APPENDIX N

BANK STABILITY EVALUATION

APPENDIX N

BANK STABILITY EVALUATION

N.1 INTRODUCTION

The removal action alternatives being considered in the EE/CA Report involve the excavation of up to 3 ft of contaminated soil from the riverbed and riverbanks of the EE/CA Reach of the Housatonic River. The excavation will require the clearing and grubbing of the dense vegetative cover that acts to stabilize the existing steep riverbanks. A slope stability analysis was completed to determine the maximum permissible inclination of the finished cut slope so as to provide adequate long-term stability of the slope. Additional slope stability analyses were completed to determine the maximum permissible inclination of the cut slope during the removal action. The methodology of these analyses, along with conclusions and recommendations resulting from the analyses, are discussed in the following subsections.

N.2 SITE CONDITIONS AND BASES FOR ANALYSIS

Height of Riverbanks. An inspection of the topography and cross sections revealed that the vertical height of the riverbanks is highly variable along the EE/CA Reach. This dimension generally varies from approximately 10 to 30 ft. For this reason, three vertical slope height values (i.e., 10, 20, and 30 ft) were assumed for the slope stability analysis.

Subsurface Conditions. A total of eight test borings, designated as BH000091 through BH000098, were completed along the EE/CA Reach as shown in Figures 2.1-7 and 2.1-9 of the EE/CA report.

Formalized logs of the eight test borings were developed based on the field logs and notes. These logs are presented in Appendix F in the EE/CA Report. The logs present geologic descriptions of the encountered subsurface materials, stratigraphic interface depths, and the measured "blow counts" and Standard Penetration Resistance (i.e., "N") values of the split-spoon soil samples.

A geotechnical laboratory testing program was also completed to supplement the field data. The laboratory test results are also presented in Appendix F of the EE/CA Report. Physical property tests completed include:

1. Natural Moisture Content (ASTM D2216)
2. Atterberg Limits (ASTM D4318)
3. Grain Size Distribution/Sieve plus Hydrometer Analyses (ASTM D421/422)
4. Specific Gravity (ASTM D854)
5. Moisture/Ash/Organic Content (ASTM D2974)

Based on an inspection of the eight test boring logs and the laboratory test data, it is apparent that the subsurface conditions along the EE/CA Reach consist of predominantly granular soils with interbedded fine-grained soil layers. These interbeds average approximately 5 ft in thickness.

Consistent with the encountered subsurface environment in which a large majority of the recovered samples consisted of silty clayey coarse to fine sand and gravel, and as a simplifying assumption, the presence of the fine-grained soil interbeds was neglected in the stability analysis. For purposes of developing a typical cross section for the stability analyses, this material was assumed to extend to a depth of 5 ft below the river bottom.

Surface Water and Groundwater Elevations. For purposes of completing the stability analyses, the average water depth in the river channel was assumed to be 3 ft. The phreatic surface (groundwater) within the riverbank was selected to be 1 ft higher than the river water level, which is consistent with available groundwater elevation information within OU 1 of the GE Housatonic River site.

Soil Shear Strength Parameters. A total unit weight of 120 pcf was selected for the silty clayey coarse to fine sand and gravel. Based on the presence of clay size particles (< .002 mm) and plasticity properties in these soils, cohesive shear strength of 30 psf was assumed for this material. The angle of internal friction of the silty clayey coarse to fine sand and gravel was correlated to the measured N values within these materials. Based on a conservative analysis of the available data from the eight test borings (i.e., N values greater than 20 blows per ft were neglected), an average N value of 10 blows per ft was calculated. This value increased to 15 blows per ft when the individual N values were corrected for effective vertical overburden pressure at the split-spoon sample location using a procedure by Liao and Whitman (07-0018). The corrected average N value of 15 blows per ft was correlated to an angle of internal friction (ø) of 32° based on the following relationship developed by Kishida (07-0019).

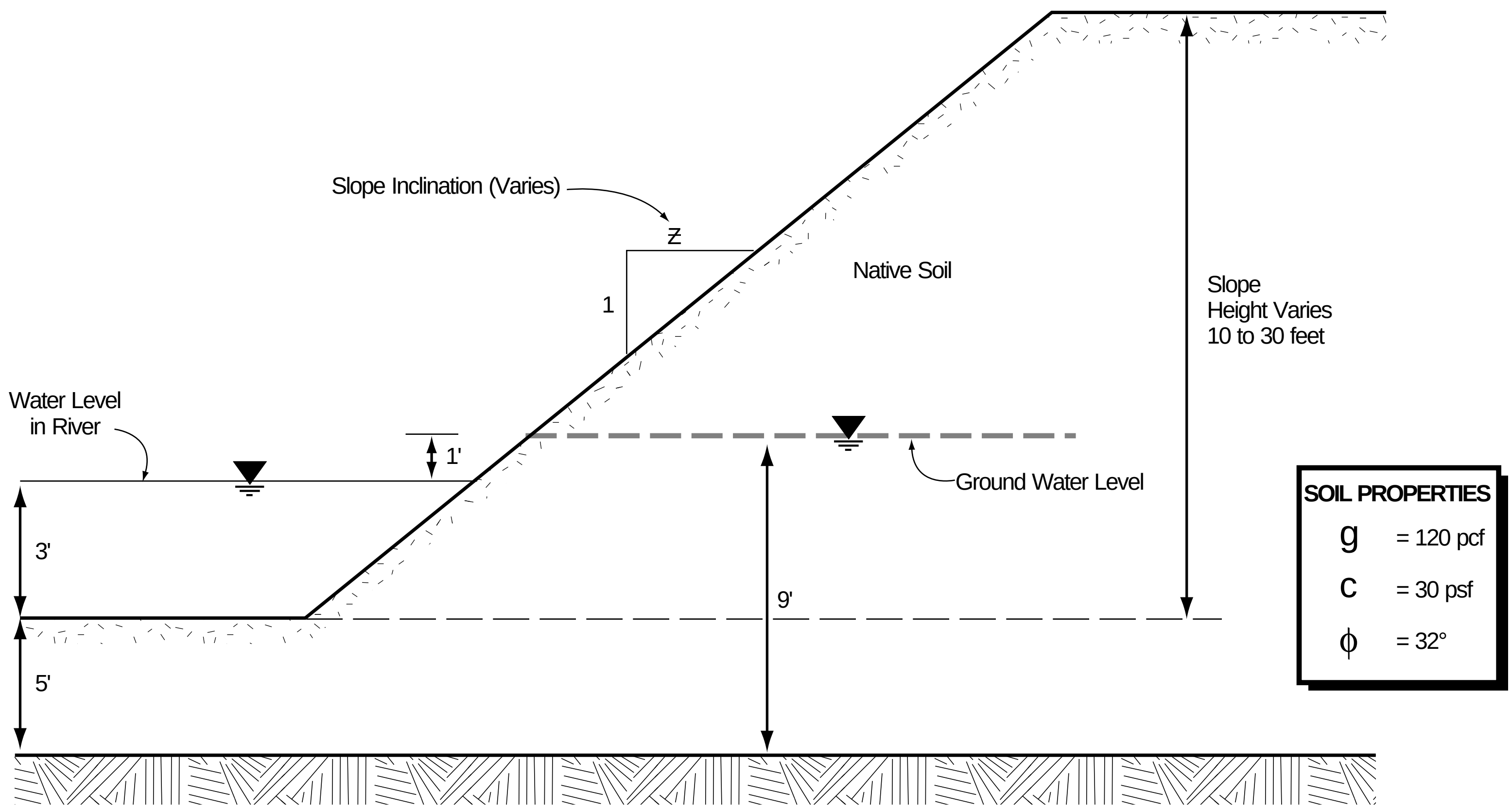
$$ø = \sqrt{20N} + 15°$$

Critical Cross Section. The generalized cross section developed for the slope stability analyses consistent with the above discussion is presented as Figure N-1.

N.3 STABILITY ANALYSES

Stability Scenarios. Consistent with a granular soil subsurface environment, three stability scenarios were analyzed:

1. Long-term, post-construction stability using drained (i.e., effective stress) soil shear strength parameters under static (i.e., no earthquake effects) conditions in which the assumed 3-ft water river depth is impounded against the slope face. For this condition, a minimum factor of safety (FS) of 1.50 is required in accordance with conventional geotechnical engineering practice.



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LEGEND

Bedrock

Native Soil

Water Level

ENGINEERING EVALUATION/COST ANALYSIS
Upper Reach of the Housatonic River
Pittsfield, Massachusetts

FIGURE N-1
GENERALIZED SLOPE STABILITY ANALYSIS
CROSS SECTION

99P-3001

2. Long-term, post-construction stability using drained soil shear strength parameters under dynamic (i.e., earthquake effects) conditions in which the assumed 3-ft water depth is impounded against the slope face. For this condition, a minimum FS of 1.00 is required per conventional geotechnical engineering practice. In this regard, a horizontal seismic coefficient of "0.10g" was selected for the Pittsfield, Massachusetts site for this analysis based on the Algermissen Study (99-0216 and 99-0217) for the 50-year recurrence interval/90% probability of nonoccurrence seismic event (see Figure N-2). In accordance with conventional geotechnical engineering practice, a vertical seismic coefficient of "0.10g" was also selected for the analysis.
3. Short-term stability using drained (i.e., effective stress) soil shear strength parameters under static (i.e., no earthquake effects) conditions in which the river channel is assumed to be dry and an equipment (excavator) surcharge load is in place at the top of slope. This scenario is representative of the end-of-construction condition in which the slope face is excavated and flattened to the required inclination and before the diverted river flow has been allowed back into the channel. For this condition, minimum FS values of 1.2 and 1.3 are required for the surcharge load and no-surcharge load scenarios, respectively, consistent with conventional geotechnical engineering practice.

Method of Analysis. The slope stability analysis was completed using the computer program SLOPE/W (99-0226) that is based on the Modified Bishop Method of Slices analysis procedure.

N.4 VEGETATED EARTHEN SLOPE CONDITION

For the condition where the finished slope will be vegetated from the riverbed to the top of the riverbank the slope angle was initially set at a 1H:1V inclination, and the minimum FS value calculated using SLOPE/W for this stability scenario. A similar analysis was completed for 2H:1V and 3H:1V inclinations. The calculated minimum FS values for the long-term static stability scenario are presented in Table N-1. Computer printouts for these runs are presented in Attachment N.1.

The minimum FS values of Table N-1 for each Vertical Slope Height (H) were plotted as shown in Figures N-3, N-4, and N-5 for H = 10, 20, and 30 ft, respectively. These graphical relationships permit determination of the maximum (i.e., steepest) permissible slope inclination of the finished cut slope to maintain a minimum FS value of 1.5 for the long-term static stability scenario for each value of H. These values are summarized in Table N-2. For the static stability condition, a slope inclination of 2.25H:1V or flatter for the finished cut slope face will yield an FS value of at least 1.5. This is true for any vertical slope height of riverbank within the EE/CA Reach.

Table N-1

Calculated Minimum FS Values for Vegetated Earthen Slope Scenario Long-Term Static Stability

Vertical Slope Height	Slope Inclination		
	1H:1V	2H:1V	3H:1V
10’	1.0	1.5	1.9
20’	0.9	1.4	1.9
30’	0.8	1.4	1.7

Table N-2

Calculated Maximum Permissible Slope Inclinations Long-Term Static Stability for Vegetated Earthen Slope Scenario

Vertical Slope Height (H)	Maximum Permissible Slope Inclination*
10’	2.0H:1V
20’	2.1H:1V
30’	2.2H:1V

*Slope inclinations of these values or flatter will provide a minimum FS of 1.5 for the static stability scenario.

The long-term dynamic and short-term static (with and without a surcharge load) stability scenarios were subsequently analyzed for the selected 2.25H:1V maximum permissible slope inclination for the 10-, 20-, and 30-ft vertical slope heights. The surcharge load was modeled using a load equivalent to the surcharge load of 2 ft of soil with a unit weight of 125 pcf as recommended by AASHTO. The computer output data for these runs are presented in Attachment N.1. These printouts document the calculated minimum FS values and the geometry of the critical failure surface for each run. The calculated minimum FS values are summarized in Tables N-3 and N-4. As is evident from these tables, all FS values exceed the minimum required values of 1.0, 1.2, and 1.3 for the long-term dynamic and short-term static (with and without surcharge loads) stability scenarios, respectively.

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Source:
Department of the Interior, U.S. Geological Survey
Probabalistic Earthquake Acceleration and Velocity Maps for the United States and Puerto Rico
by S.T. Algermissen, D.M. Perkins, P.C. Thenhaus, S.L. Hanson, and B.L. Bender
Map A. — Horizontal Acceleration (90 Percent Probability of Not Being Exceeded in 50 Years)

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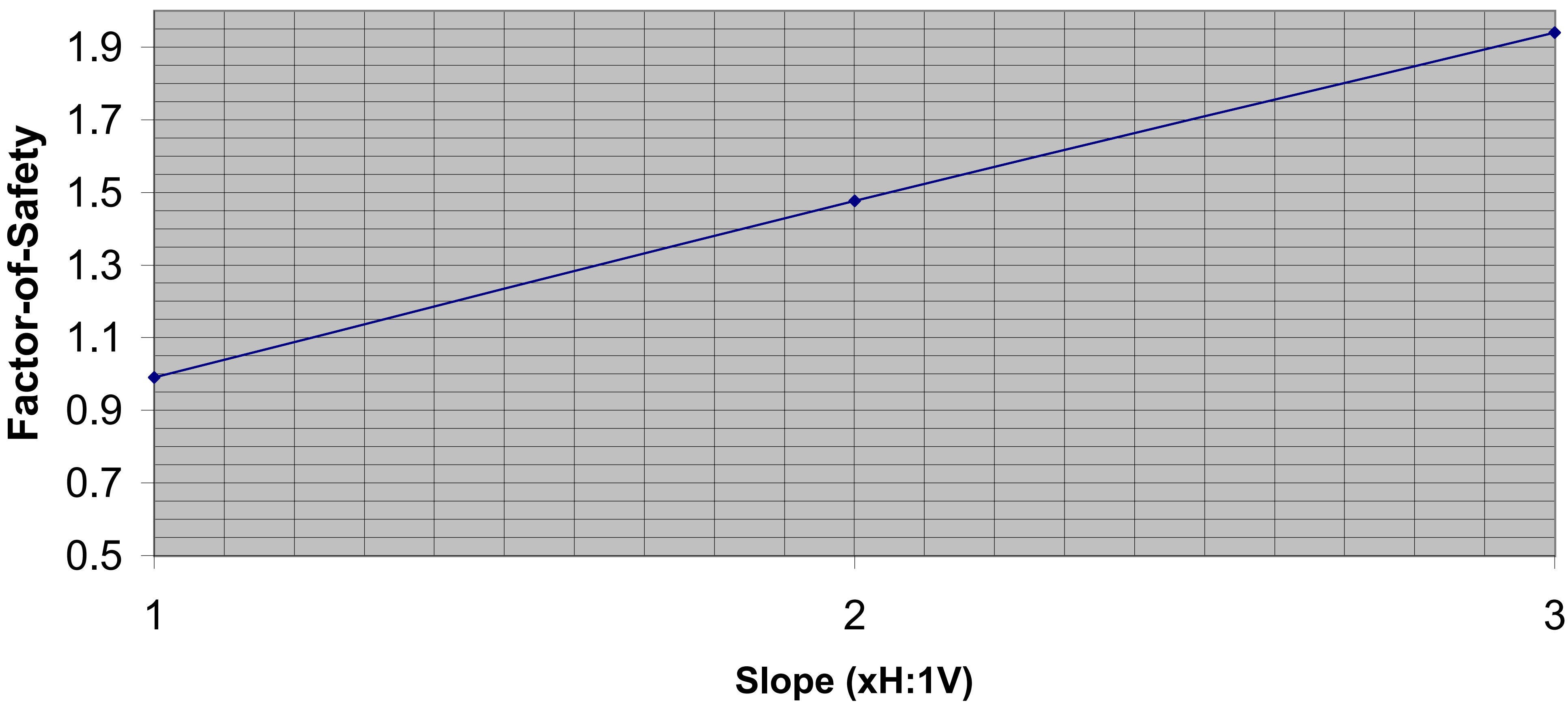
Upper Reach of the Housatonic River
Pittsfield, Massachusetts

FIGURE N-2
SEISMIC MAP

99P-1934f



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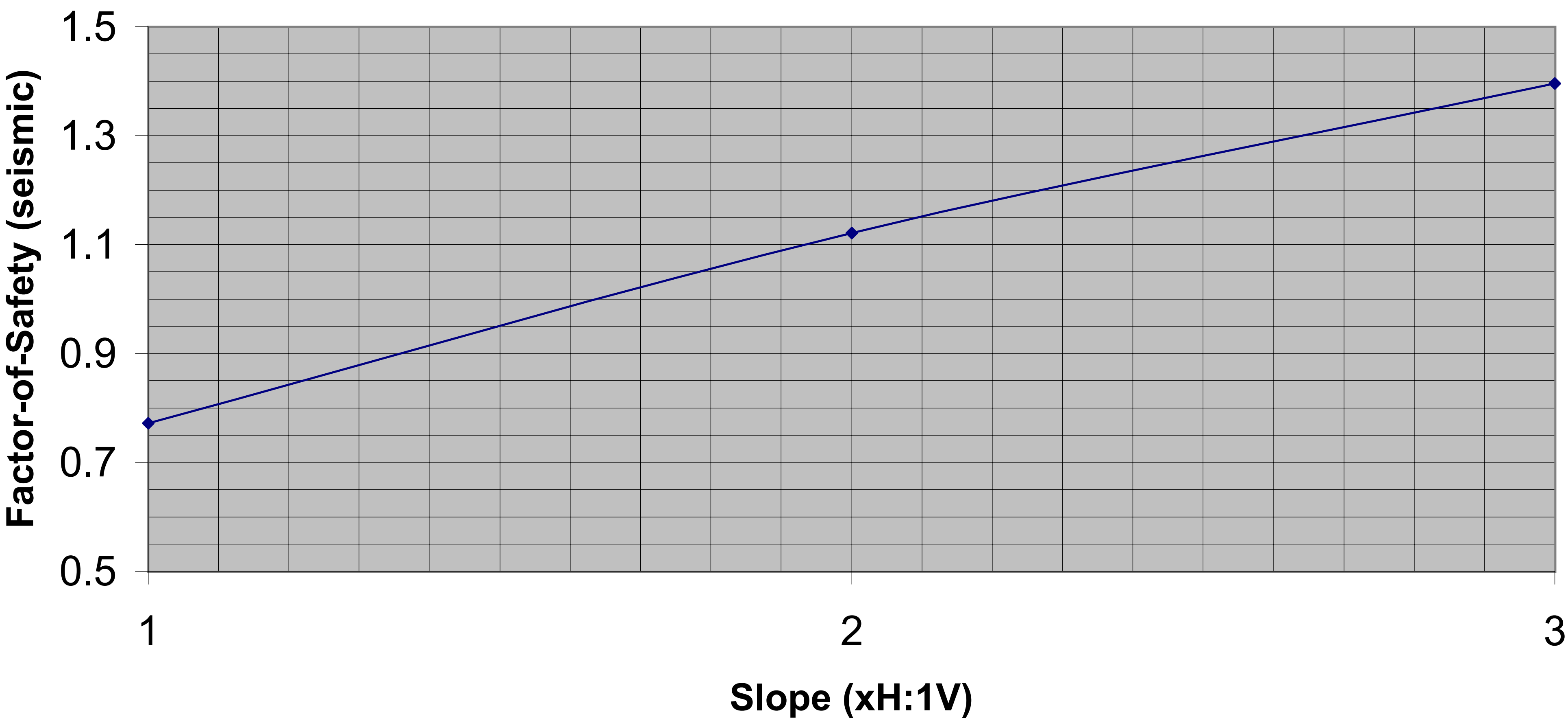
Upper Reach of the Housatonic River
Pittsfield, Massachusetts

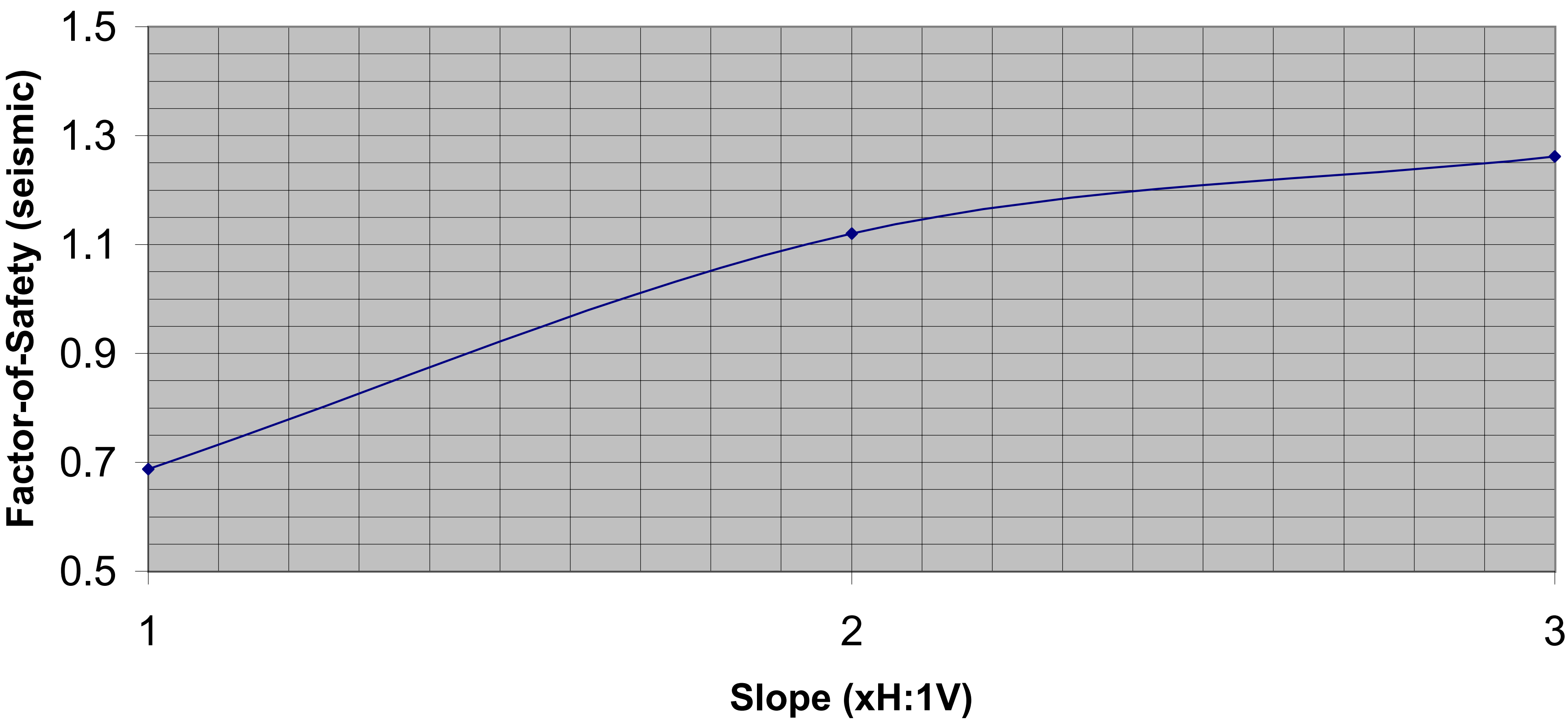
FIGURE N-3
VEGETATED EARTHEN SLOPE
SLOPE INCLINATION vs FACTOR OF SAFETY
FOR SLOPE HEIGHT OF 10 FEET AND N = 32 DEGREES



WESTON

ENGINEERING EVAULATION/COST ANALYSIS
Upper Reach of the Housatonic River
Pittsfield, Massachusetts

FIGURE N-4
VEGETATED EARTHEN SLOPE
SLOPE INCLINATION vs FACTOR OF SAFETY
FOR SLOPE HEIGHT OF 20 FEET AND N = 32 DEGREES



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ENGINEERING EVAULATION/COST ANALYSIS
Upper Reach of the Housatonic River
Pittsfield, Massachusetts

FIGURE N-5
VEGETATED EARTHEN SLOPE
SLOPE INCLINATION vs FACTOR OF SAFETY
FOR SLOPE HEIGHT OF 30 FEET AND N = 32 DEGREES

Table N-3

Calculated Minimum FS Values
for 2.25H:1V Slope Inclination
Long-Term Dynamic Stability for Vegetated Earthen Slope Scenario

Vertical Slope Height (H)	Minimum FS Value
10’	1.2
20’	1.2
30’	1.2

Table N-4

Calculated Minimum FS Values
For 2.25H:1V Slope Inclination
Short-Term Static Stability for Vegetated Earthen Slope Scenario

Vertical Slope Height (H)	With Surcharge	Without Surcharge
10’	1.3	1.3
20’	1.3	1.3
30’	1.4	1.5

Conclusion. Based on the results of the short-term and long-term static and long-term dynamic slope stability analyses discussed above, WESTON concludes that for the vegetated earthen slope scenario, for any vertical slope height encountered within the EE/CA Reach, the removal of riverbank soil contamination should be completed such that the inclination of the finished slope is no steeper than 2.25H:1V. This will provide minimum FSs of 1.5 and 1.0 for the long-term static and dynamic stability scenarios and an FS of 1.3 for the short-term static stability scenario with surcharge and 1.3 for the short-term static stability scenario without surcharge loads.

N.5 ARMORED EARTHEN SLOPE CONDITION

The lower riverbank slope from the toe of slope to a point approximately 10 ft vertically up the slope will be covered by an erosion protection layer. This erosion protection layer will be composed of gravel, riprap, or similar material sized to protect the riverbank from erosion, ice, and debris impact. For the purpose of the EE/CA it is assumed that this protective layer will consist of an 18-inch-thick blanket of riprap over a 6-inch-thick layer of sand and gravel bedding. The actual protective material size and composition may change pending results of the detailed design of the protective layer.

For this analysis assume the protective layer is composed of riprap having an in-place unit weight of 140 pcf, $\phi = 40°$, and minimal cohesive shear strength of 100 psf. The protective layer is 2 ft thick (total) and extends across the riverbed and up the riverbank for 10 vertical ft.

By trial and error it was determined that a 2H:1V slope inclination where the finished slope will be armored for the first 10 vertical feet of the riverbank and vegetated from that point to the top of the riverbank had an FS greater that 1.5 for the long-term static stability scenario. The calculated minimum FS values for the long-term static stability scenario are presented in Table N-5. Computer printouts for these runs are presented in Attachment N.2.

Table N-5

Calculated Minimum FS Values for Armored Earthen Slope Scenario
Long-Term Static Stability and 2H:1V Slope Inclination

Vertical Slope Height	FS Value
10’	1.6
20’	1.7
30’	1.6

The long-term dynamic and short-term static (with and without an excavator surcharge load) stability scenarios were subsequently analyzed for the selected 2H:1V maximum permissible slope inclination for the 10-, 20-, and 30-ft vertical slope heights under the armored earthen slope condition. The computer output data for these runs are presented in Attachment N.2. These printouts document the calculated minimum FS values and the geometry of the critical failure surface for each run. The calculated minimum FS values are summarized in Tables N-6 and N-7. As is evident from these tables, all FS values exceed the minimum required values of 1.0, 1.2, and 1.3 for the long-term dynamic and short-term static (with and without surcharge loads) stability scenarios, respectively.

Table N-6

Calculated Minimum FS Values for 2H:1V Slope Inclination
Long-Term Dynamic Stability for Armored Earthen Slope Scenario

Vertical Slope Height (H)	Minimum FS Value
10’	1.3
20’	1.3
30’	1.2

Table N-7

Calculated Minimum FS Values for 2H:1V Slope Inclination Short-Term Static Stability for Armored Earthen Slope Scenario

Vertical Slope Height (H)	With Surcharge	Without Surcharge
10’	1.6	1.6
20’	1.7	1.7
30’	1.5	1.6

Conclusion. Based on the results of the short-term and long-term static and long-term dynamic slope stability analyses discussed above, WESTON concludes that for the armored earthen slope scenario, for any vertical slope height encountered within the EE/CA Reach the removal of riverbank soil contamination should be completed such that the inclination of the finished slope is no steeper than 2H:1V. This will provide minimum FSs of 1.5 and 1.0 for the long-term static and dynamic stability scenarios and an FS of greater than 1.3 for the short-term static stability scenario with and without surcharge loads.

N.6 CONSTRUCTION CONDITION

In order to construct a 2H:1V armored earthen slope, short-term static (with and without a surcharge load) stability scenarios were analyzed for 10-, 20-, and 30-ft vertical slope heights under a bare earth slope condition. The computer output data for these runs are presented in Attachment N.3. These printouts document the calculated minimum FS values and the geometry of the critical failure surface for each run. The calculated minimum FS values are summarized in Table N-8.

Table N-8

Calculated Minimum FS Values for 2H:1V Slope Inclination Short-Term Static Stability for Armored Earthen Slope Scenario

Vertical Slope Height (H)	With Surcharge	Without Surcharge
10’	1.2	1.2
20’	1.2	1.2
30’	1.2	1.2

Conclusion. Based on the results of the short-term dynamic slope stability analyses discussed above, WESTON concludes that for the Construction Condition the removal of riverbank soil contamination should be completed such that the inclination of the slope is no steeper than 2H:1V. This will provide an FS of 1.3 for the short-term static stability scenario without surcharge loads.

N.7 WET EXCAVATION

The concept of wet excavation of the 3 ft of contaminated soil and the cutback of the slope face to the finished inclination under water (i.e., without first diverting the river water to create a dry channel condition) was evaluated.

The subsurface soils encountered during the geotechnical program were predominantly granular and therefore prone to erosion. During a removal action the existing vegetation on the slope face will be removed, exposing the underlying granular soils and erosion of the soils beneath the water surface may occur. This may, in turn, steepen this portion of the slope face as shown in Figure N-6 in the absence of engineering controls. This process could continue and eventually undermine the toe of slope. A slope instability with a failure mass that emerges along this steepened slope face could then result.

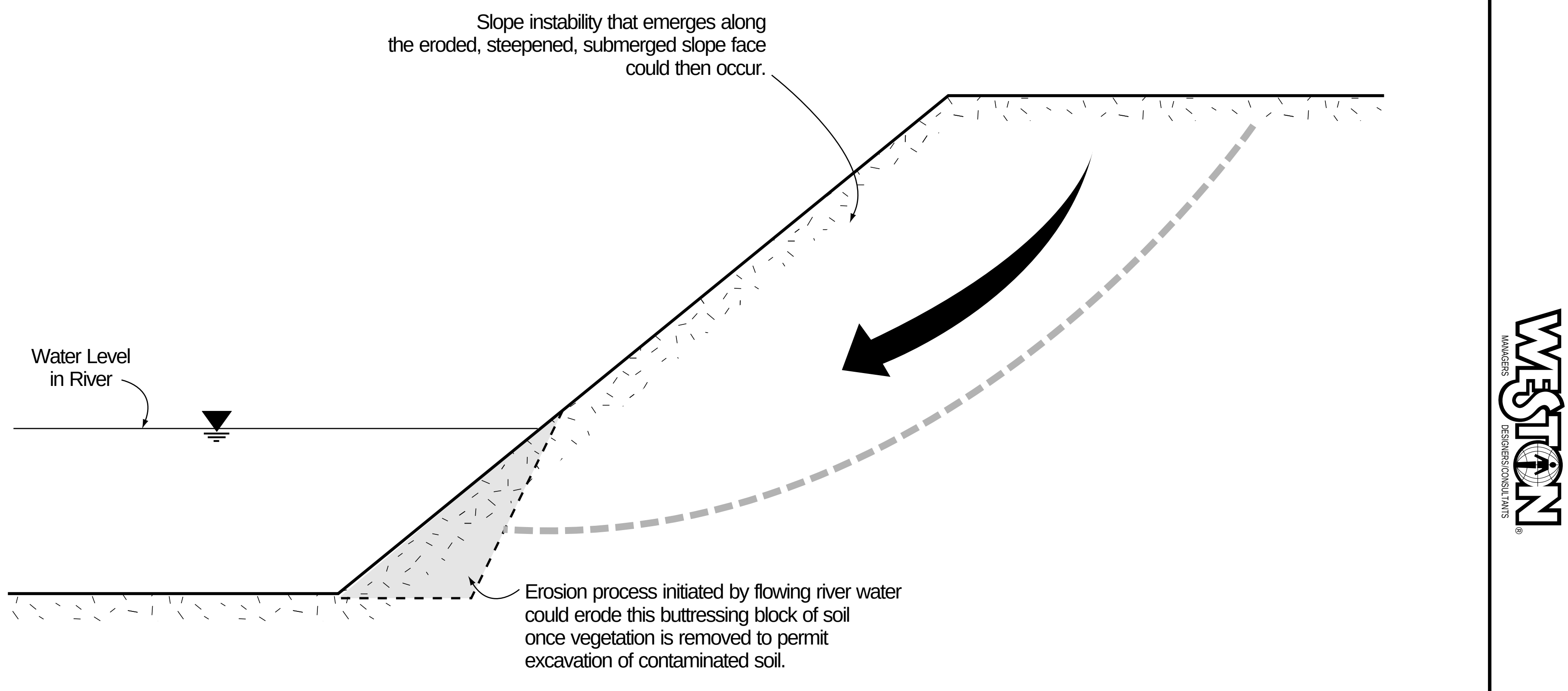
Close monitoring during the removal action of the wet toe of slope is necessary to ensure that this potential failure scenario does not occur. Should the slope begin to steepen beyond a 2H:1V slope the removal contractor must immediately stabilize the slope and begin corrective action to control the slope instability in order to continue the removal.

N.8 DETERMINATION OF RIVERBANK FINISHED GRADES

The finished cut slope grades for the riverbanks after removal was determined using the results of the above-mentioned analyses as outlined in the flow chart (see Figure N-7). Following the flow chart, the existing slope of the riverbank was determined at 50-ft intervals using computer-generated cross sections. If the slope is not steeper than 2H:1V the riverbank is excavated to the removal criteria and backfilled to the approximate original grade.

If the slope is 2H:1V or steeper, a finished grade is projected on the cross section at 2H:1V while maintaining the hydraulic capacity of the channel cross section. The projected top of slope is compared to the existing top of slope. If the projected top of slope approximates the location of the existing top of slope, the riverbank is excavated to the removal criteria and backfilled to the 2H:1V finished grade.

If the projected top of slope approaches existing structures, infrastructure, or “pushes” the top of slope more than 5 ft back, earth-retaining structures are incorporated into the riverbank. The purpose of the earth-retaining structures is to avoid impact to existing structures, infrastructure, and property. The riverbank is excavated to the removal criteria and backfilled to the proposed finished grade and retaining structures.



LEGEND

Water Level

Native Soil

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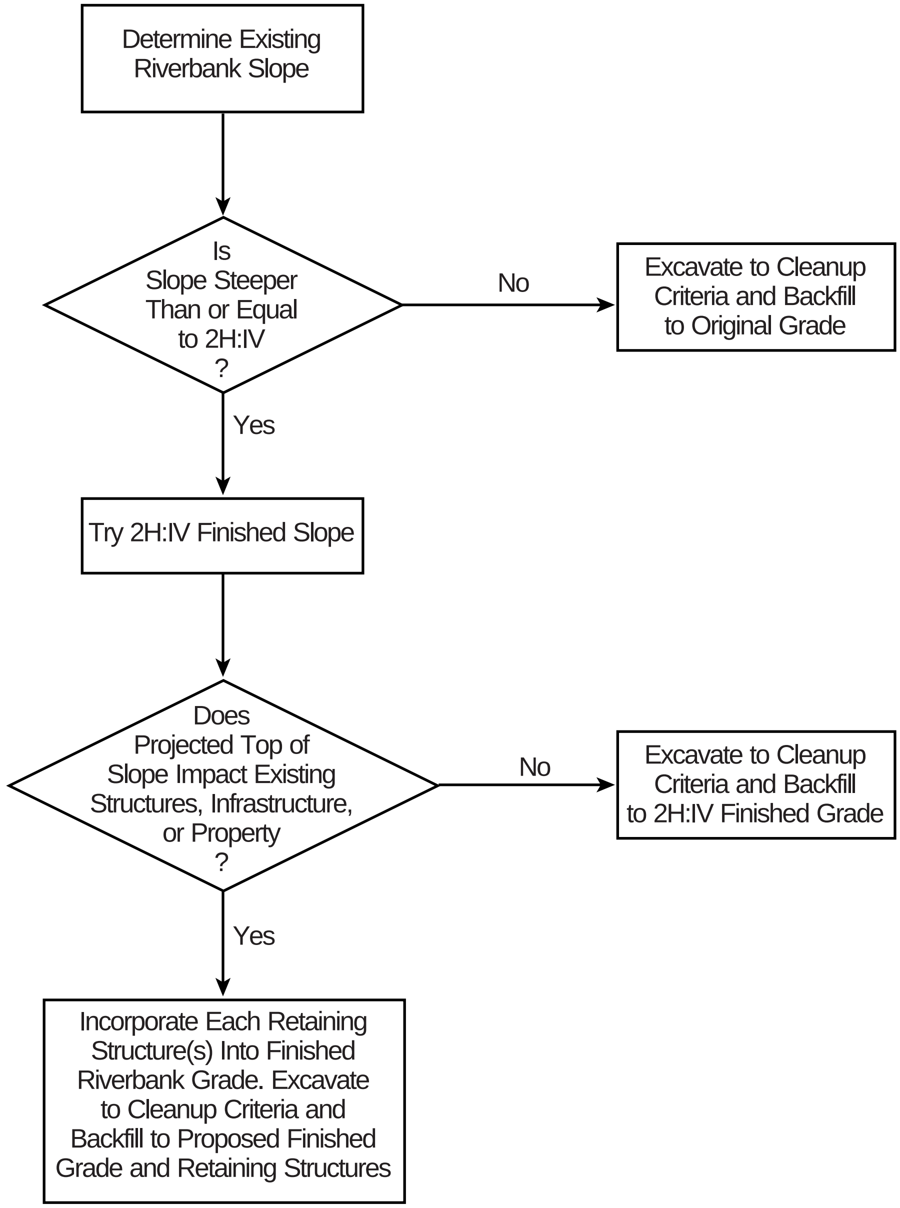
Upper Reach of the Housatonic River
Pittsfield, Massachusetts

FIGURE N-6
SCHEMATIC OF WET EXCAVATION
SLOPE INSTABILITY

99P-2091



ENGINEERING EVALUATION/COST ANALYSIS
Upper Reach of the Housatonic River
Pittsfield, Massachusetts

FIGURE N-7
FLOW CHART FOR RIVERBANK GRADING

N.9 EARTH-RETAINING STRUCTURES

WESTON's analysis of the river channel cross sections has determined the approximate location and height of retaining structures needed to stabilize the riverbanks following the removal action. These retaining structures typically will be less than 10 ft in height (exposed face). The retaining structure would typically be constructed near the toe of the bank as illustrated in Appendix L (Figure L-3), but it can be positioned at other locations along the slope length. In a few instances the retaining structures will be required in front of existing deteriorating retaining structures (e.g., old rotting timber walls) to reinforce these old walls and protect top-of-slope structures. Attachment N.4 summarizes those plan locations by project stations on both sides of the river where retaining structures will be required (reference Figures 2.6-1A-D of the EE/CA Report for retaining structure locations).

Technically feasible retaining structures include sheetpile walls, cement walls, cement stone walls, metal bin retaining walls, gabion walls, and modular block walls. Gravity-type retaining structures have a significant depth dimension (typically 5 to 16 ft depending on the wall height). Their installation on the slope will require horizontal benching of the slope to create a level subgrade surface on which the wall can be constructed (see Figure N-8).

- **Steel Sheetpile Walls** are installed using a pile-driving rig (i.e., a crane supporting vertical pile leads). Sheetpiling can be painted with corrosion and weather-resistant paint or can be purchased with a vinyl coating for aesthetic purposes.

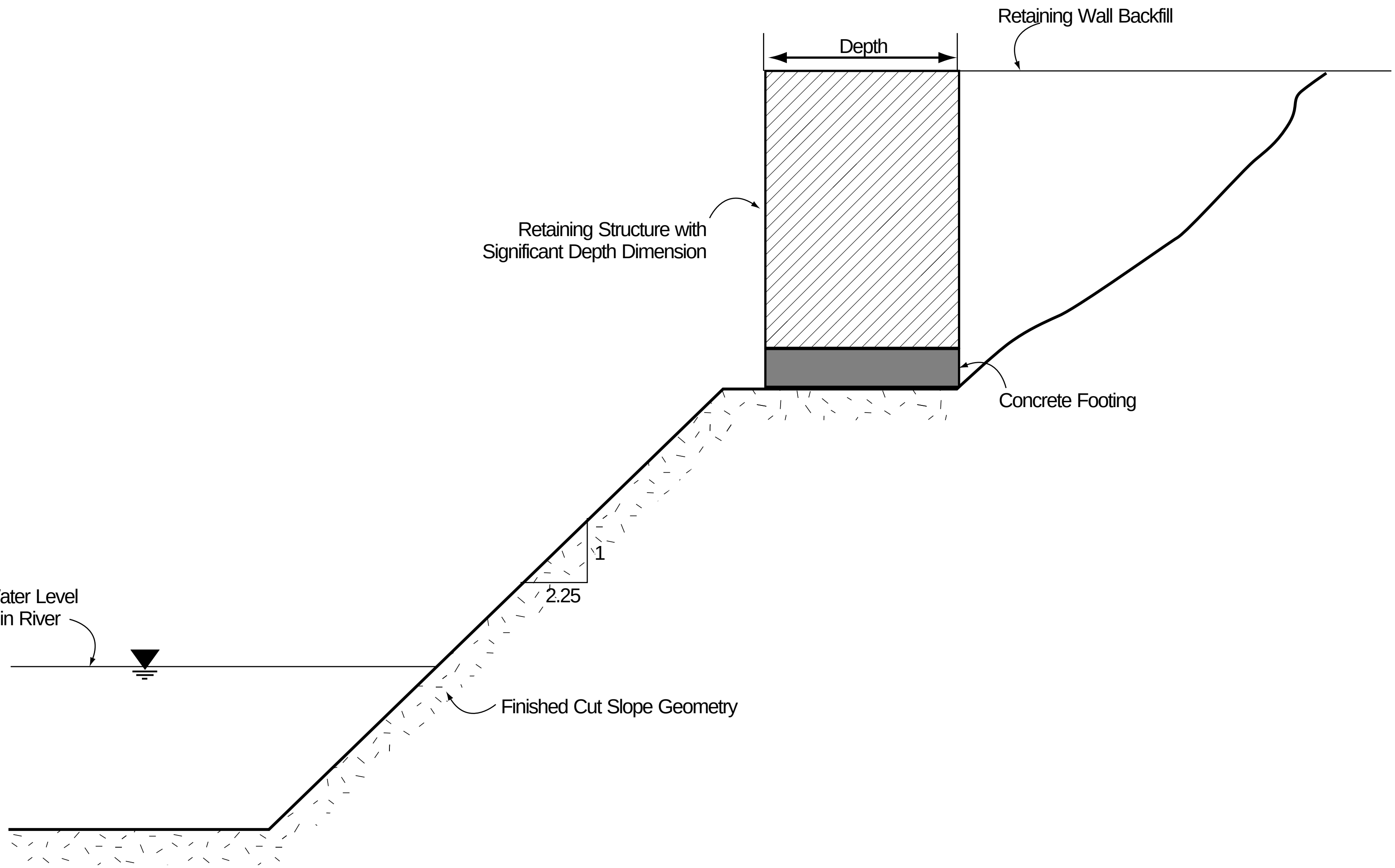
- **Concrete Walls** are gravity-type walls constructed of cement concrete.

- **Cement Stone Walls** (or masonry walls) are gravity-type walls constructed of stone and cement mortar.

- **Metal Bin Retaining Walls** are a system of adjoining four-sided, closed-face, lightweight steel bins that are bolted together at the project site. They are subsequently backfilled with granular soil to provide sufficient weight for the structure to act as a gravity retaining wall. Because the face of the bin wall is fully enclosed, loss of fill material from within the bins is prevented. In addition, the bins have sufficient flexibility so that they are not normally affected by minor ground settlements.

- **Gabion Walls** are constructed of woven steel wire baskets filled with stone and tied together with steel rings or wire. The gabion wall is a gravity-type structure.

- **Modular Block Walls** are constructed of interconnected concrete blocks and reinforced soil behind the wall. Together the blocks and soil act as a gravity wall.



LEGEND

Water Level

Native Soil

ENGINEERING EVALUATION/COST ANALYSIS
Upper Reach of the Housatonic River
Pittsfield, Massachusetts

FIGURE N-8
CONCEPT OF BENCHING FINISHED SLOPE TO PERMIT CONSTRUCTION OF RETAINING STRUCTURE WITH DEPTH

99P-2944k

N.10 EROSION PROTECTION OF FINISHED CUT SLOPE FACE

The finished slope face of the riverbanks will require protection against potential erosion from river flow, ice, and debris impact. This protection will consist of appropriately sized riprap (stone), concrete revetments (articulated or hand placed), or similar material installed from the toe of slope to an elevation approximately 10 ft above the toe of slope. At higher elevations along the remainder of the slope face, an erosion resistant, deep-rooted vegetative cover may be used to provide veneer protection for rainfall that directly contacts the slope.

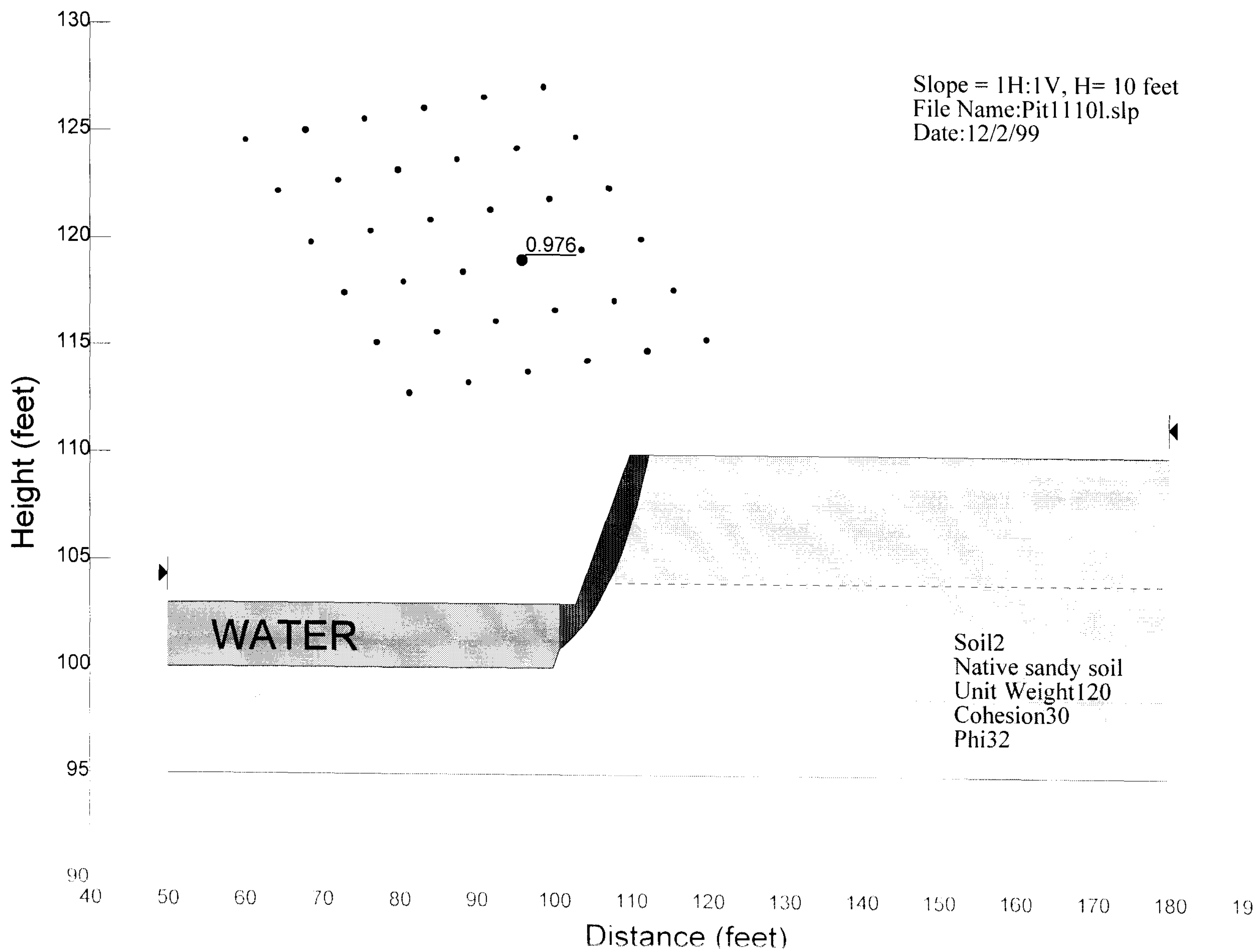
N.11 LIMITATION OF THIS ANALYSIS

The bank stability analysis presented here is preliminary and is intended for use in the EE/CA only. A detailed geotechnical investigation and design program should be undertaken to determine the final stability requirements and design of retaining structures. Furthermore, placement of a gravity-type retaining structure mid-slope will act as a surcharge on the slope, which may affect slope stability. This condition was not evaluated as part of this analysis.

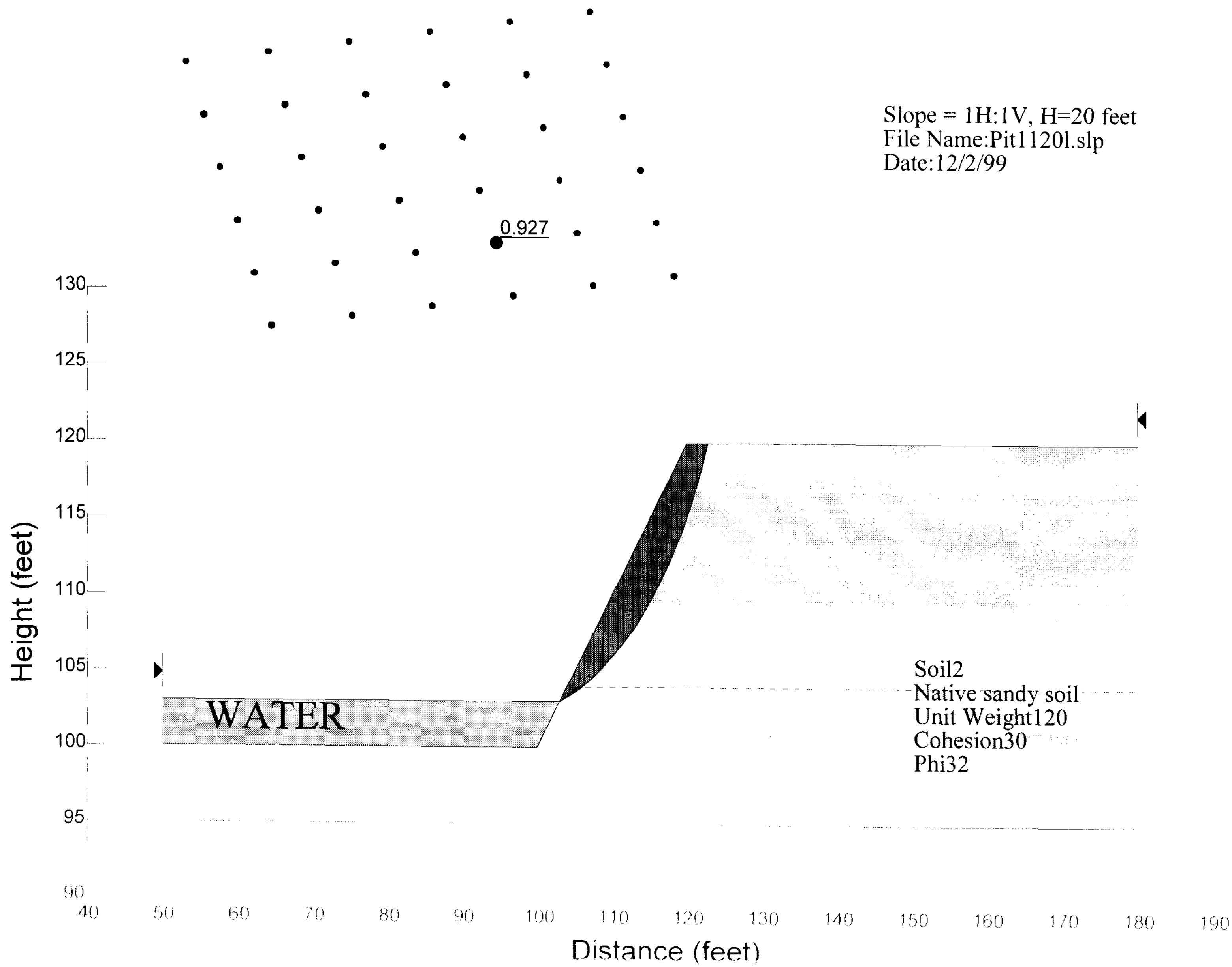
ATTACHMENT N.1

STABILITY ANALYSES FOR VEGETATED EARTHEN SLOPE CONDITION

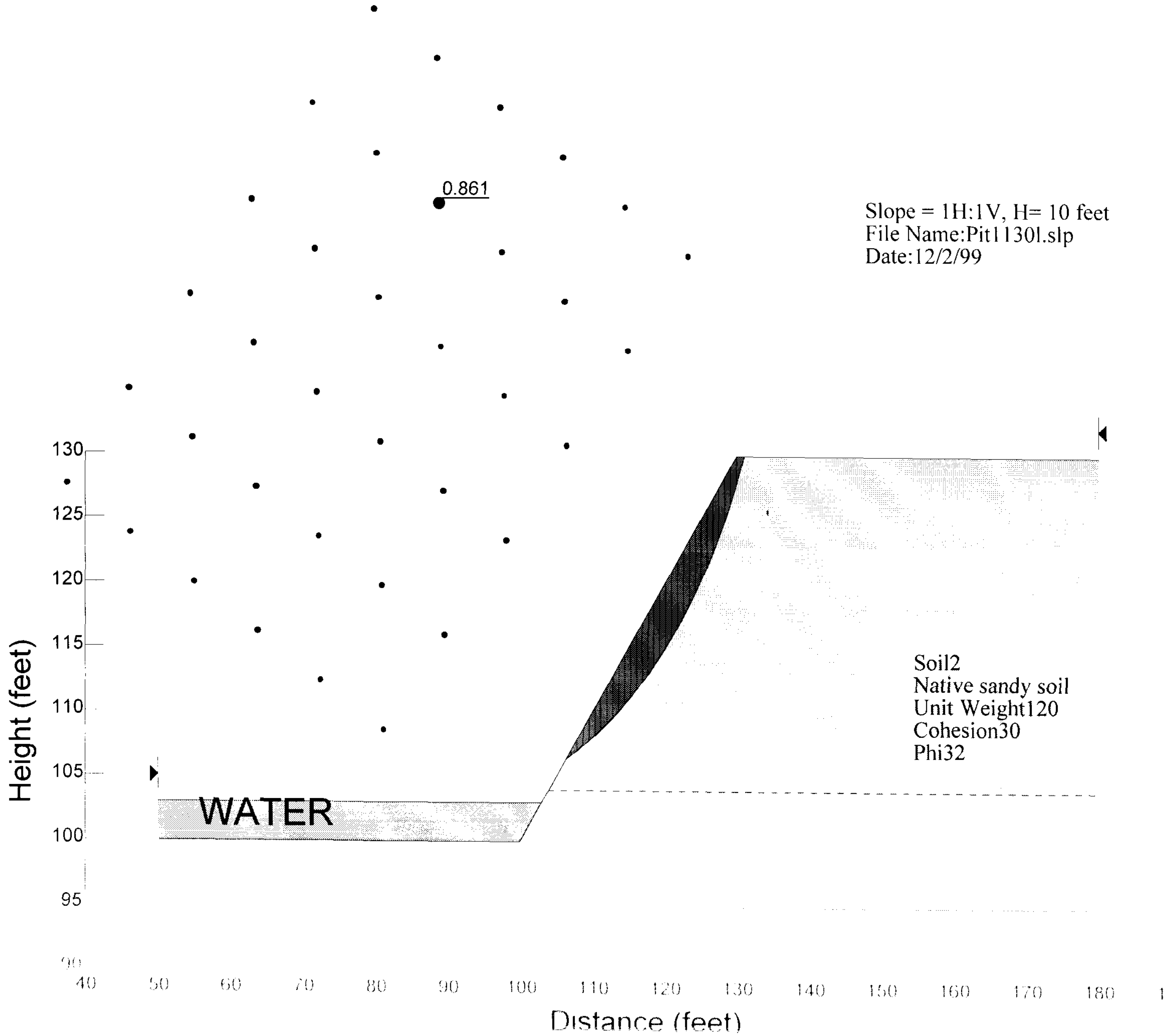
PITTSFIELD SLOPE STABILITY EVALUATION



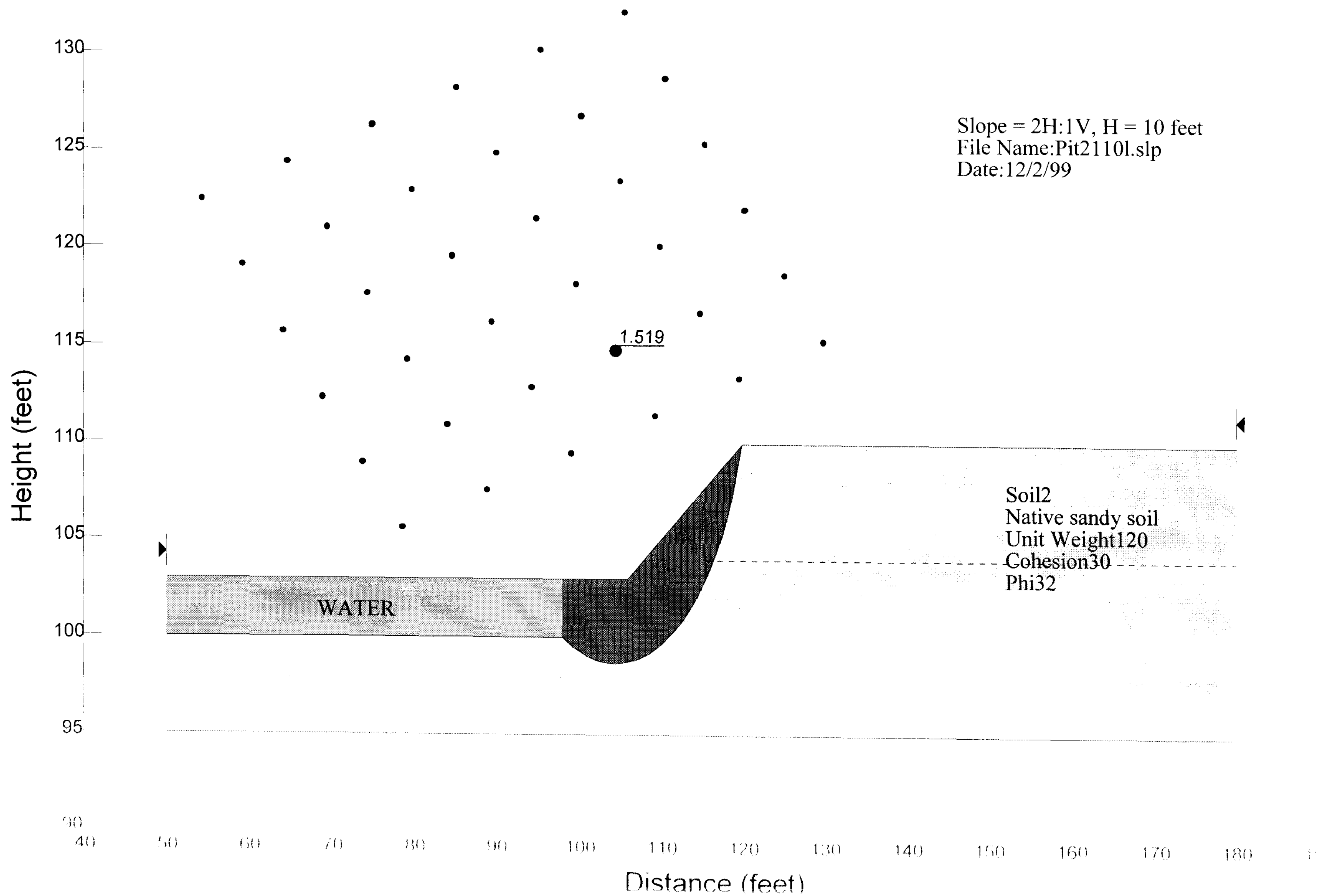
PITTSFIELD SLOPE STABILITY EVALUATION



PITTSFIELD SLOPE STABILITY EVALUATION



PITTSFIELD SLOPE STABILITY EVALUATION



PITTSFIELD SLOPE STABILITY EVALUATION

Slope = 2H:1V, H = 20 feet
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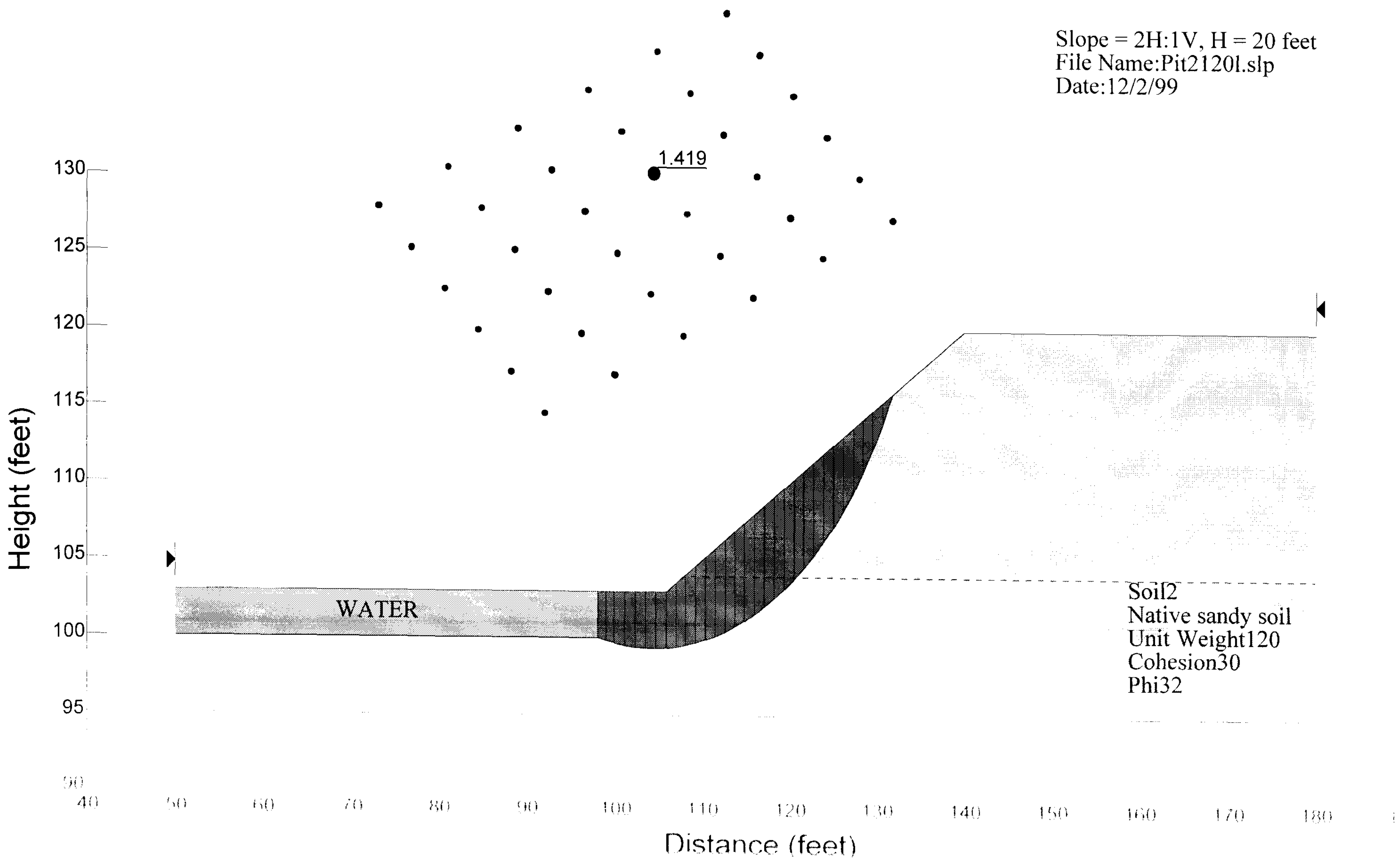
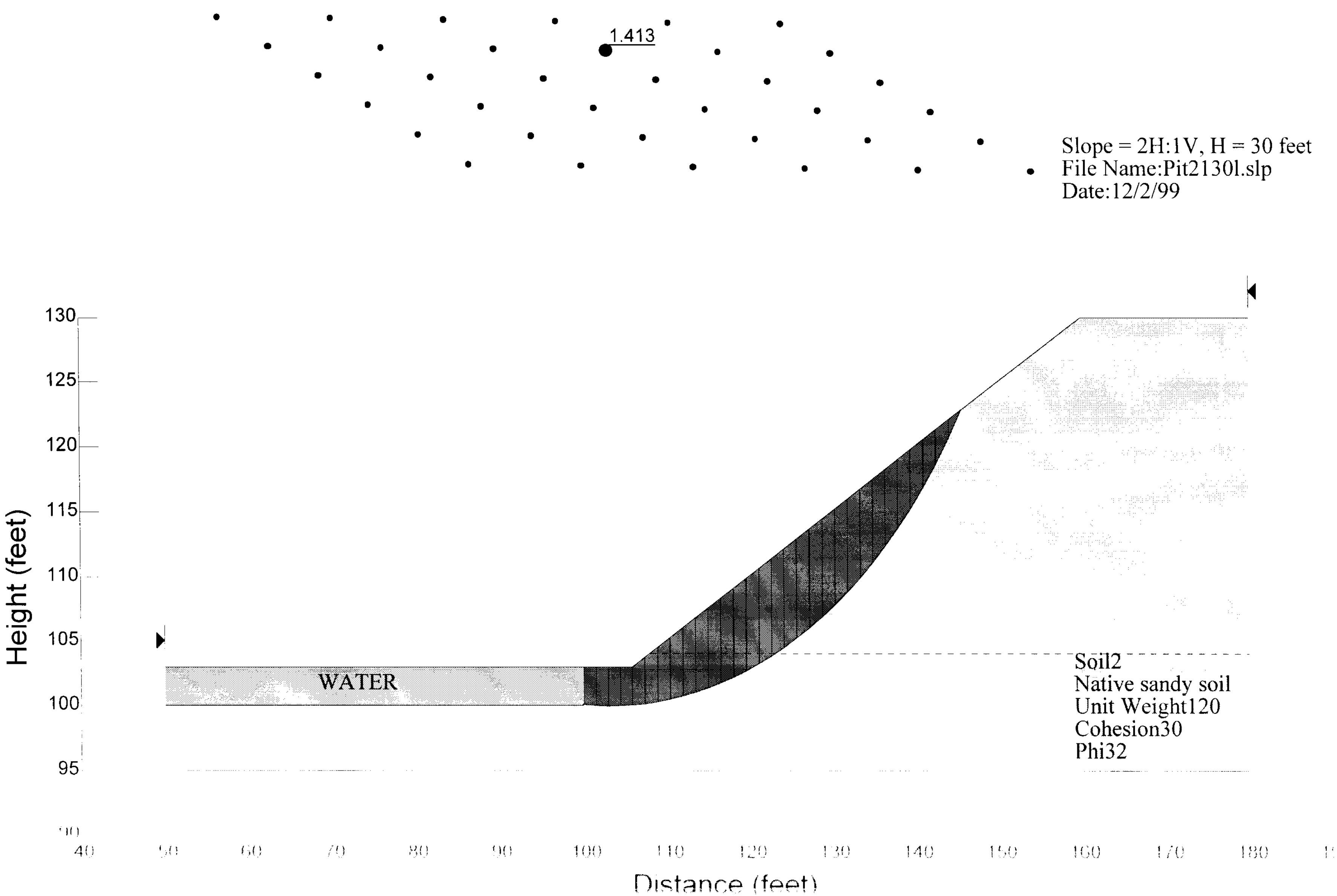
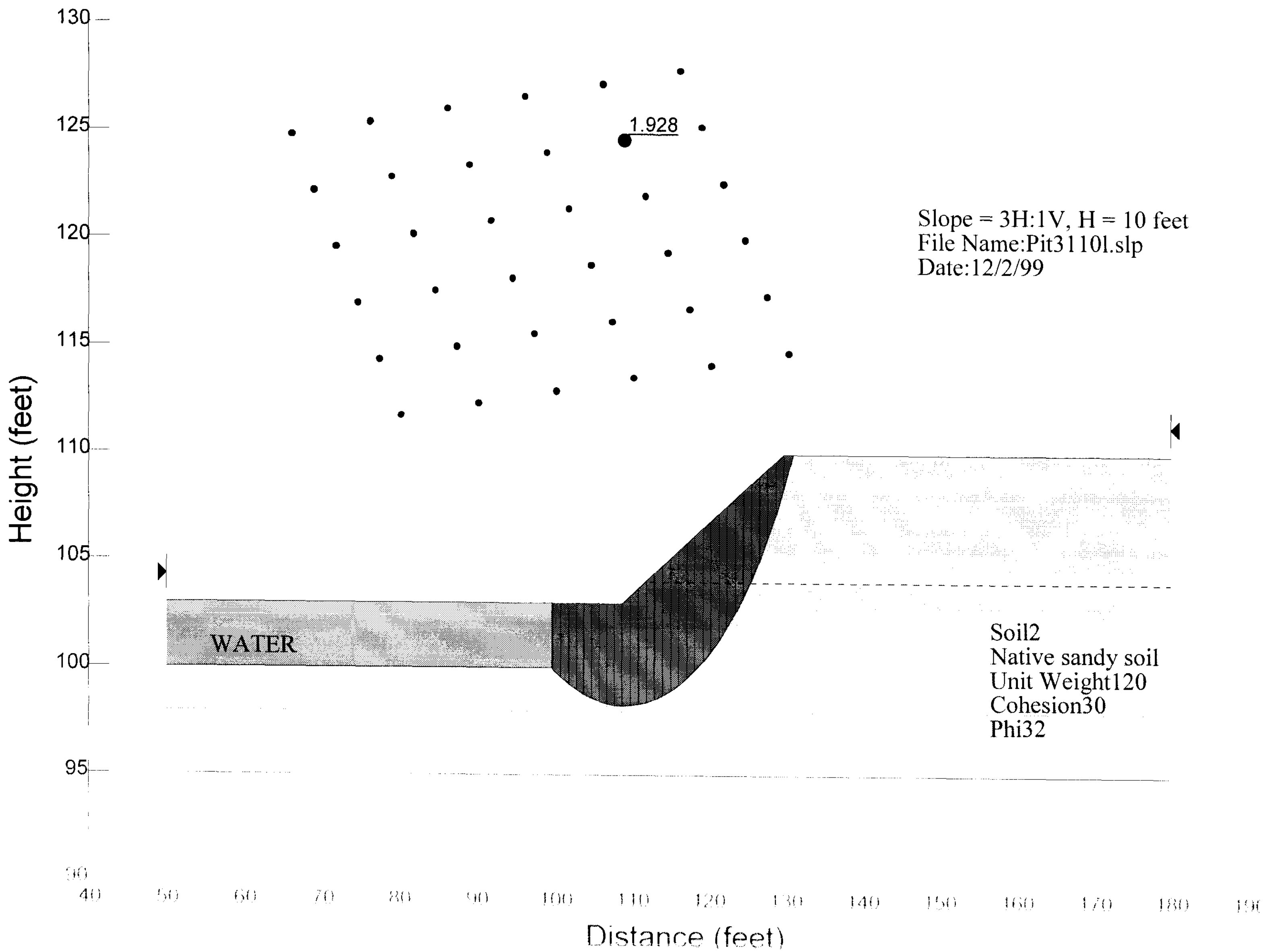
Date:12/2/99



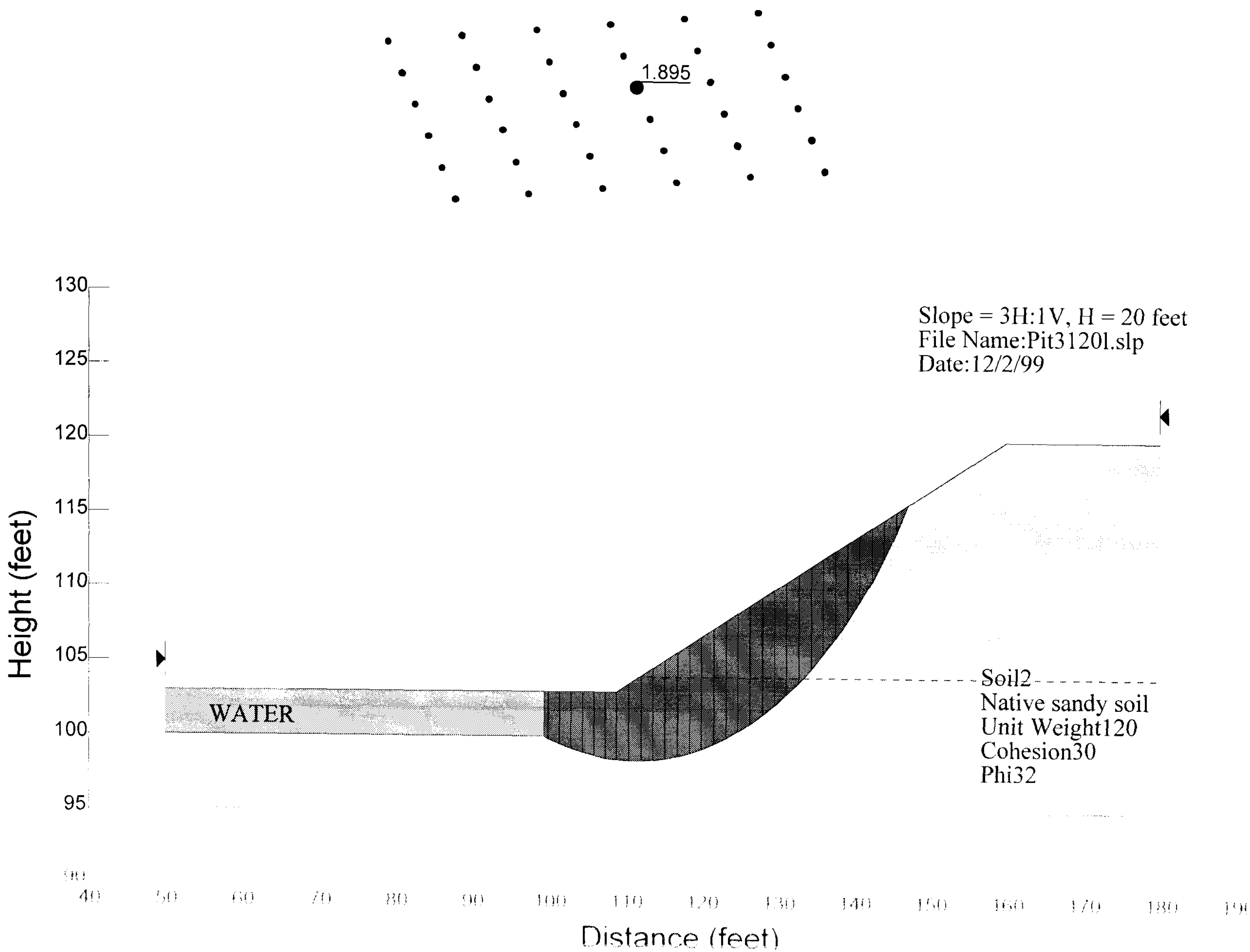
PITTSFIELD SLOPE STABILITY EVALUATION



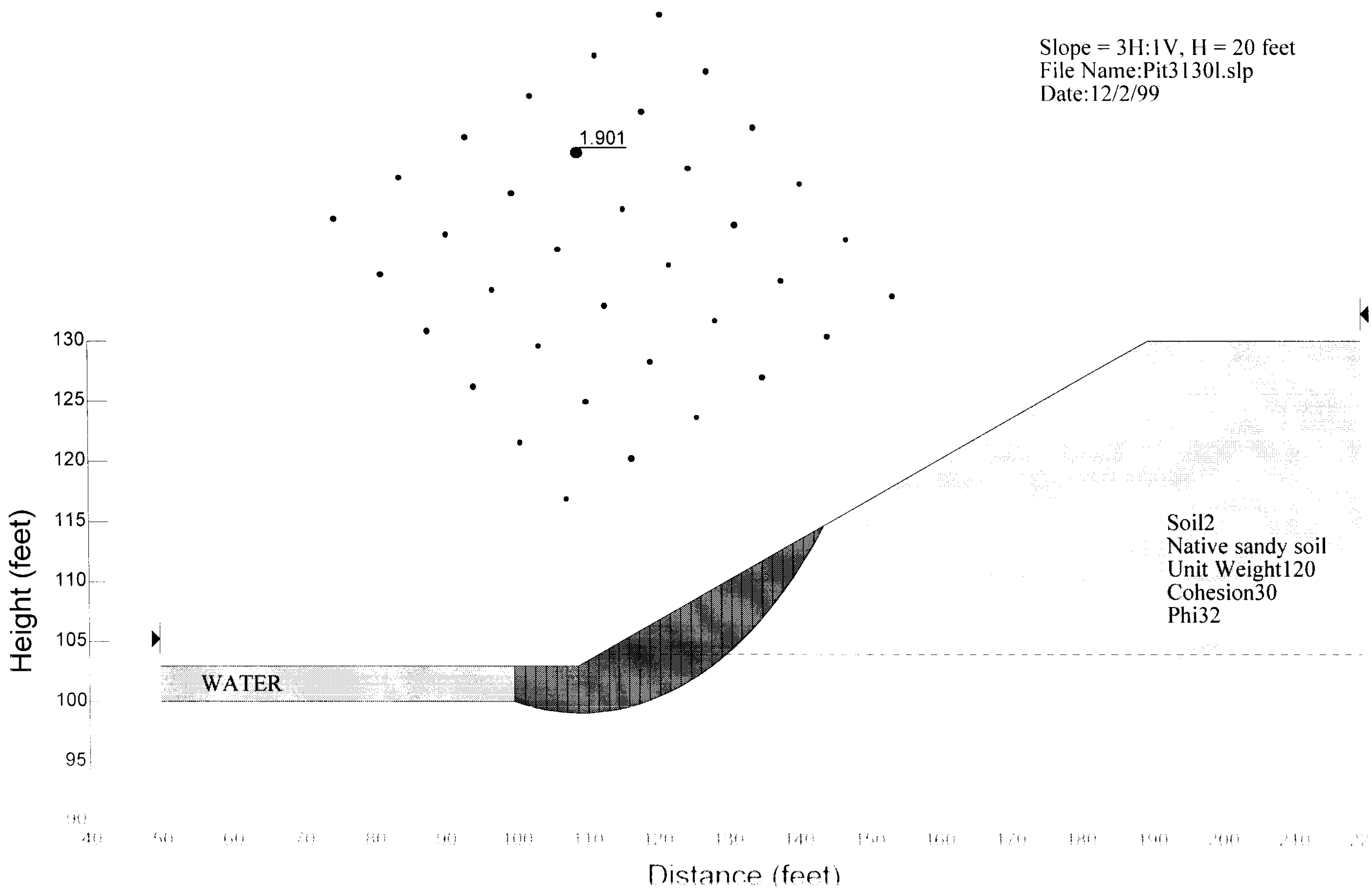
PITTSFIELD SLOPE STABILITY EVALUATION

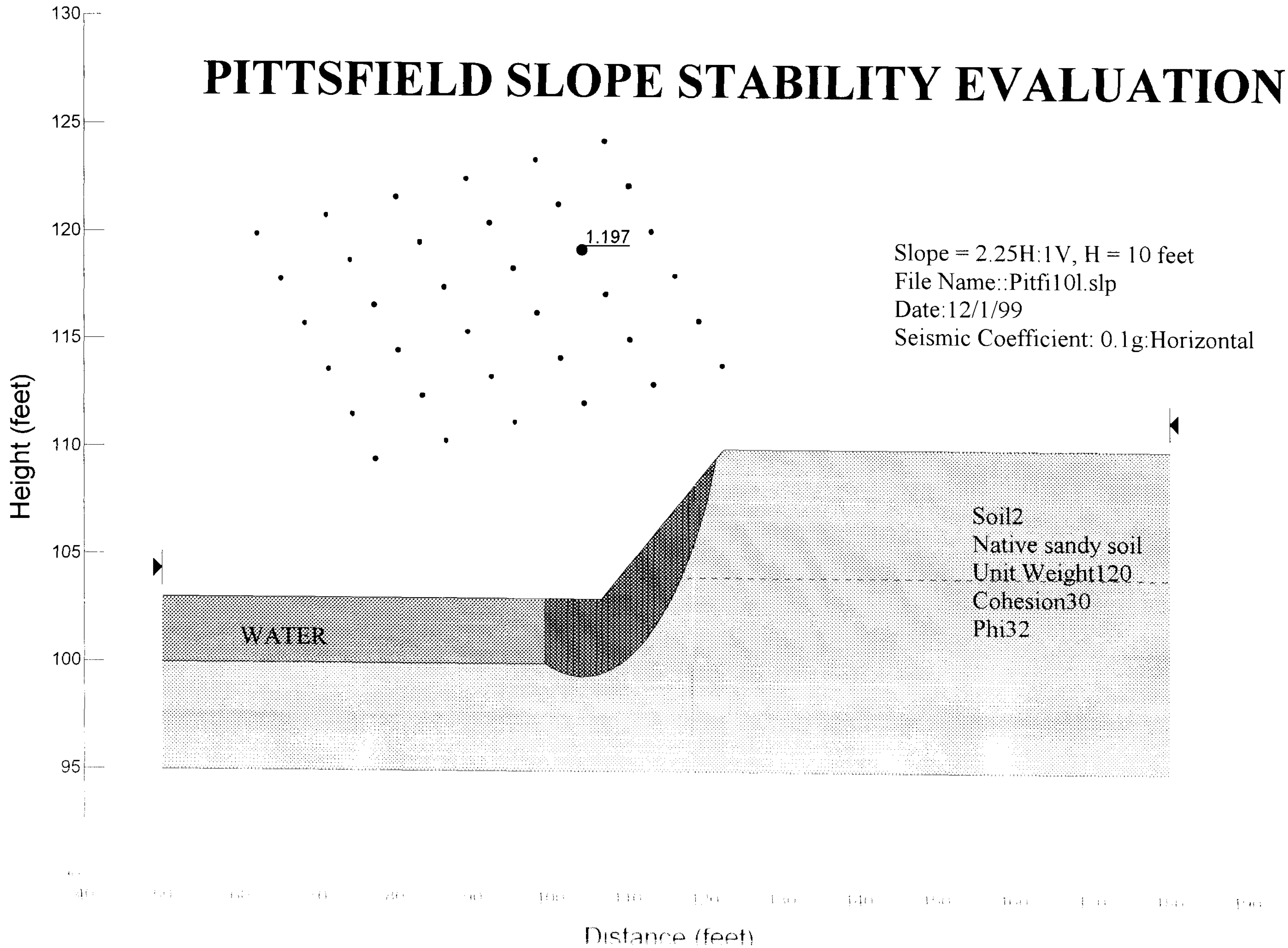


PITTSFIELD SLOPE STABILITY EVALUATION



PITTSFIELD SLOPE STABILITY EVALUATION




PITTSFIELD SLOPE STABILITY EVALUATION
1.197
Slope = 2.25H:1V, H = 10 feet
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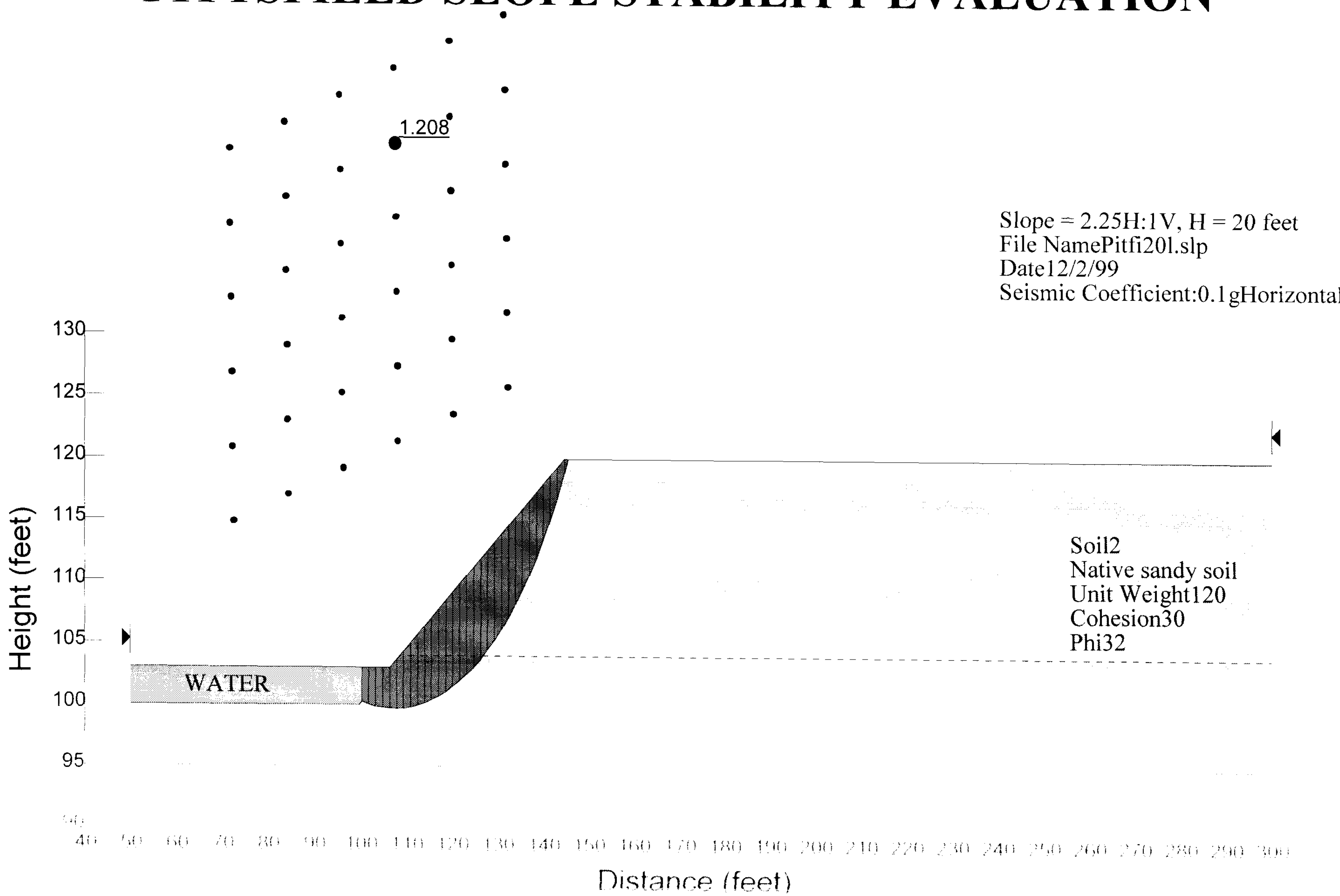
Date:12/1/99
Seismic Coefficient: 0.1g:Horizontal
Soil2
Native sandy soil
Unit Weight120
Cohesion30
Phi32
WATER
130
125
120
115
110
105
100
95
Height (feet)
Distance (feet)

PITTSFIELD SLOPE STABILITY EVALUATION



PITTSFIELD SLOPE STABILITY EVALUATION

Slope = 2.25H:1V, H = 30 feet
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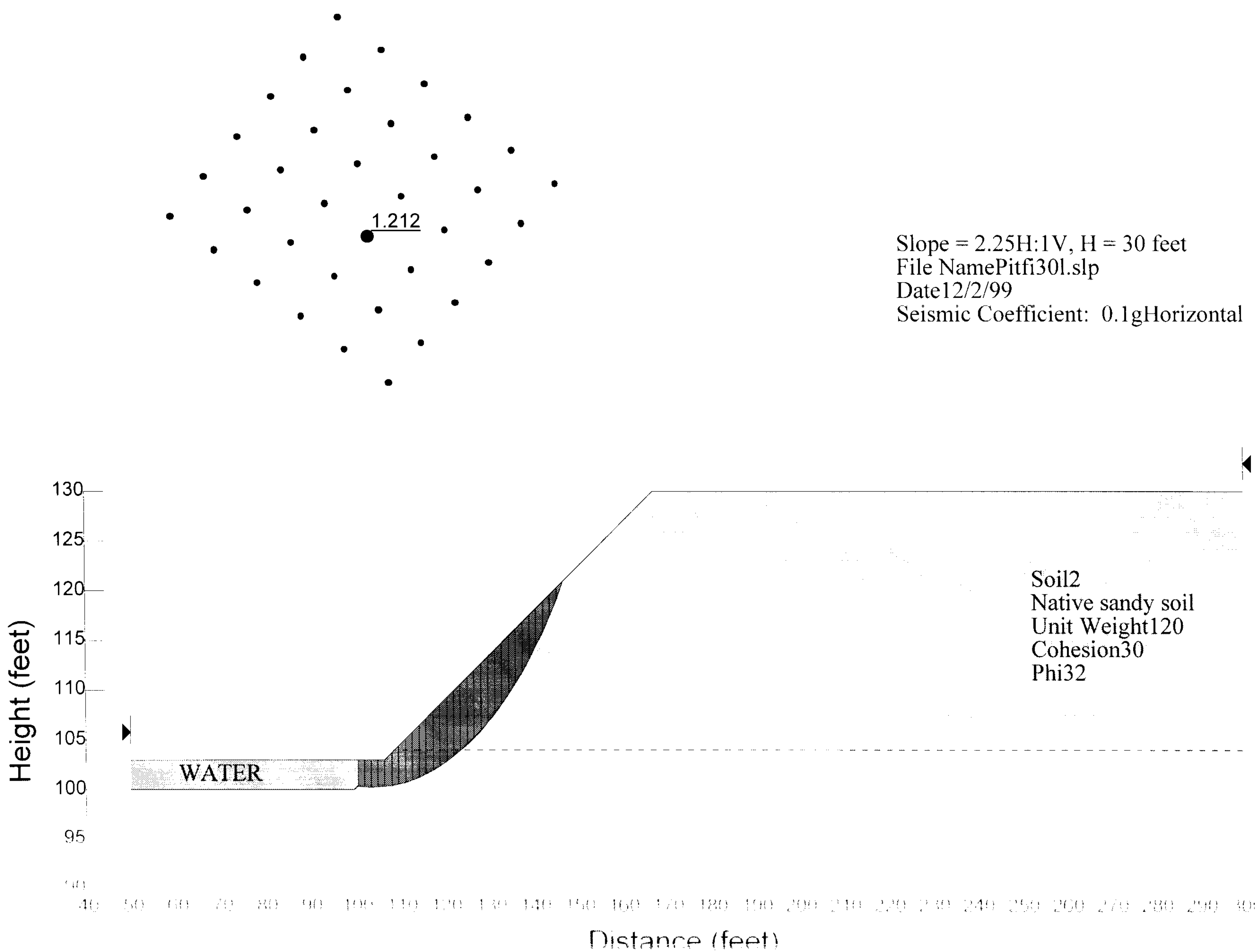
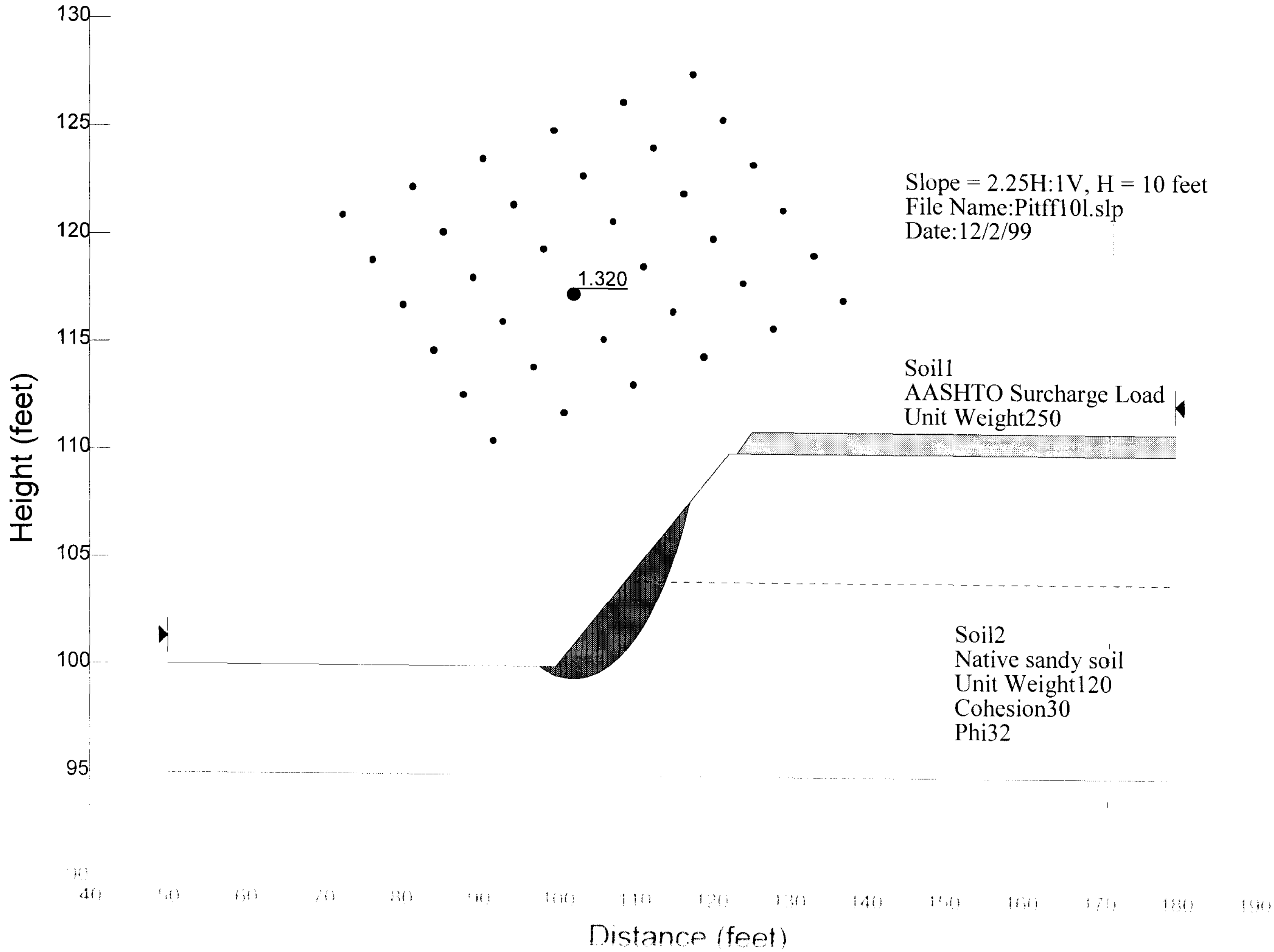
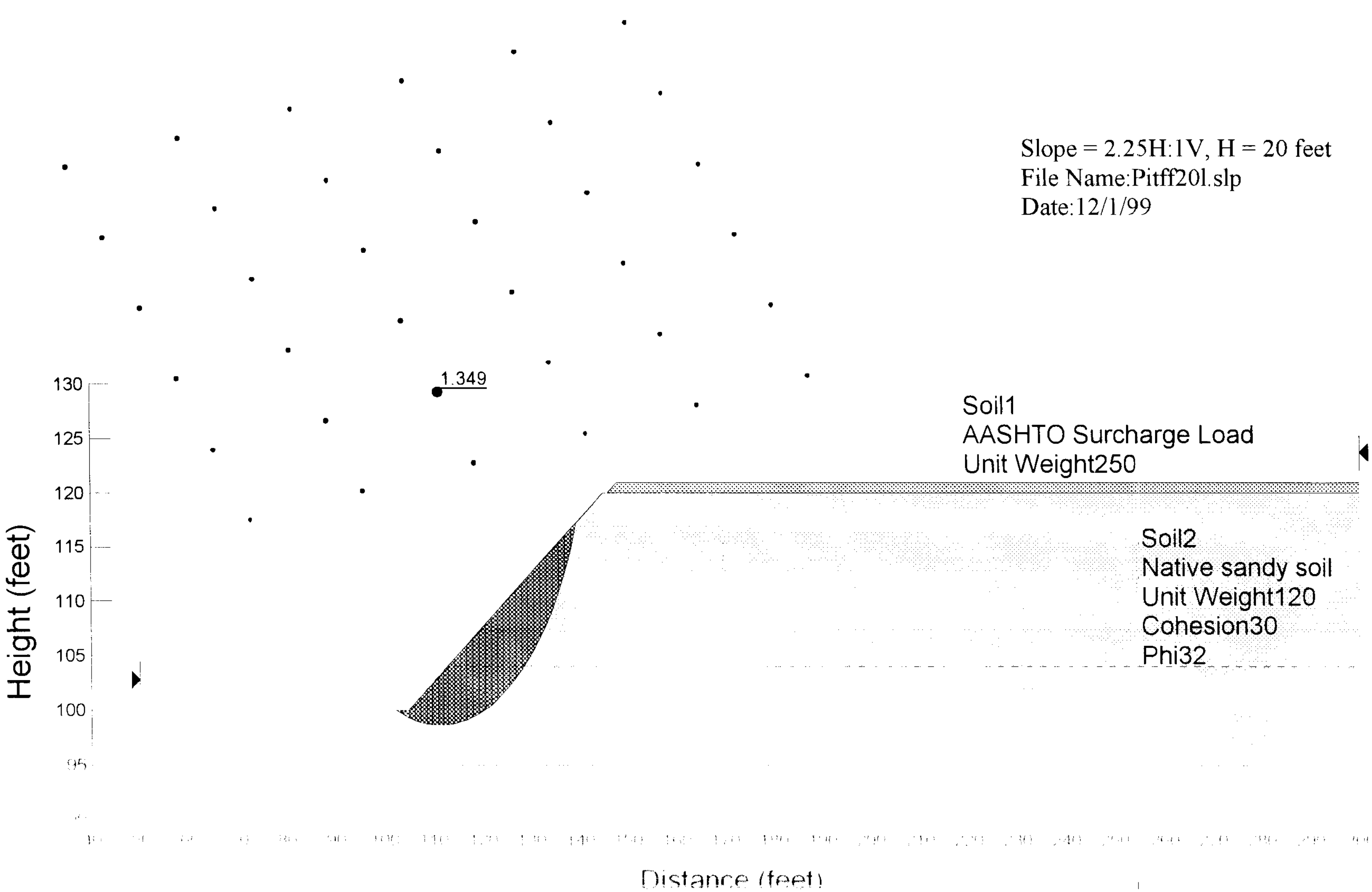
Date12/2/99
Seismic Coefficient: 0.1gHorizontal



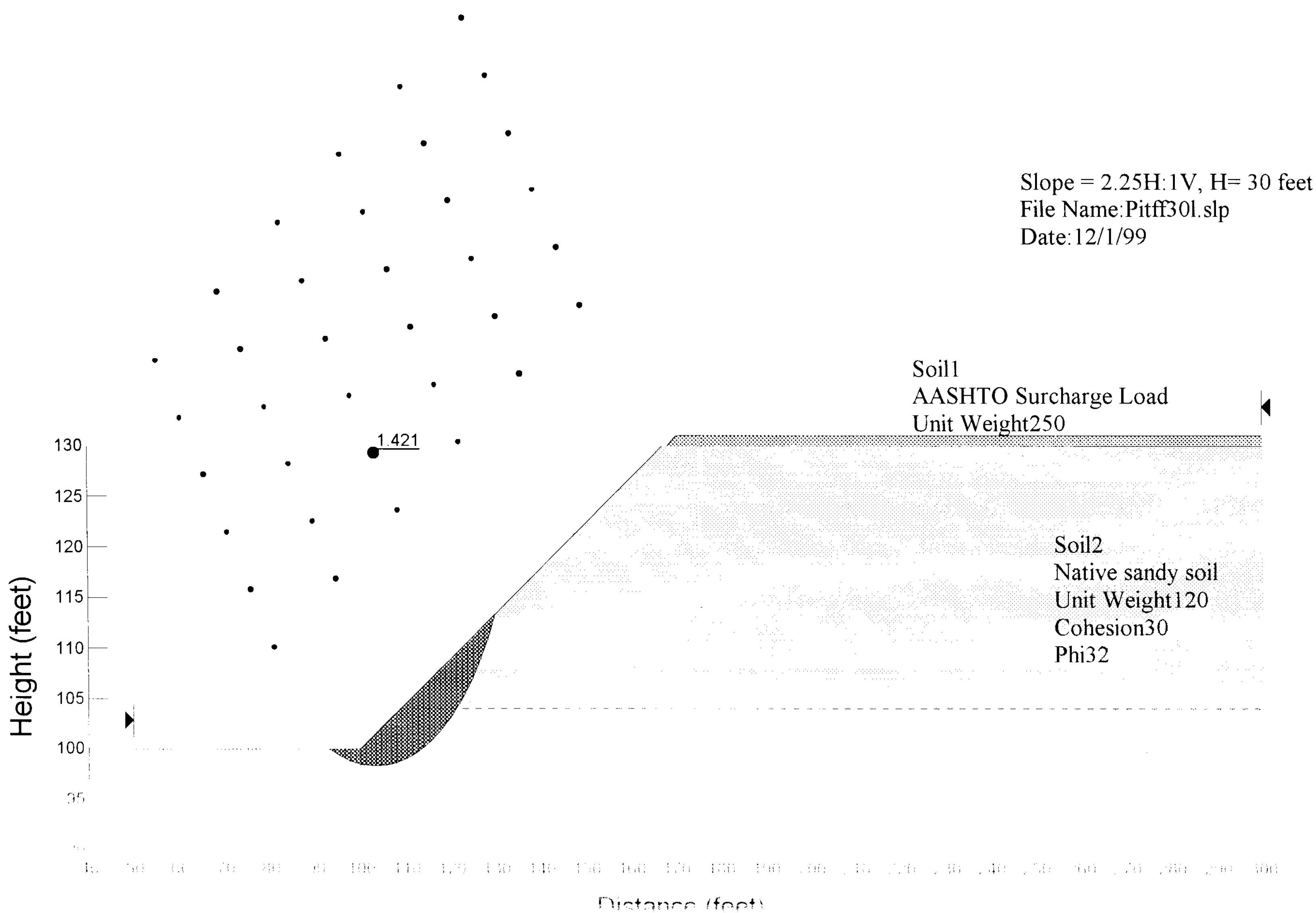
PITTSFIELD SLOPE STABILITY EVALUATION



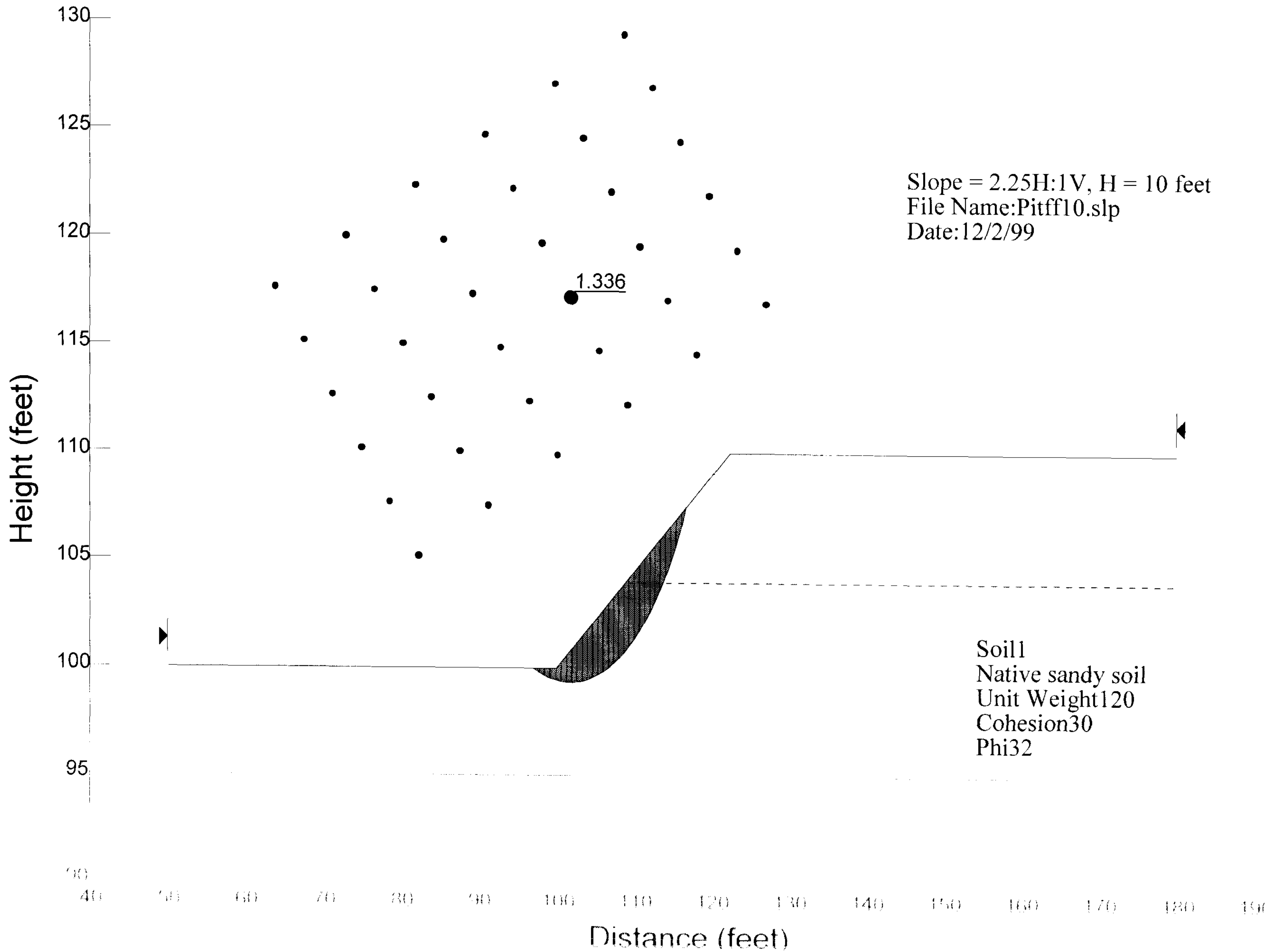
PITTSFIELD SLOPE STABILITY EVALUATION



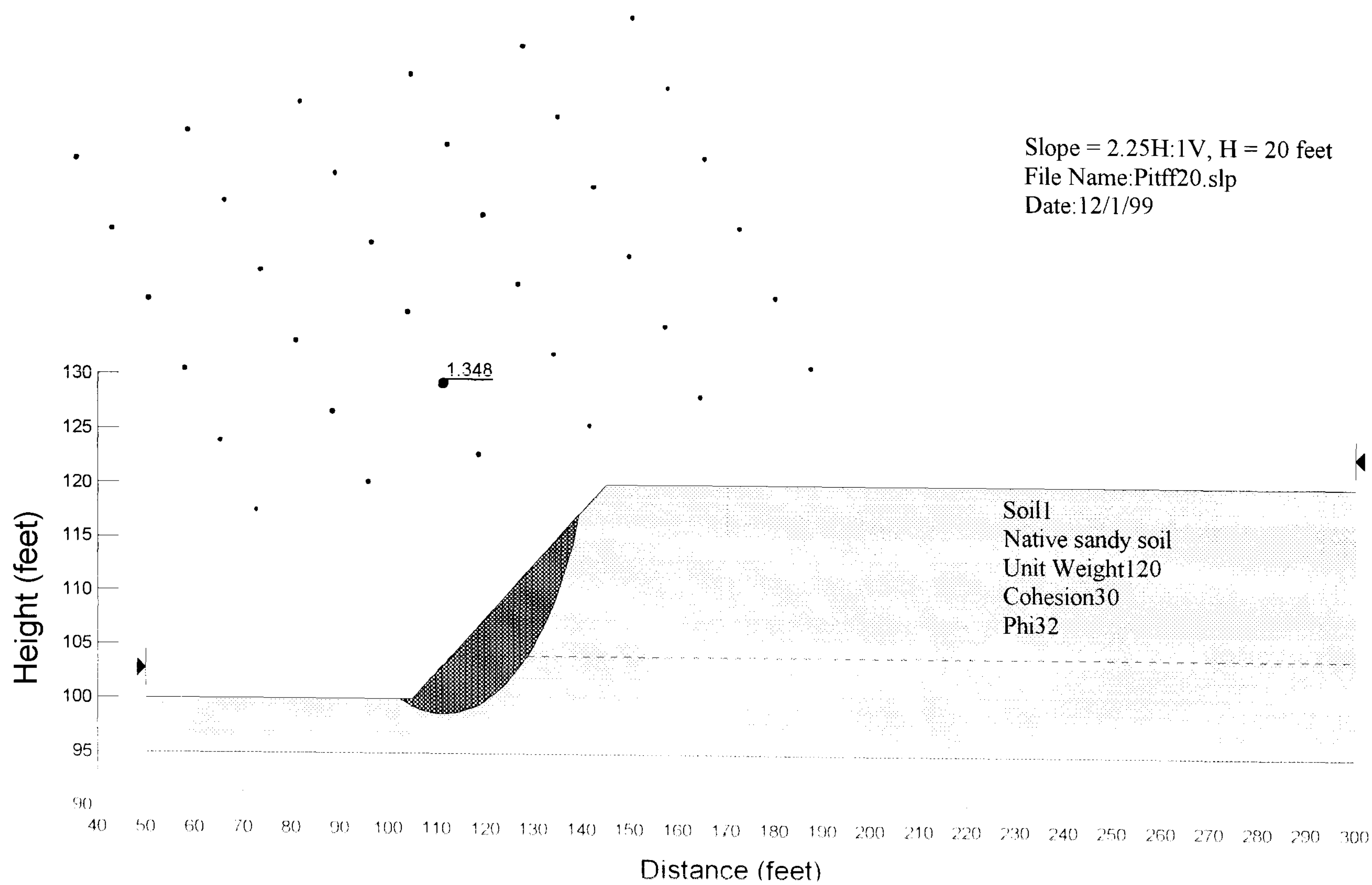
PITTSFIELD SLOPE STABILITY EVALUATION



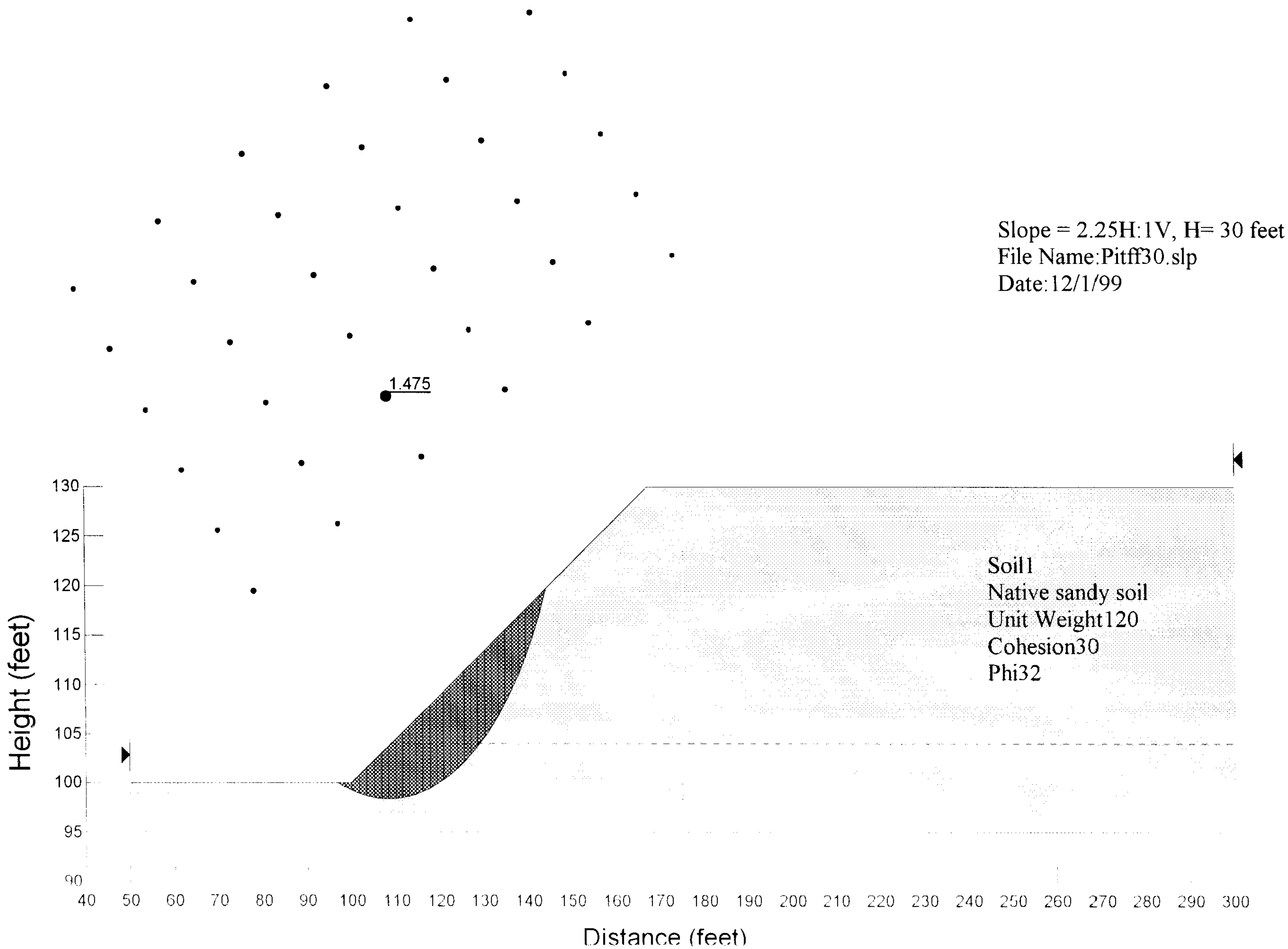
PITTSFIELD SLOPE STABILITY EVALUATION



PITTSFIELD SLOPE STABILITY EVALUATION



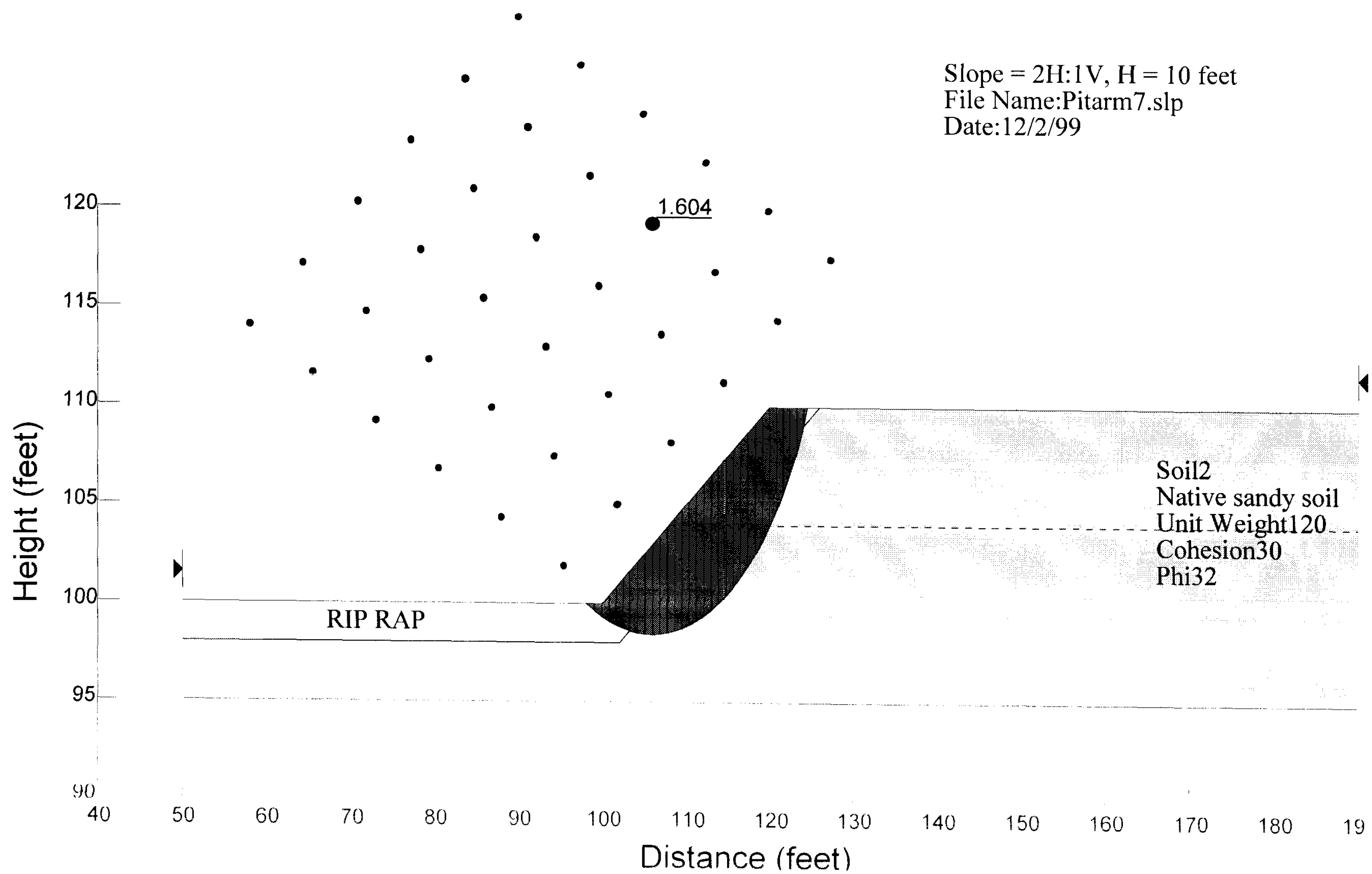
PITTSFIELD SLOPE STABILITY EVALUATION



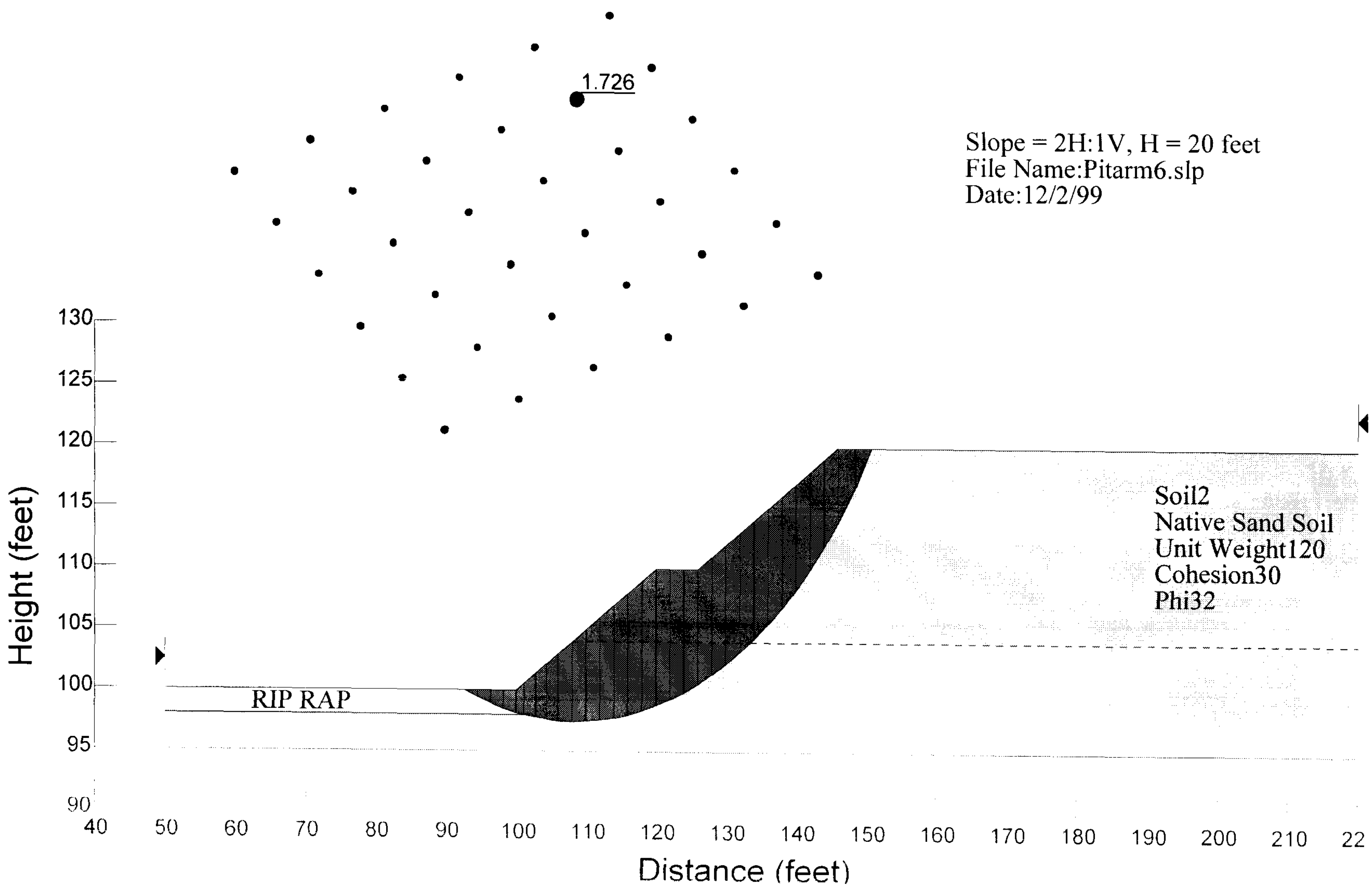
ATTACHMENT N.2

STABILITY ANALYSES FOR ARMORED EARTHEN SLOPE CONDITION

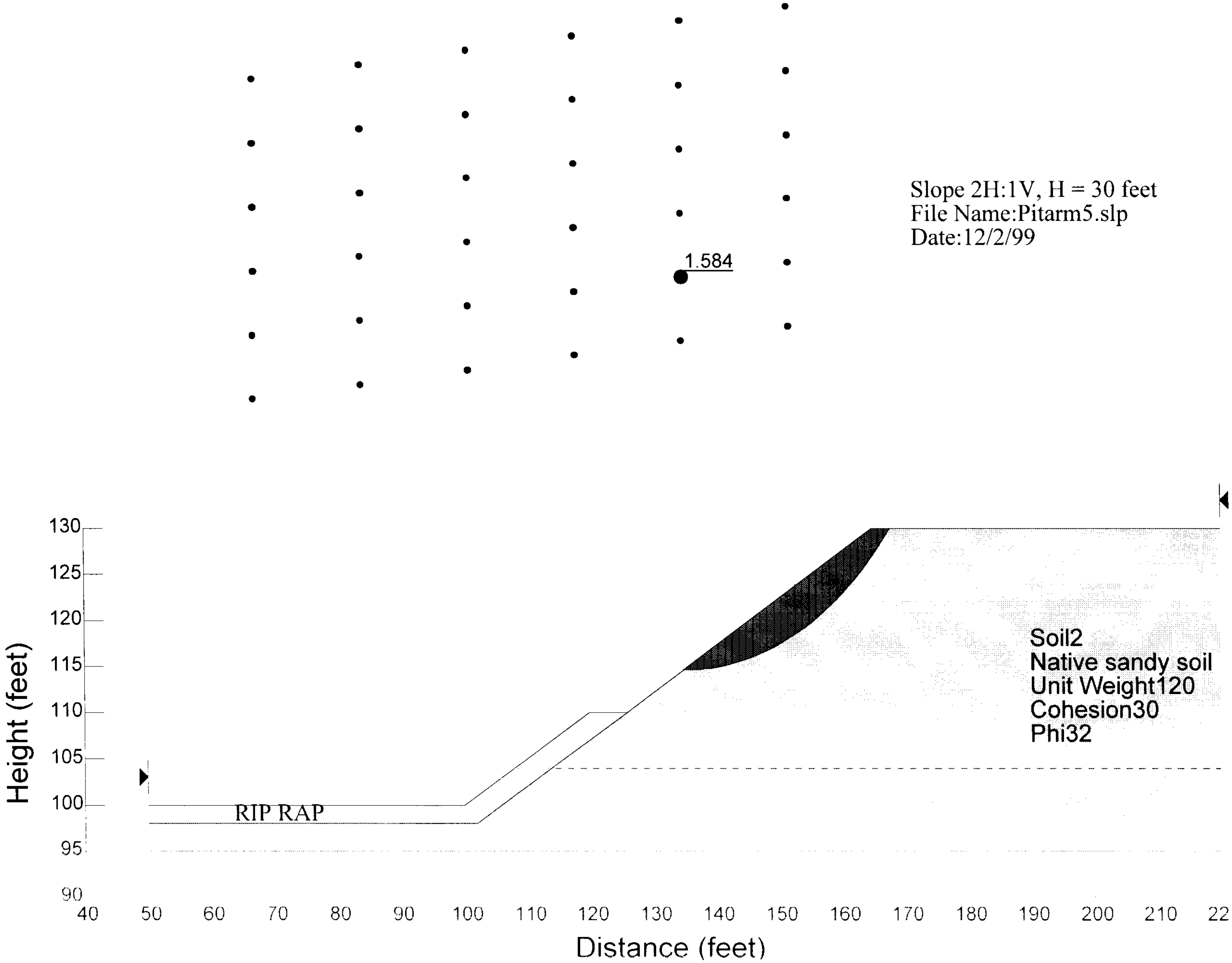
PITTSFIELD SLOPE STABILITY EVALUATION



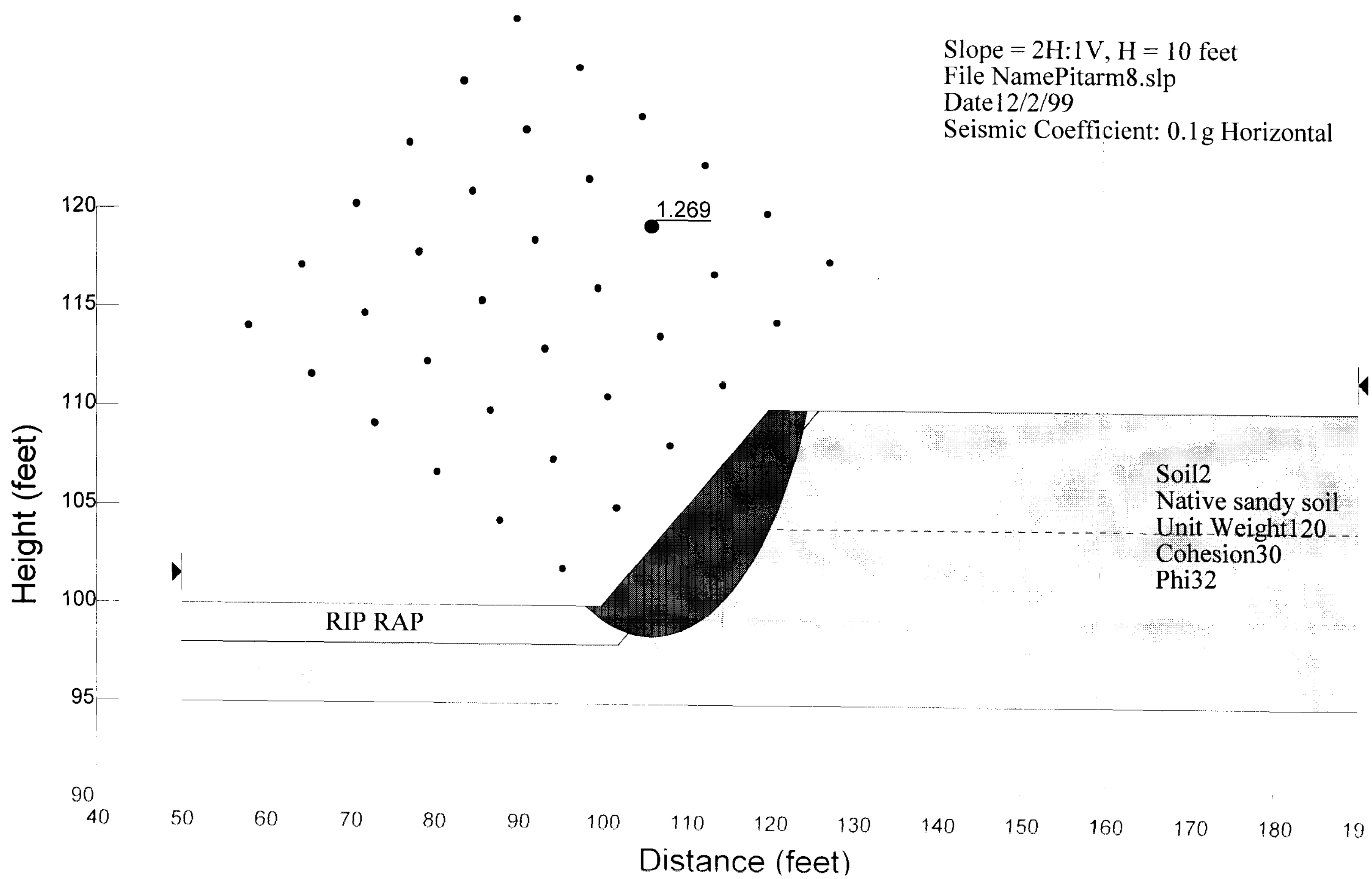
PITTSFIELD SLOPE STABILITY EVALUATION



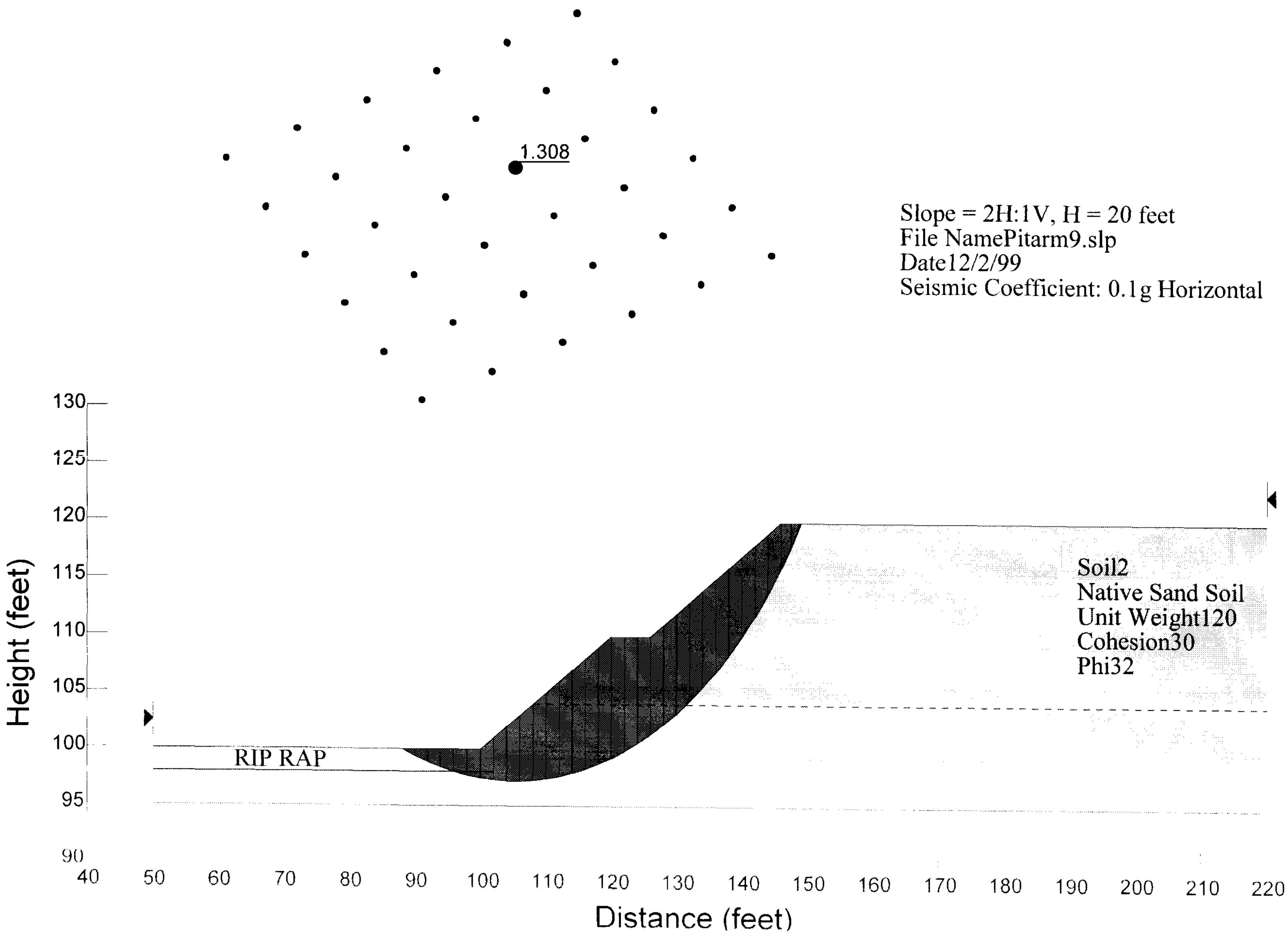
PITTSFIELD SLOPE STABILITY EVALUATION



PITTSFIELD SLOPE STABILITY EVALUATION

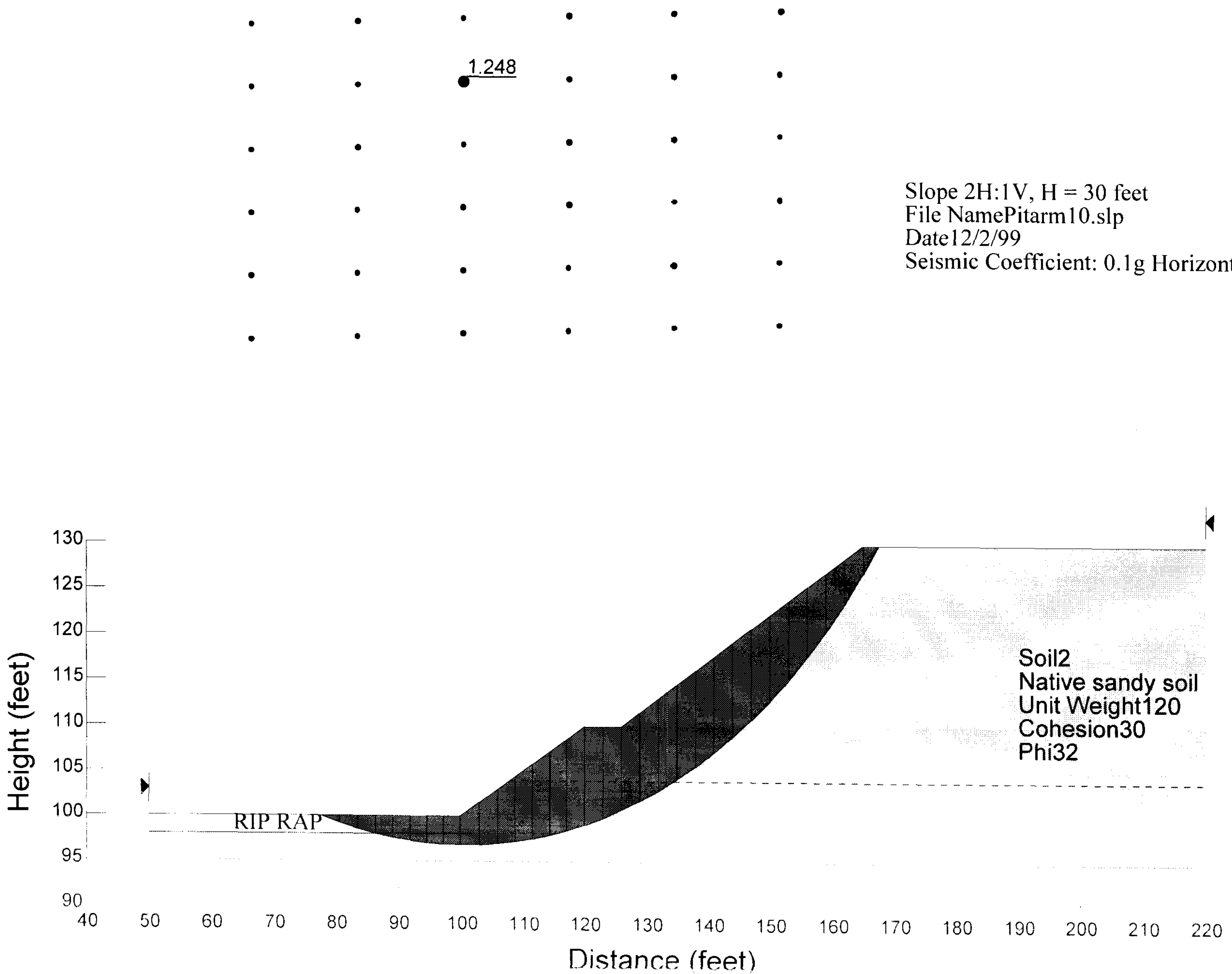


PITTSFIELD SLOPE STABILITY EVALUATION

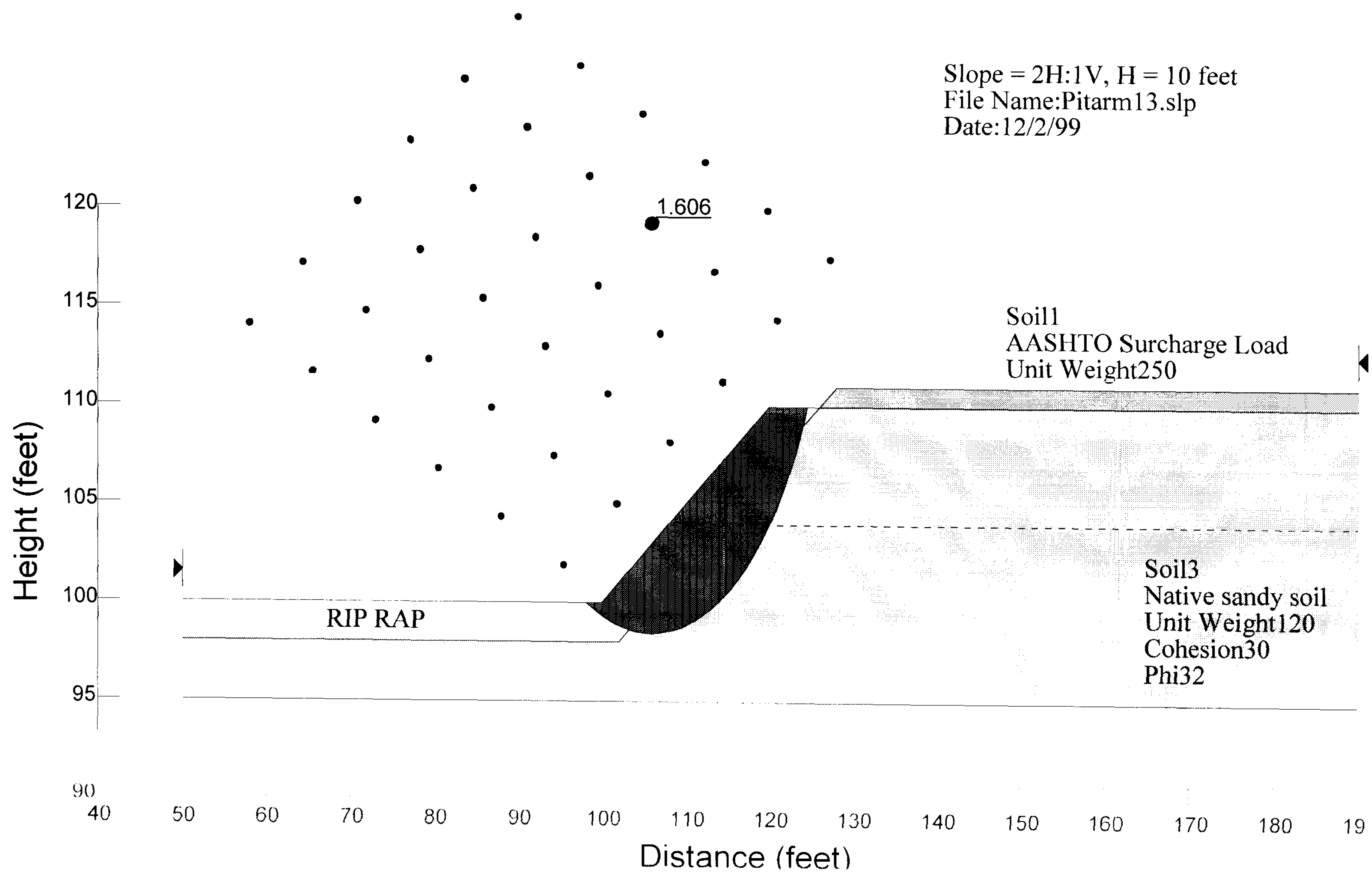


PITTSFIELD SLOPE STABILITY EVALUATION

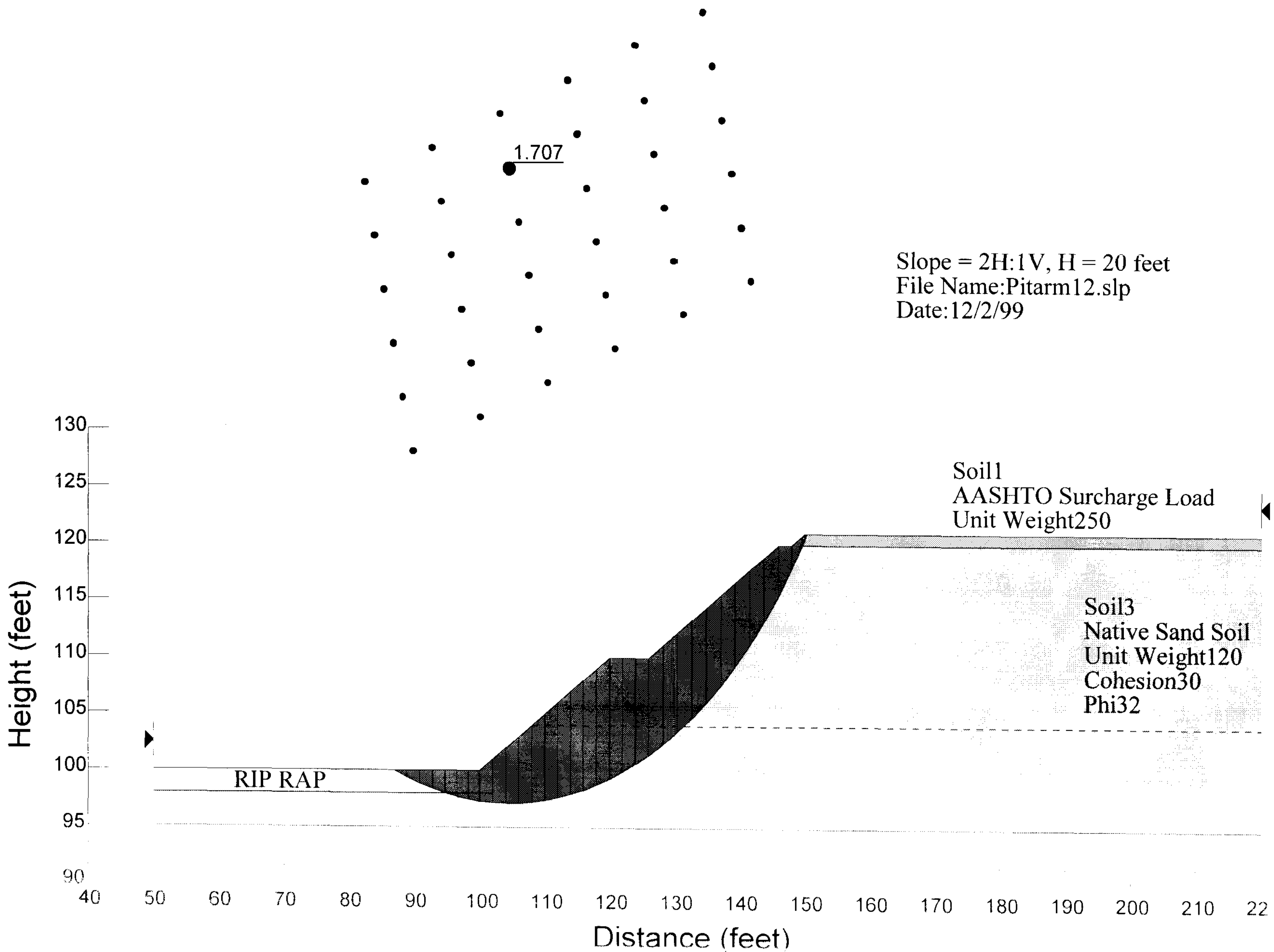
Slope 2H:1V, H = 30 feet
File NamePitarm10.slp
Date12/2/99
Seismic Coefficient: 0.1g Horizontal



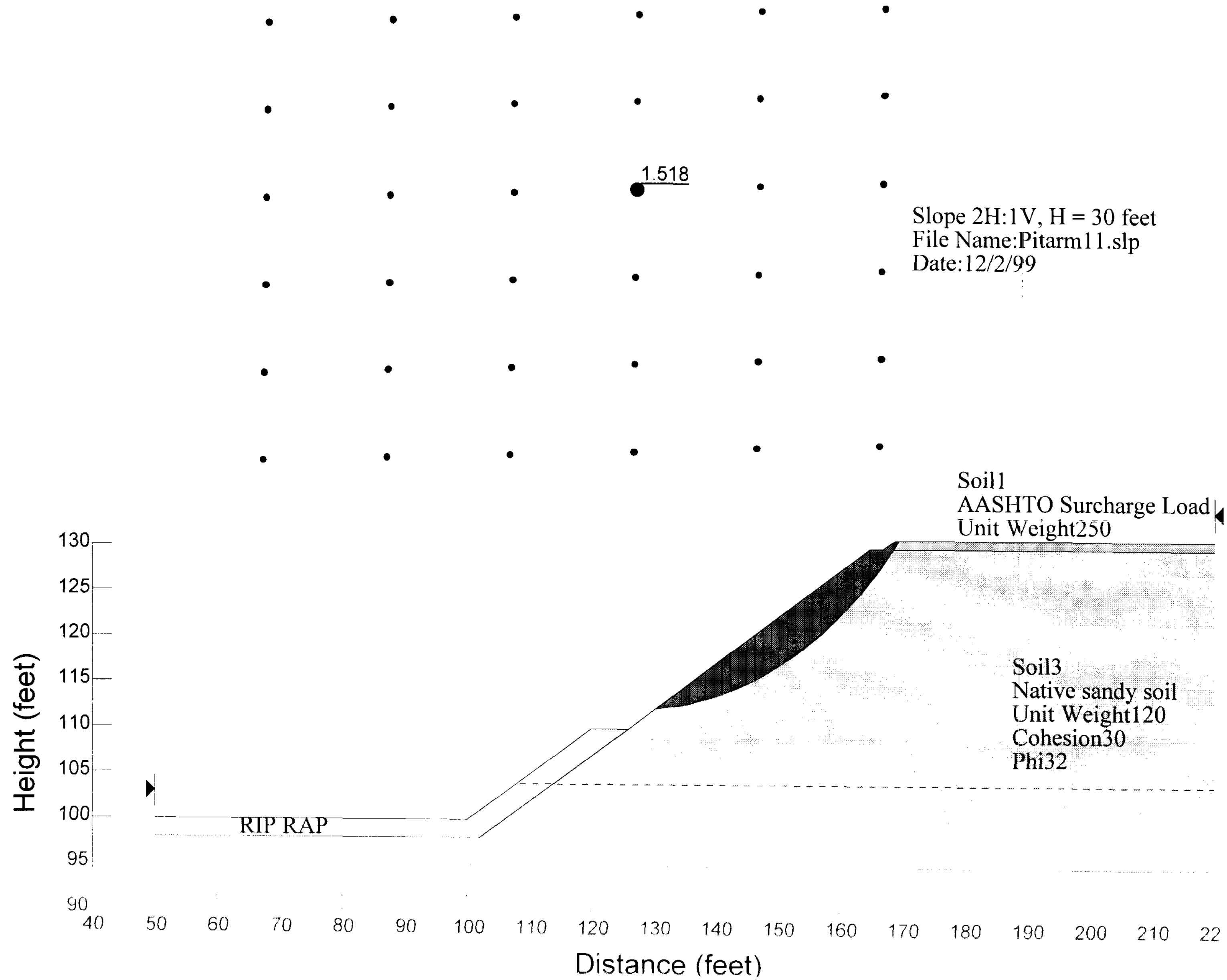
PITTSFIELD SLOPE STABILITY EVALUATION



PITTSFIELD SLOPE STABILITY EVALUATION



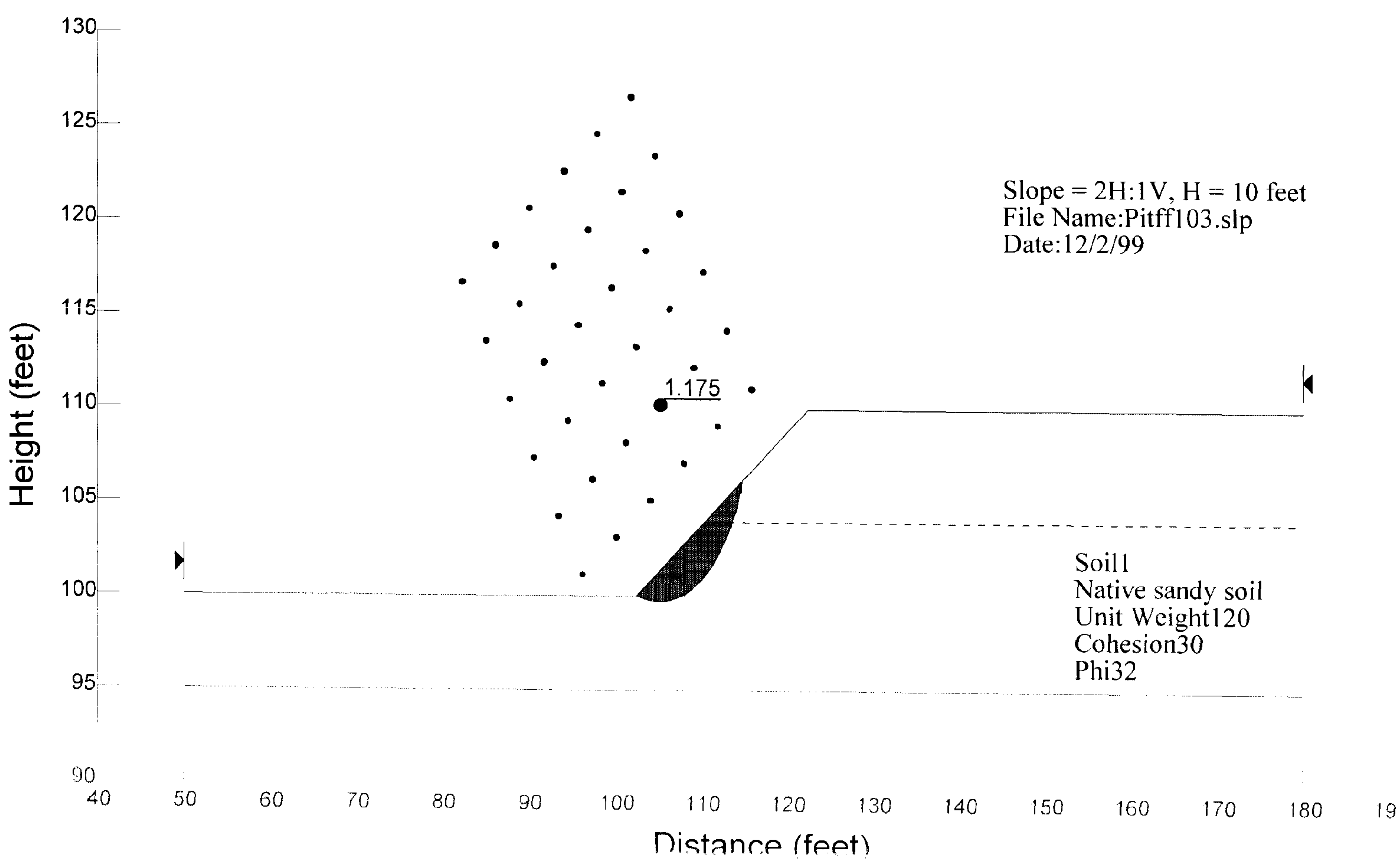
PITTSFIELD SLOPE STABILITY EVALUATION



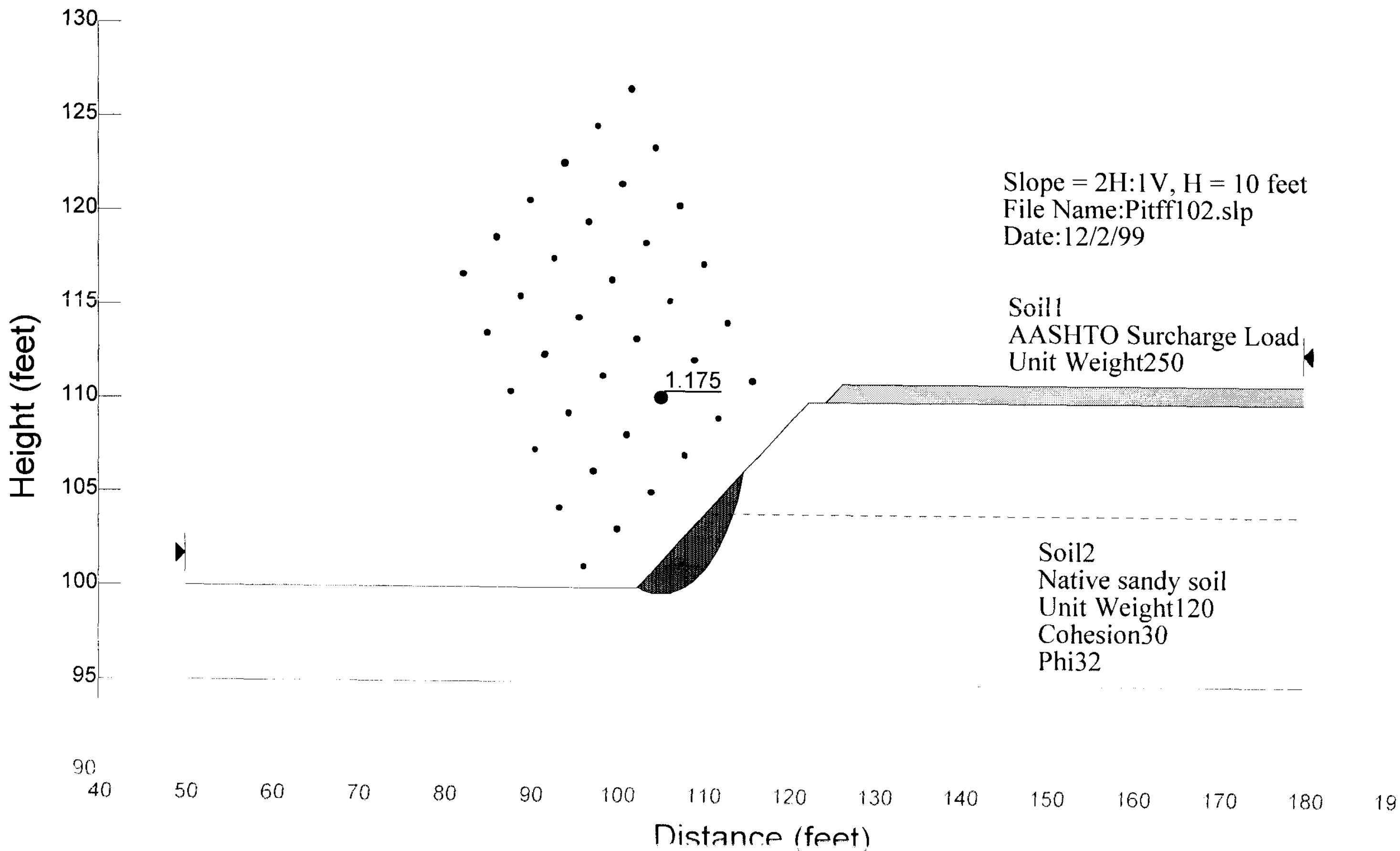
ATTACHMENT N.3

STABILITY ANALYSES FOR CONSTRUCTION CONDITION

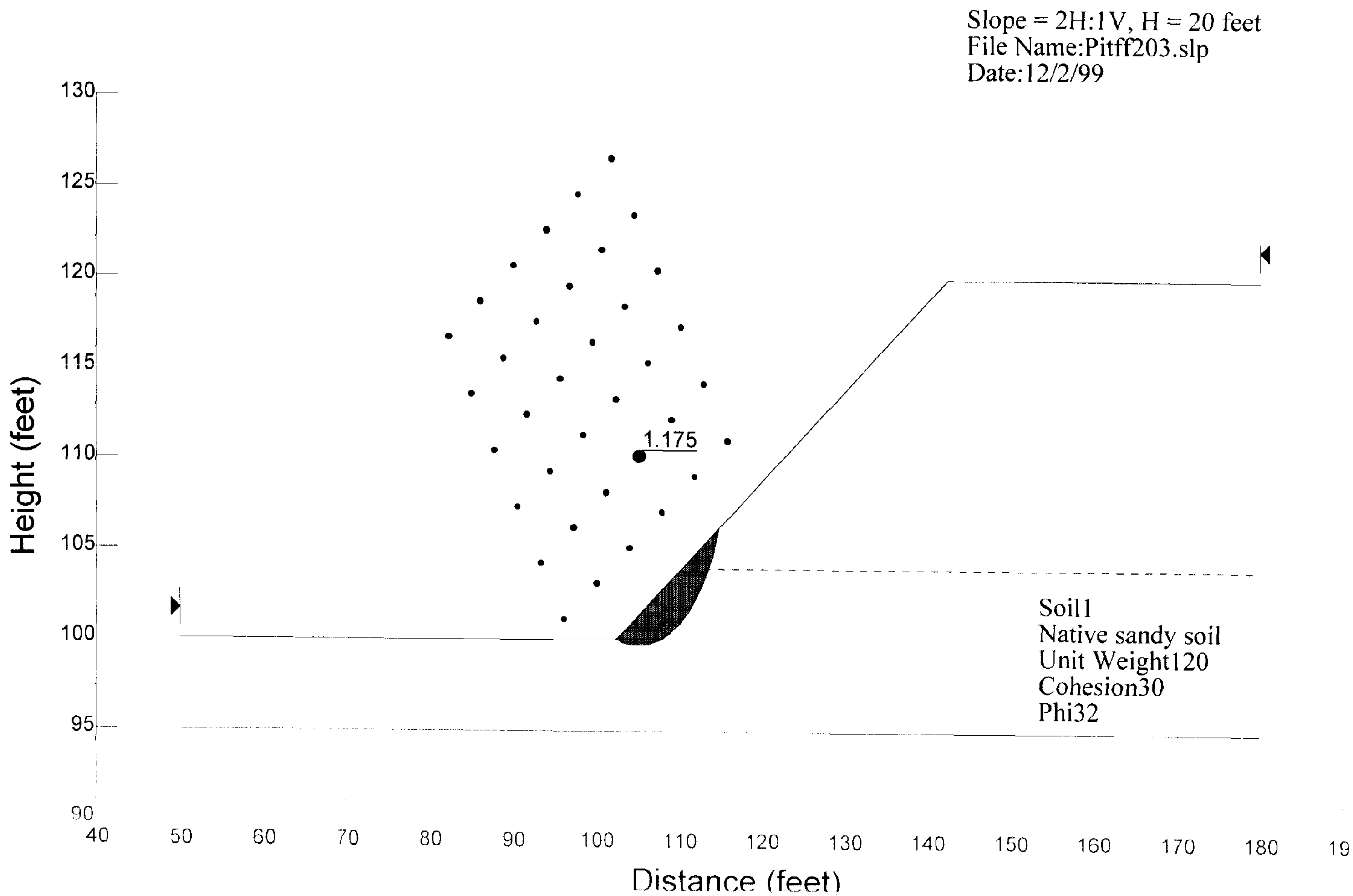
PITTSFIELD SLOPE STABILITY EVALUATION



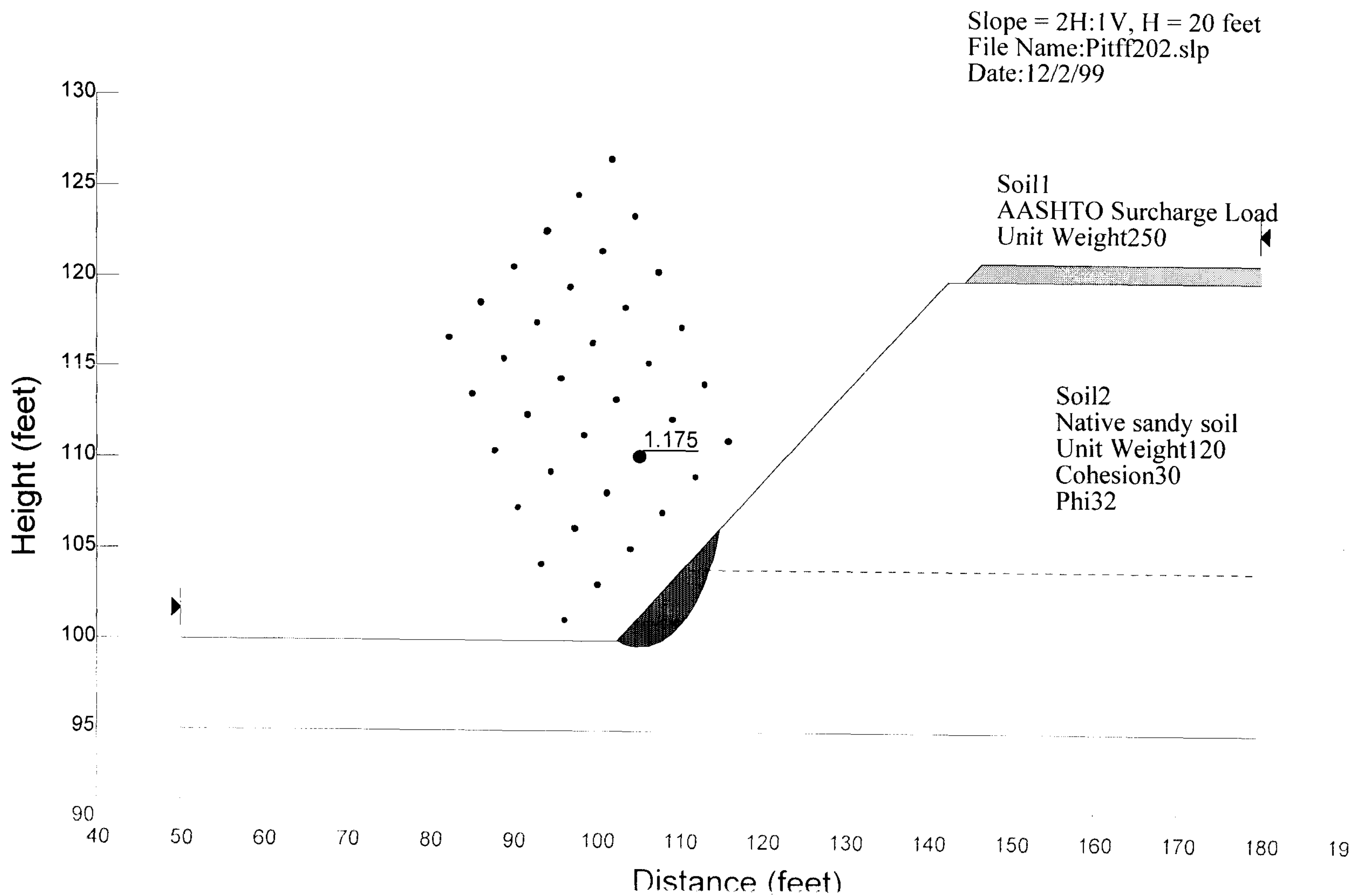
PITTSFIELD SLOPE STABILITY EVALUATION



PITTSFIELD SLOPE STABILITY EVALUATION

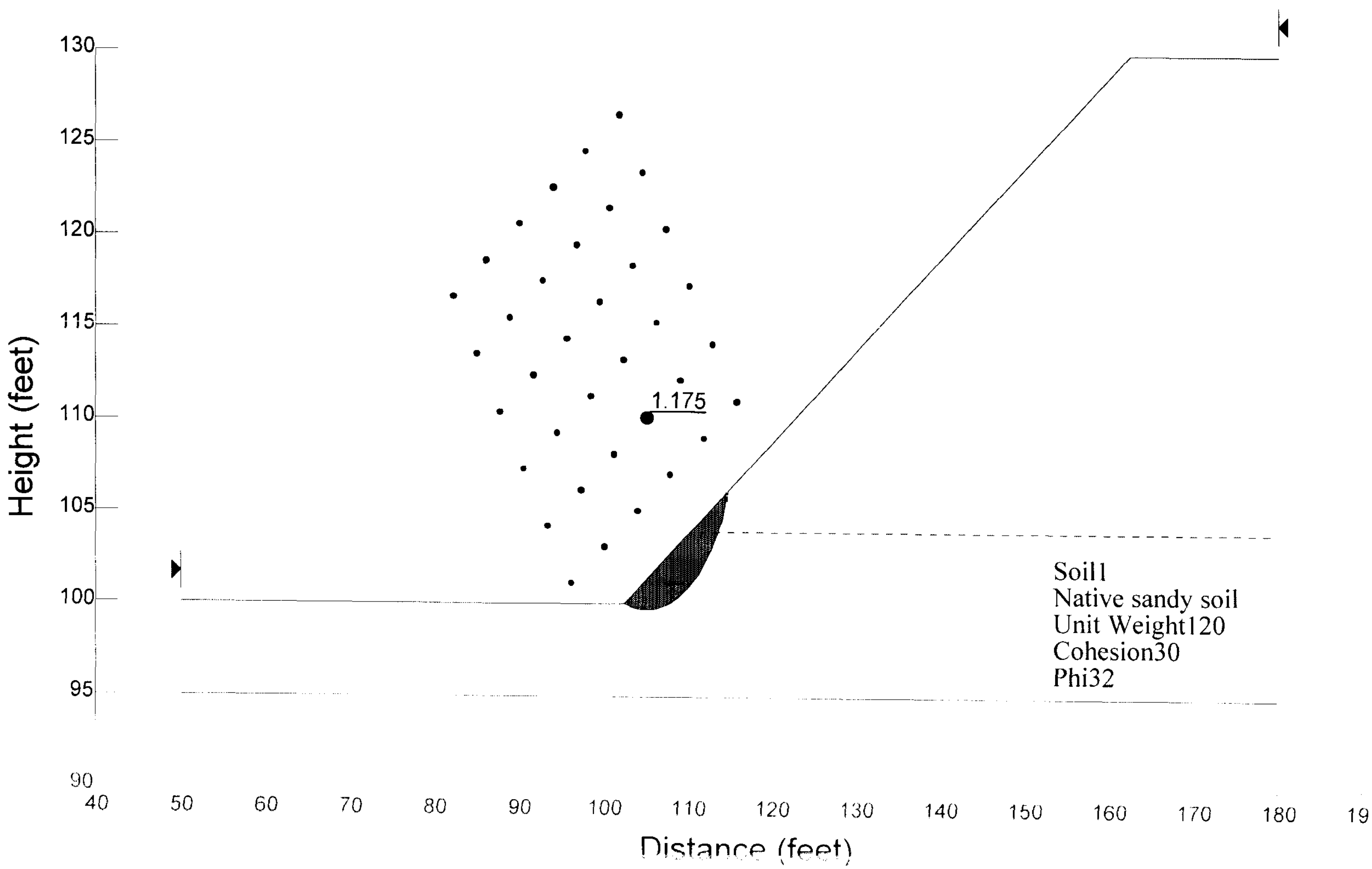


PITTSFIELD SLOPE STABILITY EVALUATION



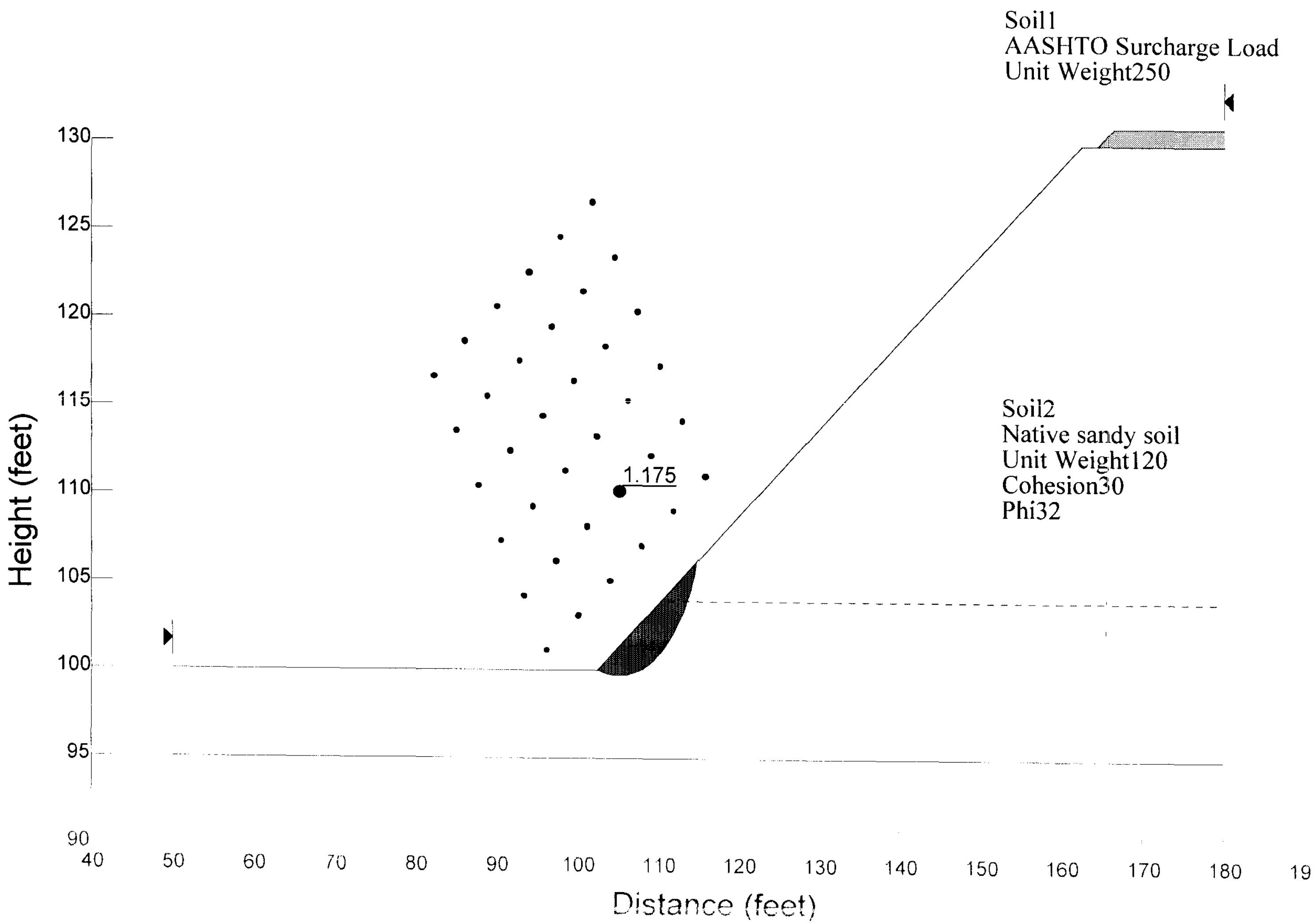
PITTSFIELD SLOPE STABILITY EVALUATION

Slope = 2H:1V, H = 30 feet
File Name:Pitff303.slp
Date:12/2/99



PITTSFIELD SLOPE STABILITY EVALUATION

Slope = 2H:1V, H = 30 feet
File Name:Pitff302.slp
Date:12/2/99



ATTACHMENT N.4

PROJECTED EARTH RETAINING STRUCTURE LOCATIONS

EAST BANK* STATION	WEST BANK* STATION
2+50 to 28+50	0+00 to 2+50
29+50 to 46+00	4+50 to 6+50
52+00 to 56+00	7+25 to 7+75
59+25 to 59+75	8+50 to 9+50
60+50 to 66+50	11+00 to 19+50
67+75 to 68+25	20+50 to 33+50
70+50 to 71+00	36+50 to 37+00
	38+00 to 44+50
	46+00 to 46+50
	49+00 to 53+00
	53+75 to 54+25
	56+50 to 57+00
	59+25 to 59+75
	60+25 to 60+75
	68+00 to 71+50

*Refer to Figures 2.1-6 A-D for river stationing and earth-retaining structure locations.