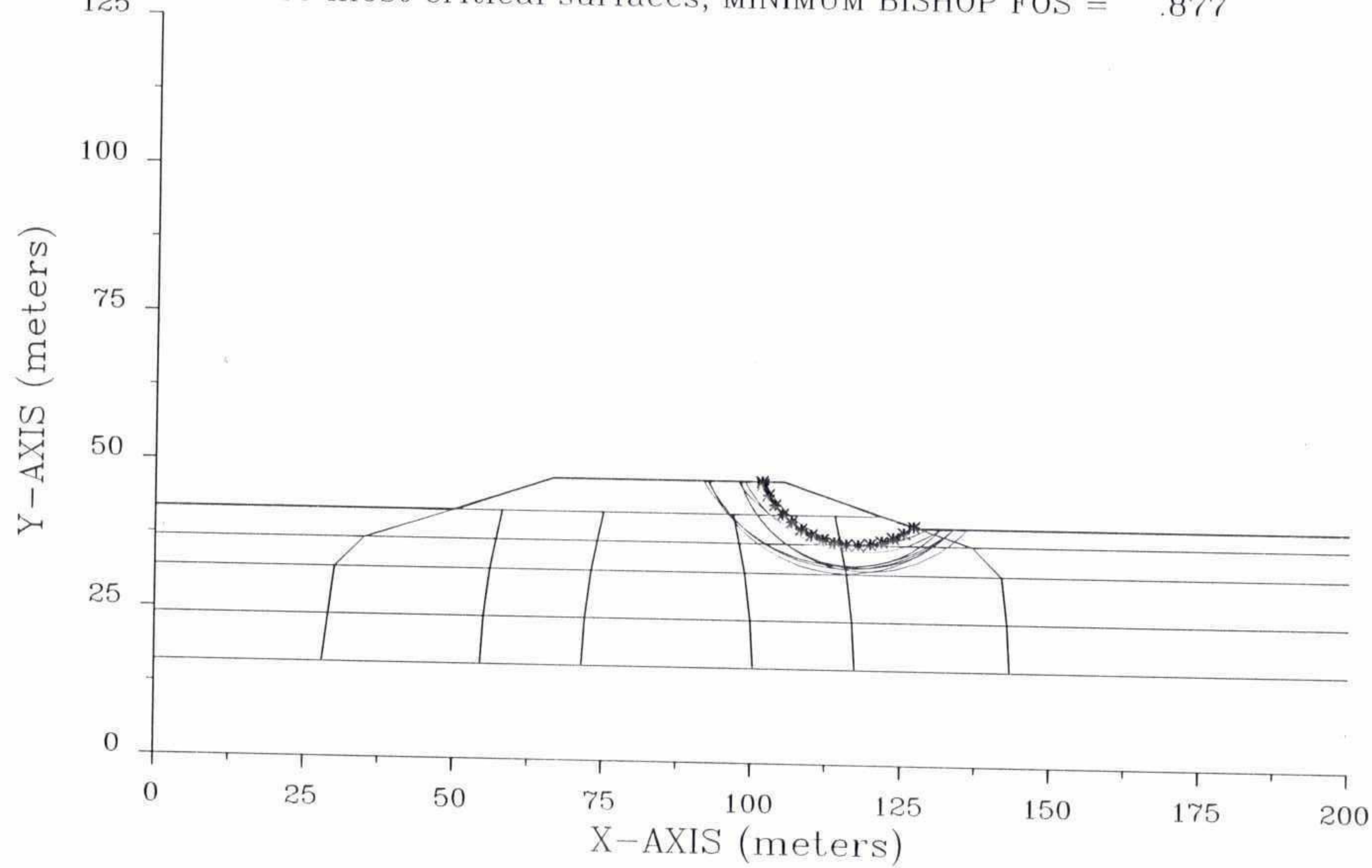


238

DRX41U0S 12:15

SEGMENT 4 STAGE 1, U=0

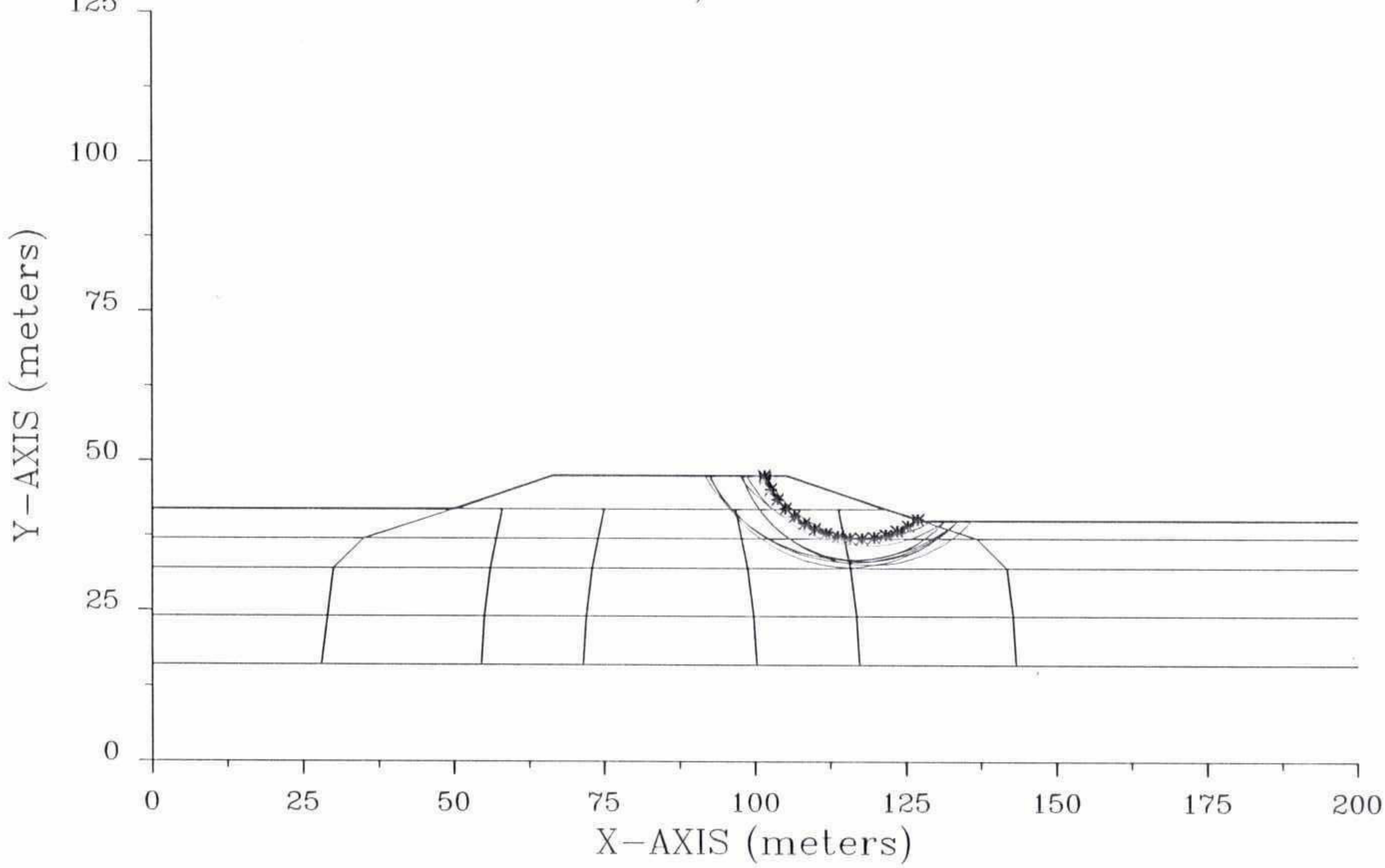
10 most critical surfaces, MINIMUM BISHOP FOS = .877



DRX41U2S 12:17

SEGMENT 4 STAGE 1, U=20

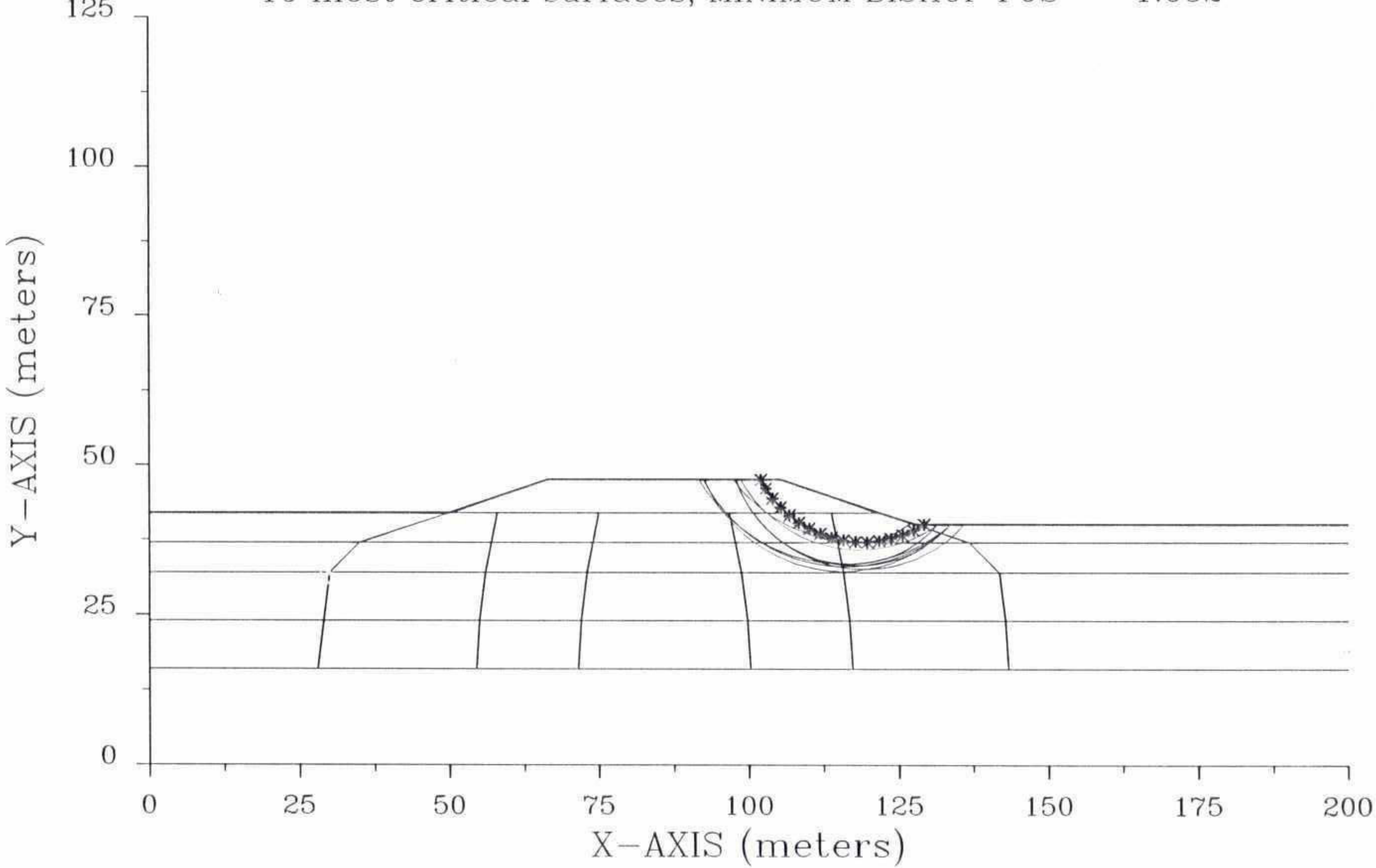
10 most critical surfaces, MINIMUM BISHOP FOS = .948



DRX41U5S 12:19

SEGMENT 4 STAGE 1, U=50

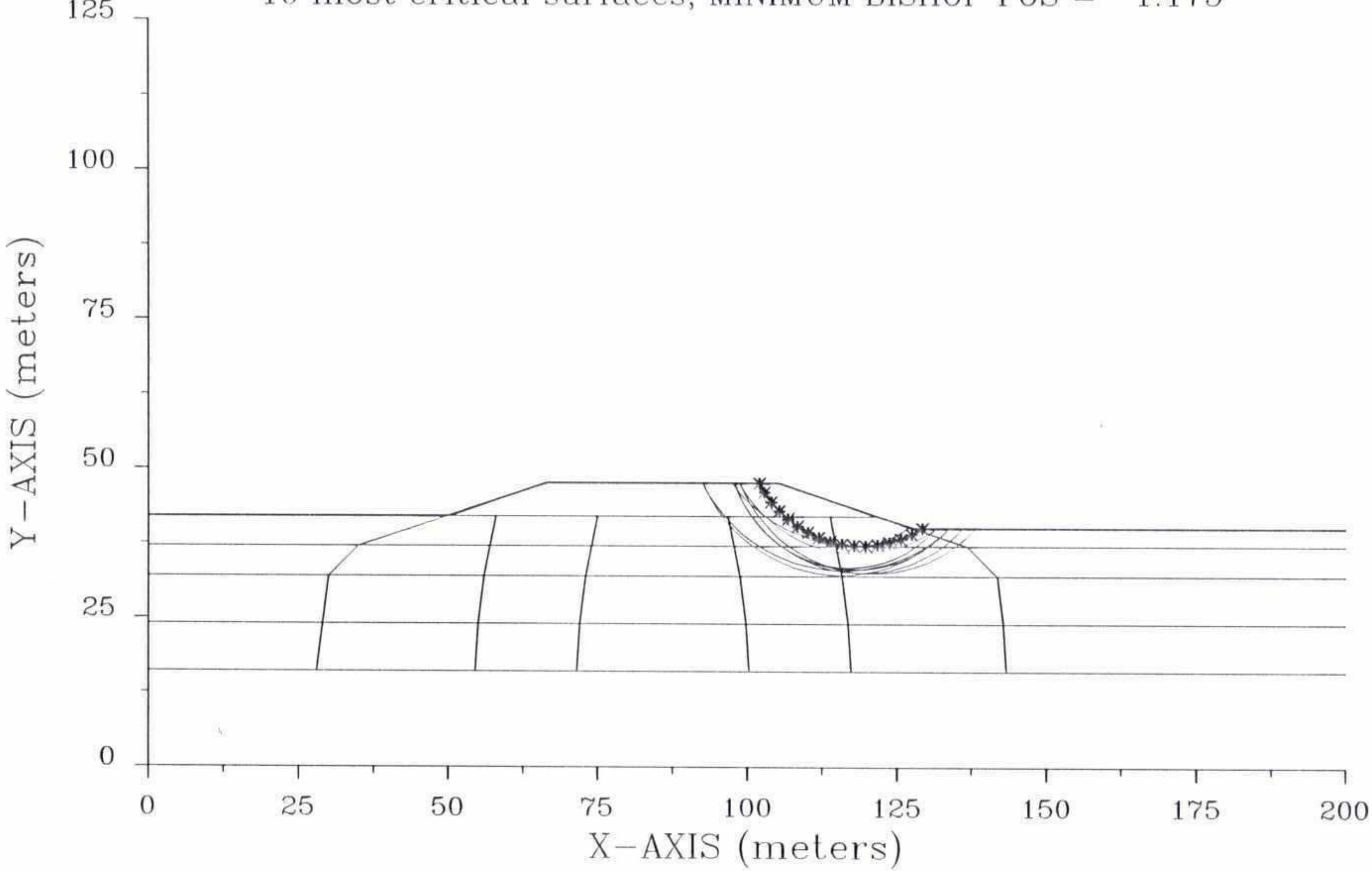
10 most critical surfaces, MINIMUM BISHOP FOS = 1.052



DRX41U9S 12:20

SEGMENT 4 STAGE 1, U=90

10 most critical surfaces, MINIMUM BISHOP FOS = 1.175

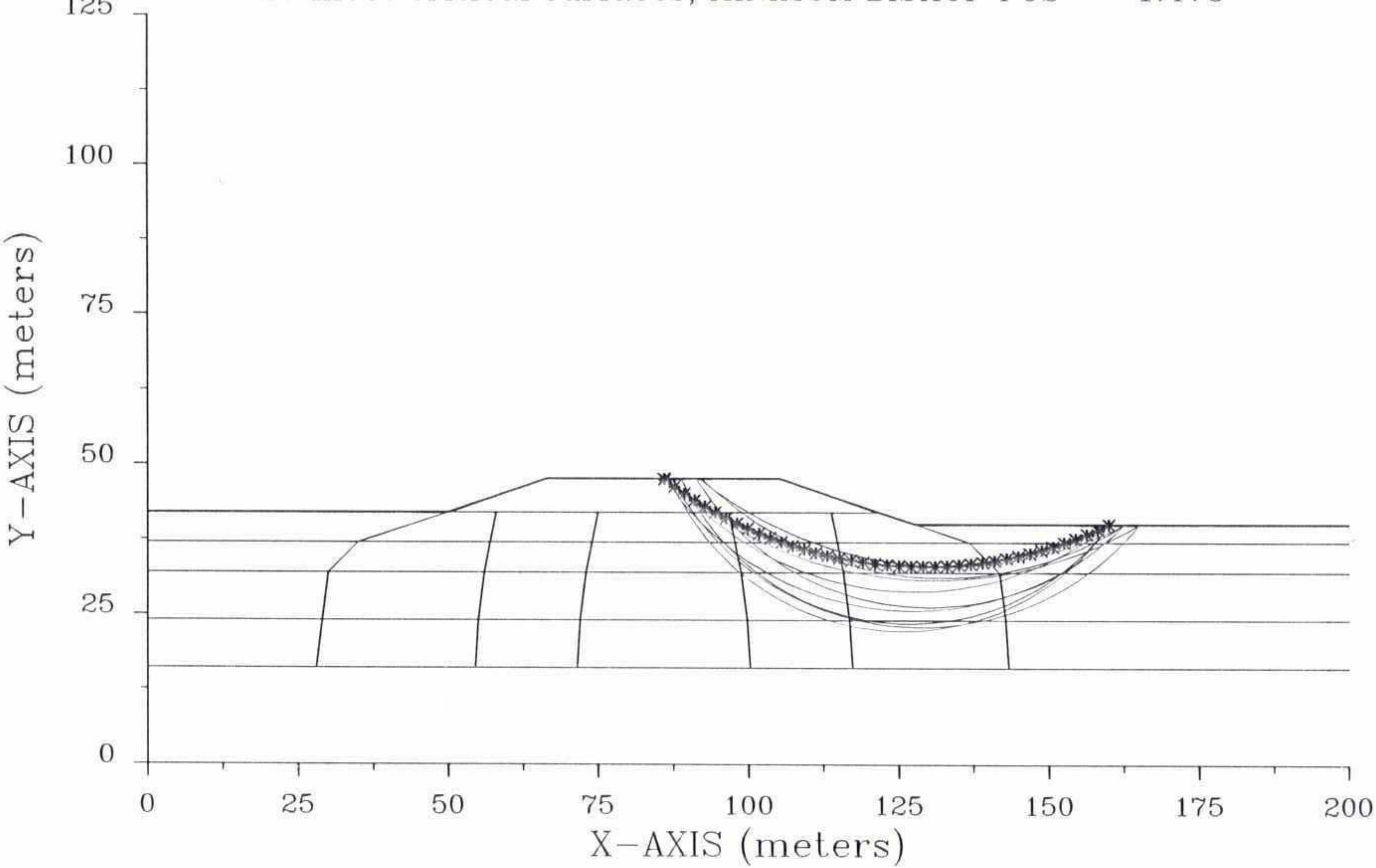


237

DRX41U0D 18:28

SEGMENT 4 STAGE 1, U=0

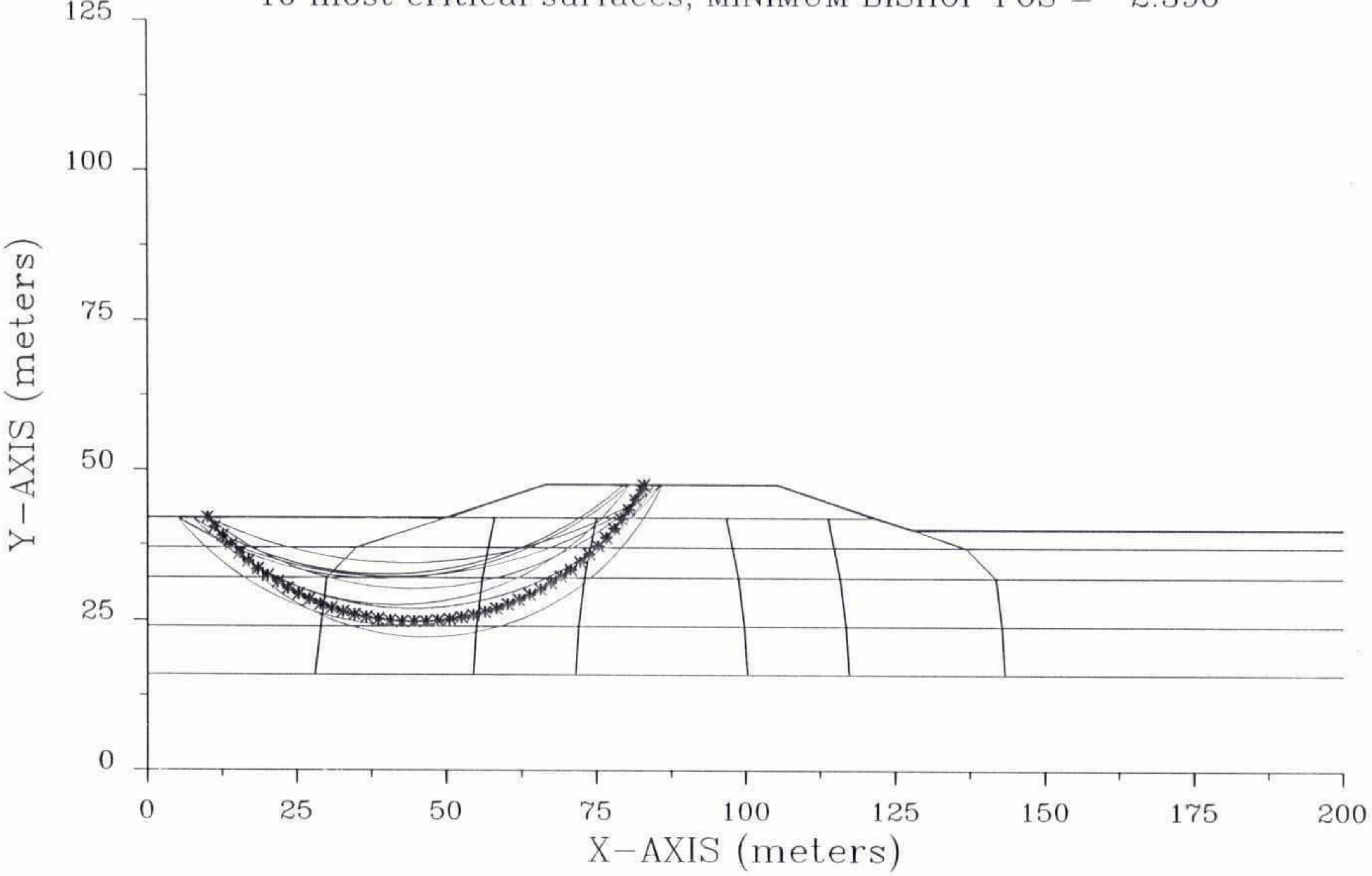
10 most critical surfaces, MINIMUM BISHOP FOS = 1.478



DCX41U9D 10:25

SEGMENT 4 STAGE 1, U=90

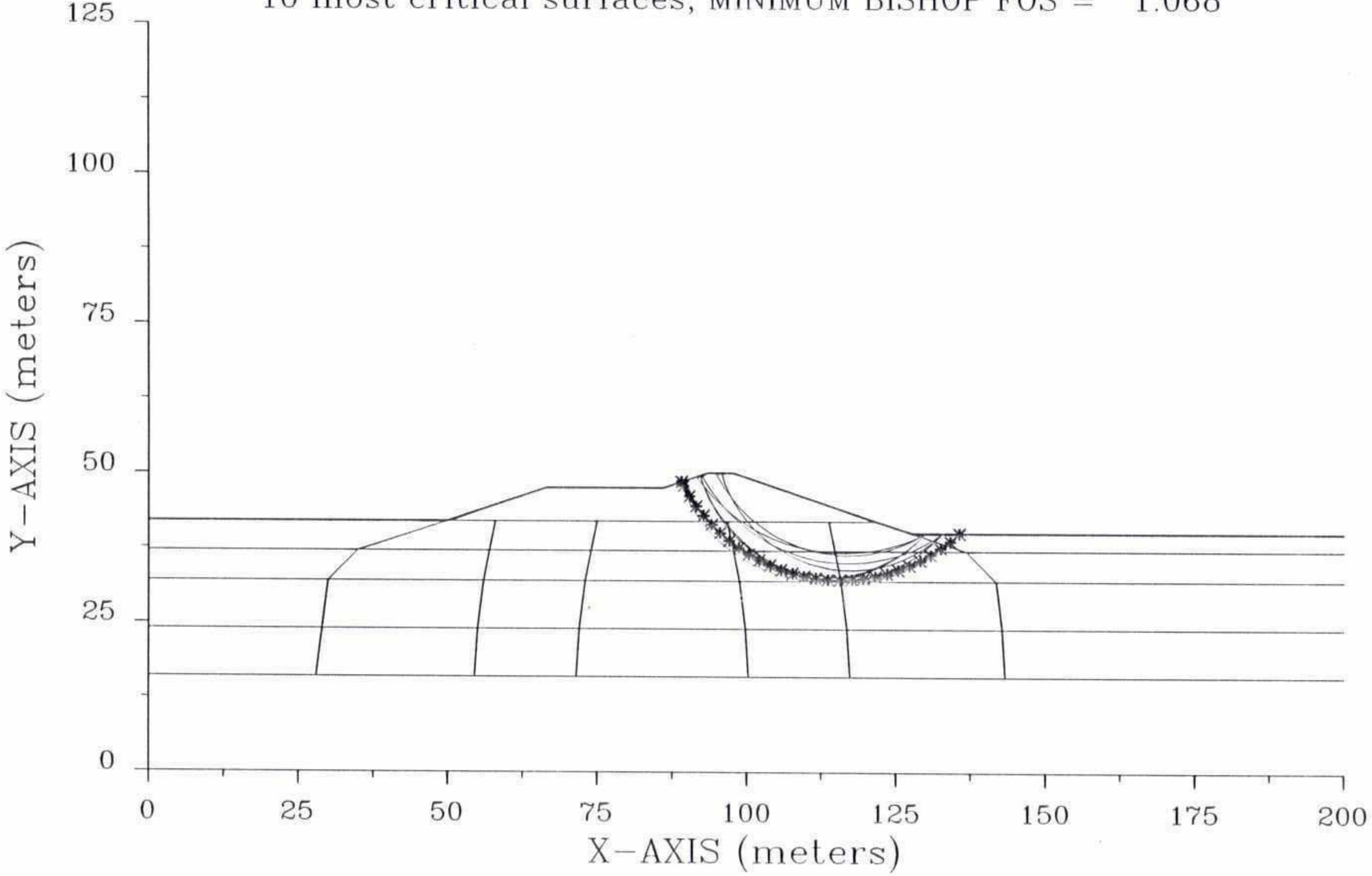
10 most critical surfaces, MINIMUM BISHOP FOS = 2.396



DRX42U0S 12:22

SEGMENT 4, STAGE 2, U=0

10 most critical surfaces, MINIMUM BISHOP FOS = 1.068

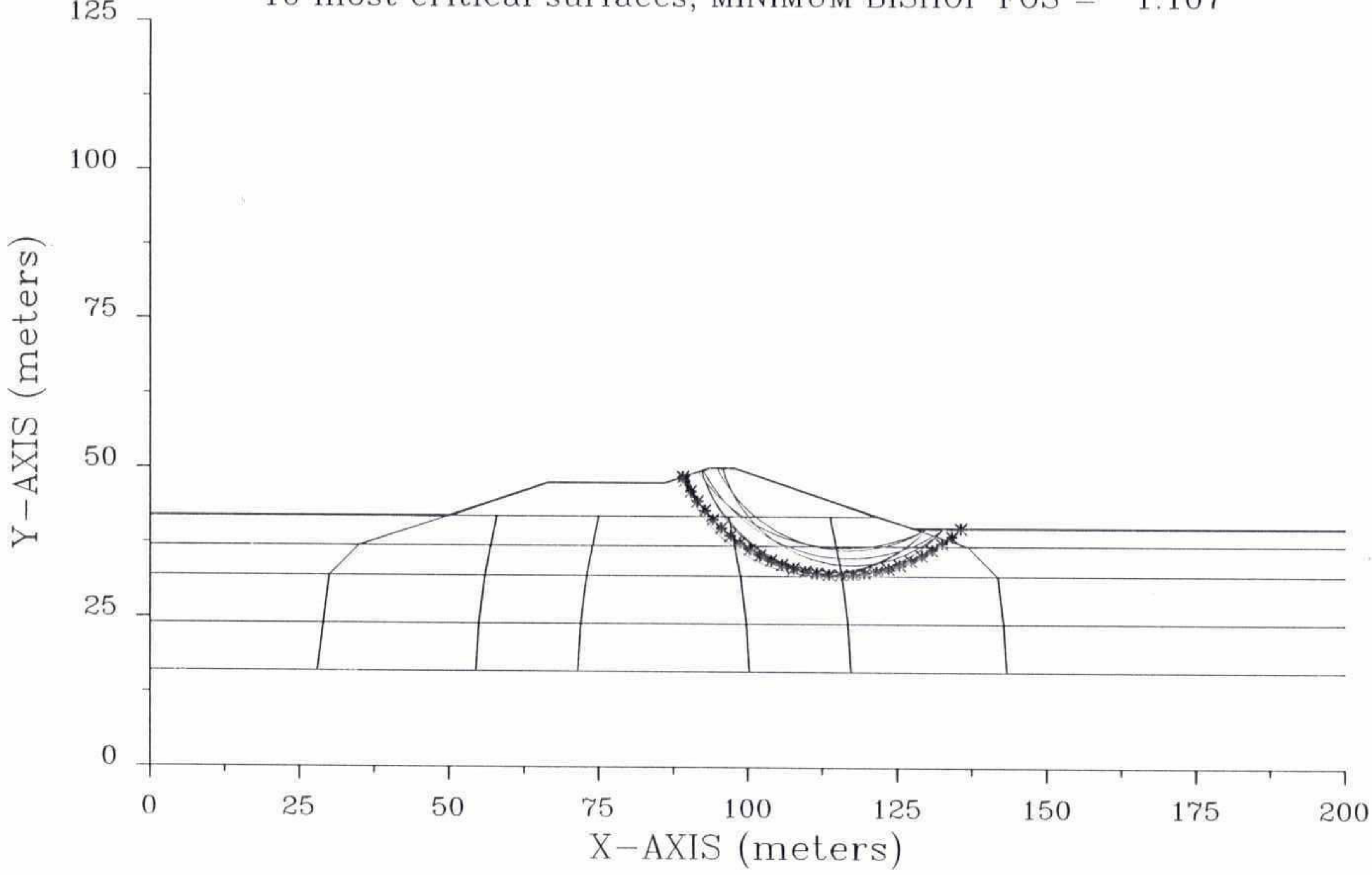


292

DRX42U2S 14:02

SEGMENT 4, STAGE 2, U=20

10 most critical surfaces, MINIMUM BISHOP FOS = 1.107

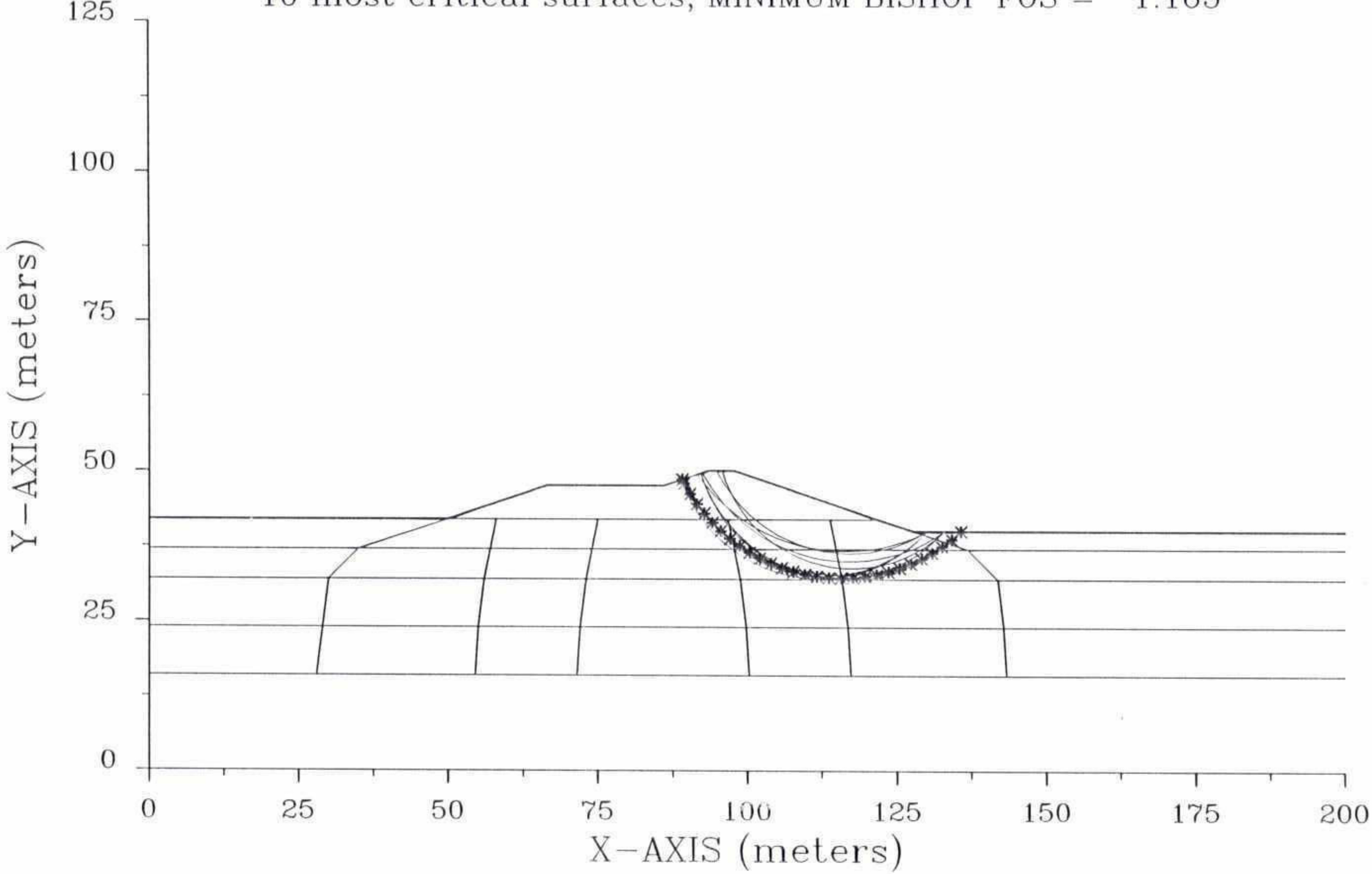


292

DRX42U5S 14:03

SEGMENT 4, STAGE 2, U=50

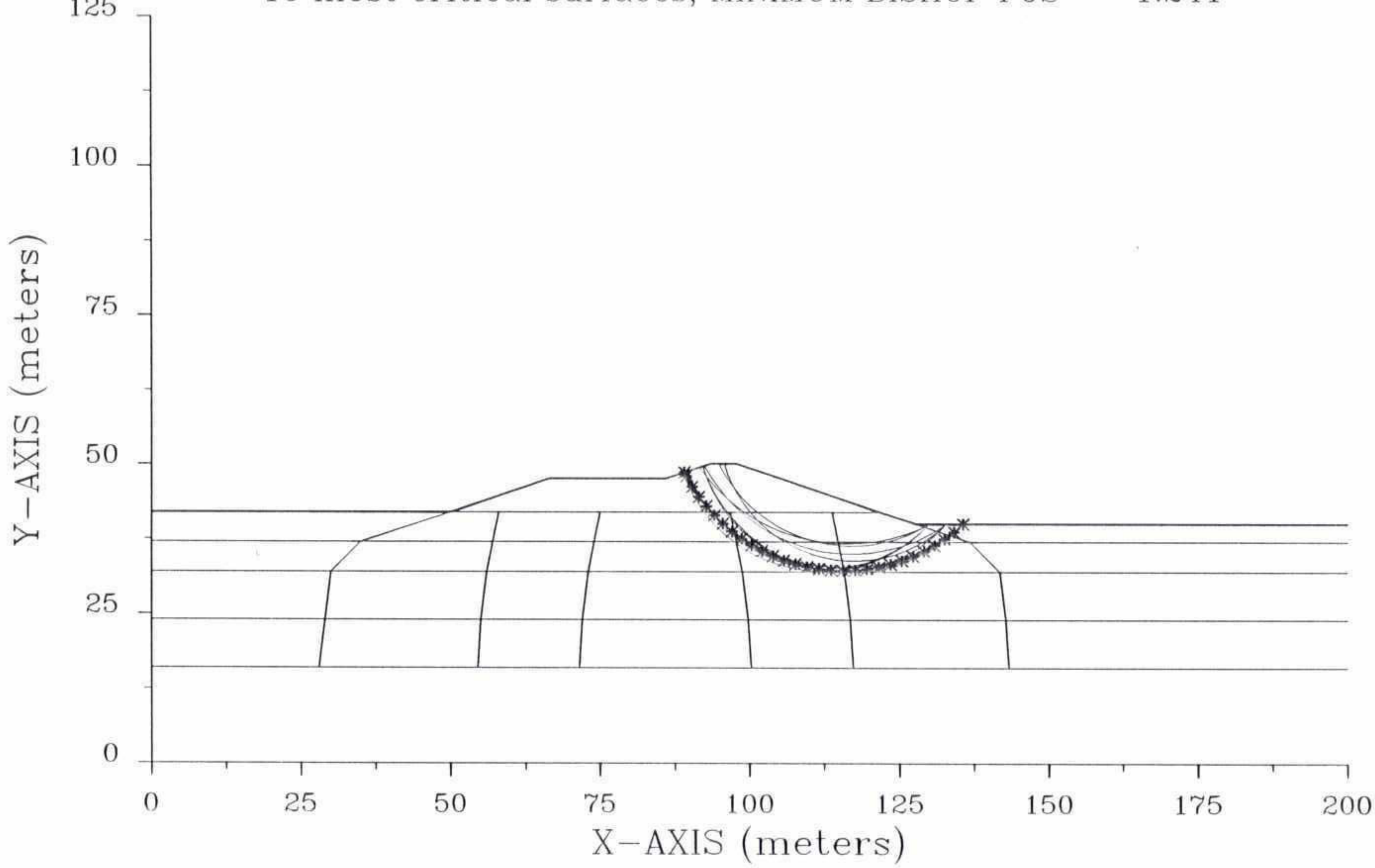
10 most critical surfaces, MINIMUM BISHOP FOS = 1.165



DRX42U9S 14:11

SEGMENT 4, STAGE 2, U=90

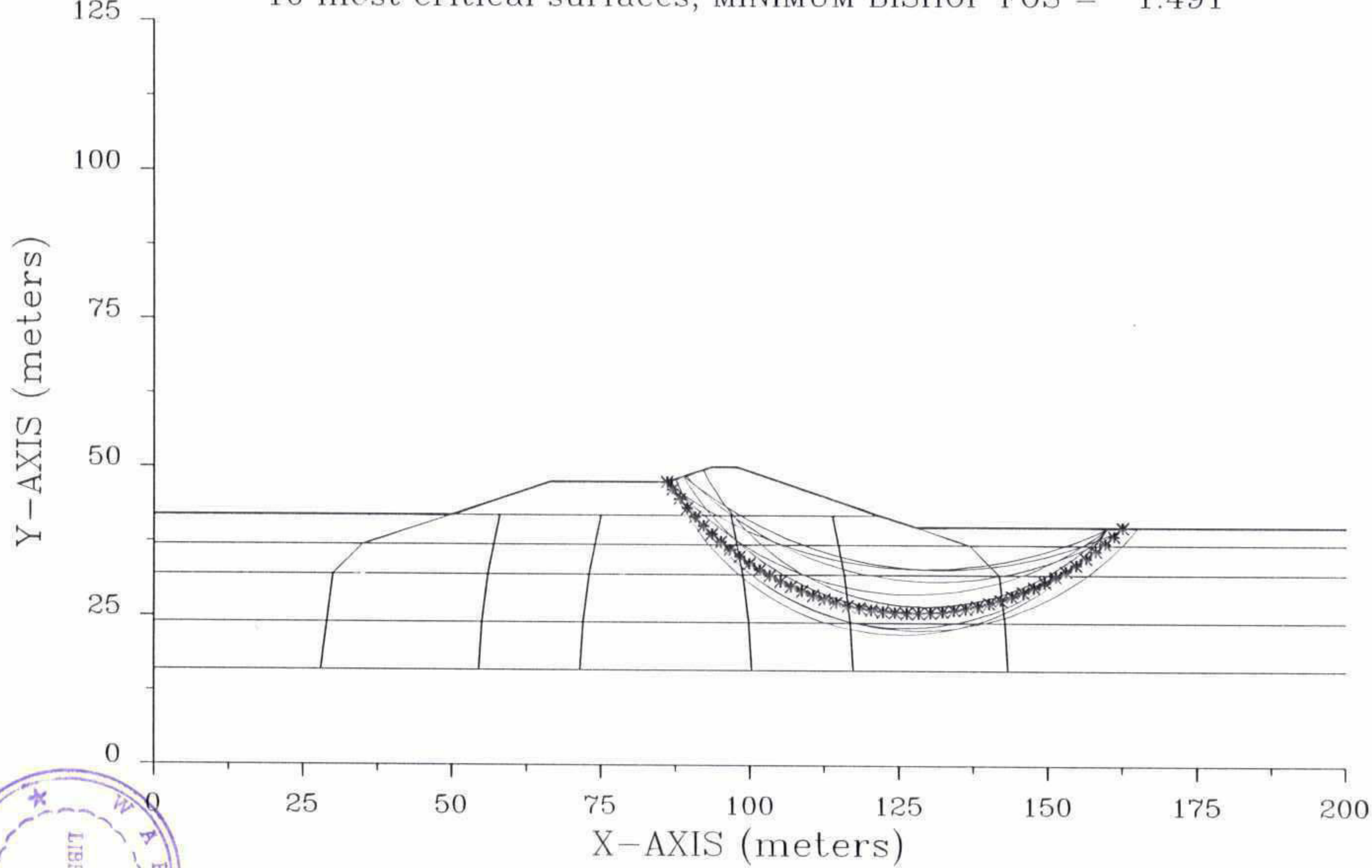
10 most critical surfaces, MINIMUM BISHOP FOS = 1.241



298

SEGMENT 4, STAGE 2, U=0

10 most critical surfaces, MINIMUM BISHOP FOS = 1.491

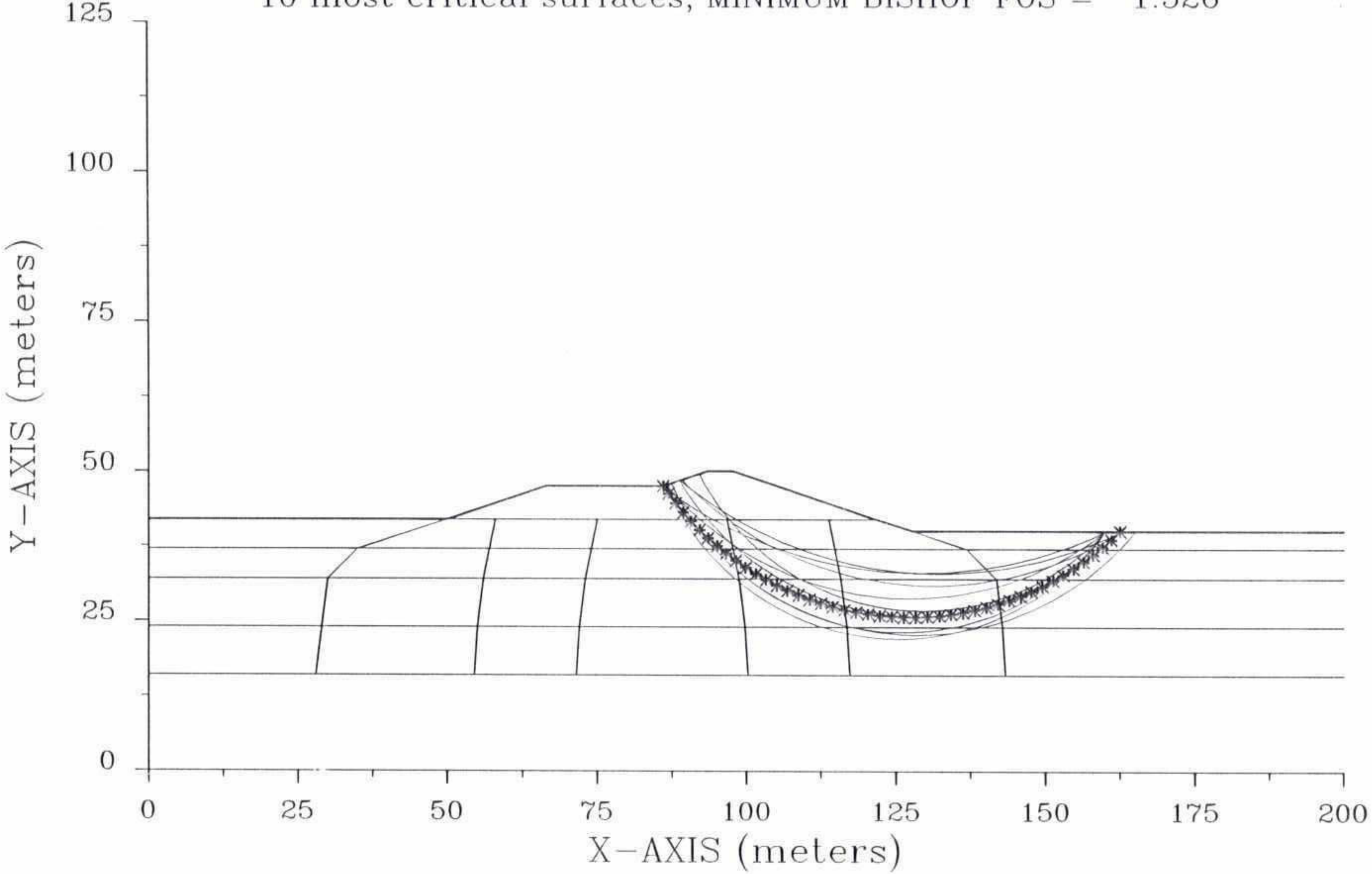


D 6c

DRX42U2D 14:18

SEGMENT 4, STAGE 2, U=20

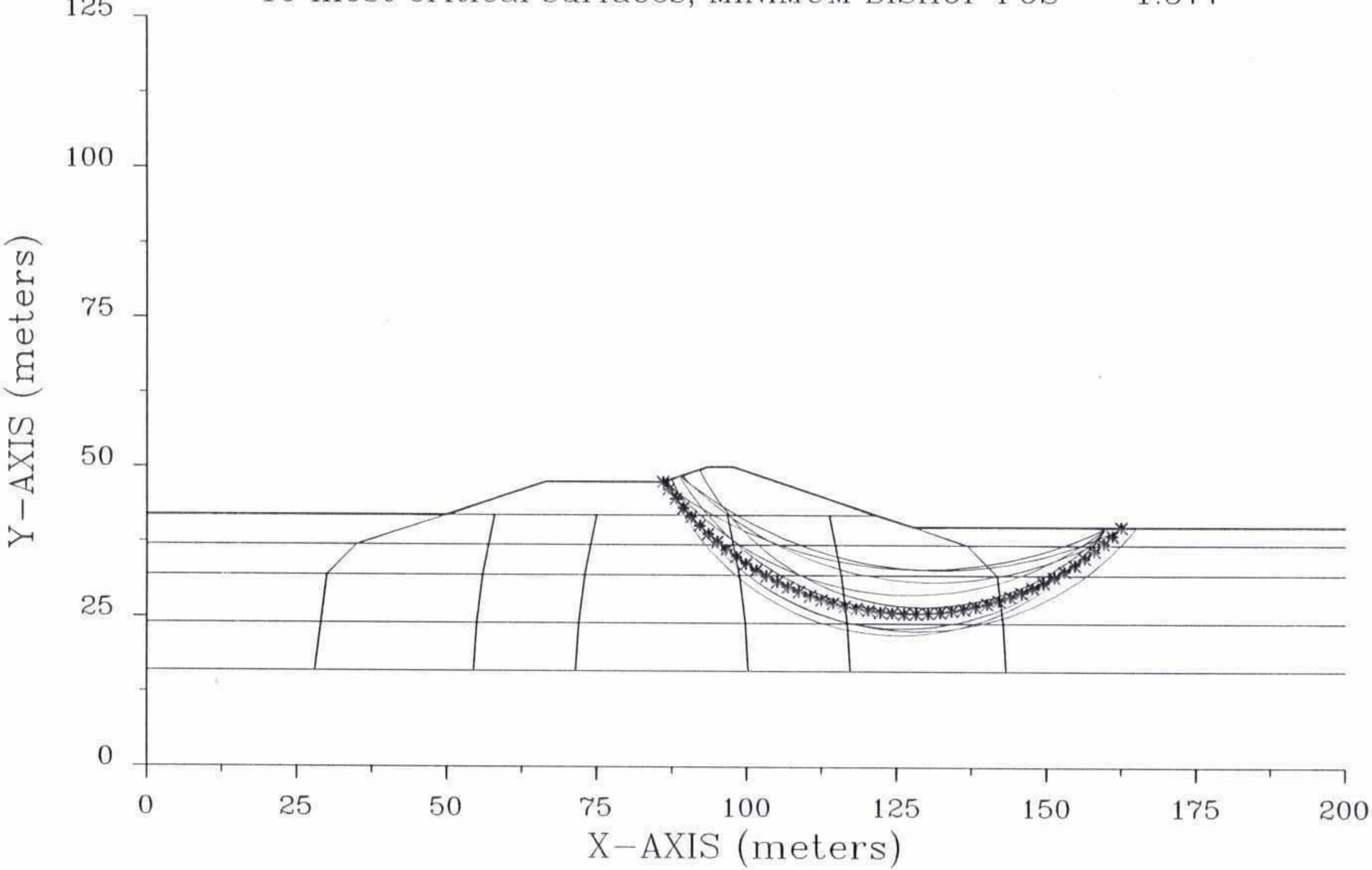
10 most critical surfaces, MINIMUM BISHOP FOS = 1.526



298

SEGMENT 4, STAGE 2, U=50

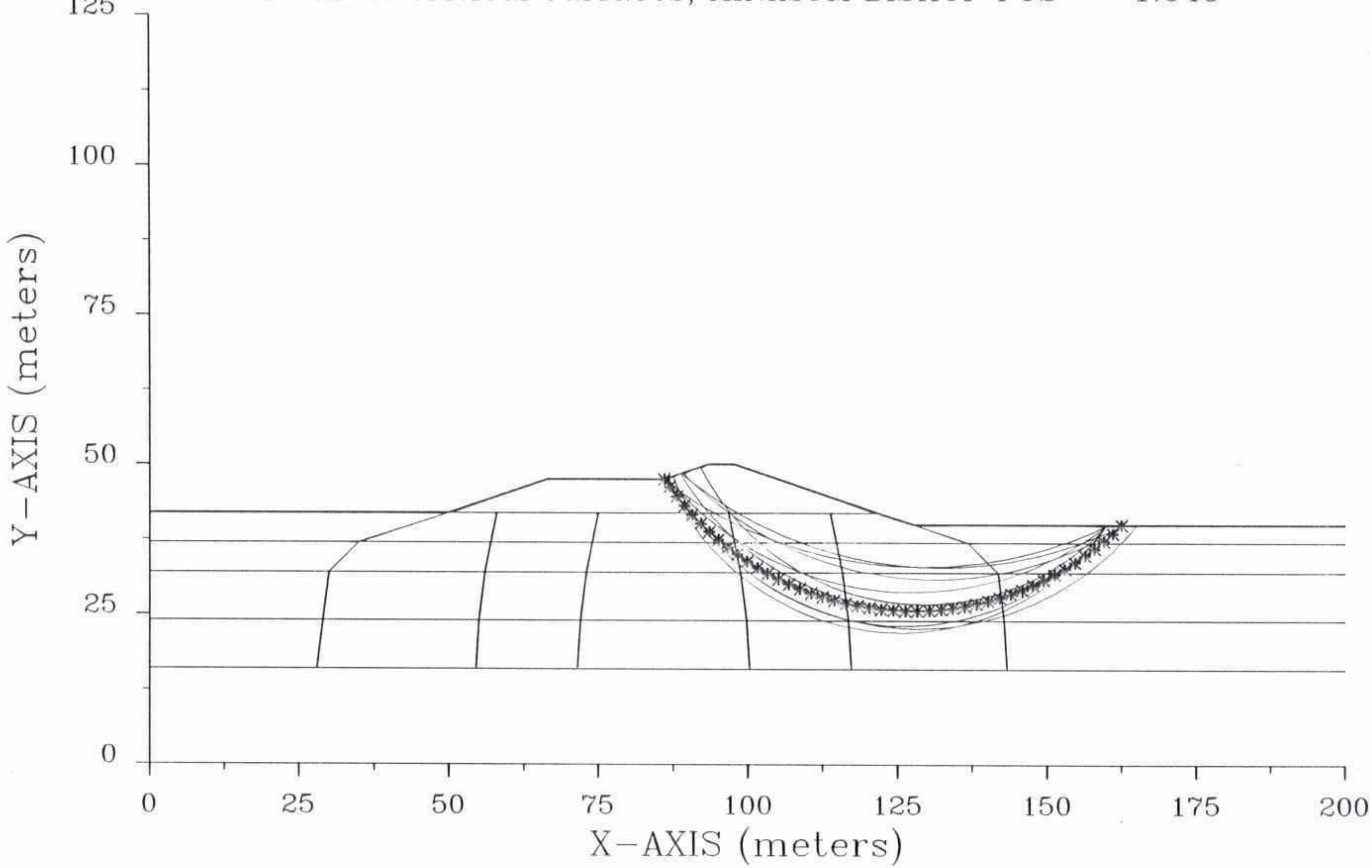
10 most critical surfaces, MINIMUM BISHOP FOS = 1.577



DRX42U9D 14:13

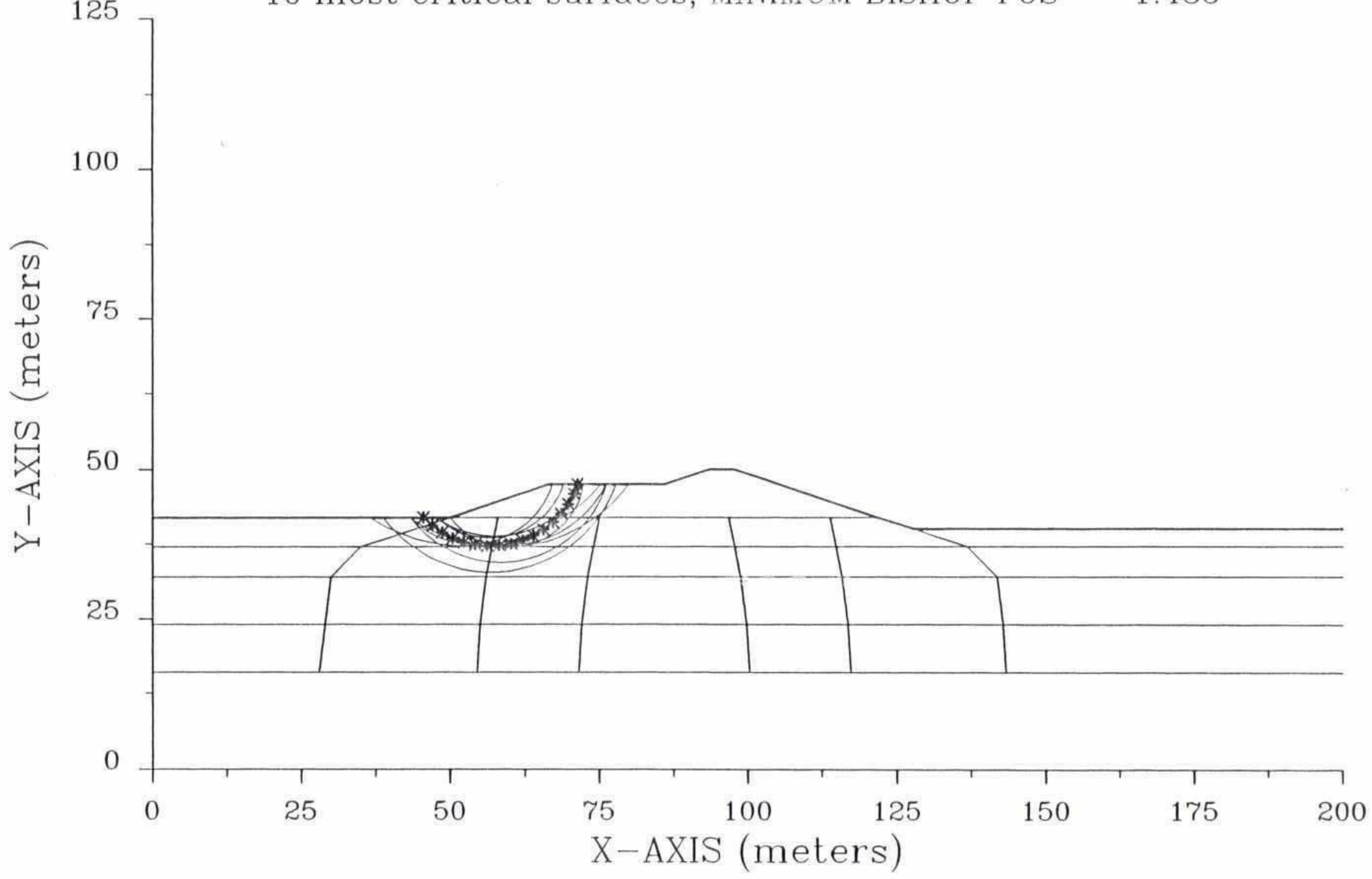
SEGMENT 4, STAGE 2, U=90

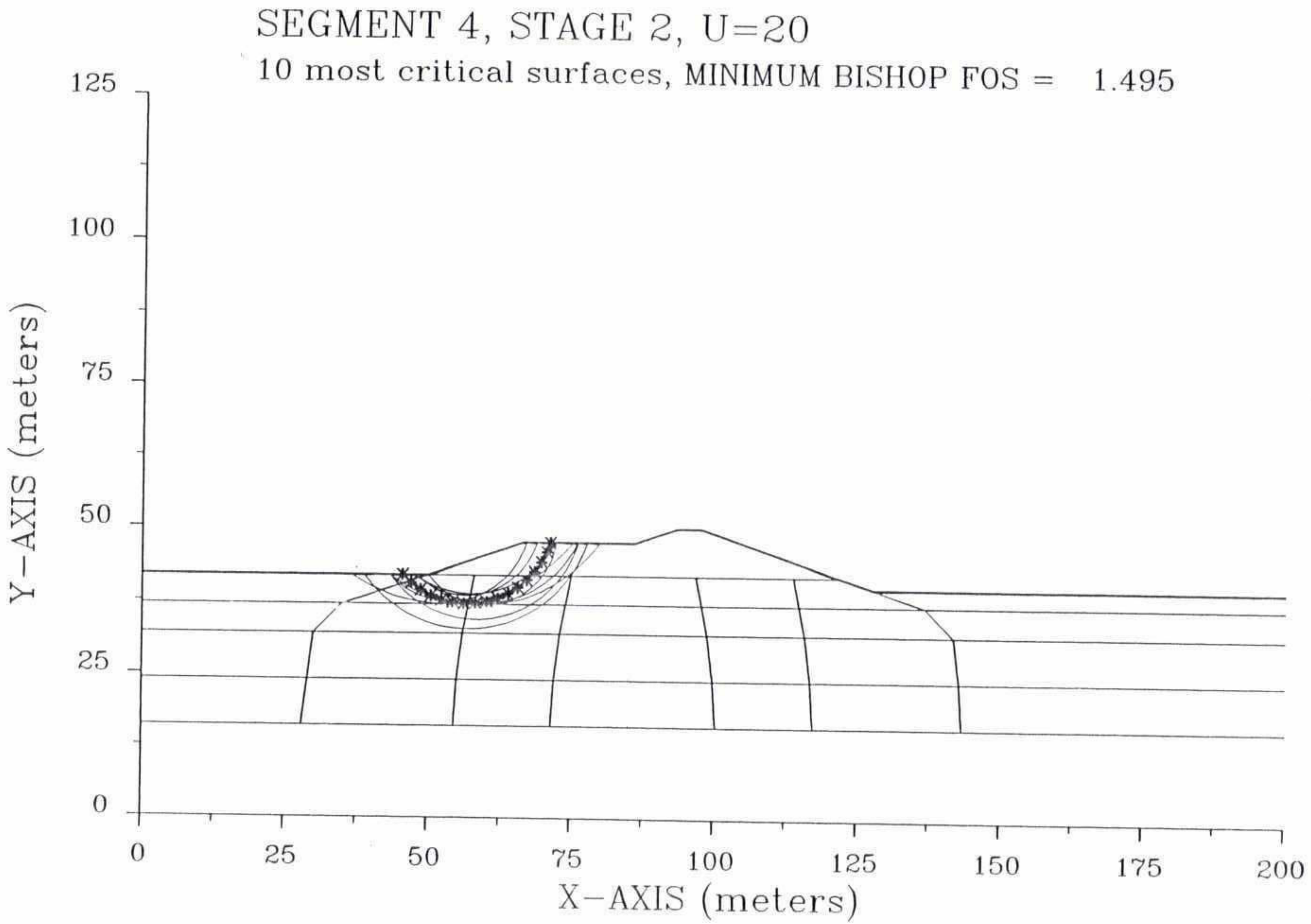
10 most critical surfaces, MINIMUM BISHOP FOS = 1.645



SEGMENT 4, STAGE 2, U=0

10 most critical surfaces, MINIMUM BISHOP FOS = 1.435



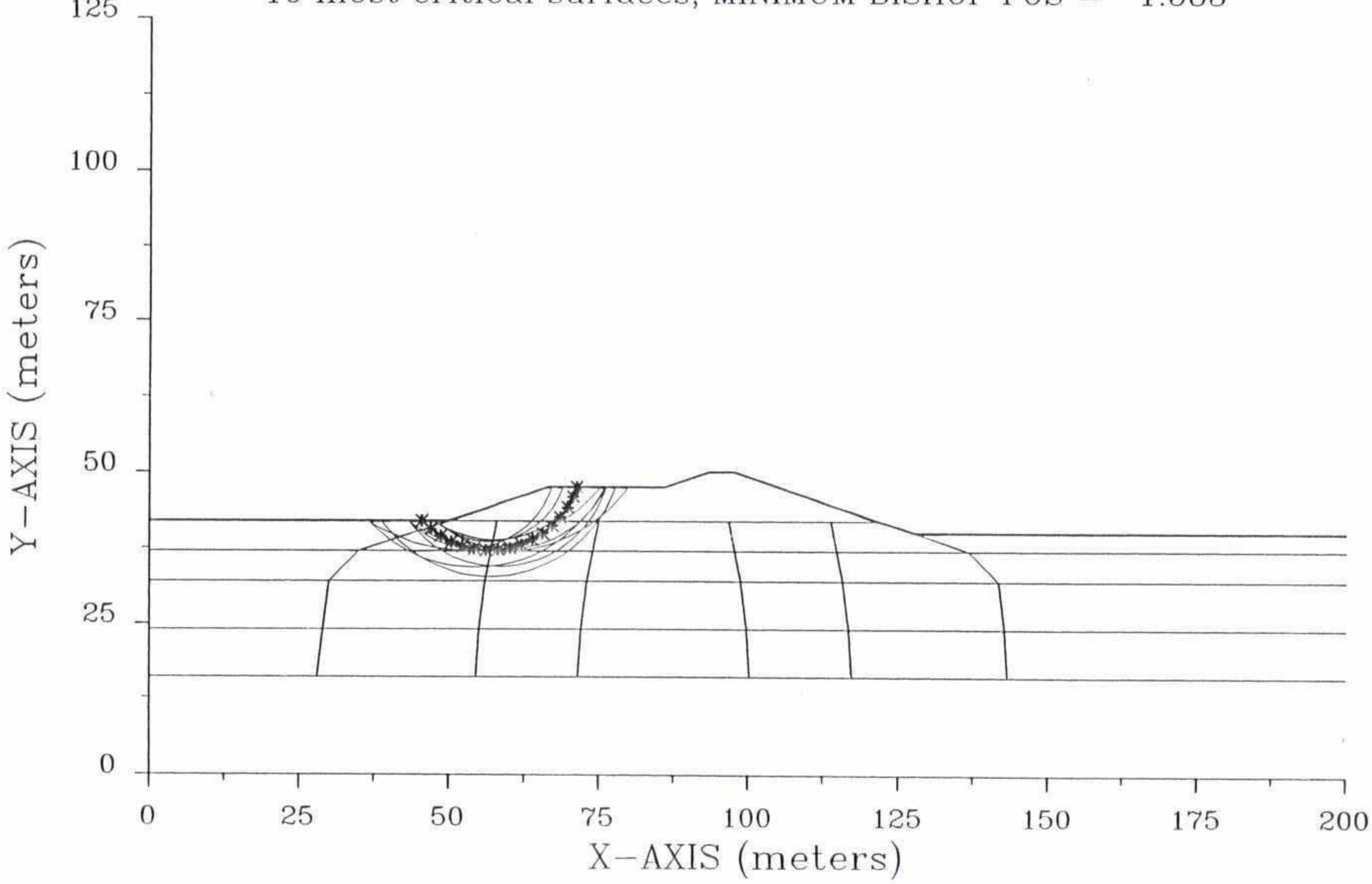

SEGMENT 4, STAGE 2, U=20
10 most critical surfaces, MINIMUM BISHOP FOS = 1.495
125
100
75
50
25
0
Y-AXIS (meters)
0
25
50
75
100
125
150
175
200
X-AXIS (meters)

282

DCX42U50 14:22

SEGMENT 4, STAGE 2, U=50

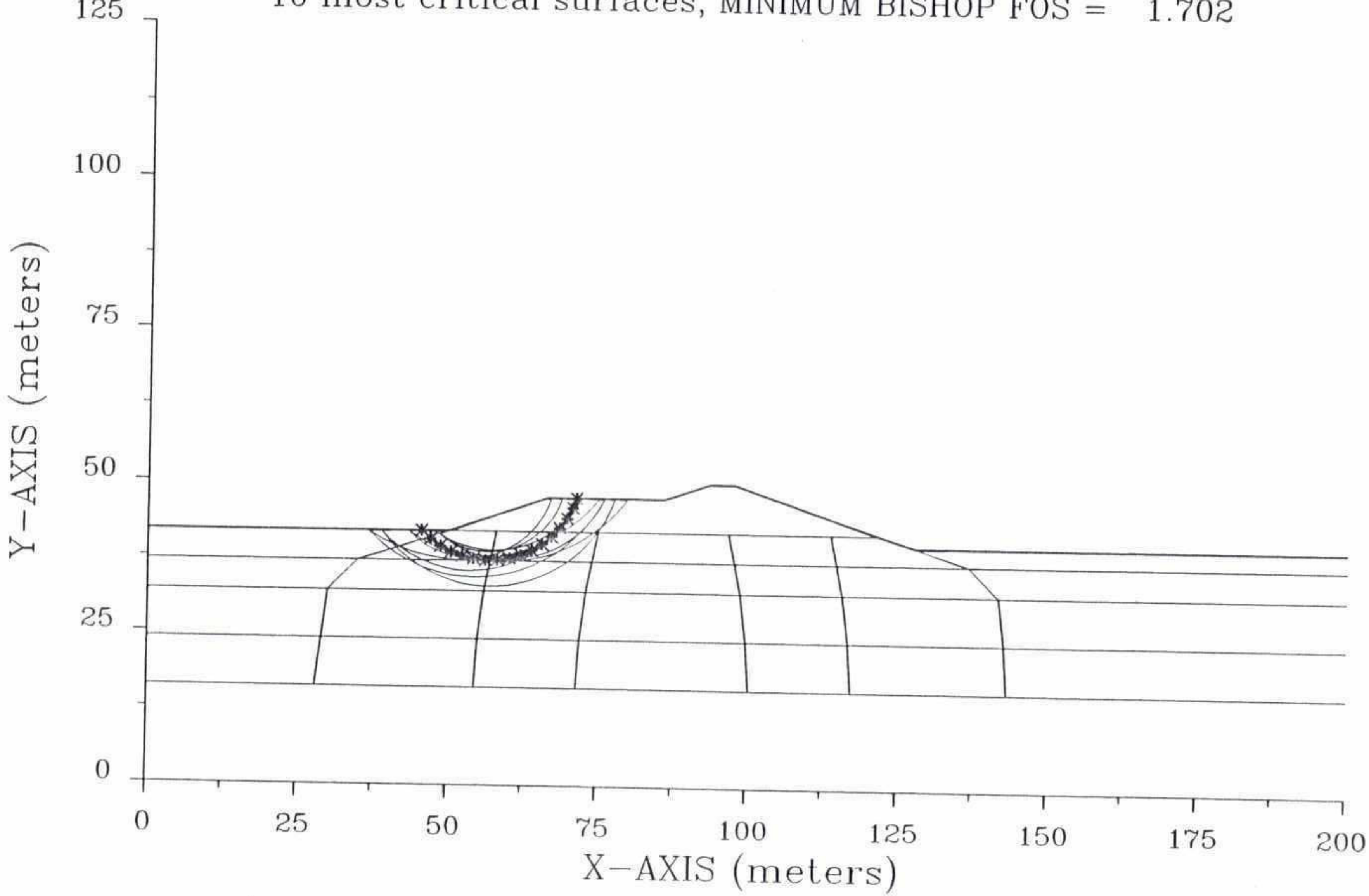
10 most critical surfaces, MINIMUM BISHOP FOS = 1.583



DCX42U90 14:24

SEGMENT 4, STAGE 2, U=90

10 most critical surfaces, MINIMUM BISHOP FOS = 1.702

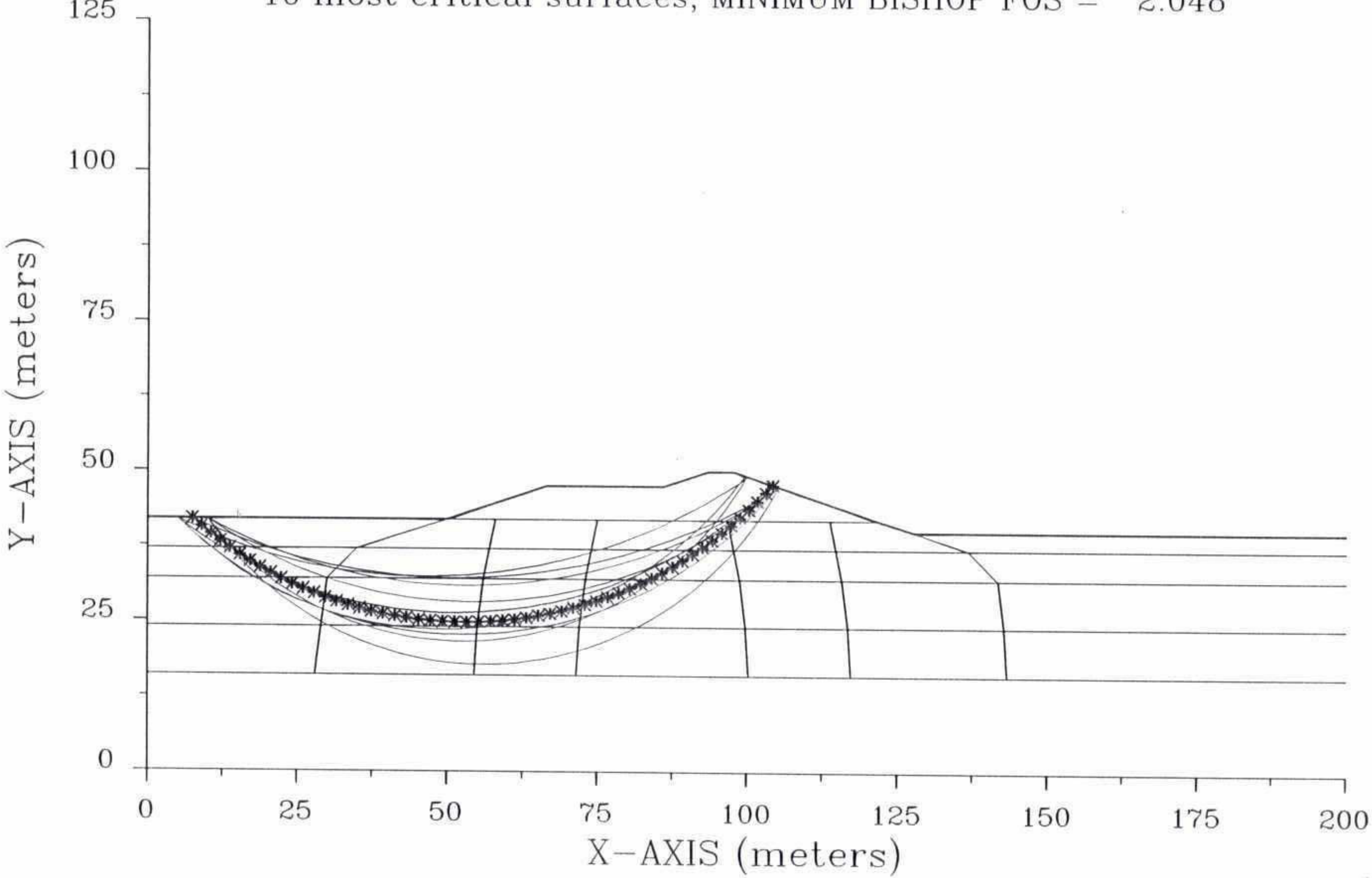


282

DCX42U0D 18:54

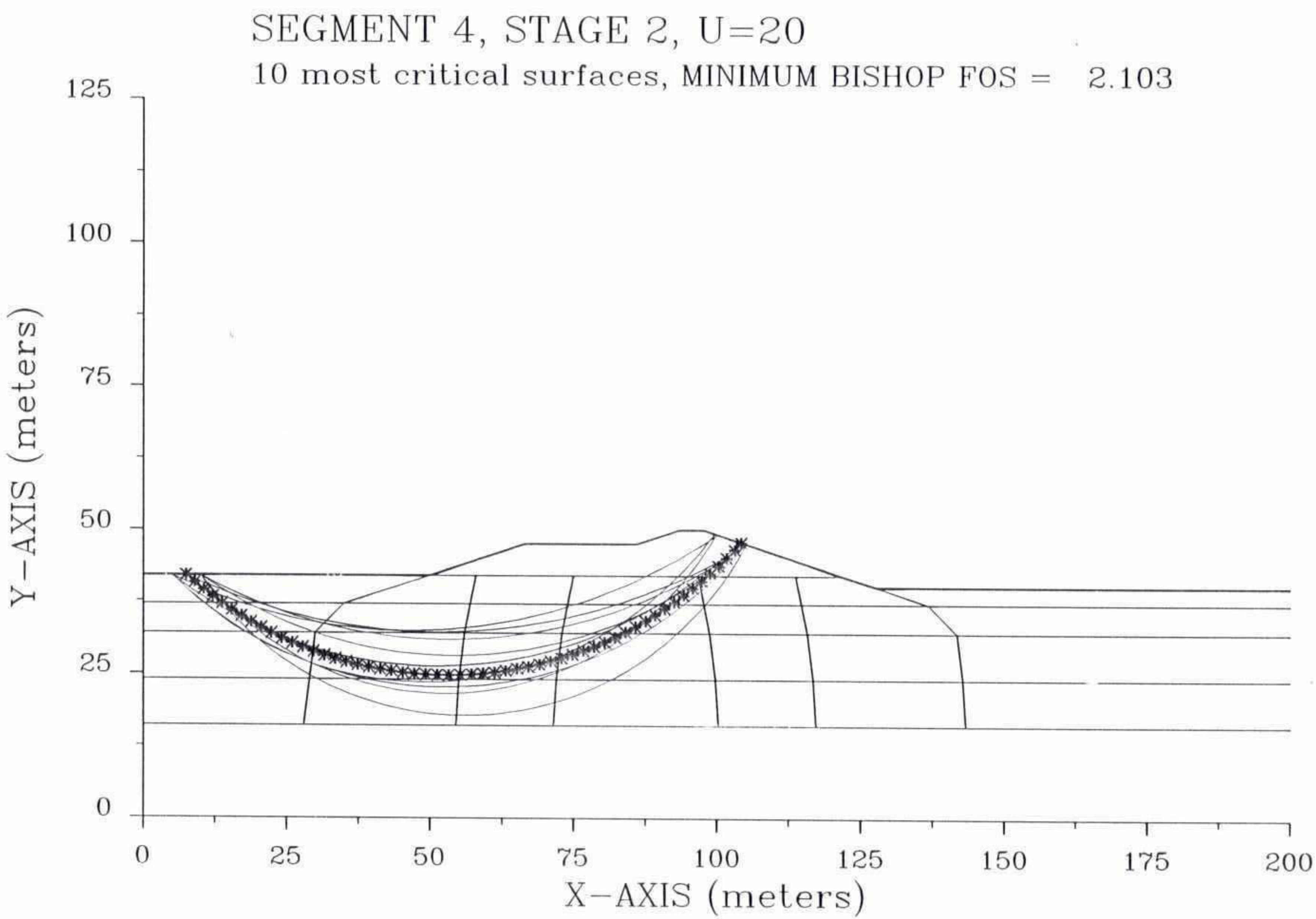
SEGMENT 4, STAGE 2, U=0

10 most critical surfaces, MINIMUM BISHOP FOS = 2.048



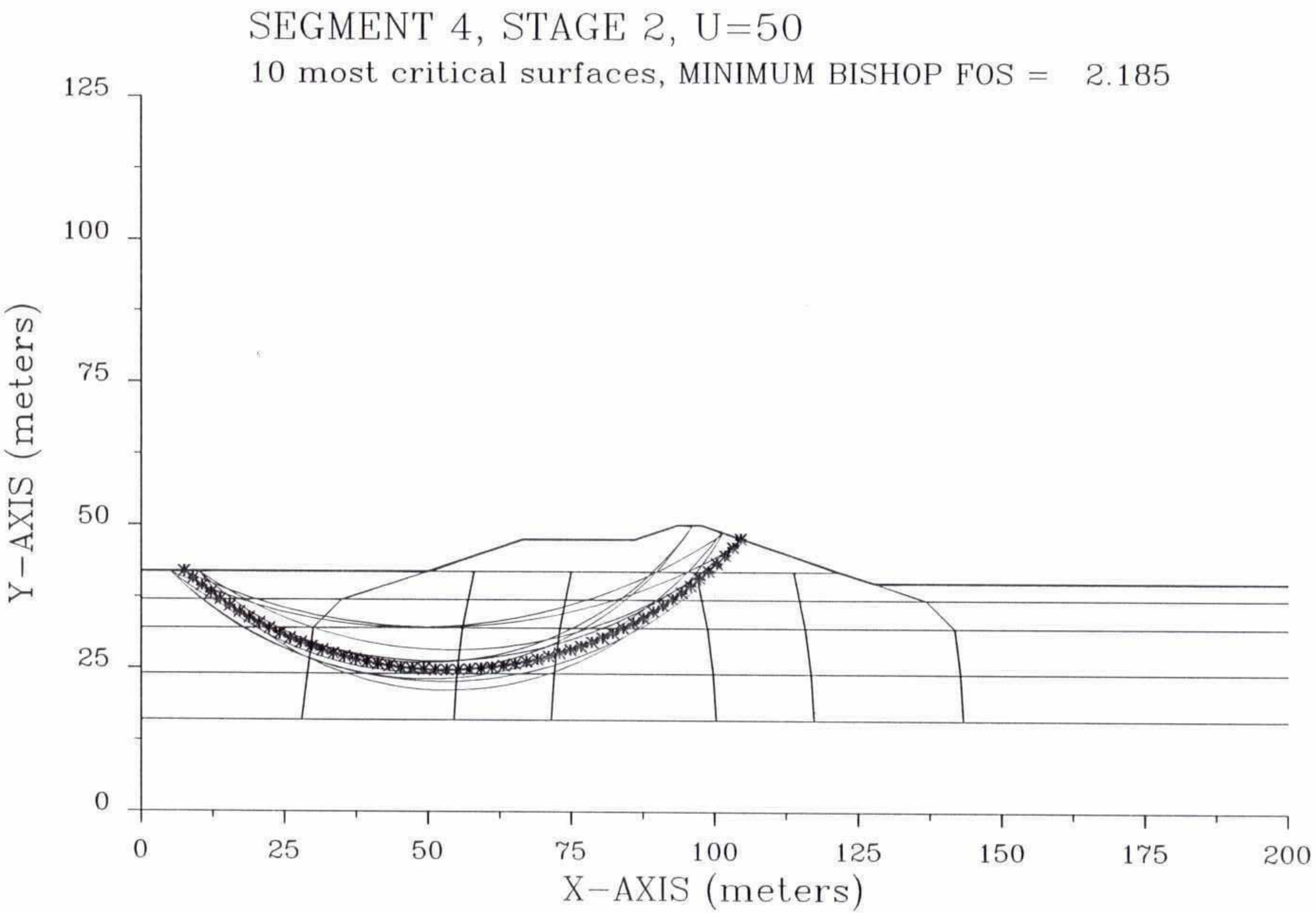
276

DCX42U2D 18:58



228

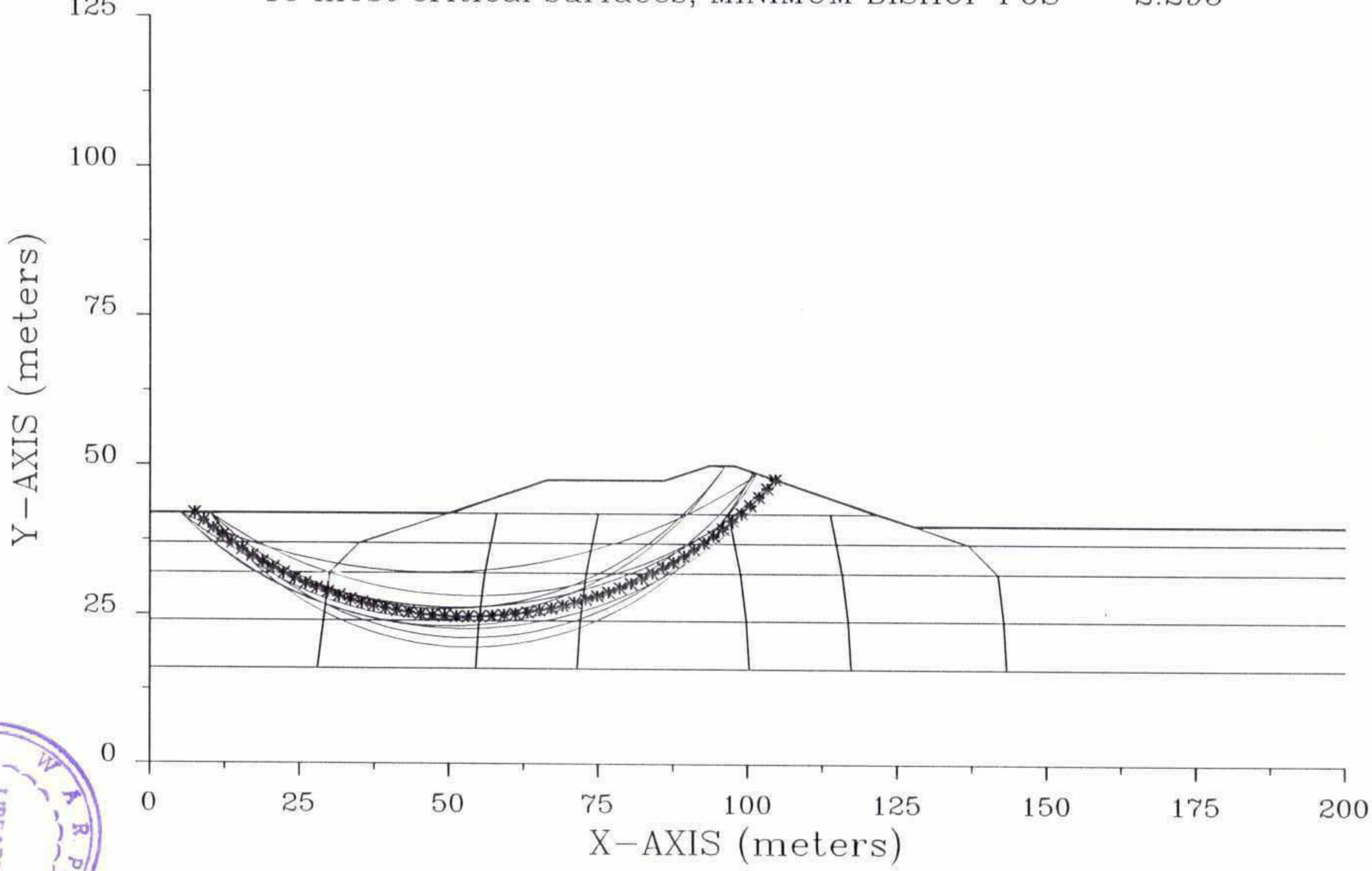
DCX42U5D 19:01



DCX42U9D 19:03

SEGMENT 4, STAGE 2, U=90

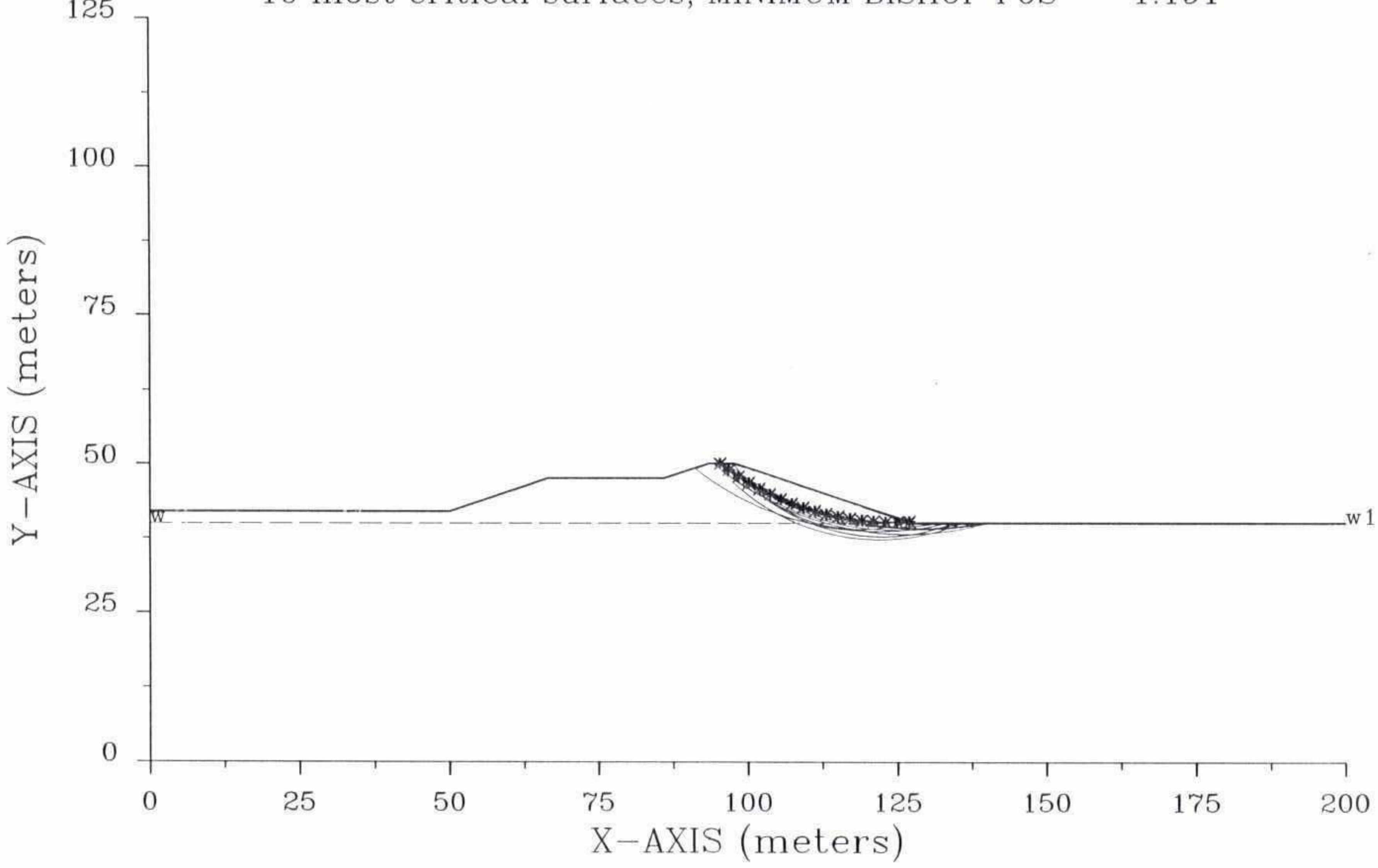
10 most critical surfaces, MINIMUM BISHOP FOS = 2.295



DRSDR20 16:34

FINAL EMBANKMENT DRAINED CASE PHI=20

10 most critical surfaces, MINIMUM BISHOP FOS = 1.191

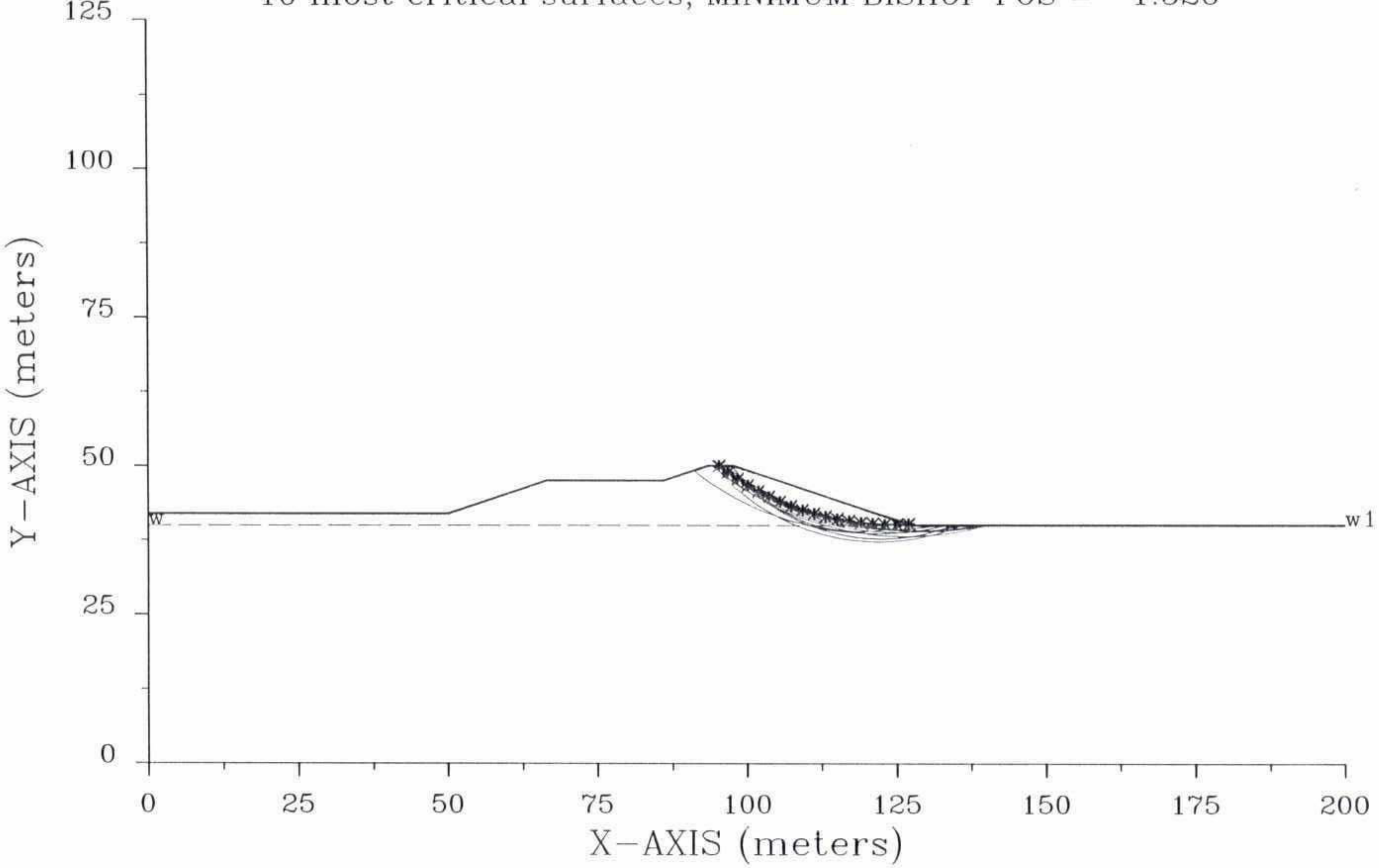


269

DRSDR25 16:33

FINAL EMBANKMENT DRAINED CASE PHI=25

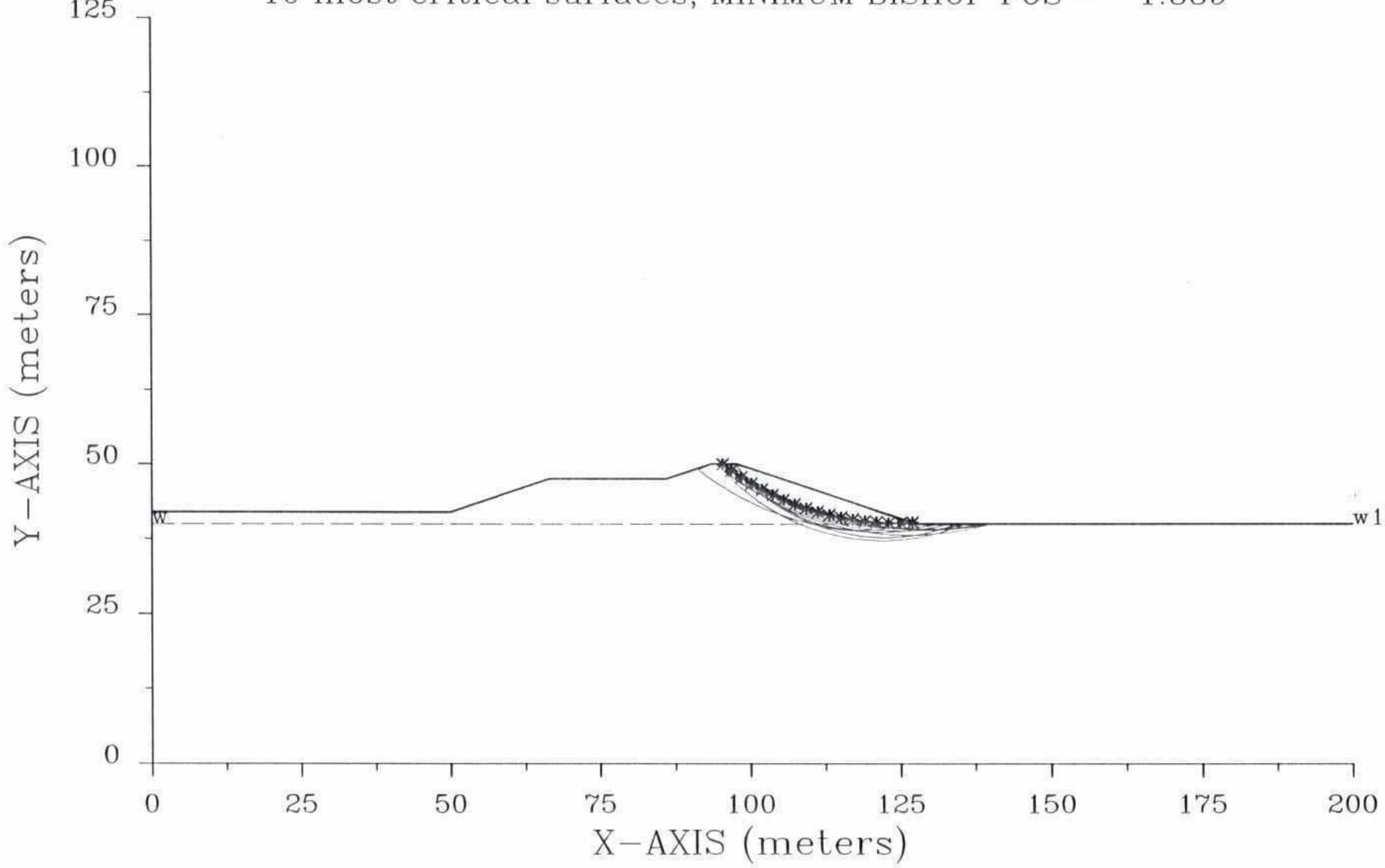
10 most critical surfaces, MINIMUM BISHOP FOS = 1.526



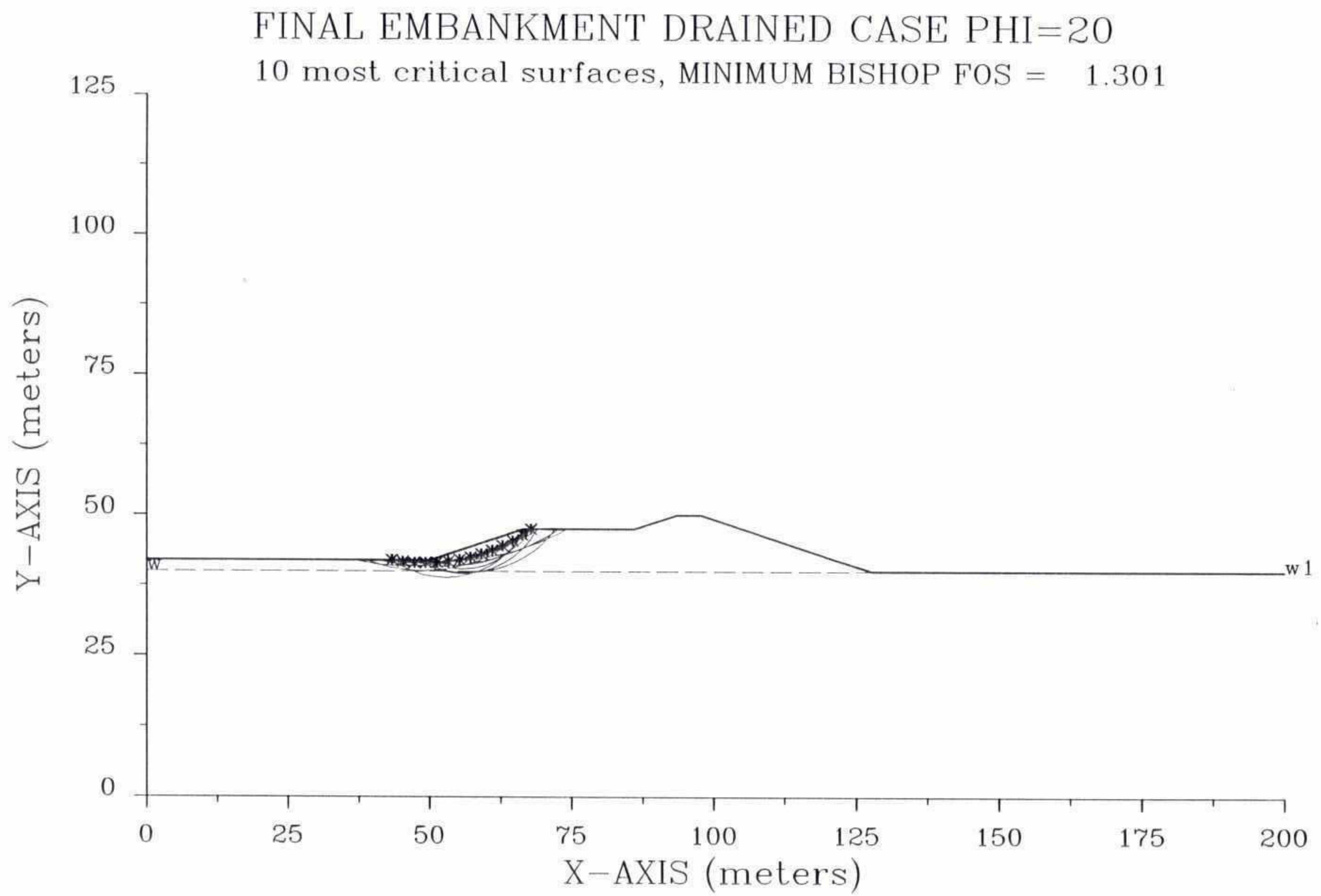
DRSDR30 16:30

FINAL EMBANKMENT DRAINED CASE PHI=30

10 most critical surfaces, MINIMUM BISHOP FOS = 1.889

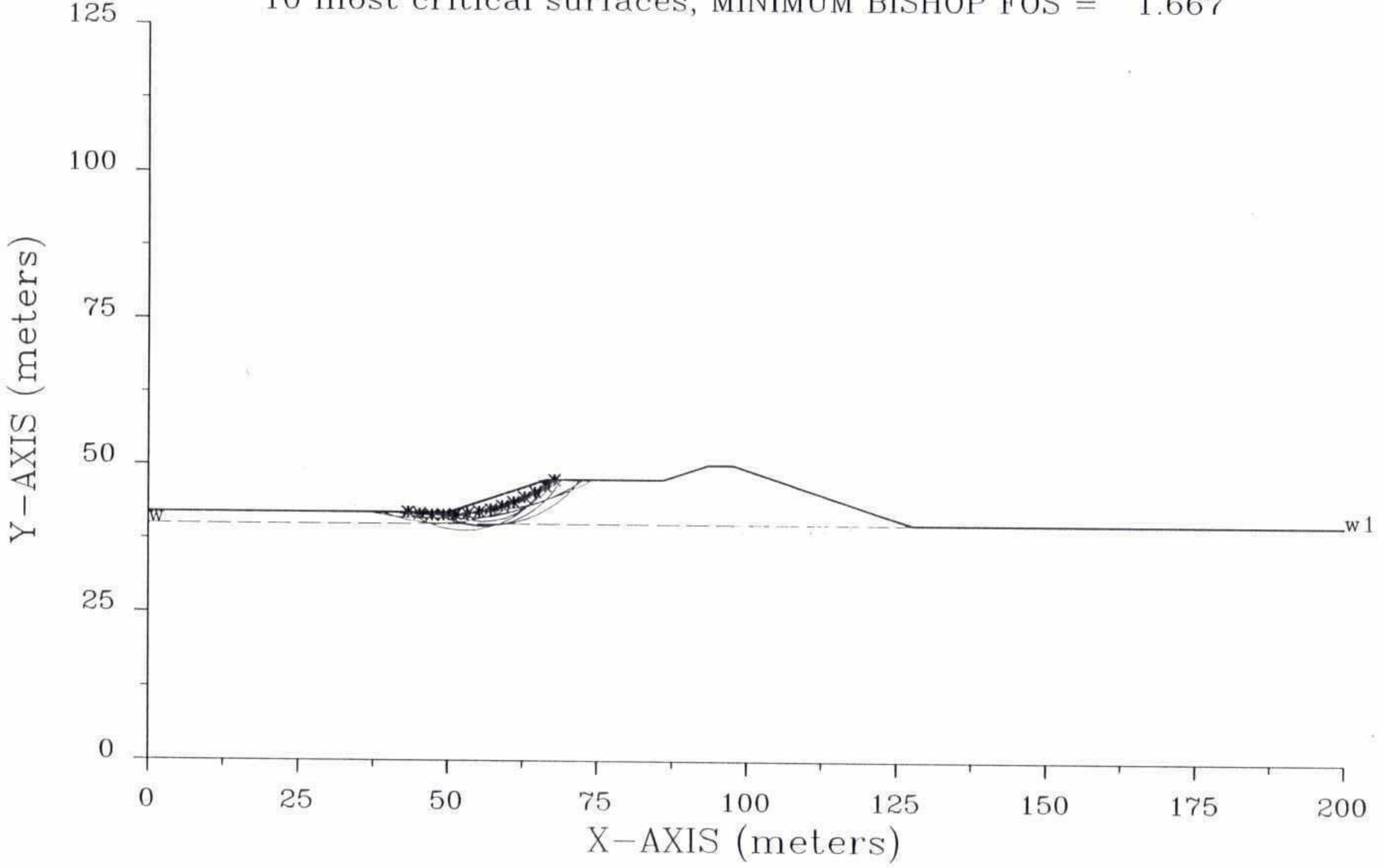


DCSDR20 8:47


FINAL EMBANKMENT DRAINED CASE PHI=20
10 most critical surfaces, MINIMUM BISHOP FOS = 1.301
Y-AXIS (meters)
125
100
75
50
25
0
X-AXIS (meters)
0
25
50
75
100
125
150
175
200
W
W1

FINAL EMBANKMENT DRAINED CASE PHI=25

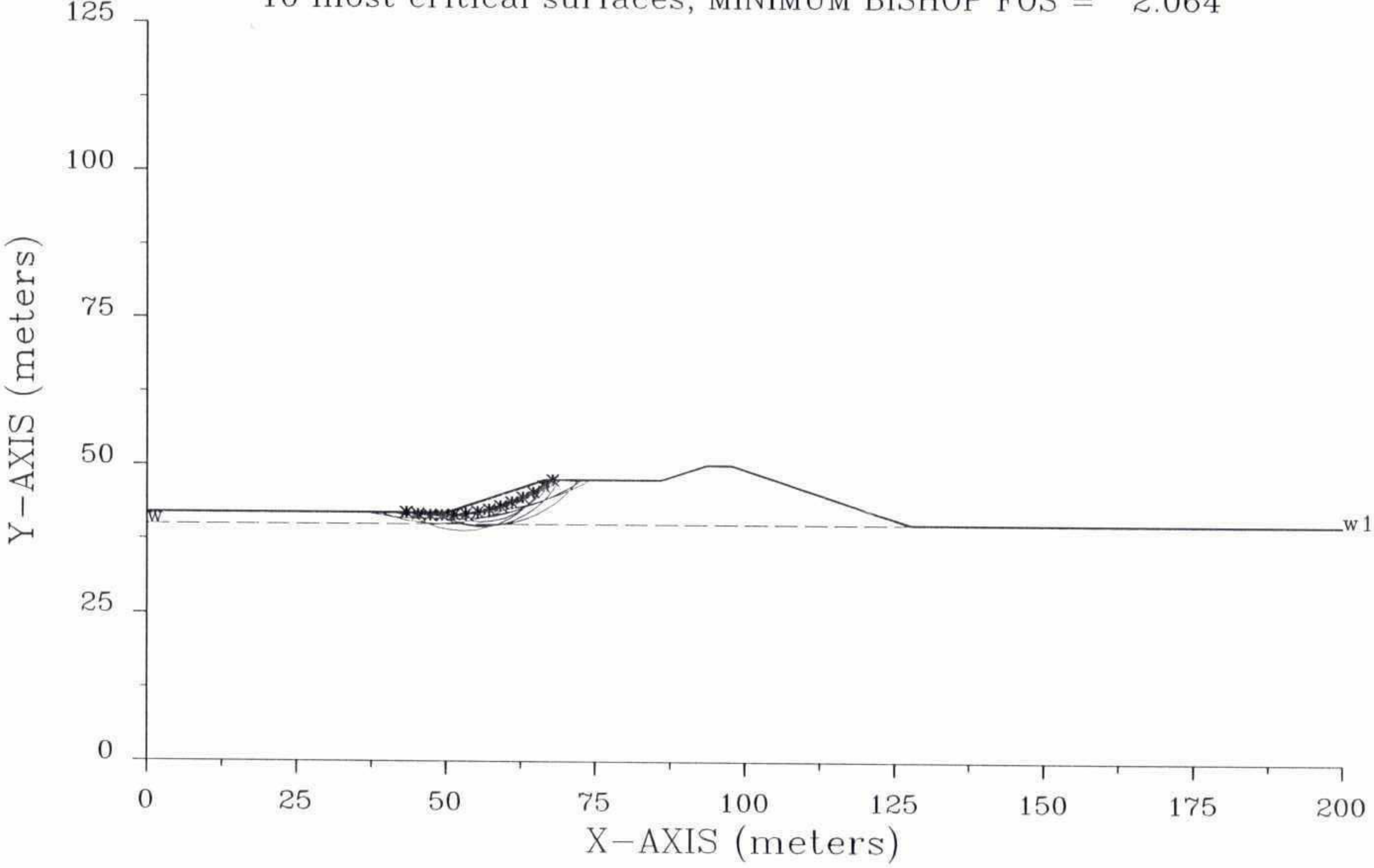
10 most critical surfaces, MINIMUM BISHOP FOS = 1.667



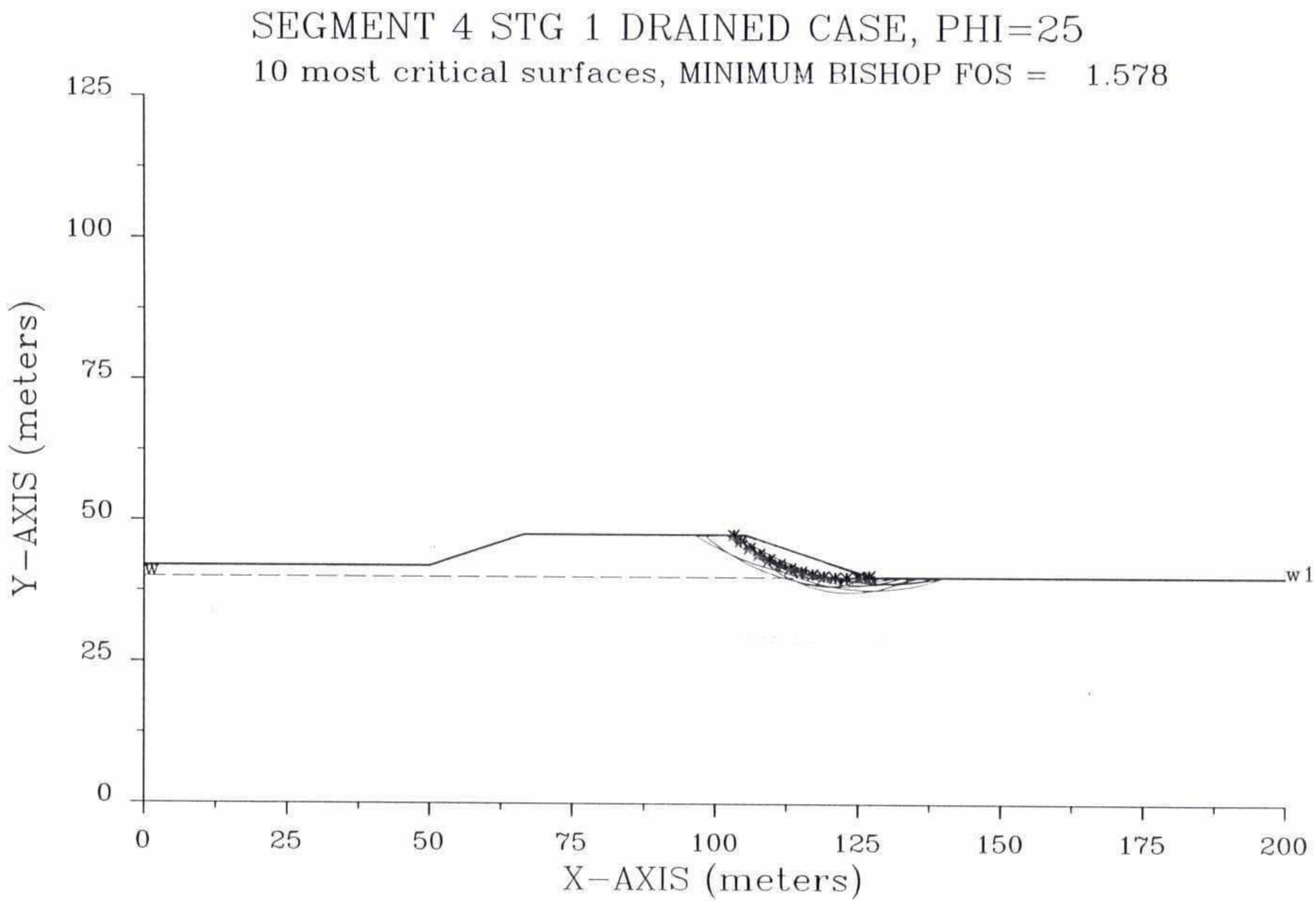
DCSDR30 8:46

FINAL EMBANKMENT DRAINED CASE PHI=30

10 most critical surfaces, MINIMUM BISHOP FOS = 2.064



DRX41D25 9:22

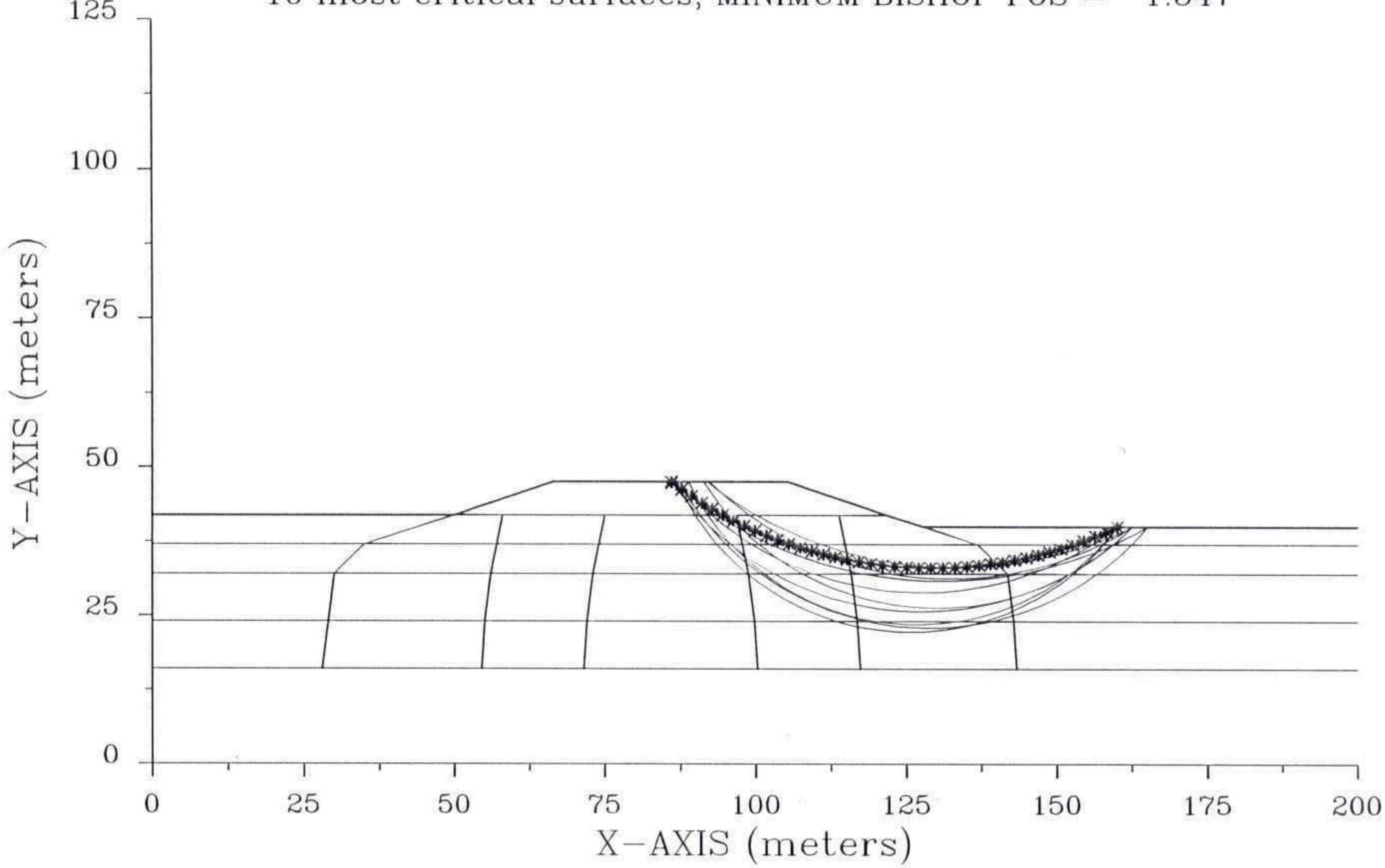


Jed

DRX41U2D 18:31

SEGMENT 4 STAGE 1, U=20

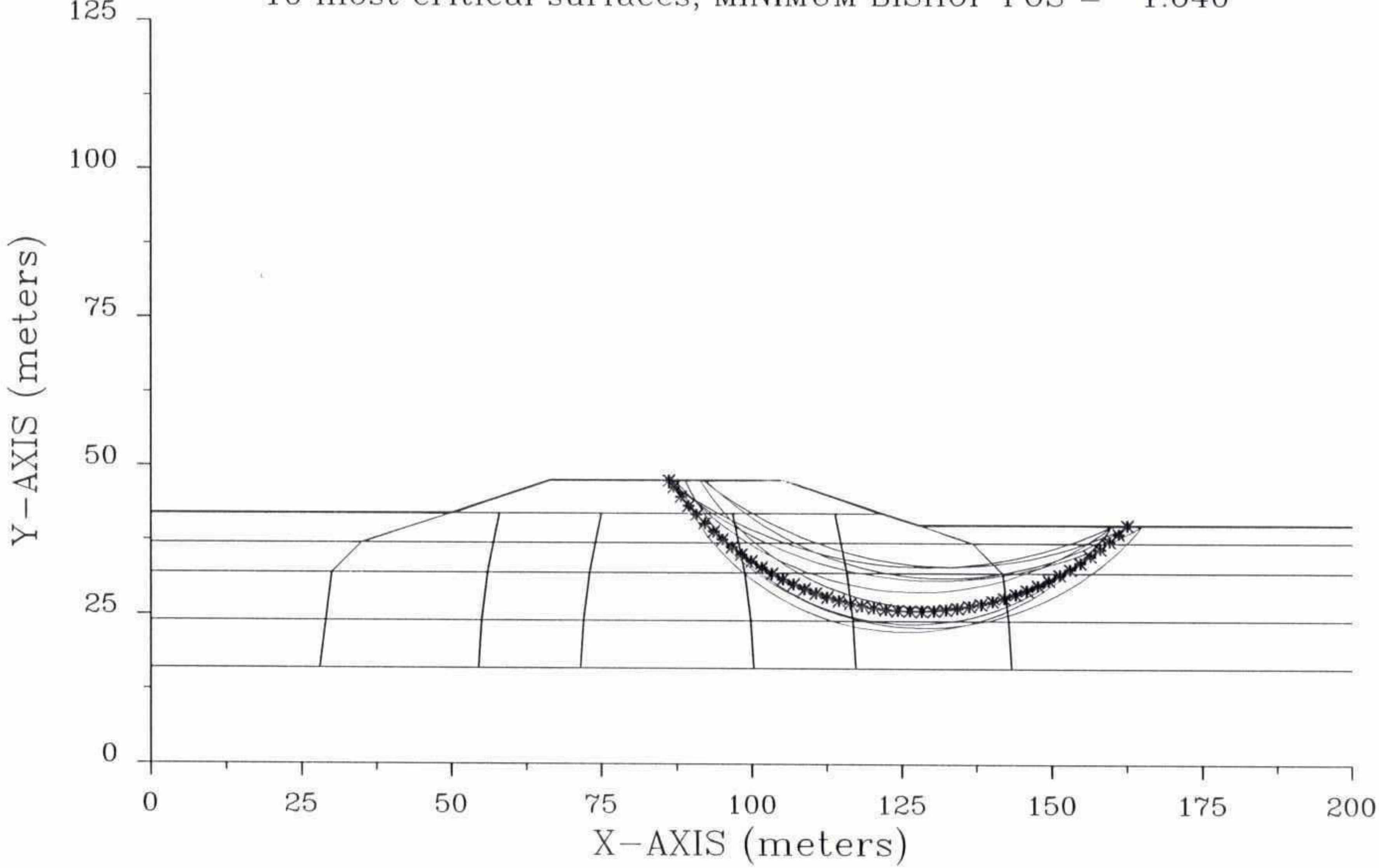
10 most critical surfaces, MINIMUM BISHOP FOS = 1.547



DRX41U5D 18:34

SEGMENT 4 STAGE 1, U=50

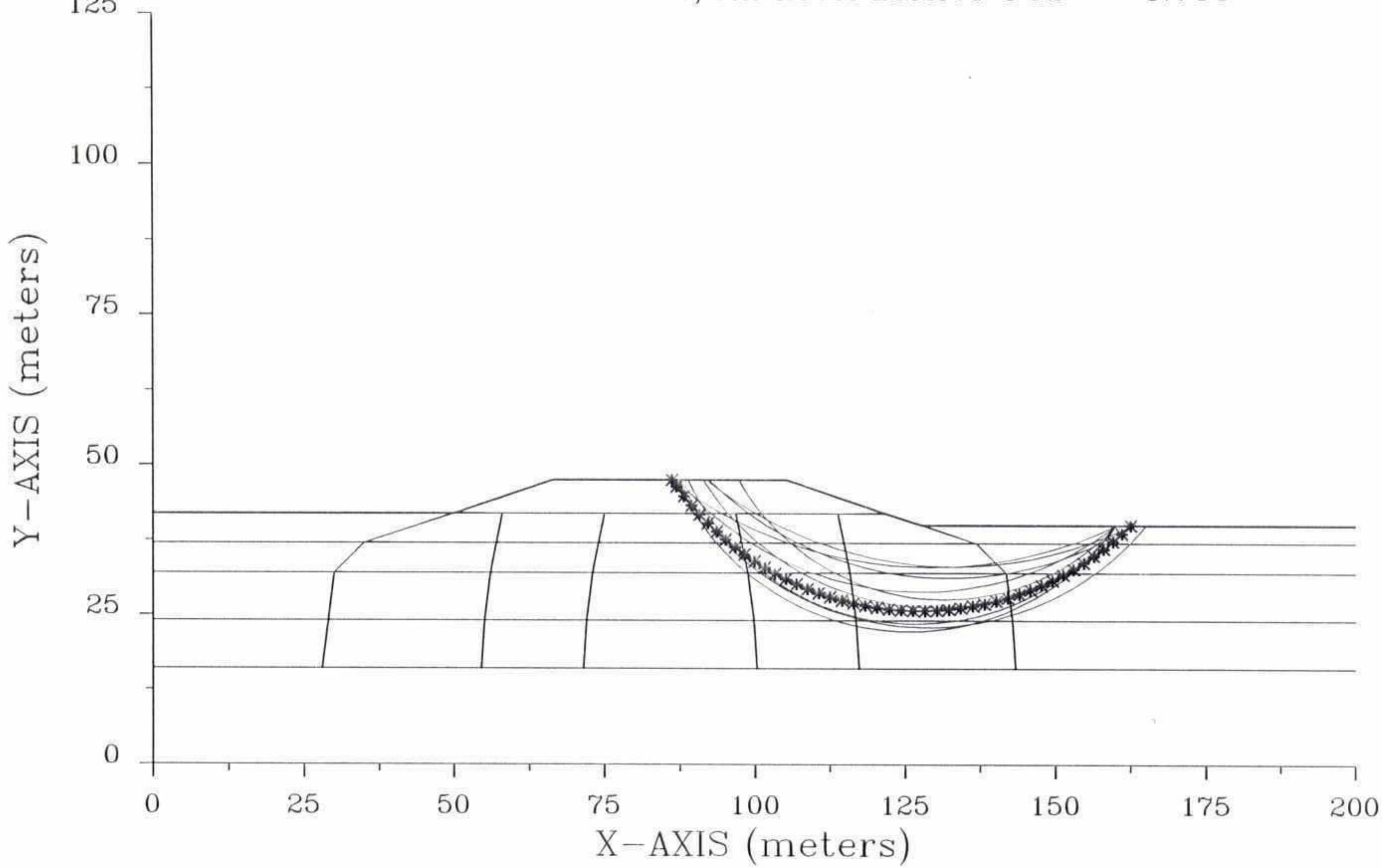
10 most critical surfaces, MINIMUM BISHOP FOS = 1.640



DRX41U9D 18:35

SEGMENT 4 STAGE 1, U=90

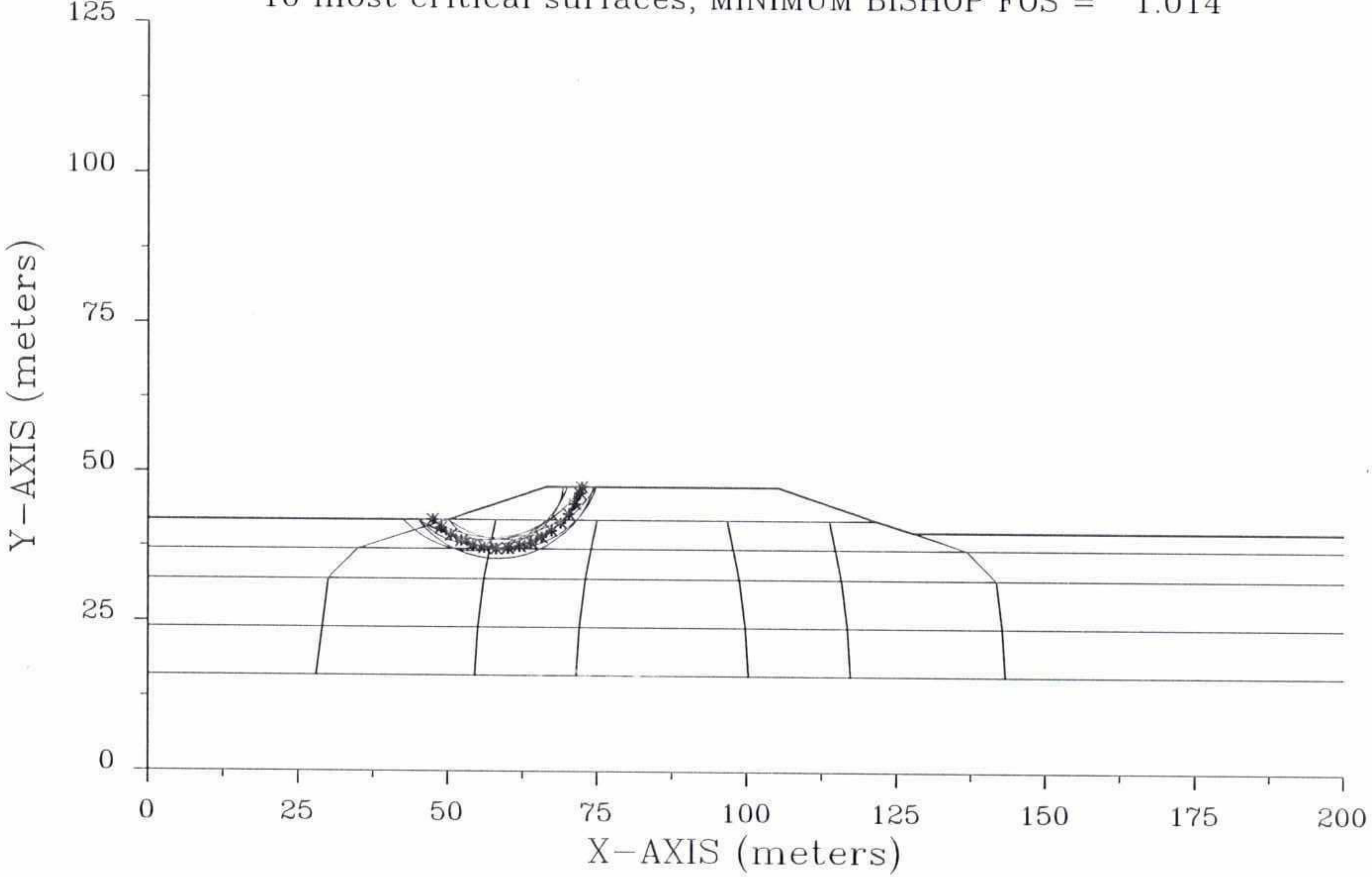
10 most critical surfaces, MINIMUM BISHOP FOS = 1.760



DCX41U0S 18:49

SEGMENT 4 STAGE 1, U=0

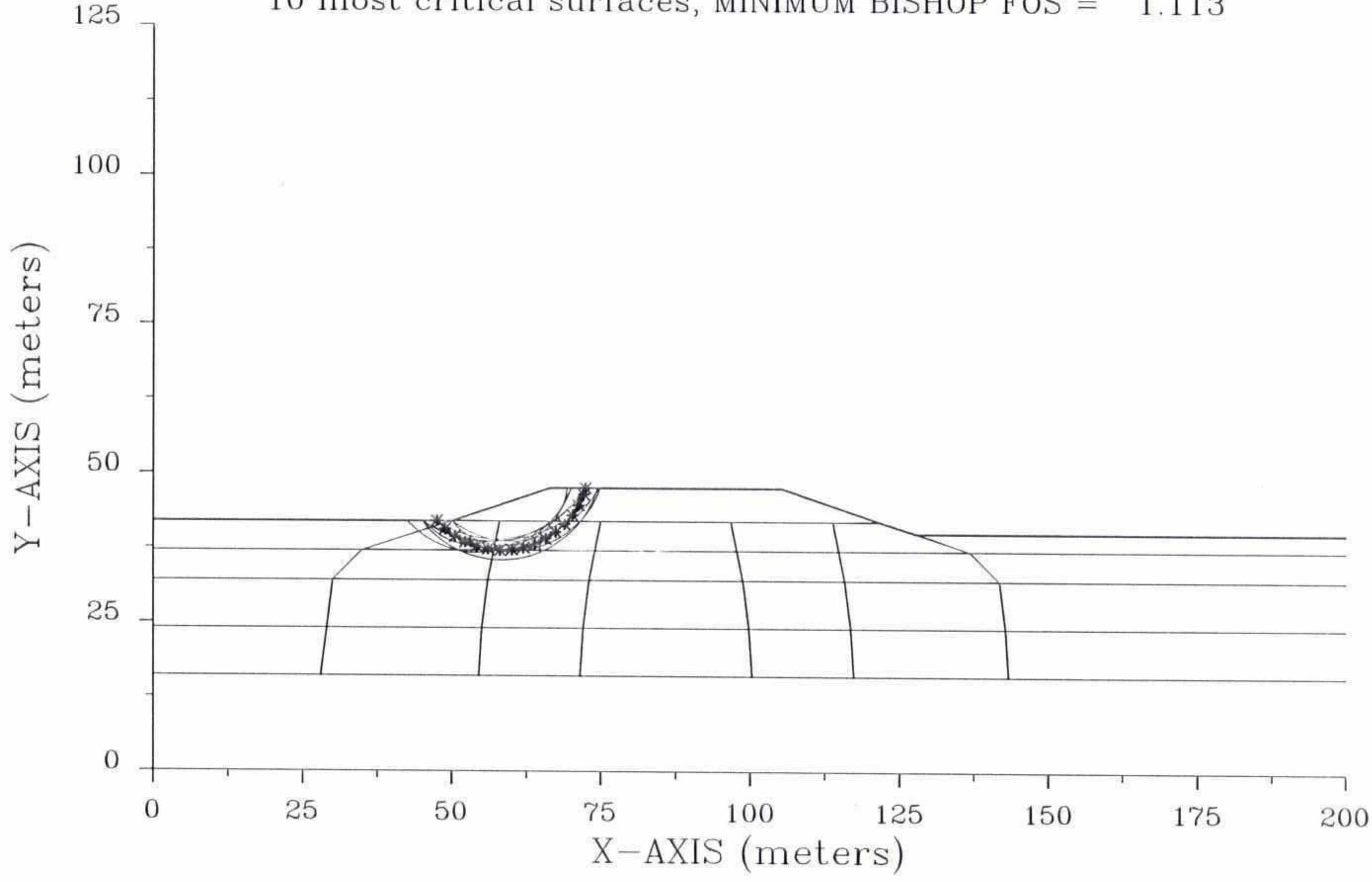
10 most critical surfaces, MINIMUM BISHOP FOS = 1.014



DCX41U2S 18:45

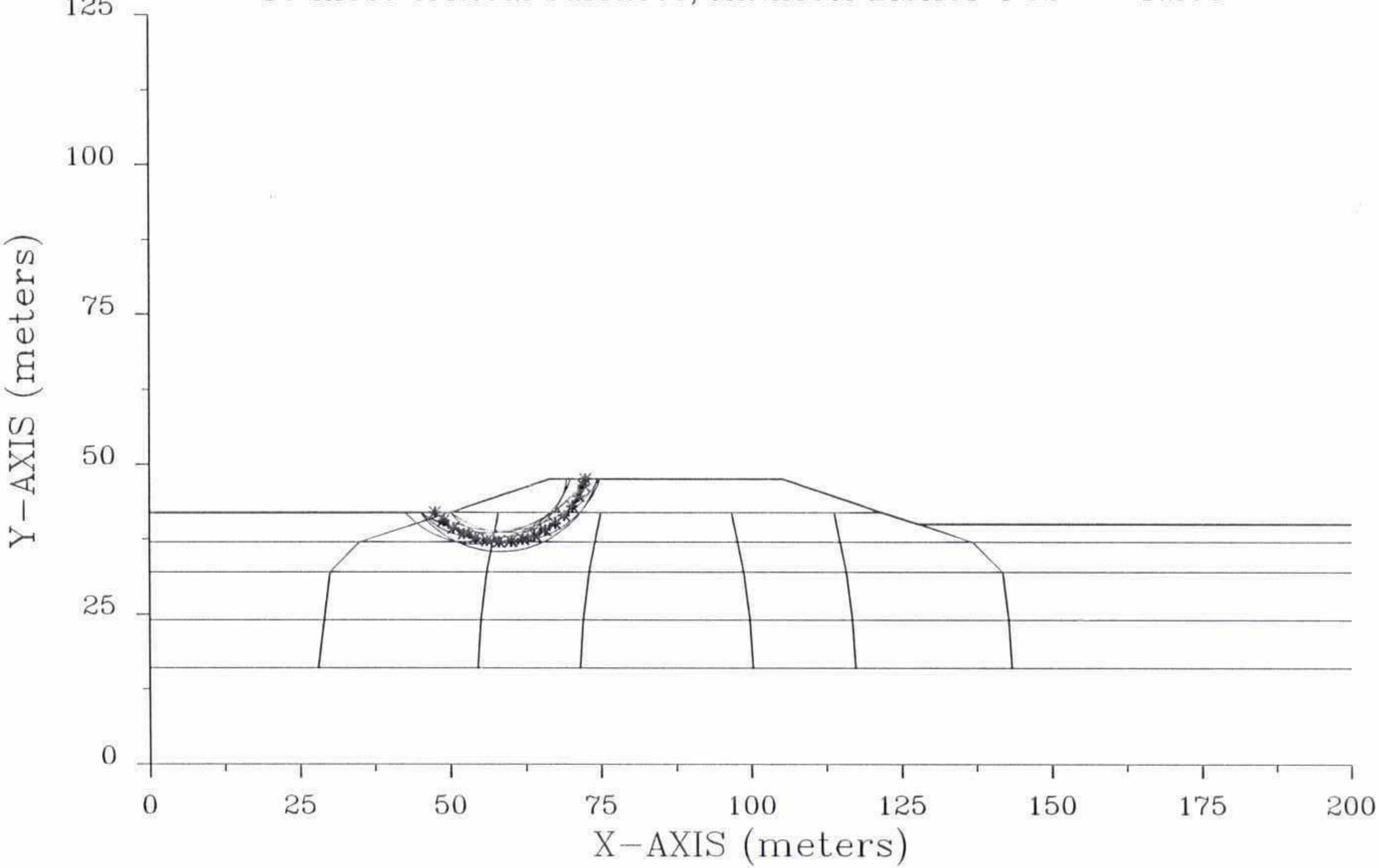
SEGMENT 4 STAGE 1, U=20

10 most critical surfaces, MINIMUM BISHOP FOS = 1.113



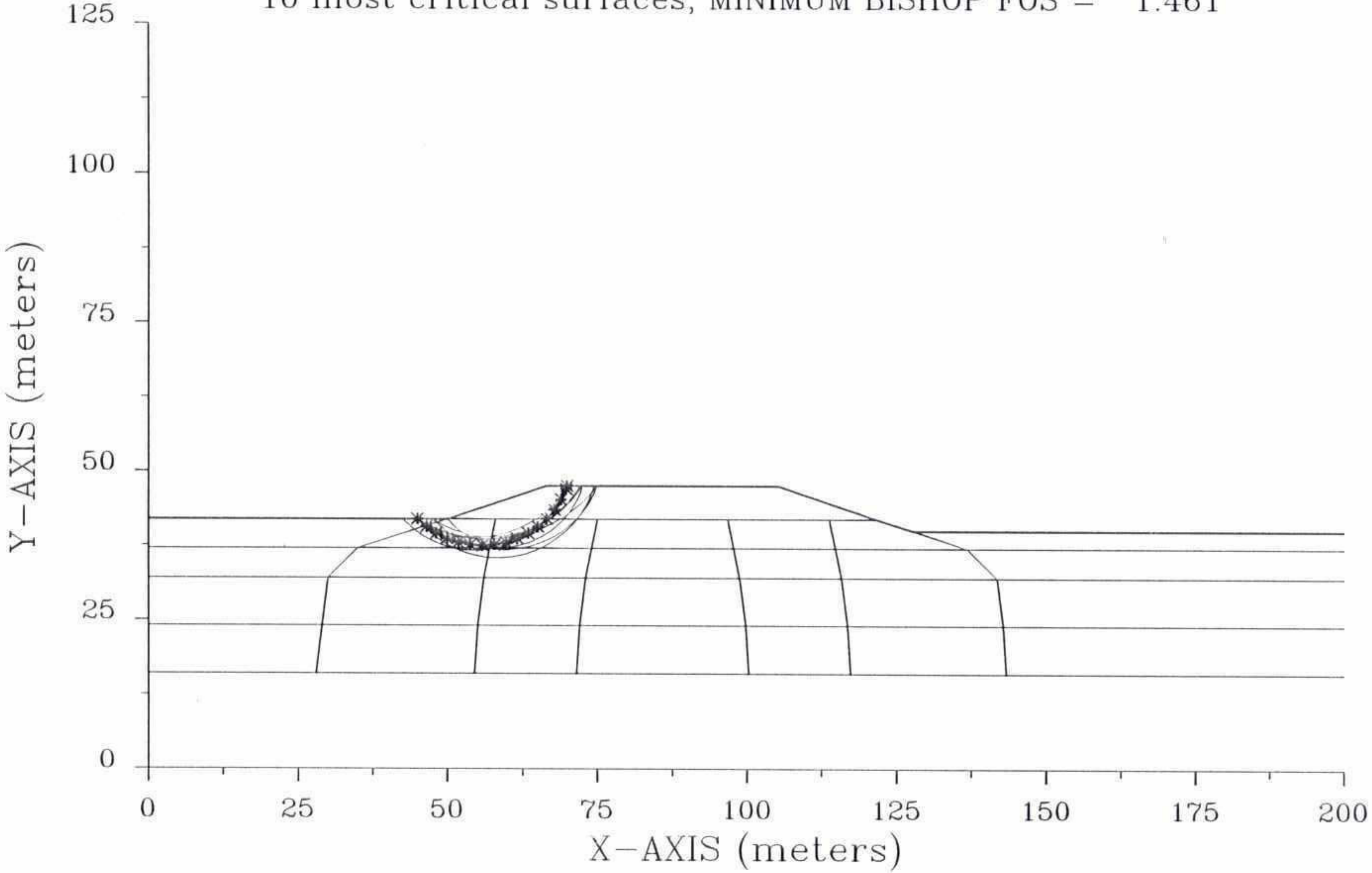
SEGMENT 4 STAGE 1, U=50

10 most critical surfaces, MINIMUM BISHOP FOS = 1.268



SEGMENT 4 STAGE 1, U=90

10 most critical surfaces, MINIMUM BISHOP FOS = 1.461

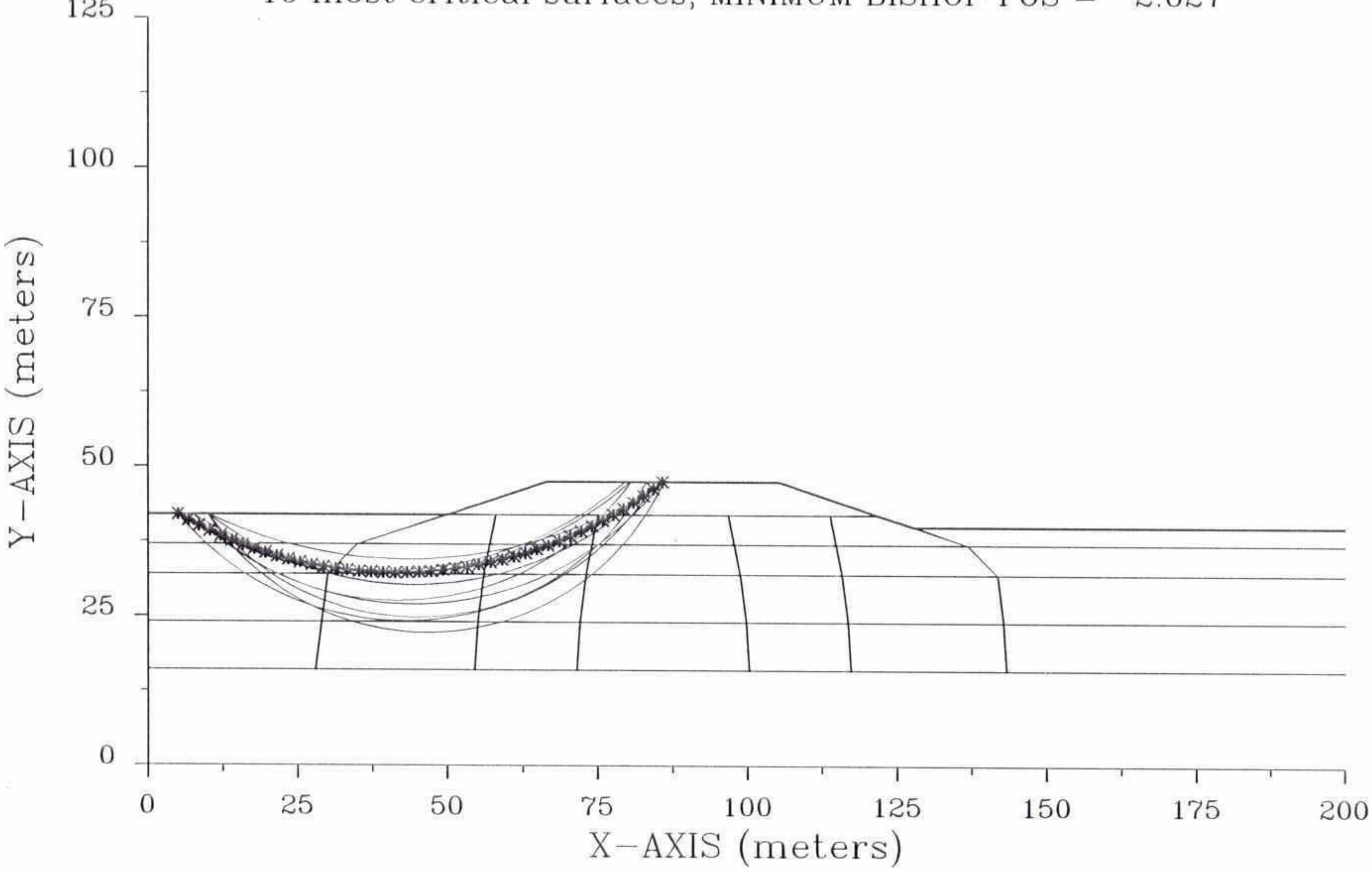


200

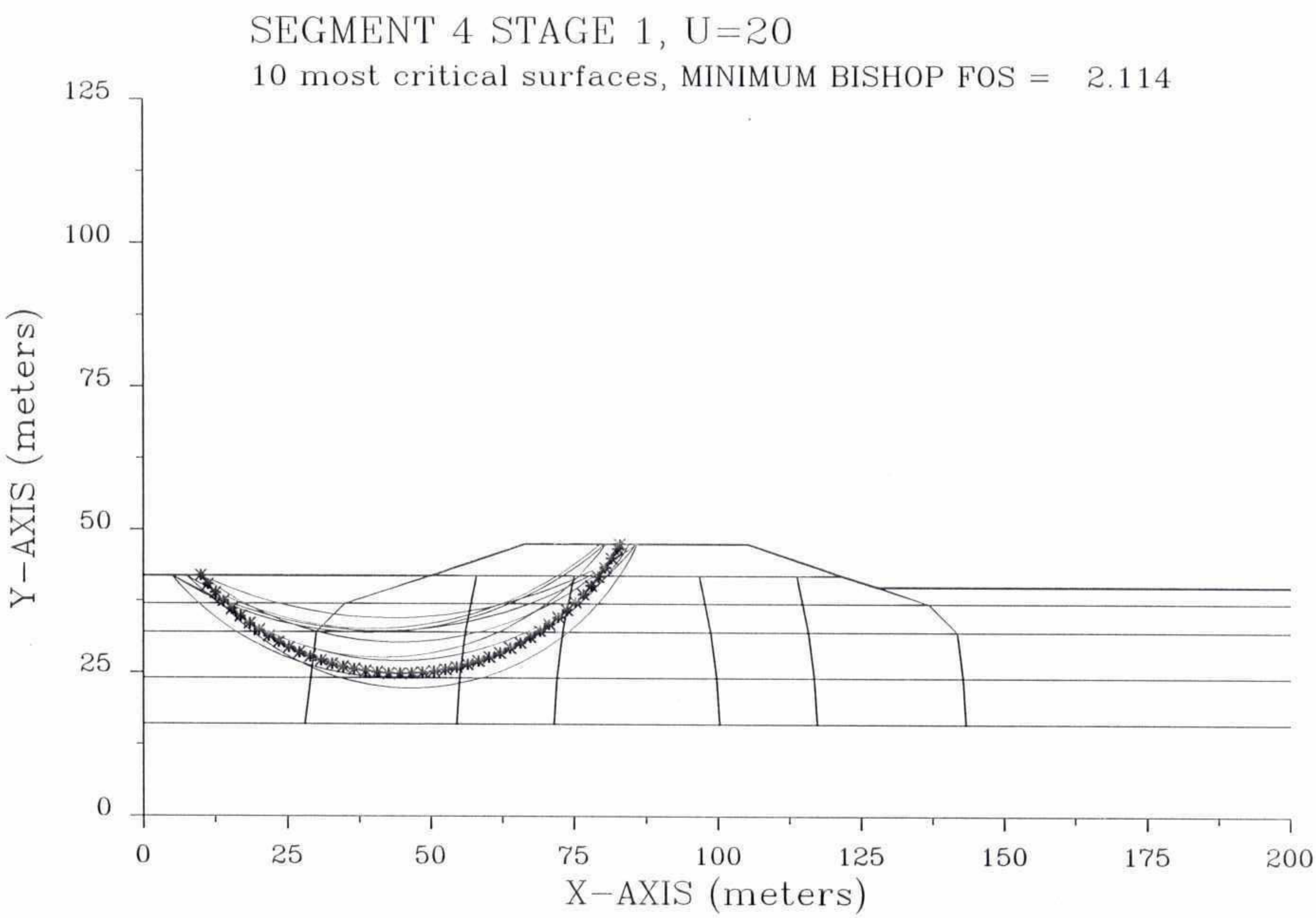
DCX41U0D 10:24

SEGMENT 4 STAGE 1, U=0

10 most critical surfaces, MINIMUM BISHOP FOS = 2.027



DCX41U2D 18:43

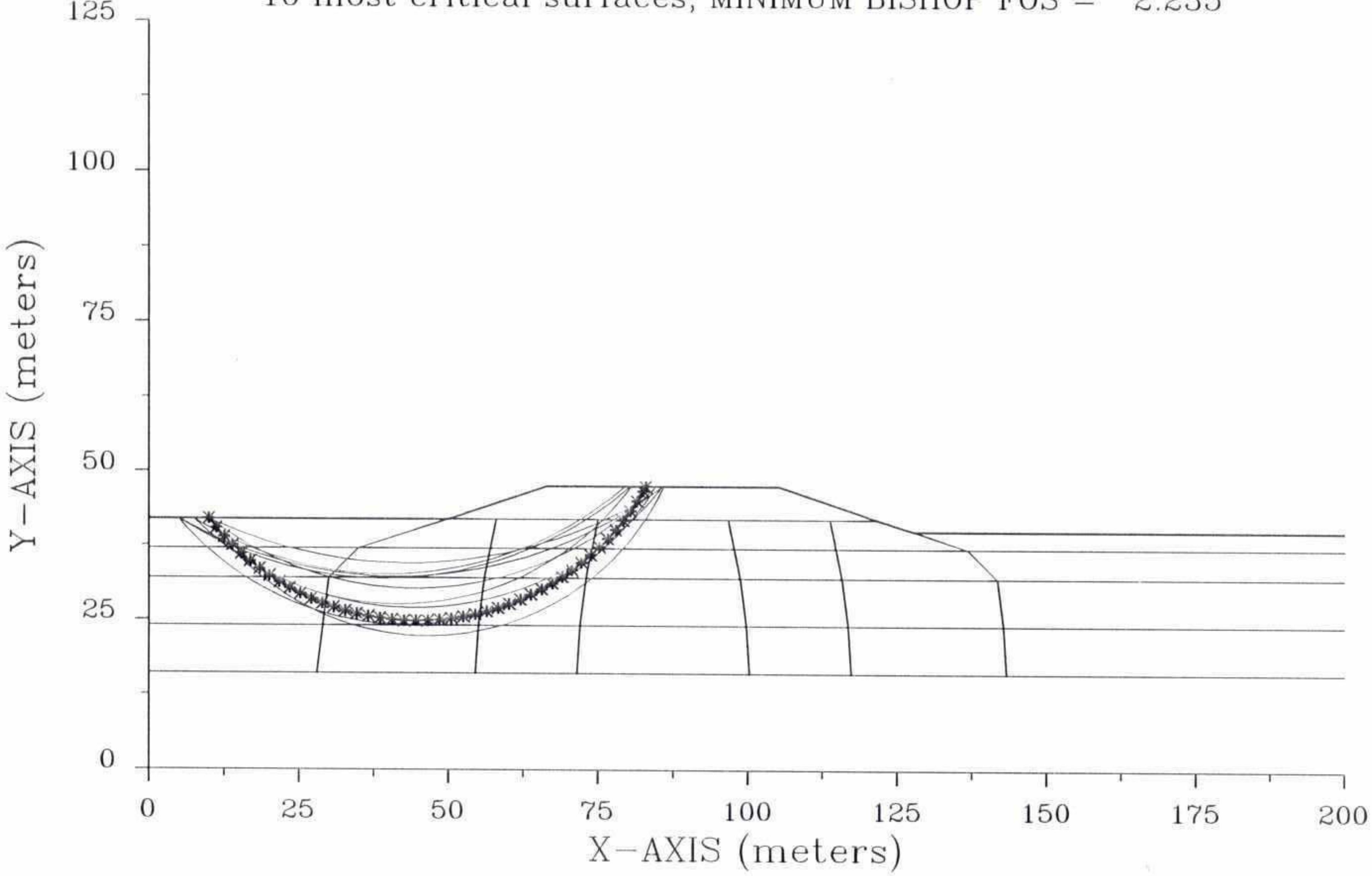


202

DCX41U5D 18:41

SEGMENT 4 STAGE 1, U=50

10 most critical surfaces, MINIMUM BISHOP FOS = 2.235



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APPENDIX D-1

ENGINEERING DRAWINGS

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APPENDIX D-2

CONSTRUCTION SPECIFICATIONS

PRELIMINARY SPECIFICATION

EARTHWORK

SECTION 02200

PART 1-GENERAL

1.01 SCOPE OF WORK

The Contractor shall furnish all labor, materials, tools, supervision, transportation, and installation equipment necessary to perform all excavation, backfill, and grading required to complete the work shown on the Construction Drawings and specified herein. The work shall include, but not necessarily be limited to: clearing and grubbing of vegetation from the slope of the embankment; removal and stock piling of topsoil or other existing unsatisfactory material; excavation of borrow material from on-site and off-site borrow pits; bench excavation for strip drain installation; preparation of existing surfaces to receive fills; placement and compaction of fill for the reconstructed embankment, in conjunction with the installation of geosynthetic reinforcement layers; disposal of surplus material; and all related work such as slope protection.

1.02 RELATED SECTIONS

Section 02300 - Vertical Strip Drain Installation

Section 02400 - Geosynthetic Reinforcement

1.03 PAYMENTS

A. No additional payment shall be made for losses due to settlement, compaction, erosion, or replacement of rejected material.

B. No additional payment shall be made for dewatering and slope protection that may be necessary as a result of the Contractor's construction procedures.

1.04 REFERENCES

A. Remedial Design Report, Dhaka Integrated Flood Protection Project, Dhaka, Bangladesh, July 1992.

B. Construction Quality Assurance Plan for the Western Embankment Remedial Design.

C. Latest version of American Society for Testing and Materials (ASTM) standards:
   1. ASTM D 422. Standard Method for Particle-Size Analysis of Soils.
   2. ASTM D 698. Standard Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using a 5.5-lb (2.49-kg) Rammer and 12-in. (305-mm) Drop.
   3. ASTM D 1556. Standard Test Method for Density of Soil In Place by the Sand-Cone Method.
   4. ASTM D 2216. Standard Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures.
   5. ASTM D 2487. Standard Test Method for Classification of Soils for Engineering Purposes.

6. ASTM D 2922. Standard Test Method for Density of Soil and Soil-Aggregate In Place by Nuclear Density Methods (Shallow Depth).
7. ASTM D 3017. Standard Test Method for Water Content of Soil and Rock In Place by Nuclear Methods (Shallow Depth).
8. ASTM D 4220. Standard Practices for Preserving and Transporting Soil Samples.
9. ASTM D 4318. Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils.

1.05 DEFINITIONS

A. *Owner*. The party that has hired the Contractor to execute the work under the terms of the contract. The activities of the Owner in the contract specifications may be performed by the Owner or other party representing the Owner such as, but not limited to, the Engineer or CQA Consultant.

B. *Contractor*. The individual, firm, or corporation undertaking the execution of the work under the terms of the contract. The Contractor may elect to use Subcontractors, however, the Contractor is responsible for the completion of all work.

C. *Engineer*. The party representing the Owner and having direct supervision of the execution of the contract.

D. *CQA Consultant*. The party, independent from the Owner, Contractor, or Manufacturer that is responsible for observing and documenting construction activities under the terms of the contract. The CQA Consultant is responsible for issuing a CQA report, sealed by a Registered or Licensed Professional Engineer approved by the Owner.

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1.06 SUBMITTALS

A. The Contractor shall submit to the Owner for review the proposed methods of construction, including stripping, filling, compaction, and backfilling. The review shall be for method only. The Contractor shall remain responsible for the adequacy and safety of the methods.

B. For the soil type specified in Part 2 of this Section, the Contractor shall submit to the Owner the following information and sample fourteen (14) days prior to starting construction:
   1. The proposed material source and location of borrow pits.
   2. The results of grain-size analyses conducted on the proposed material in accordance with ASTM D 422.
   3. The results of liquid and plastic limit tests conducted on the proposed material in accordance with ASTM D 4318.

C. If work is interrupted for reasons other than inclement weather, the Contractor shall notify the Owner twenty-four (24) hours prior to the resumption of work.

D. The Contractor shall submit to the Owner the results of a grain-size analysis and a liquid and plastic limits test for every 2,000 cubic yards of each soil type brought to the site. The tests shall be conducted in accordance with the methods defined in Part 1.06B of this Section. Any material that does not conform to these Specifications shall be rejected and replaced.

1.07 CONSTRUCTION QUALITY ASSURANCE

A. The compacted fill shall be constructed in accordance with the requirements of this Specification and the design drawings.

B. The construction of the compacted fill shall be monitored as outlined in the CQA Plan.

C. The Contractor shall be aware of the activities outlined in the CQA Plan and account for these CQA activities in the construction schedule.

PART 2-EMBANKMENT FILL MATERIAL

A. The fill material for the embankment reconstruction shall be on-site and off-site borrow soils.

B. The fill for the embankment shall consist of relatively homogeneous, natural soils that are free of debris, foreign objects, roots, and organics. No material larger than 1 in. (25 mm) shall be allowed. The embankment fill shall be classified according to the USCS as a SM, SC, or CL material. The fill material shall have a maximum liquid limit of 50.

PART 3-EXECUTION

3.01 FAMILIARIZATION

A. The Contractor shall notify the Owner in writing at least seven (7) days in advance of intention to perform the work of this Section.

B. Prior to submission of the bid to perform any work of this Section, the Contractor shall become thoroughly familiar with the site, the site conditions, and all portions of the work falling within this Section.

C. If the Contractor has any concerns regarding the site, he should notify the Owner in writing within forty-eight (48) hours of the site visit. Failure to notify the Owner will be construed as Contractor's acceptance of the related work of all other Sections.

3.02 FIELD QUALITY CONTROL

A. The frequency of quality control testing is outlined below. The quality control testing shall be performed by the CQA Consultant or an approved independent testing laboratory. The Contractor shall take this testing frequency into account in planning his construction schedule.
   1. Routine testing frequencies for material evaluation and construction quality evaluation are:
      a. Natural Moisture Content (ASTM D 2216) at one (1) per 1,000 yd^3 (minimum one (1) per source).
      b. Particle-Size Analysis (ASTM D 422) at one (1) per 2,000 yd^3 (minimum one (1) per source).
      c. Liquid and Plastic Limits (ASTM D 4318) at one (1) per 2,000 yd^3 (minimum one (1) per source) for soils with 5 percent or more particles passing the No. 200 sieve.
      d. Standard Proctor (ASTM D 698) at one (1) per 2,000 yd^3 (minimum one (1) per source).
   2. The following quality control testing shall apply to installed material:
      a. Moisture Content (ASTM D 3017) at one (1) per 200 yd^3. This test shall be calibrated by performing one (1) Laboratory Determination of Water (Moisture) Content (ASTM D 2216) per ten (10) moisture content (ASTM D 3017) tests.
      b. In Situ Dry Density (ASTM D 2922) at one (1) per 200 yd^3. This test shall be calibrated by performing one (1) sand cone density test (ASTM D 1556) per ten (10) Nuclear Density (ASTM D 2922) tests.

B. The location of routine in-place moisture content and dry density test shall be determined using a non-biased sampling plan.

C. A special testing frequency shall be used at the discretion of the Owner and/or his Engineer when visual observations of construction performance indicate a potential problem.

D. If a defective area is discovered in the fill, the Owner shall immediately determine the extent and nature of the defect. If the defect is indicated by an unsatisfactory test result, the Owner shall determine the extent of the defective area by additional tests, observations, a review of records, or other means that the Owner deems appropriate. If the defect is related to adverse site conditions, such as overly wet soils or surface desiccation, the Owner shall define the limits and nature of the defect.

E. After determining the extent and nature of a defect, the Contractor shall correct the deficiency to the satisfaction of the Owner. The cost of corrective actions shall be borne by the Contractor.

F. Additional testing shall be performed to verify that the defect has been corrected before any additional work is performed by the Contractor in the area of the deficiency.

3.03 STRIPPING AND STOCKPILING TOPSOIL

A. Stripping and stockpiling will be required to obtain topsoil for reclamation of the vegetative cover and construction of the top-soil for the reconstructed embankment. Clearing and grubbing of vegetation, and removal of highly organic soils will be required prior to placement of fill material in order to provide a firm subgrade to facilitate compaction of the fill.

B. Following clearing and grubbing, the upper 6 in. (150 mm) of topsoil shall be removed from within the footprint of the area where fill material is to be placed, and stockpiled at an on-site location approved by the Owner.

C. Once the top 6 in. (150 mm) of material is removed, additional undercutting may be required to clear highly organic soil and soil containing a significant amount of roots or other unsuitable materials.

D. The topsoil stockpile shall be maintained in a neat condition with a maximum 10 ft (3 m) height and maximum 3H:1V side slopes.

3.04 EMBANKMENT FILL

A. Compacted fill material shall be placed to the lines and grades shown on the Construction Drawings.

B. The fill material shall meet the requirements of Part 2 of this Section.

C. Do not place any fill or geosynthetic material until the subgrade has been evaluated and approved by the Owner's representative.

D. During the dumping and spreading of fill material, remove all roots, stones, and debris that are uncovered or observed.

E. Maintain the moisture content of the soil as close as practical to the optimum to facilitate getting the required density. If the moisture content is too high, spread the material and permit it to dry. Assist the drying process by disking or harrowing if necessary. If the moisture content is too low, sprinkle each layer uniformly with water. Work the moisture into the soil by disking, harrowing, or other approved method.

F. Fill material shall be placed in loose lifts that result in compacted lift thickness of not more than 1 ft (0.3 m). Lifts shall be properly benched into the existing slope, as directed by the Engineer.

G. Each lift shall be compacted to at least 90 percent of the maximum dry density and at a moisture content of ± 2 percent of the optimum moisture content, as measured according to ASTM D 698. The dry density and moisture content shall be measured in accordance with the quality control testing, as specified in Section 3.02B.

H. The Geosynthetic Reinforcement materials shall be placed between the compacted soil layers, as shown on the construction plans.

I. Topsoil and Hydroseed shall be placed on the locations shown on the construction plans. Final slope of the hydroseed shall be maintained at 2H:1V.

3.05 PUMPING AND DRAINAGE

A. At all times during construction, the Contractor shall provide and maintain proper equipment and facilities to remove all water entering the work area and keep it dry so as to obtain a satisfactory subgrade to allow the construction of the compacted fill and installation of the geosynthetic reinforcement material.

B. The Contractor shall maintain proper grades to drain surface water away from the work area. Water entering the work area from surface runoff shall be collected in shallow ditches around the perimeter of the work area and drained to drainage swales in order to maintain the work area free from standing water.

C. Drainage shall be disposed of only in an area approved by the Owner. Drainage shall be disposed of in a manner which prevents flow or seepage back into the work area.

D. The Contractor shall install silt fences around all areas downslope of soil disturbance. Other areas requiring silt fences shall be identified by the Engineer during construction. Silt fences shall not be removed until the contained areas are successfully revegetated.

[END OF SECTION]

PRELIMINARY SPECIFICATION

VERTICAL STRIP DRAIN INSTALLATION

SECTION 02300

PART 1-GENERAL

1.01 SCOPE OF WORK

The Contractor shall furnish all labor, materials, tools, supervision, transportation, and installation equipment necessary to perform all vertical strip drain installation required to complete the work shown on the Construction Drawings and specified herein.

1.02 RELATED SECTIONS

Section _____ - Geotextile

Section 02200 - Earthwork

Section 02400 - Geosynthetic Reinforcement

1.03 PAYMENTS

A. Payment shall be made based on the total vertical footage of vertical strip drain required as shown on the Construction Drawings.

B. No additional payment shall be made for replacement of rejected material.

1.04 REFERENCES

A. Remedial Design Report, Dhaka Integrated Flood Protection Project, Dhaka, Bangladesh, July 1992.

B. Latest version of American Society for Testing and Materials (ASTM) standards:
1. D 422. Standard Method for Particle-Size Analysis of Soils.
2. D 4751-87. Apparent Opening Size of a Geotextile, Determining.
3. D 4632-86 (1990). Breaking Load and Elongation of Geotextiles (Grab Method).
4. D 4716-87. Constant Head Hydraulic Transmissivity (In-Plane Flow) of Geotextiles and Geotextile Related Products.
5. D 4355-84. Deterioration of Geotextiles from Exposure to Ultraviolet Light and Water (Xenon-Arc Type Apparatus).
6. D 4594-86. Effects of Temperature on Stability of Geotextiles.
7. D 4833-88. Index Puncture Resistance of Geotextiles, Geomembranes, and Related Products.
8. D 4595-86. Tensile Properties of Geotextiles by the Wide Width Strip Method.
9. D 4533-85 (1990). Trapezoid Tearing Strength of Geotextiles.
10. D 4491-89. Water Permeability of Geotextiles by the Permittivity Method.
11. D 4354-89. Sampling of Geosynthetics for Testing.
12. D 4759-88. Specification Conformance of Geosynthetics, Determining.
13. D 4873-88. Identification, Storage, and Handling of Geotextile.
14. D 4439-87. Geosynthetics.
15. D 1785-89. Poly Vinyl Chloride (PVC) Plastic Pipe, Schedules 40, 80, and 120.
16. D 2855-90. Making Solvent Cemented Joints with Poly Vinyl Chloride (PVC) Pipe and Fittings.

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17. F 758-90. Smooth-Wall Poly Vinyl Chloride (PVC) Plastic Underdrain Systems for Highways, Airport, and Similar Drainage.

1.05 DEFINITIONS

A. *Owner*. The party that has hired the Contractor to execute the work under the terms of the contract. The activities of the Owner in the contract specifications may be performed by the Owner or other party representing the Owner such as, but not limited to, the Engineer or CQA Consultant.

B. *Contractor*. The individual, firm, or corporation undertaking the execution of the work under the terms of the contract. The Contractor may elect to use Subcontractors, however, the Contractor is responsible for the completion of all work.

C. *Engineer*. The party representing the Owner and having direct supervision of the execution of the contract.

D. *CQA Consultant*. The party, independent from the Owner, Contractor, or Manufacturer that is responsible for observing and documenting construction activities under the terms of the contract. The CQA Consultant is responsible for issuing a CQA report, sealed by a Registered or Licensed Professional Engineer registered in the State of Ohio.

1.06 SUBMITTALS

A. The Contractor shall submit to the Owner for review the proposed method of vertical strip drain installation. The review shall be for method only. The Contractor shall remain responsible for the adequacy and safety of the method.

B. The Contractor shall submit to the Owner at least 14 days before starting strip drain installation, the results of quality control testing carried out on the vertical strip drain material. The testing shall include, as a minimum, the test results for the parameters listed in Section 2.01. The Contractor shall submit certification that the vertical strip drain material meets the specification shown in Section 2.01.

C. If work is interrupted for reasons other than inclement weather, the Contractor shall notify the Owner twenty-four (24) hours prior to resumption of the work.

1.07 CONSTRUCTION QUALITY ASSURANCE

A. The vertical strip drains shall be installed in accordance with the requirements of this Specification.

B. The materials and installation shall be monitored by the Owner to confirm satisfaction of the requirements of this Specification.

C. The Contractor shall be aware of the monitoring activities and account for these activities in the installation schedule.

PART 2-VERTICAL STRIP DRAIN MATERIAL

2.01 MATERIAL PROPERTIES

The prefabricated wick drain material shall consist of a continuous plastic drainage core wrapped in a nonwoven geotextile material. The geotextile wrap shall be tight around the core, and shall be securely seamed in a manner that will not introduce any new materials nor present an obstruction that will impede flow in the channels of the core. The prefabricated vertical strip drain material used shall meet the following specifications:

Item	Units	Value
Drain Body		100% Polypropylene
Filter Jacket		100% Polypropylene
Weight (including filter jacket)	gm/m	93
Width (including filter jacket)	mm	100
Thickness (including filter jacket)	mm	3
Water Discharge Capacity	m^3/sec	1.2×10^{-5}
Water Permeability (k)	m/sec	6.5×10^{-4}
Free Volume	mm^3/mm	180
Tensile Strength (Filter)	kN	1.3

The Contractor shall submit a 5-ft sample of the vertical drain material to the Engineer prior to usage and shall allow three weeks for the Engineer to evaluate the material. The sample shall be stamped or labeled by the manufacturer as being representative of the drain material having the specified trade name. Approval of the sample material by the Engineer shall be required prior to site delivery of the wick drain material.

The Contractor shall state which wick drain product he intends to install at the time of the preconstruction conference. The drains shall be free of defects, rips, holes, or flaws. During shipment, the drain shall be protected from damage, and during storage on-site, the storage shall be such that the drain is protected from sunlight, mud, dirt, dust, debris, and detrimental substances. Manufacturer certification shall be provided for all drain material delivered to the project.

The perforated poly vinyl chloride (PVC) pipe shall meet the following specifications:

Item	Units	Value
Diameter	mm	100
Thickness		Schedule 40
Holes	mm	4.8 to 9.7
Hole Spacing (2 rows)	mm	82.6 ± 6.4

The Contractor shall provide samples of the perforated PVC pipe, geotextile filter, and sand to the Engineer. The Contractor shall allow at least 3 weeks for the Engineer to test and approve the materials.

PART 3-EXECUTION

3.01 CONSTRUCTION

Where shown on the plans, or as directed by the Engineer, vertical drains shall be installed subsequent to construction of temporary berms or excavations to create a relatively level, stable platform on which to install the vertical strip drains. Drain locations shall be staked prior to installation of the drains. The Contractor shall take all reasonable precautions to preserve the survey stakes.

The Contractor shall demonstrate that his equipment, methods, and materials produce a satisfactory installation in accordance with these specifications. For this purpose, the Contractor will be required to install several trial drains at locations within the work area, as designated by the Engineer. Trial drains conforming to these specifications will be paid for at the same unit price as the production drains.

The vertical drains shall be installed in the locations shown on the plans, or as directed by the Engineer. Drains that deviate from the

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plan location by more than 6 in., or that are damaged, or improperly installed will be rejected. Rejected drains may be removed or abandoned in place, at the Contractor's option. Replacement drains shall be offset approximately 18 in. from the location of the rejected drain. All rejected drains will be replaced at the Contractor's expense.

Drains shall be installed vertically, within a tolerance of not more than 0.25 in. per ft. The equipment shall be carefully checked for plumbness, and the Contractor shall provide the Engineer with a suitable means of verifying the plumbness of the mandrel and of determining the depth of the drain at any time.

Splices or connections in the vertical drain material shall be done in a professional manner so as to ensure continuity and no diminishing of the flow characteristics of the wick material. Splices shall be a minimum of 6 in. in length. The prefabricated drain shall be cut as directed by the Engineer to maintain positive drainage after placement of the overlying fill.

It may be necessary to preauger or use some other method to clear obstructions and to facilitate the installation of the drains through the working platform or a stiffer natural deposit, above the compressible soil strata. The depth to which preaugering is used shall be subject to the approval of the Engineer, but should not extend more than 0.6 m (2 ft) into the underlying compressible soils.

Where obstructions are encountered within the compressible strata, which cannot be penetrated by augering or spudding, the Contractor shall abandon the hole. At the direction of the Engineer, the Contractor shall then install a new drain within 18 in. of the obstructed drain. A maximum of two attempts shall be made, as directed by the Engineer, for each obstructed drain. If the drain still cannot be installed to the design tip elevation, the drain location shall be abandoned and the installation equipment shall be moved to the next drain location. The Contractor shall be paid for such frustrated drains at the unit price bid per foot for production drains.

The vertical strip drains shall be connected to the perforated pipe and sand drainage layer as described in the engineering drawings. The perforated pipe, sand drainage layer, and geotextiles constitute the horizontal drainage layer and shall be paid for on a unit price basis per meter of drainage layer installed and approved by the Engineer.

In the sections where no horizontal drainage layers are installed, the Contractor shall provide additional vertical strip drain material for future horizontal connections. This additional strip drain material will be provided at a different unit rate from the installed vertical strip drain.

3.02 EQUIPMENT

Vertical drains shall be installed with equipment which will cause a minimum of disturbance to the embankment and subsoil during the installation. The prefabricated drains shall be installed using a mandrel or sieve that will be advanced through the compressible soils to the required depth using constant load, or constant rate of advancement methods, only. Use of vibratory or falling weight impact hammers will not be allowed. Jetting shall not be permitted for installation of the drain, except, with the approval of the Engineer, to lubricate the mandrel when working in highly plastic clays.

The mandrel shall protect the prefabricated drain material from tears, cuts, and abrasions during installation and shall be withdrawn after the installation of the drain. The drain shall be provided with an "anchor plate" or rod at the bottom, to anchor the drain at the required depth at the time of mandrel removal. The projected crosssectional area of the mandrel and anchor combination shall not be greater than 12 in^2.

At least three weeks prior to the installation of the vertical strip drains, the Contractor shall submit to the Engineer, for review and approval, details of the sequence and method of installation. The submittal shall, at a minimum, contain the following specific information:

1. Size, type, weight, maximum pushing force, and configuration of the installation rig;
2. Dimensions and length of mandrel;
3. Details of drain anchorage;
4. Detailed description of proposed installation procedures;
5. Proposed methods for overcoming obstructions; and
6. Proposed methods for splicing drains.

Approval by the Engineer will not relive the Contractor of his responsibilities to install drainage wicks in accordance with the plans and specifications. If, at any time, the Engineer considers that the method of installation does not produce a satisfactory drain, the Contractor shall alter his method and/or equipment as necessary to comply with the plans and specifications.

The Contractor shall also submit to the Engineer, at least 3 weeks prior to installation, quality control and conformance test results demonstrating that the vertical strip drain and perforated pipe materials meet the project specifications and the requirements of the engineering drawing.

[END OF SECTION]

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ATTACHMENT A

SUBSURFACE INVESTIGATION SERVICES

AND

FIELD INVESTIGATION PLAN

FOR

DHAKA INTEGRATED FLOOD PROTECTION PROJECT

DHAKA, BANGLADESH

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I. INTRODUCTION

A. Scope

The Agreement for Subsurface Investigation Services and Field Investigation Plan covers services described in this document in connection with a subsurface investigation for existing flood protection works in the Dhaka area. Investigations will be located along and across the alignment of the existing western embankment starting at Tongi Bridge and proceeding in a westerly and southerly direction.

The term "Owner" which is used in this document refers to the appropriate authority of the Dhaka Integrated Flood Protection Project. The term "Consultant" which is used in this document refers to the authorized personnel from Technoconsult International Limited (TCIL) and their associates, Associated Consulting Engineers Limited, Desh Upodesh Limited, and the individual consultants from Louis Berger International Incorporated.

B. Terms of Reference

Mr. Roger B. North, Consultant, has prepared Annexure I, Subsurface Investigation Services and Field Investigation Plan, for the implementation of the field and laboratory testing components of the design of the Phase I Embankment of the Dhaka Integrated Flood Protection Project (DIFPP). The document has been reviewed by Mr. Robert D. Berlin, Consultant. Its contents, inclusive of all the terms and conditions have been discussed with and accepted by the Contractor.

C. Intent

Several contractors will be simultaneously involved in undertaking the field and laboratory testing works described herein. It is therefore essential that minimum and uniform quality standards, methods and report documents are used by all contractors. Sophisticated testing and analyses will be performed as part of this project and it is of critical importance to obtain high quality samples from the field program, and high quality results from the laboratory testing.

It is the intent of this document to provide a framework for the work to be completed in a consistent manner. The Contractor should understand and be aware of the contents of this document and should ensure that involved personnel implement appropriate sections. All work will conform to the standards of the American Society for Testing and Materials (ASTM) except as described differently herein or as amended by the Consultant.

D. Objectives

Soft soils are present along the embankment alignment, and catastrophic failure of the embankment has occurred in several locations due to the inability of the underlying soils to support the embankment. The embankment has also sustained considerable damage due to erosion, vectors, poor construction, and poor construction quality control and quality assurance. Sections of the embankment have been repaired. However, some repaired sections have failed after being repaired and other sections of the embankment are considered to be at risk of further catastrophic failure.

Louis Berger International, Inc. and their associates performed an investigation in February 1990 to determine the causes of failure and to develop remediation options. The findings of this investigation were presented in "Interim Report No. 1, Dhaka Integrated Flood Protection, FAP-88", dated May 1991.

The report characterizes the damaged sections into three Classes: I, II, and III, as those requiring immediate (I), short-term (S), and medium- to long-term (M) remedial actions, respectively. The lengths of embankment in the different classes were as follows:

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Class	Action Level	Embankment Length (m)	Embankment Length (%)
I	I	4,700	16.1
II	S	3,050	10.4
III	M	3,150	10.8
		10,900	37.3

The objectives of the present field and laboratory testing program and subsequent analyses will be to:

- better characterize the extent of the problems;
- refine the estimates of the limits of the Class I, II and III areas;
- determine the interface between the embankment fill soils and the original soils;
- characterize the stratigraphy along the embankment alignment;
- perform in situ tests to obtain quantitative soil parameters;
- obtain disturbed and undisturbed soil samples for identification and laboratory testing;
- perform laboratory tests to obtain quantitative soil parameters; and
- use the data collected to perform remedial designs.

E. Timing

The Contractor should be prepared to commence the field program on signing the agreement and to complete the field program within 7 weeks. Laboratory testing should start within one week of signing the agreement and must be completed by 14 May 1992. Therefore, time is of critical importance to this project. The Contractor must initiate, perform, complete and report work in a timely manner.

F. Field and Laboratory Testing Program Organization

The Consultant recognizes that the proposed field and laboratory testing program comprises a quantity of work which is beyond the capacity of a single contractor. The Consultant anticipates that at least two contractors will be engaged to perform the field program. The Consultant also anticipates that these contractors will also be capable of performing most of the routine laboratory tests. However, the Consultant appreciates that additional laboratory tests (namely: consolidation, unconsolidated undrained triaxial (UU), consolidated undrained triaxial with pore pressure measurement (CU), and triaxial permeability) will require the services of contractors other than the field program contractors. The Consultant will stipulate which other contractor(s) will perform these additional tests and the Contractor entering into this agreement will be responsible for transporting and delivering the soil samples to the laboratory testing contractor(s) in a timely manner.

G. Contractor's Undertakings

1. TCIL's Interests

The Contractor shall act at all times so as to protect the interests of TCIL and will take all reasonable steps to keep expenses to a minimum, consistent with sound engineering practice.

2. Units and Language

The Contractor will prepare all reports and documents using the metric system and the English language, unless otherwise directed by the Consultant.

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3. Records and Confidentiality

The Contractor shall keep accurate and systematic records of the work performed. Except with the prior written consent of TCIL, the Contractor will not make public any information obtained during the execution of this project.

4. Indemnification and Insurance

The Contractor shall be liable for loss or damage to persons or property which are caused by the Contractor, or subcontractor's, actions. The Contractor shall hold the Consultant, TCIL and Owner harmless from any actions arising from such loss or damage.

The Contractor shall take out and maintain adequate professional liability insurance as well as adequate insurance against third party liabilities and losses.

5. Variation of Agreement

This agreement may be varied by consent between the parties. All such variations shall be in writing and signed by the duly appointed representatives of the parties.

H. Measurement and Payment

1. General

The Contractor, at his own cost and expense, shall do all work and provide all labor, materials, tools, supplies, machinery and other equipment and plant necessary for the execution of the project.

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2. Items, Quantities and Standards

The items and estimated quantities of each item that will be measured for payment are contained in Table 1.1. The Contractor is aware that these quantities are estimates only; the actual quantities will be different and may be increased or decreased at the direction of the Consultant. The Consultant does not guarantee that the Contractor will perform a particular quantity of work for any item. The Contractor will be reimbursed for services on a unit rate basis for work actually completed and approved. The Contractor will be reimbursed at the unit rates agreed between the Contractor and TCIL and presented as Appendix II.

The Consultant reserves the right to inspect the work in progress and to reject and refuse payment for any work which does not conform to the standards and methods listed herein or to recognized engineering practice. TCIL may at any time terminate this agreement and/or engage another contractor(s) to perform any of the works if, in his sole opinion, the work is not being undertaken in a manner consistent with the intent of this project.

3. Contractor's Responsibility

TCIL will reimburse contractors for such work that the Consultant directs will be performed by other contractors on work associated with the Contractor's activities (for example laboratory tests performed by BUET on soils sampled by the Contractor). It will be the responsibility of the Contractor and the other contractor(s) to coordinate invoicing to TCIL.

4. Mobilization Payment

The Contractor will be paid a mobilization payment of Tk 2,00,000 (Taka Two Lacs only) within 7 days of signing the agreement or within 7

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days of TCIL receiving its mobilization payment from the Owner, whichever is the later. The mobilization payment will be deducted from progress payments in equal amounts from the first two monthly progress payments.

5. Monthly Progress and Final Payments

The Contractor should submit monthly progress invoices and a final invoice after completion of the works. Subject to the Consultant's approval of each invoice, which will be subject to satisfactory submission of reports as described in Section 6 herein, TCIL will make payment within 7 days of receipt of payment from the TCIL.

I. Document Organization

The remainder of this document is organized as follows:

- Section 2 details the soil borings that will be drilled and the procedures that will adopted to drill the borings.
- Section 3 describes the field sampling procedures that will be followed to obtain disturbed and undisturbed samples.
- Section 4 describes the in situ field tests that will be performed and the procedures that will be followed.
- Section 5 describes the laboratory testing that will be performed and the procedures to be followed.
- Section 6 describes the periodic and final reports which are to be submitted.
- Section 7 contains a list of references.

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TABLE 1.1

FIELD AND LABORATORY TESTING PROGRAM

ITEMS OF WORK AND ESTIMATED QUANTITIES

ITEM No.	ITEM	UNIT	ESTIMATED QUANTITY
1	Mobilization of rotary drill rig and all equipment, supplies, tools, personnel and all other things needed for performance of work and demobilization upon completion of work.	Each	2 - 4
2	Moving rotary drilling equipment and all equipment from one boring location to another when the distance between borings is greater than 250 meters.	Each	27
3	Drilling 82, minimum 120 mm nominal diameter borings including field identification of soils, recording ground-water levels, maintaining drilling records etc.	Linear meters	2050
4	Performing Standard Penetration Tests with field identification and preservation of samples, including transport to laboratory.	Each	2000
5	Collecting undisturbed samples (shelby tube) of minimum 70 mm internal diameter, performing miniature vane on sample, identification of soils, and preservation of samples including transport to laboratory in Dhaka.	Each	600
6	Transport of shelby tubes to River Research Institute in Faridpur.	Each	100
7	Vane shear testing in boring made for sample collection.	Each	300
8	Vane shear testing adjacent to sample collection boring.	Linear meter	1000
9	Performing in situ rising head or falling head permeability test in open boring.	Each	20
10	Performing in situ permeability test in boring using double packer rings.	Each	40
11	Installing piezometer with protective casing in boring.	Each	20
12	Laboratory soil identification of samples.	Each	1500
13	Moisture content determination.	Each	1500
14	Liquid limit, plastic limit and plasticity index determination.	Each	1500
15	Wet and dry unit weights.	Each	1500
16	Sieve and hydrometer gradation analyses.	Each	1000
17	Specific gravity determinations.	Each	250
18	Consolidation test with unload-reload cycle.	Each	200
19	Unconsolidated undrained triaxial tests (UU), with 3 specimens per sample.	Each	500
20	Consolidated anisotropically (K_o) undrained compression triaxial test with pore pressure measurement (CU), with 3 specimens per sample.	Each	70
21	Direct shear test.	Each	-
22	Triaxial permeability test.	Each	50
23	Production of records and reports.	Item	1

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II. SOIL BORINGS

It is proposed to drill eighty-two borings (minimum 120-mm (5-in.) nominal diameter) along the embankment alignment at the stations and locations shown on the boring schedule in Appendix I. The borings will be drilled from the top of the embankment and on the side slopes of the embankment. Many of the borings will be drilled in areas where embankment failures have occurred.

The borings will be located by the Consultant. The Consultant will also establish the ground surface elevation and/or the elevation of a reference marker at each boring position. The Contractor will use the reference elevations in performing and reporting field activities. The Contractor will drill the boring within 0.5 m (1.5 ft) of the marked position; greater deviations will require the approval of the Consultant. Companion in situ shear vane testing will be performed within 0.5 m (1.5 ft) of a boring after the boring is completed.

The intent is to drill each boring through the silt and clay stratum, which is thought to underlie the embankment alignment, and a minimum of 3 m (10 ft) into a continuous sand stratum. This depth may be increased if conditions at a particular location are thought to consist of alternating clay and sand strata having thicknesses greater than 3 m (10 ft). It is anticipated that boring depths will not exceed 35 meters (120 ft). The actual depths will be dependent on the field conditions.

A. Boring Method

The borings will be drilled vertically using a wash boring technique and rotary equipment capable of pushing undisturbed sample shelby tubes by hydraulic pressure. Casing will be installed in each boring as it is advanced to stabilize the upper part of the boring and to provide a drilling fluid return conduit. Water will be used as the drilling fluid; however, if the sides of the boring are unstable bentonite shall be mixed

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with the drilling fluid to increase its density, and/or the casing will be advanced to a greater depth.

If the casing is driven by a free falling hammer the number of blows of the hammer and the drop height of the hammer will be recorded for each 0.3 m (1 ft) of casing length. This resistance to penetration information may be helpful as an additional guide to determining the subsurface stratigraphy.

In situ tests will be performed in the borings and both disturbed and undisturbed soil samples will be obtained from the borings.

Ground-water levels should be obtained in each boring as it is being advanced, as a minimum when ground water is initially encountered and subsequently at the commencement and end of each day's drilling. In addition, ground-water levels should be checked after breaks (eg. equipment failure, delays, and meal times) and as casing is installed to different depths.

Upon completion, each boring will be left open sufficiently long to obtain stabilized ground-water levels, no less than 24 hours, prior to backfilling. At the end of each days drilling and during the time that a boring is left open upon completion the boring shall be provided with a temporary protective cover.

As a minimum the following information should be recorded for each ground-water level recording:

- the elevation of the ground surface (or other reference position);
- the height of the top of the casing above the ground surface; and
- the distance from the top of the casing to the water in the boring. This should be recorded with a weighted tape or other device approved by the Consultant. The measurement will be made twice to ensure reliability. The measurements should agree to within 6 mm (0.25 in.).

Piezometers or wells will be installed in some borings to record piezometric and ground-water level variations with time. ASTM D 4570 shall be followed when measuring water level readings in boreholes or monitoring wells. Decisions will be made at the time of drilling whether to install a ground-water monitoring device and the type of device to install.

In order to obtain the detailed information that is being sought by this field investigation it is imperative that the top elevation and depths of the borings, samples, casing, etc., be known at all times. Therefore the lengths of all drilling equipment must be accurately known and recorded. Drilling rods and casing should all be of standard lengths. 'Odd' length equipment should neither be brought to the site nor used. The Consultant may require that any such equipment be removed from the site.

All equipment must be maintained in good and proper working condition. The Consultant may direct that inadequately maintained or calibrated equipment not be used.

B. Sampling and Testing Intervals

Different sampling and in situ testing intervals and sequences will be used in depending on the subsurface materials and circumstances. Four different conditions have been identified for which different sequences are proposed, as follows, and as detailed in Table 2.1.

Sequence 1. Embankment fill soils will be sampled and tested continuously in all borings using a combination of SPT's and undisturbed samples.

Sequence 2. The silt/clay stratum will be sampled and tested continuously in areas where deep seated failures have occurred until the failure surface has been reached. SPT's and undisturbed samples will be taken.

Sequence 3. The silt/clay stratum will be sampled and tested at intervals in areas where deep seated failures have not occurred and below the failure surface in areas where they have occurred. SPT's and undisturbed samples will be taken.

Sequence 4. The underlying sand stratum will be sampled and tested at intervals using SPT's.

The sequences, test orders, and intervals described above and shown in Table 2.1 should not be construed as fixed; they may be changed as the field program progresses to reflect experience gained and may be changed in individual borings in response to individual conditions. In developing Table 2.1 it has been assumed that both SPT's and undisturbed samples will have a length of approximately 0.45 m (1.5 ft).

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TABLE 2.1
IN SITU SAMPLING AND TESTING

SAMPLE AND IN SITU TEST ORDER (5)	CONTINUOUS SAMPLING		INTERVAL SAMPLING	
	SEQUENCE 1	SEQUENCE 2	SEQUENCE 3	SEQUENCE 4
	Embankment soils (1)	Silt/clay stratum above failure surface (2)	Silt/clay stratum below failure surface and in non-failure areas (3)	Sand (4)
1	SPT	SPT	SPT	SPT
2	Undisturbed	Undisturbed	Undisturbed	Gap
3	SPT	Undisturbed	Undisturbed	SPT
4	SPT	SPT	SPT	Gap
5	Undisturbed	Undisturbed	Gap	SPT
6	SPT	Undisturbed	SPT	Gap
7	SPT	SPT	Undisturbed	SPT
8	Undisturbed	Undisturbed	Undisturbed	Gap

Notes: 1. Sequence 1, approximate undisturbed sample interval 1.5 m (5 ft).
2. Sequence 2, approximate SPT interval 1.5 m (5 ft).
3. Sequence 3, approximate SPT interval 1.5 m (5 ft) and approximate interval between pairs of undisturbed samples 3.0 m (10 ft).
4. Sequence 4, approximate SPT interval 1.5 m (5 ft).
5. Assumed SPT sample length approximately 0.45 m (1.5 ft) and undisturbed sample length approximately 0.45 m (1.5 ft).

C. Sample Descriptions

In order to achieve consistency between different parties it is essential that a consistent system of soil description be adopted. A description must be provided for every sample obtained. Sample descriptions should follow ASTM D 2488. The following descriptors may be used to indicate material composition of secondary constituents:

- trace < 10%,
- little 10 - 20%,

- some 20 - 35%, and
- and 35 - 50%.

Specimens must be classified using the following material limits:

- clay < 0.002 mm,
- silt 0.075 - 0.002 mm, <#200 sieve,
- fine sand 0.425 - 0.075 mm, #40 - #200 sieve,
- medium sand 2.0 - 0.425 mm, #10 - #40 sieve,
- coarse sand 4.75 - 2.00 mm, #4 - #10 sieve,
- fine gravel 19.0 - 4.75 mm, 0.75 in. - #4 sieve, and
- coarse gravel 75 - 19 mm, 3 in. - 0.75 in. sieve.

In addition the following descriptors should be used when appropriate (abbreviated explanations/examples are provided, see ASTM D 2488 for complete explanations) and those underlined should always be given:

- group name (e.g. Fill, Alluvium, Residuum),
- color,
- consistency (very soft, soft, firm, hard, very hard),
- moisture condition (dry, moist, wet),
- dilatancy (none, slow, rapid),
- structure (stratified (>6 mm layers), laminated (<6 mm layers), fissured, slickensided (polished failure planes), blocky, lensed (note thickness of lenses), homogeneous),
- dry strength (none, low, medium, high, very high),
- cementation (weak, moderate, strong),
- angularity of coarse-grained material (angular, subangular, subrounded, rounded),
- particle shape (flat, elongated, flat and elongated),
- organic material, clay, silt, or peat,
- odor (particularly for organic deposits),

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- toughness (low, medium, high), and
- additional data (roots, holes, mica, etc).

In the description the group name (if given) and the primary component should be capitalised. The first letter of secondary components should be capitalised. Descriptors should be separated by commas. For example:

> FILL. Light brown, red and grey, fine to medium SAND, little Gravel, some Silt and Clay, moist, roots of diameter less than 6 mm. Fines have moderate plasticity.

D. Records

The driller, or person responsible for each boring, shall maintain for each boring, at the time of drilling, as a minimum the following details:

- the project name and location;
- the boring reference and location;
- the date and time of drilling;
- personnel information, as a minimum the name of the driller and any supervisory staff;
- the make and type of drilling equipment used;
- a sketch of the boring location including its position relative to site features;
- soil descriptions of samples recovered;
- sample records as described in Section 3;
- details of casing installation;
- details of ground-water readings;
- details of piezometer or observation well installation;
- details of other subsurface and drilling information, such as presence of hard or soft layers, change of materials, change in return water color;

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- details of equipment breakdown, and intervals between continuous drilling (e.g. meal breaks, shift changes and normal end of day and start of day breaks); and
- other such information as may assist in interpreting the subsurface conditions.

To assist in the correlation and interpretation of the data the Consultant has prepared a standard Field Boring Log to be used to record the boring information. A copy of this log must be given to the Consultant at the completion of each boring. The Contractor may also use its own field record in addition to the above log. The Consultant may also, at his discretion, develop additional standard forms during the field program, that the Contractor will be required to complete, to assist in record keeping.

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III. SAMPLING

Field sampling will be undertaken, at the intervals described in Section 2.2 above, to obtain representative samples of the different subsurface materials encountered. Both disturbed and undisturbed samples will be obtained. Each sample, whether disturbed or undisturbed, will be identified as described in ASTM D 420 showing, as a minimum, the following information:

- the project name and location;
- the boring reference;
- the date;
- the sample reference; and
- the depth and interval below ground level of the sample;

Samples shall be transported to the laboratory, at least on a daily basis, for storage and testing, in accordance with ASTM D 4220. In particular, undisturbed samples will be classified as Group D (samples that are highly sensitive and which will be subjected to, inter alia, consolidation, permeability and stress-strain tests) and shall be transported and stored in cushioned purpose made containers which will permit the samples to remain in a vertical position.

A. Disturbed Samples

Disturbed soil samples will either be obtained by hand sampling from cuttings or from the split-barrel sampler used to perform standard penetration tests. Hand samples will be collected at the Consultant's or driller's discretion for visual identification and possibly classification type laboratory tests.

Standard penetration testing and split-barrel sampling (SPT) will be performed in accordance with ASTM D 1586. A split-barrel sampler, 1.38-in. (34.9-mm) internal diameter and 2.0-in. (50.8-mm) external diameter, is driven into the soil using a hammer weighing 140 lb (63.5

kg) and falling freely from a height of 30 in. (0.76 m). The number of blows of the hammer required to advance the split-barrel each of three consecutive 6-in. (150-mm) increments is recorded unless one of the following has occurred:

- a total of 50 blows is applied in any one of the three 150-mm (6-in.) increments;
- a total of 100 blows have been applied; or
- there is no observable advance of the sampler during the application of 10 successive hammer blows.

The standard penetration resistance is recorded as the sum of the blows required to drive the split-barrel sampler the final 300 mm (12 in.).

Essential procedures in performing this test are:

- clean the boring to the sampling elevation;
- maintain the liquid level in the boring above the <u>piezometric</u> ground-water level;
- lower the sampler to the bottom of the boring; and
- perform the test using a trip-hammer without interruption.

After removing the sampler from the boring it will be opened and the following performed:

- the soil recovery will be measured, discounting any cuttings, and recorded;
- the soil will be described by the driller on a field log; and
- a sample of the soil will be preserved for laboratory testing.

B. Undisturbed Samples

Undisturbed samples will be taken to obtain samples suitable for laboratory consolidation, permeability and triaxial stress-strain tests. Undisturbed samples will be obtained in clay and silt deposits using thin-wall (shelby) tube and piston sampling techniques. The aim of this type of sampling is to obtain soil samples of the highest quality that have been subjected to a minimum degree of sample disturbance.

The tubes will be circular in cross section and must be free of rust and dirt. The tubes shall have a minimum diameter of 70 mm (3 in.) and a retrieved sample length of at least 0.45 m (1.5 ft). The drilling equipment must be capable of both pushing the tubes into the soil and extracting the tubes from the soil at a steady rate. Tubes should not be "hammered" into or out-of the soil.

The sampling procedures are described in ASTM D 1587. Essential sampling procedures are:

- clean the borehole to the sampling elevation;
- maintain the liquid level in the boring at or above the piezometric ground-water level during sampling;
- lower the thin-wall sampler (or piston sampler) to the bottom of the hole;
- push the sampler into the soil using a continuous motion at a rate of approximately 75 to 150 mm/sec (3 to 6 in./sec);
- withdraw the sampler from the soil carefully to minimize sample disturbance; and
- remove disturbed cuttings from the upper end of the tube and measure the length of sample;
- determine the shear strength of the sample using a hand operated miniature vane, in accordance with ASTM D 4648 on the soil in the bottom of the tube;
- remove approximately 1 in. (25 mm) of soil from each end of the tube for identification purposes and preserve separately; and
- seal both ends of the sample with wax and attach a protective cap to both ends of the tube.

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IV. FIELD TESTING

In addition to the SPT tests, described in Section 3.1, and hand operated miniature shear vane tests mentioned in Section 3.2, in situ permeability tests and shear vane tests shall be performed. The actual test schedule will be determined at the time of drilling to reflect the actual field conditions encountered and the test equipment available.

A. Permeability Tests

In situ permeability tests will be performed using either open hole rising or falling head tests or double packer tests. Borings in which permeability tests are performed must be drilled with clear water to reduce the possibility of forming a smeared zone along the walls of the boring. When the boring has reached the desired depth the boring is thoroughly cleaned by washing with clear water until a clean surface of undisturbed material is present along the sides and at the bottom of the boring.

In open hole tests, rising head tests will be preferred to falling head tests since with falling head tests there is a possibility of clogging the soil pores with sediment from the water in the boring. However, falling head tests will have to be performed if there is a risk that the soil at the bottom of the hole will become quick due to the hydrostatic pressure gradient. The bottom of the boring must be sounded after each rising head test to determine if the bottom of the hole has heaved. The tests will be performed twice at each depth to ensure a consistent result.

Rising head tests will be performed by bailing water from the boring and recording the rate of rise of water in the boring at intervals until the rate of level change becomes small. Falling head tests are performed by filling the casing with water and recording the rate of fall of the water level as the water seeps into the soil.

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The appropriate reading intervals will depend on the permeability of the soils but frequent enough to establish an equalization diagram. The test time should be greater than 5 minutes. Water level readings are usually taken 1, 2, and 5 minutes after the start of the test and at 5 minute intervals thereafter until the rate of change has become negligible.

Appropriate procedures for packer tests will be determined based on the equipment which may be available.

For either test the following minimum information must also be recorded:

- the depth from the ground surface to the ground-water (piezometric) level before and after the test;
- the inside diameter of the casing;
- the height of the casing above the ground surface;
- the installed length of casing;
- the diameter of the boring below the casing;
- the depth to the bottom of the boring below the top of the casing;
- the zone being tested;
- the depth to the standing water level from the top of the casing; and
- a description of the material being tested.

B. <u>Vane Shear Tests</u>

In situ vane shear tests will be performed in cohesive soils to obtain shear strength information. Vane shear tests will be performed in accordance with ASTM D 2573. It is the intent that vane shear be performed in borings advanced adjacent to the borings used to collect samples. However, at the Consultant's discretion, tests may be performed in the same boring used to collect samples.

The Consultant recognizes that different types of vane shear equipment are available in Dhaka and the Contractor's equipment may not comply with the requirements of ASTM D 2573. Approval of such equipment will be at the Consultant's discretion and appropriate test methods will be agreed. The following notes apply to the desired method of testing using equipment with a drive gear mechanism.

The vane shear test should be performed in such a way that friction between the soil and rods and within the gear housing is determined. The recorded vane shear strength will be adjusted to compensate for friction. The vane shear test will be conducted at a rate of angular rotation of less than 0.1 degree/sec (6 degree/min). It should be anticipated that time to failure will be on the order of 5 to 10 minutes. If rapid failures occur the angular rotation rate will be reduced. After the maximum torque has been determined the vane will be rotated rapidly through at least 10 complete revolutions to remould the soil and the remoulded resistance determined immediately using the same procedure as initially. The vane shear tests will generally be conducted at an interval of 1 m (3.3 ft) in companion borings.

The following minimum information must be reported for each test:

- boring number;
- date and time of test;
- name of person performing test;
- size and shape of vane;
- depth of vane tip;
- torque readings at 1 degree intervals during undisturbed test and 2 degree intervals during remoulded test and maximum torque readings;
- time to failure;
- rate of rotation; and
- notes on deviation from standard practice.

Prior to performing any tests on the project the equipment must be calibrated and friction components determined. The friction components

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must be rechecked at regular intervals during the project. The appropriate interval will be determined by the Consultant depending on the type of equipment, its performance and treatment in the field.

V. LABORATORY TESTING

Laboratory tests will be performed to evaluate the properties of embankment and subgrade soils. The laboratory program is designed to obtain parameters to analyze the stability of the existing embankments and to design appropriate remedial measures.

The tests detailed in Table 1.1 and described in the subsections below are planned. References of the ASTM designations given in the text are presented in Section 6. Other tests or variations of the proposed tests may be deemed necessary during execution of the Field Investigation Plan depending on the soils encountered and the results obtained. The actual laboratory test schedule will be determined by the Consultant as the field work is performed.

All equipment used, including gauges, balances, load cells, etc. must be calibrated and the documentation made available for inspection.

In order to assist in the compilation and analysis of the laboratory data the Consultant will prepare forms for all or some of the tests and the Contractor will be required to submit the data to the Consultant upon the completion of each test.

A. Moisture Content

Moisture content determinations will be routinely performed on all samples. The tests will be performed in accordance with ASTM D 2216.

B. Wet and Dry Unit Weights

Wet and dry unit weights will be routinely determined on all undisturbed samples.

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C. Atterberg Limits Tests

Atterberg Limits tests will be routinely performed on all plastic inorganic and organic samples. The tests will be conducted in accordance with ASTM D 4318.

D. Particle Size Analysis

Particle size determinations will be routinely made on cohesive and cohesionless samples. The tests will be conducted in accordance with ASTM D 422. Sieve analyses will be performed on all samples and hydrometer analyses on samples passing the No. 10 (2.00-mm) sieve.

Sieve tests will be performed using, as a minimum, the following sieve sizes: 3/4 in. (19 mm), 1/2 in. (12 mm), 3/8 in. (9.5 mm), No. 4 (4.75 mm), No. 10 (2.00 mm), No. 20 (0.85 mm), No. 40 (0.425 mm), No. 60 (0.250 mm), No. 100 (0.150 mm), and No. 200 (0.075 mm).

Hydrometer tests will be performed using a hydrometer of type 151H or 152H for which a composite correction graph has been determined. The composite correction graph, showing the date of calibration, should be submitted to the Consultant for approval prior to performing any hydrometer tests.

E. One-Dimensional Consolidation Test

One-dimensional consolidation tests will be performed on cohesive soil samples. The tests will be performed in accordance with ASTM D 2435. Precautions must be taken to minimize disturbance to the soil samples during preparation of the samples. Specific gravity determinations will be made in accordance with ASTM D 854 on samples adjacent to the test specimen. The following procedures will be used for the consolidation test:

- the specimen will be subjected to a load increment ratio (LIR) of one to produce a doubling of pressure on the soil. Values of 12, 25, 50, 100, 200, 400, 800 and 1,600 kN/sq. m (0.12, 0.25, 0.5, 1.0, 2.0, 4.0, 8.0 and 16.0 tons/sq. ft), or a similar sequence will be used. Smaller increments may be used at low pressures in the region of the preconsolidation pressure if poorly defined curves are consistently produced;
- the loading will be continued to a pressure of at least four times the preconsolidation pressure;
- an unload-reload cycle will be performed at a pressure approximately 1.5 to 2 times greater than the preconsolidation pressure. A load increment (decrement) ratio of two will be used during unload-reload cycles and unloading will span two pressure decrements;
- an unloading schedule will be performed after application of the maximum load using an LIR of two.
- the change in height of the sample will be read for each load application, as a minimum, at times of 0.1, 0.25, 0.5, 1, 2, 4, 8, 15 and 30 minutes and, as necessary, at 1, 2, 4, 8, and 24 hours. Each load will be left in place until at least two cycles of secondary compression have occurred; however, load increments may be applied at 24 hour intervals if primary consolidation has completed;
- time-deformation curves will be produced for each load increment using <u>both</u> the log of time method and the square root of time method. The coefficient of consolidation and the time for primary consolidation will be computed from the average of the log of time and the square root of time methods and secondary compression coefficients will be determined from the log of time curves;
- the consolidation readings will be adjusted to determine the deformations at the end of primary consolidation for each load;
- a curve of cumulative work done <u>at the end of primary consolidation</u> (vertical effective stress * cumulative strain) (y-algebraic scale) versus vertical effective (x-algebraic scale) will be prepared (Becker, 1987);

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- a curve of strain at the end of primary consolidation versus log of pressure will be prepared;
- the recompression ratio (RR), virgin compression ratio (CR), and preconsolidation pressure (P_c) will be determined, using the methods of Casagrande, Schmertmann (1955) and Becker (1987); and
- a graph of coefficient of consolidation versus log of pressure will be prepared.

F. Unconsolidated, Undrained Triaxial Compression (UU) Test

UU tests will be performed on cohesive samples in accordance with ASTM D 2850. Three specimens will be tested from each specimen at different confining pressures. The pressures will be approximately 0.5, 1, and 2 times the in situ vertical total stress.

G. Consolidated, Undrained Triaxial Compression (CU) Test

Neither in situ vane shear tests nor laboratory UU tests can relate changes in strength to increases in consolidation in a rational manner. In addition, both shear vane and UU tests may give unreliable estimates of shear strength due to sample disturbance and the applied shear system. CU type testing will be performed to overcome these deficiencies. These tests will be performed generally in accordance with ASTM D 4767. However, to obtain data that can be used and interpreted rationally, the tests will be run to obtain normalized soil parameters using the SHANSEP method developed by Ladd and Foott (1974).

The following general procedures will be used. However, the Consultant will develop specific procedures in consultation with the contractor(s) who will perform these tests:

- test specimens will be consolidated anisotropically under a K_o stress system based on the stress history data gathered from consolidation tests;

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- some specimens will be consolidated to different pressures above the preconsolidation pressure and then sheared (i.e. at an overconsolidation ratio (OCR) of 1). Other specimens will be consolidated to pressures above the preconsolidation pressure and then unloaded to different stress levels, to simulate overconsolidation (i.e. to produce specimens with OCR's >1), and then sheared;
- all specimens will be sheared in compression (unless the capability exists to perform extension tests) with pore pressure measurements; and
- shear strength data will be presented in terms of p, q stress path plots; and
- in addition, at least the following plots will be required for each test:
  - $(p_1 - p_3)$ versus axial strain;
  - (p_1'/p_3') versus axial strain;
  - change in pore pressure versus axial strain; and
  - pore pressure parameter "A" versus axial strain.

H. Triaxial Permeability Test

Triaxial permeability tests will be performed on cohesive soil samples. The tests will be conducted generally in accordance with ASTM D 5084. Two test methods may be used. One method follows ASTM D 5084 and permits only vertical drainage to obtain the vertical permeability. The other method developed by GeoSyntec Consultants only permits radial drainage and prevents vertical drainage. The Consultant will develop specific procedures in consultation with the contractor(s) who will perform these tests.

I. Hydraulic Conductivity Ratio (HCR) Test

The Hydraulic Conductivity Ratio (HCR) test is a test method for evaluating filtration performance of geotextiles. It is used to evaluate characteristics of the soil smear zone adjacent to geotextiles. No ASTM test procedure has been formally prepared for this test at this time. However, GeoSyntec Consultants has developed detailed test procedures for this test. The Consultant will develop specific procedures in consultation with the contractor(s) who will perform these tests and will distribute the test procedures at that time.

The HCR test is performed in a flexible-wall triaxial permeability device. The test specimen is prepared such that the cross-section simulates the construction of the drainage system anticipated in the field. In general, this would include a composite of a drainage layer, geotextile and soil. The drainage material and soil are prepared such that the density and moisture conditions simulate those anticipated from the field application. The specimen is subsequently percolated and back-pressure saturated with water to simulate saturation. After saturation, the specimen is isotropically or anisotropically consolidated at an effective confining stress equal to that anticipated from the field application. After primary consolidation is complete, the test is initiated by permeating the pore fluid through the soil, through the geotextile, and then into the drainage material. The hydraulic conductivity of the composite system is plotted as a function of time, and as a function of the volume of fluid passing through the specimen. The hydraulic conductivity ratio (HCR) is defined as:

$$HCR = \frac{k_{sg}}{k_s}$$

where: k_s is the hydraulic conductivity measured at the outset of the test, and k_{sg} is the stabilized hydraulic conductivity, or the hydraulic conductivity measured after a specified time period (for example, after 30 days). Clogging of the geotextile is indicated by significant reduction in the hydraulic conductivity, which results in a low value of HCR.

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VI. REPORTING

The Contractor will be required to prepare a final report which, as a minimum, contains the following:

- details of the field and laboratory work performed;
- details and descriptions of the methods employed;
- data from the work performed;
- test results;
- analysis of test results where appropriate;
- conclusions and recommendations.

The report format shall contain a mixture of text, tables, graphs and location plans.

In addition, the Contractor will be required to submit such reports and documents necessary to enable the Consultant to remain current with the testing program and to verify the Contractor's monthly invoices.

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VII. REFERENCES

ASTM D 420 Standard Guide for Investigating and Sampling Soil and Rock.

ASTM D 422 Standard Test Method for Particle-Size Analysis of Soils.

ASTM D 854 Standard Test Method for Specific Gravity of Soils.

ASTM D 1586 Standard Method for Penetration Test and Split-Barrel Sampling of Soils.

ASTM D 1587 Standard Practice for Thin-Walled Tube Sampling of Soils.

ASTM D 2216 Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures.

ASTM D 2435 Standard Test Method for One-Dimensional Consolidation Properties of Soils.

ASTM D 2488 Standard Practice for Description and Identification of Soils (Visual-Manual Procedure).

ASTM D 2573 Standard Test Method for Field Vane Shear Test in Cohesive Soil.

ASTM D 2850 Standard Test Method for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression.

ASTM D 4220 Standard Practices for Preserving and Transporting Soil Samples.

ASTM D 4318 Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils.

ASTM D 4648 Standard Test Method for Laboratory Miniature Vane Shear Test for Saturated Fine-Grained Clayey Soil.

ASTM D 4750 Standard Test Method for Determining Subsurface Liquid Levels in a Borehole or Monitoring Well.

ASTM D 4767 Standard Test Method for Consolidated-Undrained Triaxial Compression Test on Cohesive Soils.

ASTM D 5084 Standard Test Method for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter.

Becker, D.E., Crooks, J.H.A., Been, K., and Jeffries, M.G. (1987). "Work as a Criterion for Determining In Situ and Yield Stresses in Clays." Journal of Canadian Geotechnical Journal, Volume 24, pp 549-564.

Ladd, C.C., and Foott, R. (1974). "New design procedure for stability of soft clays." Journal of Geotechnical Engineering Division, ASCE, 100(7), pp 763-786.

Schmertmann, J.H. (1955). "The undisturbed consolidation of clay." Transaction ASCE, 120, pp 1201 to 1233.

APPENDIX I

BORING SCHEDULE

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BORING REFERENCE (WAB-)	STATION	POSITION CL = Center line R = River Side C = Country Side
201	0+900	CL
202	0+950	CL
203	1+000	CL
204	1+050	R26
205	1+050	R13
206	1+050	CL
207	1+050	C13
208	1+050	C23
209	1+050	C36
210	1+100	CL
211	1+150	CL
212	1+200	CL
213	1+400	CL
214	4+100	CL
215	4+400	CL
216	4+700	CL
217	6+200	CL
218	6+275	CL
219	6+350	CL
220	7+050	R26
221	7+050	R14
222	7+050	CL
223	7+050	C13
224	7+050	C23
225	7+050	C36
226	7+750	CL
227	7+850	CL
228	8+000	CL
229	9+100	CL
230	9+850	CL
231	10+600	CL
232	10+850	R26
233	10+850	R14
234	10+850	CL
235	10+850	C13
236	10+850	C23
237	10+850	C36
238	11+000	CL
239	13+750	CL
240	13+900	R26
241	13+900	R14
242	13+900	CL
243	13+900	C13
244	13+900	C23
245	13+900	C36

BORING REFERENCE (WAB-)	STATION	POSITION CL = Center line R = River Side C = Country Side
246	14 + 350	CL
247	14 + 550	CL
248	14 + 900	R26
249	14 + 900	R14
250	14 + 900	CL
251	14 + 900	C13
252	14 + 900	C23
253	14 + 900	C36
254	15 + 250	CL
255	15 + 800	CL
256	16 + 500	R26
257	16 + 500	R13
258	16 + 500	CL
259	16 + 500	C13
260	16 + 500	C23
261	16 + 500	C36
262	17 + 200	CL
263		CL
264		CL
265		CL
266		CL
267		CL
268		CL
269		CL
270		CL
271		CL
272		CL
273		CL
274		CL
275		CL
276		CL
277		CL
278		CL
279		CL
280		CL
281		CL
282		CL