INDUSTRI–PLEX SITE REMEDIAL TRUST

WOBURN, MASSACHUSET

100% DESIGN REPORT
PART I

INDUSTRI-PLEX SITE
WOBURN, MASSACHUSETTS

Volume 4 of 8

Prepared by:

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Mt. Laurel, New Jersey

In Association with:

Jordan, Jones and Goulding
Normandeau Associates
Sasaki Associates
Husch & Eppenberger
Environmental Systems
Group Monsanto

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Prepared for:

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CHAPTER 11
PERMEABLE COVER

11.1 REMEDIAL DESIGN REQUIREMENTS

11.1.1 Consent Decree Requirements

The Remedial Design Action Plan (RDAP) is included as Appendix I of the Consent Decree. Page 1 of the RDAP states the following:

> "The remedial action for soils, sediments, and sludges contaminated with Hazardous Substances, other than those emitting odors (the East Hide Pile), shall include site grading, capping with a permeable soil cover, excavation, dredging, and/or consolidation for all areas containing Hazardous Substances at concentrations above established action levels (arsenic = 300 ppm, lead = 600 ppm, chromium = 1,000 ppm)...."

The RDAP also states that:

> (p.2) "Settlers shall design and implement remedial action for soils contaminated with Hazardous Substances above the action level for metals that shall consist of site grading and capping together with Institutional Controls..... Areas already covered adequately by buildings, roadways, parking lots, or other ground covering features, would not receive cover material, instead allowing the structures themselves to act as the protective cap.
>
> For small areas on-site, such as the landscaped areas between buildings and parking lots, Settlers may propose location-specific alternatives to capping consisting of excavation of contaminated soil and consolidation on-site with similarly contaminated soils, or placement of a protective layer such as asphalt to cap the contaminated soils.
>
> Settlers shall design and implement the remedial actions for contaminated soils in accordance with the following requirements:
>
> (1) cap design and construction activities shall be in accordance with regulations and/or guidance on cap design for permeable covers as summarized in

Attachment A hereto; provided that an alternative permeable cap design including a permeable synthetic fabric and a soil layer less than 30 inches in depth, may be used in all areas of the Site where Settlers demonstrate to EPA and the Commonwealth that the alternative cap design will perform as well as or better than the permeable cap design summarized in Attachment A"

Attachment A to the RDAP states that:

"Permeable covers shall be designed and constructed to include at a minimum the following:

(a) A vegetated top layer which shall be,

(1) of a minimum thickness of six (6) inches;

(2) capable of supporting vegetation that minimizes erosion and minimizes continued maintenance;

(3) planted with a persistent species with roots that will not penetrate into the contaminated soil;

(4) designed and constructed with a top slope of between 3 percent and 5 percent after settling and subsidence or, if designed and constructed with less than 3 percent, a drainage plan to ensure that the ponding of surface water does not occur or, if designed and constructed with a slope of greater than 5 percent, an expected soil loss of less than 2 tons/acre/year using the USDA universal soil loss equation; and

(5) designed and constructed with a surface drainage system capable of conducting effective run-off across the cap.

(b) A base layer that shall be:

(1) of a minimum thickness of twenty four (24) inches of appropriate fill material; and

(2) designed and constructed to prevent clogging."

An alternative permeable cover design concept utilizing a 16-inch thick borrow cover overlaying a geotextile was developed in the Alternate Cover Design Report (Golder, 1989). This design was subsequently approved by the EPA and MDEP in a letter dated September 11, 1989. The function and requirements of the permeable cover was presented in the Pre-Design Investigation (PDI) Task S-3 Interim Final Report (Golder, 1990a). This discussion is included as Appendix 11-A as a reference.

11.1.2 Remedial Design Work Plan Requirements

The Remedial Design Work Plan (RDWP; Golder, 1990b) establishes that the 100% Design Report is to include the following permeable cover design elements:

1. Definition of West Hide Pile stabilization method;
2. Final definition of cover extent;
3. Type of cap and cover on each Landowner's property;
4. Location and concentration of Hazardous Substances on each Landowner's property; and,
5. Location of areas where excavation of Hazardous Substances and replacement with clean backfill would be appropriate.

11.2 EXTENT AND TYPES OF COVER

To design the cover system, the extent and the type of cover on each individual Landowner's property had to be defined. These steps are discussed below.

11.2.1 Definition of Cover Extent

The minimum extent of cover for soils was established by determining the limit of arsenic, lead and/or chromium (hereafter referred to as "metals") at concentrations at or above the Consent Decree action levels. Data obtained in the Remedial Investigation/Feasibility Study (RI/FS), Pre-Design Investigation (PDI) and Groundwater/ Surface-Water Investigation Plan (GSIP) was used for this purpose. The cover was also extended to include the areas delineated in the Consent Decree as the East, West, East-Central and South Hide Piles, irrespective of metals concentrations. The following additional detailed criteria were used to define the limit of cover:

1. The limit of cover was extended to the boreholes with concentrations below action levels in the top 30 inches ("no hit points") closest to boreholes where metals above action levels were detected in the top 30-inches, as described below.

2. Straight lines were drawn connecting "no hit points".

3. A margin of at least 20 feet was maintained at all locations between the cap edge and any PDI borehole/test pit containing metals above action levels in the top 30 inches. This margin was increased to 50 feet for RI/FS holes, the locations of which were not surveyed.

4. Where the cap extent was limited by the Site boundary, the cap limit was drawn from the nearest "no hit point" within the Site to the Site boundary, according to the following criteria:

a. Extrapolating to the next "no hit point" outside the Site boundary, where available.

b. A straight line perpendicular to the Site boundary.

c. Extrapolating by continuing the trend of the line joining "no hit points" within the Site.

Decisions between the criteria b. and c. above were made on a conservative basis, i.e. adopting the criterion which gave the greatest cap area.

For wetland sediments, the extent of the remedy (dredging and/or cover) was determined in a similar manner to the soil cover and included all areas containing metals above action levels within the specific wetland. Although Wetland 3B does include some sediment containing lead in excess of Consent Decree Action levels, remediation of this wetland is not to occur at the specific direction of the USEPA (USEPA letter dated March 4, 1992 to ISRT). For stream sediments, the remedy was extended to the first downstream location determined not to contain metals above action levels. Stream and wetland sediment remediation is discussed, in detail, in Chapter 13.

The borehole data used to interpret the extent of soil cover and the limit of streams and wetland remedy is summarized in Sheets 11-2A through 11-2D. The resulting cover limits are also presented on the same drawings.

11.2.2 Landowner Consultation

Section XVI of the Consent Decree requires that "each landowner will be afforded opportunities to review and comment on the design of those portions of the Work that will affect the Landowner's property, including the design of any caps or covers to be placed on the property". The elements of the consultation process are defined by the Consent Decree as follows:

> "(2) Prior to sixty percent (60%) completion of the Remedial Design phase of the Work, as described in the RD/AP, Settlers shall prepare a map or maps showing the known locations and concentrations of Hazardous Substances on each Landowner's property and reasonable interpolations of such data delineating the contaminated areas in a form approved by EPA in consultation with the Commonwealth. Settlers shall provide to each Landowner a copy of the map or maps showing the Landowner's property.
>
> (3) Not later than completion of the 60% Remedial Design phase, Settlers shall notify each Landowner, in a form approved by EPA in consultation with the Commonwealth, of: (a) the types of cap or cover (e.g., pavement, soil cap, or synthetic/soil cover) that, consistent with the ROD, the requirements of the RD/AP, and the overall Remedial Design for the Site, it would be feasible to place on each area of the Landowner's property containing Hazardous Substances in excess of action levels; and (b) any area(s) of the Landowner's property in which, consistent with those same requirements, it would be appropriate to excavate contaminated soil and backfill with clean material. Landowners shall have not less than thirty (30) days from receipt of such notice and the maps required by subparagraph (2) above to notify Settlers, in writing, of their preferences as to the type(s) of cap or cover to be placed on the specified locations or of their preference for excavation and backfilling of designated areas on their respective properties"

In accordance with the above requirements, an information package was prepared for each Landowner consisting of the following items:

1. Cover letter describing the above Consent Decree requirements and the Landowner's rights and obligations with respect to cover preferences.

2. Drawing providing an overview of the Site boundaries and the approximate extent of the areas of the Site which will receive cover.

3. Drawing of Landowner's specific property, its limits and the sampling points that were used during the RI/FS and/or Pre-Design Investigation Studies.

4. Table presenting the results of soil and sediment sample analyses for arsenic, lead and chromium, and hide residue on the Landowner's specific property.

5. Drawing of Landowner's specific property defining the various areas that are to receive cover, or that currently have cover equivalents in place.

6. Table detailing, for each area of the Landowner's property, the current condition of the area; the cover proposed for that area by the ISRT; and any alternative cover available.

7. Drawings showing typical cross-sections of the various cover options applicable to the specific Landowner's property.

A typical package is reproduced as Appendix 11-B. EPA approved the Landowner notification package in a letter to the ISRT dated February 6, 1991. The contents of the notification package were described at a Landowner's meeting in Woburn on February 7, 1991 immediately prior to mailing of the packages. Landowners were also invited to schedule individual meetings with the ISRT to discuss their particular properties. Packages were mailed to all Site landowners, with the exception of Woburn Industrial Associates and Chestnut Hill Realty Trust for whom it was not possible to obtain current mailing addresses. Copies of all Landowner packages were also provided to the EPA.

The cover options presented to the Landowners included the following, as appropriate to the particular local conditions:

1. The alternate soil cover, placed above existing grade, comprising a nonwoven geotextile and 16-inches of clean soil.

2. Asphalt cover, placed above existing grade, comprising a nonwoven geotextile, 6-inches of granular subbase, 4-inches of asphalt binding course and 2-inches of asphalt wearing surface.

3. For small areas, such as the landscaped areas between buildings and parking lots, excavation of 16-inches and placement of the alternate at grade soil cover, as permitted by the RD/AP.

4. For small areas, such as the landscaped areas between buildings and parking lots, excavation of 12-inches and placement of an at grade asphalt cover in accordance with (b) above, as permitted by the RD/AP.

5. The impermeable cover on the East Hide Pile.

6. Existing cover equivalents (roads, buildings, parking lots and railroad lines).

Formal responses indicating cover preferences were received from a number of the Landowners within the 30 day period specified in the Consent Decree. Certain Landowners did not respond within the 30 day period or proposed alternate approaches. Landowners were formally advised of the availability of the 95% Design Report and afforded a further opportunity to provide comments on the design.

Meetings have been held with specific Landowners, at their request, throughout the design process to discuss their cover preferences and these have been incorporated in the design to the extent possible and consistent with design requirements. Where Landowners have exercised their right under the Consent Decree to select an alternative cover to that proposed by the ISRT, associated additional costs are payable by the Landowner.

11.3 DESIGN CONSIDERATIONS

11.3.1 Slope Stability

Slope stability analyses were conducted of the West Hide Pile for existing and remediated conditions. Sections L-L′ and J-J′ were analyzed for existing conditions, with section L-L′ representing the area where the geometry is most critical. Remediated conditions were analyzed for Section L-L′, which was found to have the lowest factor of safety for existing conditions. A similarly critical section, I-I′, was also analyzed for the remediated condition. The locations of these cross-sections are identified on Figure 11-1. The results of the slope stability analyses are presented in Appendix 11-C for existing conditions and in Appendix 11-D for remediated conditions.

Two series of slope stability analyses were conducted both for existing and remediated conditions: one representing the long-term groundwater condition without a perched water table and the other including the effect of a possible perched water table. Groundwater levels were based upon borehole observations as presented in the PDI Task S-2 Interim Final Report (Golder, 1990c).

Because the materials that form the hide pile were found to behave as cohesionless soils, the critical failure mechanism was generally shallow surface sloughing with semi-planar failure surfaces parallel to the slope where the surficial soils are locally weaker or where water tables are present. The analysis of planar failure surfaces in cohesionless soils is most appropriately conducted using the infinite slope theory (Lambe & Whitman, 1969). In order to model possible deeper seated failure mechanisms for the long-term groundwater condition, analysis was carried out using the PCSTABL5M version of the computer program STABL developed by Purdue University (Purdue University, 1988). Circular failure surfaces were analyzed using the simplified Bishop method, applying restrictions to the failure initiation and termination zones to preclude the shallow surface sloughing mode, which was analyzed separately as described above. The PCSTABL5M program permits analysis of large numbers of potential circular failure surfaces per run and automatically searches for the most critical; in this case 400 surfaces per run were analyzed. The slope stability results are summarized on Table 11-1.

The cross-sections originally analyzed for the 30% and 60% Design Reports have been updated in this report to account for the additional topographic information available. The cross-sections analyzed for the individual slope stability runs are illustrated in Appendices 11-C and 11-D. The soil parameters used for the analysis were those recommended in the PDI Task S-2 Interim Final Report (Golder, 1990c) as described below.

A unit weight (saturated) of 100 pcf and an effective angle of shearing resistance of 25 degrees with zero cohesion was used for the Surficial Material.

Unit weights determined from undisturbed Shelby tube samples of Fill and Hide Residue prior to extrusion indicated a range of values of 65 to 130 pcf, reflecting variations in the local degree of compaction and the degree of saturation associated with perched water tables. A conservative value of 125 pcf was selected for slope stability calculations.

Shear strength parameters for the Fill and Hide Residue were assessed from the results of triaxial tests and SPT 'N' values. For heterogeneous materials of this nature, the most reliable triaxial strength parameters were obtained by considering all of the test results together to define a single failure envelope that accounts for the volumetric changes associated with the development of shearing ("steady state" shear strength) rather than by assessing distinct values for each test from the Mohr circles. A large number of results were conveniently assessed in this way by plotting the failure points on a p′-q plot where:

$$p' = \frac{\sigma 1' + \sigma 3'}{2} \qquad q = \frac{\sigma 1' - \sigma 3'}{2}$$

$\sigma 1'$ and $\sigma 3'$ being the major and minor principal effective stresses at failure. The conventional Mohr-Coulomb failure parameters c' and ϕ' are related to the slope (ψ) and intercept (d) of the p′-q plot as follows:

$$\sin \phi' = \tan \psi$$

$$c' = \frac{d}{\cos \phi'}$$

A p′-q plot for the Fill and Hide Residue is presented in Appendix 11-C. The data is reasonably consistent, between consolidated undrained tests with pore pressure measurement and consolidated drained tests, for both undisturbed samples and specimens remolded at field water content and density. A "best fit" line through the data gives an effective angle of friction of 37 degrees and an effective cohesion of 2 psi. A lower bound line corresponds to an effective angle of friction of 34 degrees and zero effective cohesion.

The SPT results for Fill and Hide Residue are plotted against depth in Appendix 11-C. The data shows a range of values, as would be expected for a heterogeneous material of this nature, with 'N' values almost constant with depth. Using the work of Schmertmann (1975) to account for overburden effects and noting low 'N' values which are likely to have been affected by piping, suggests an effective angle of shearing resistance of 35 degrees.

It is considered unwise to rely on a cohesive strength component for such a heterogeneous material and a prudent allowance was also made for possible future degradation of material properties as a result of continuing anaerobic decomposition of the hide materials. Considering this and the above discussions, effective shear strength parameters of zero cohesion and 34 degrees friction angle were selected for the Fill and Hide Residue.

A unit weight (saturated) of 120 pcf and an effective angle of shearing resistance of 36 degrees with zero cohesion were selected for the Outwash Sand. A unit weight (saturated) of 125 pcf and an effective angle of shearing resistance of 37 degrees with zero cohesion were selected for the Glacial Till.

11.3.1.1 Existing Conditions

The case of shallow surface sloughing was analyzed for the existing conditions case using the infinite slope model and the average slope of Section L-L' equal to 40 degrees. An apparent factor of safety of 0.6 was obtained for the long-term groundwater condition. For perched water table conditions, analyses were performed for seepage emerging from and parallel to the slope; apparent factors of safety of approximately zero and of 0.3 were obtained.

The results obtained from the surface sloughing analyses of Section L-L′ suggest that the combination of conditions adopted for this analysis are quite conservative. This cross-section is currently stable possibly due to a locally higher shear strength of the material due to surficial vegetation adding frictional stability, or increased interparticle friction due to capillary suction. Visual inspection of this section of the hide pile suggests that minor surface sloughing has occurred in the past as evidenced by the angle that vegetation emerges from the slope.

The case of shallow surface sloughing was also analyzed for the long-term groundwater condition using the infinite slope model and the average slope of Section J-J′ equal to 26 degrees. A factor of safety of 0.8 was obtained.

Circular analyses were conducted for Sections L-L′ and J-J′ for the long-term groundwater condition. Factors of safety of 1.3 and 1.6 were obtained, respectively, indicating that Section L-L′ is more critical than Section J-J′, and confirming that, as anticipated, deeper failure surfaces are less critical than surface sloughing for this type of material. Circular analyses were also conducted for Section L-L′ adopting a conservative perched water table, affecting the upper half of the slope with a phreatic surface at approximately 75 percent of the slope height. A factor of safety of 1.3 was calculated, comparable to that for the long-term groundwater case.

For the type of profile determined for the West Hide Pile, in which no weaker layers have been detected interlayered with other soils, circular surfaces are expected to be more critical than non-circular surfaces. However, as a verification, a limited number of feasible non-circular potential failure surfaces were also analyzed for the long-term groundwater condition on Section L-L', using PCSTABL5M with the Spencer method. These analyses yield a minimum factor of safety of 1.4, higher than for the circular mechanism.

The computer data for each PCSTABL5M run are included in Appendix 11-C with cross-sections showing the critical failure surfaces.

11.3.1.2 Remediated Conditions

Slope stability analyses were conducted on remediated conditions for the most critical cross-section determined for existing conditions, that is, Section L-L', and a less critical cross-section I-I'. Both cross-sections were analyzed with alternate unit weights of the 125 and 115 pcf for the Fill and Hide Residue in the long-term groundwater condition, to reflect the variability of this material. Analyses were also conducted for Section L-L' for perched water conditions using a unit weight of 125 pcf.

As discussed above, the critical failure mechanism prior to remediation is shallow surface sloughing associated with cohesionless, Surficial Materials. This failure mechanism is precluded with the proposed remediated plan, since compacted granular fill will be placed to flatten the slopes. Proof rolling of the Surficial Material on the existing slope surface will be undertaken prior to placement of the fill in areas where the existing slope is 2.5H:1V or flatter, which can be achieved by drum and rear wheel drive rollers. The surface of existing slopes steeper than 2.5H:1V can not be proof rolled; however, the thickness of compacted fill in front of these slopes will be significant and will be sufficient to prevent sloughing.

The cross-section geometries analyzed for the remediated condition are illustrated with their individual slope stability run outputs in Appendix 11-D. Additional materials used in the remediated condition, or any modifications made to the soil parameters previously used in the existing condition analyses, are discussed below. The conservative assumption that the cover materials provide weight only (unit weight of 120 pcf) and have zero shear strength was made.

Select fill material to be used for grading purposes beneath the cover was analyzed with a unit weight of 125 pcf and an effective angle of shearing resistance of 33 degrees with zero cohesion. Materials chosen for this fill will have to meet these standards as required in the construction specifications.

For Section L-L' only, two different effective angles of shearing resistance of 25 and 32 degrees were used for the Surficial Material. An angle of 25 degrees was selected for areas where compaction would be difficult, that is, the steep portion of the slope and the area beyond the toe of the remediated slope. An angle of 32 degrees was selected for the area from the toe of the existing slope to the toe of the remediated slope and beyond the crest of the existing slope, because a higher degree of compaction is obtainable in these areas. Section I-I' conservatively used 25 degrees for the effective angle of shearing resistance for the Surficial Material. A unit weight of 100 pcf (saturated) and zero cohesion was used for the Surficial Material in both cross-sections.

The factor of safety for the long-term groundwater condition for Sections L-L' and I-I' was determined to be 1.6 and 1.7, respectively. Using alternate unit weights of 125 pcf and 115 pcf for the Fill and Hide Residue had no effect on the factor of safety for this condition.

The factor of safety for the perched water table condition was determined to be 1.6. This condition was analyzed with the same conservative perched water table adopted for existing conditions. The perched water table was considered not to continue horizontally once it emerged from the Surficial Material into the new fill material, due to the good drainage of the material specified for the fill.

The analysis of the slope stability of the West Hide Pile in the remediated condition concludes that the proposed grading plan is acceptable. The computer output data from the PCSTABL5M runs and cross-sections showing the critical failure surfaces for the remediated conditions are included in Appendix 11-D.

11.3.1.3 Cover Interface Friction

The permeable cover comprises a nonwoven geotextile overlain by 16 inches of clean soil. The internal stability of the cover on slopes is a key design consideration since the interface between the cover soil and geotextile is most critical.

A testing program using representative soils and geosynthetic samples was undertaken to verify the friction angle selected for the design. A detailed discussion of the laboratory testing program is presented as Appendix 11-E. The results of this program indicate that the minimum residual interface friction angle between a soil cover of the type that has been specified for the West, East-Central, and South Hide Piles, and the geotextile or geocomposite is 30 degrees.

For the slopes less than 33 percent, under the assumption of infinite slope, the calculated factor of safety with respect to sliding of the cover is at least 1.7 for the friction angle of 30 degrees.

For slopes steeper than 33 percent, a geogrid tensile reinforcement is incorporated above the geotextile. The calculated factor of safety with respect to sliding of the cover is 1.7 for an inclination of 21.8 degrees (40 percent). Calculations for the geogrid reinforced cover are presented in Appendix 11-F.

Additional interface friction testing will be performed using the actual borrow sources for cover material and geosynthetics prior to construction, as outlined in the specifications.

11.3.2 Soil Erosion

Calculations of soil loss based on the USDA Universal Soil Loss Equation as presented in the USEPA document entitled "Evaluating Cover Systems for Solid and Hazardous Waste" (Lutton, 1982, Revised Edition) are included in Appendix 11-G. These calculations show an expected soil loss of 0.76 tons per acre per year, below the specified 2 tons per acre per year. Establishing vegetative cover as quickly after construction as possible should further aid in the prevention of soil loss.

11.3.3 Frost Penetration

The permeable cover is designed to satisfy the performance factors specified in the RDAP, as follows:

1. Assurance that direct contact with contaminated soils will be eliminated;
2. Long-term performance not to be impaired by the effects of the freeze/thaw cycle;
3. Erosion limitation;
4. Durability and long-term reliability; and,
5. Adequacy of quality assurance during installation.

In approving the cover section design, EPA noted "It was and is the intent of the ROD to minimize the effects of freeze/thaw by keeping the waste below the average depth of frost."

The Alternate Cover Design Report concluded that the depth of frost penetration for an average winter at the site would be contained within a cover thickness of 16 inches. The report also concluded that incorporation of the geotextile in the cover precludes upward migration of contaminated material by frost heave effects when the depth of frost penetration exceeds the average. The geotextile also prevents physical damage to the cover by heaving of gravel sized particles or larger objects.

In the specific case of steeply sloping portions of the permeable cover (notably parts of the hide piles), a geosynthetic drainage/capillarity break layer is included at the base of the cover to prevent sloughing due to formation of water films during thawing after frost penetration in excess of the average. The design elements of the permeable cover are further discussed in Section 11.4.2.

11.3.4 Settlement

The allowable settlement for which a structure has to be designed depends on its specific characteristics and function. The permeable cover to be constructed on the West, East-Central and South Hide Piles is not a structure sensitive to settlements, because it is very flexible and will not support other structures. Therefore the assessment of the effects of settlements included in this section considers strains that could occur in the cover and the maintenance of appropriate drainage.

Calculations of the maximum differential settlement of the cap as a consequence of variations in the thickness of the hide piles are presented in Appendix 11-H. These calculations are based on one-dimensional compressibility which is appropriate for the present case of a wide, flexible loaded area. Additional calculations are presented in Appendix 11-H of the maximum differential settlement of the cap as a consequence of the heterogeneity in the properties of the soils. These calculations were based on Schmertmann's method as directed by USEPA; this method strictly applies for a rigid axisymmetric load of finite extent. In order to use the method, the hide pile was approximated as a circle of equivalent area. Schmertmann's method also relies upon static cone resistance data. Such data were not available in the present case and was approximated from the SPT 'N' values using the correlations presented by Robertson, et al., (1983) and Kasim, et al. (1986). Settlements were calculated for maximum and minimum 'N' value profiles in order to compute maximum differential settlements.

The maximum differential settlement obtained by the one-dimensional method is 0.05 feet in a distance of 60 feet, while the maximum predicted by the Schmertmann method is 0.004 feet in a distance of 194 feet. This indicated that neither the integrity of the cap nor the drainage gradients would be adversely affected by the maximum credible settlements which are conservatively estimated by the one-dimensional method. Preloading of the hide piles prior to construction of the cap is therefore not necessary.

11.4 PERMEABLE COVER DESIGN

11.4.1 Permeable Cover Grading

The permeable cover grading plan is largely controlled by the existing topography of the Site. In general, the Site may be divided into predominantly undeveloped and developed areas. The grading plan in the undeveloped area is controlled by topographic highs consisting of the South, East-Central, West and East Hide Piles, and numerous debris/spoil piles scattered throughout the Site. The developed areas of the Site are characterized by low relief and man-made structures such as buildings and pavements. The existing topography of the Site is presented on Sheets 11-1A to 11-1D.

The location and type of cover for each Landowner is presented as Sheets 11-3A to 11-3D. Cover equivalents are shown on Sheet 11-4 and discussed in Section 11.4.1.5. Prior to construction of the permeable cover, various features require decommissioning or abandonment. These features are illustrated on Sheets 11-5 and 11-6.

11.4.1.1 Cover Contours

The grading plan showing the top of cover contours is presented in Sheets 11-7A to 11-7D. Top of cover contours are drawn for each hide pile and in areas where debris/spoil piles are to be regraded. Cover contours are generally drawn at 2 foot intervals increasing to 4 foot intervals in areas of high relief. In more developed areas of low relief, shading is used to represent different cover types placed above existing grade. At grade covers and transition locations are presented in Sheets 11-3A to 11-3D. Six profiles through the Site illustrate areas of cut and fill and are presented as Sheets 11-14 to 11-19.

The grading plan is designed to minimize cut and fill, optimize constructability and to maintain the existing Site drainage pattern to the extent possible.

The basic permeable cover, consisting of a geotextile, overlain by 16 inches of clean soil above existing grade, is used to the fullest practical extent as shown in Sheets 11-3A to 11-3D. Cut and fill are necessary in the following circumstances:

1. In areas of at grade cover and transitions adjacent to existing buildings and pavements, excavation and replacement is required. This aspect is discussed in detail in Section 11.4.1.5.

2. The regrading of various debris/spoil piles produces cut material. Judgement was used to select individual piles to be regraded. Selection was based on cover constructability, drainage considerations, aesthetics and possible future land use.

3. Construction of a clean utility corridor within the cover area produces cut material. The utility corridor is further discussed in Section 11.4.5.

4. Minor regrading to the northwest corner of the South Hide Pile to limit land take on the adjacent property.

5. Wetland sediment remediation and mitigation produces material requiring disposal. Wetland sediment remediation is discussed in Chapter 13. Wetland mitigation is discussed in Chapter 14.

6. Filling is required to stabilize the slopes of the hide piles. The grading plan and cross-sections of the hide piles, presented as Sheets 11-7 to 11-13, illustrate the fill areas.

In all cases, the amount of cut in areas of known hide residues is minimized to the extent possible consistent with construction of the remedy.

The cover design optimizes constructability by maximizing the use of long straight contours and smooth constant slopes. The design minimizes cutting of the hide piles and slopes are generally maintained at 33 percent or flatter. Steeper slopes of up to 40 percent have been adopted in limited areas to minimize impact to adjacent structures or wetlands.

Transitions are required at the limit of cover and at the Site boundary to tie the cover with the existing grade. The typical transition at the cover limit is shown as Detail 10 on Sheet 11-24. The transition consists of feathering out the cover to the existing grade at a maximum slope of 33 percent. A similar transition is utilized where cover extends to the Site boundary as shown in Detail 9 on Sheet 11-24.

Bedrock outcrops exist at several locations throughout the Site. The proposed remedy for bedrock outcrops within the cover area is to grub vegetation and clean by suitable means to ensure that any soil which may contain metals above action levels is removed. The surrounding soil cover is extended up to the outcrop.

11.4.1.2 South Hide Pile Grading

The South Hide Pile is located between Commerce Way and the MBTA line, south of Atlantic Avenue and north of Boston Edison Right-of-Way No. 9. The South Hide Pile is bounded on the north and east by existing paved areas and to the south by the Site boundary. Relatively flat, sparsely vegetated ground is present to the west. The location and extent of the hide pile, as defined by the Consent Decree, is presented on Sheets 11-2C and 11-2D.

The peak elevation of the South Hide Pile is around 94 feet with adjacent ground ranging between elevation 64 and 66 feet. Slopes of 1.5H:1V, increasing locally to 1:1, exist currently along the north, east and south sides of the hide pile. The existing slope flattens on the west side to approximately 5H:1V.

The grading plan for the South Hide Pile is presented on Sheets 11-7C and 11-7D and cross-sections are included as Sheet 11-8. The major design considerations in this instance were to reduce the slopes to 40 percent or flatter and to minimize the resulting land take in the developed areas to the north and east of the hide pile.

A maximum slope of 40 percent was achieved with a 2-foot high gabion retaining wall at the toe of the slopes on the northern side. The gabion retaining wall functions to reduce land take, facilitate the cover tie-in with the existing paved area, and provide drainage. The gabion retaining wall is illustrated as Detail 6 on Sheet 11-25. Slopes of 40 percent are also designed along the eastern side and 33 percent slopes are maintained on the southern side of the hide pile. A rip-rap toe drain is provided to transition from the permeable cover to the existing paved area on the east side and the Site boundary on the south side of the hide pile. The rip-rap toe drain is shown as Detail 5 on Sheet 11-25.

11.4.1.3 East-Central Hide Pile Grading

The East-Central Hide Pile is located north of Atlantic Avenue and south of the Western Branch of the Aberjona River (also referred to as Wetland 2A). The majority of the East-Central Hide Pile is located west of the Aberjona River. The location and extent of the hide pile, as defined in the Consent Decree, is presented on Sheets 11-2A and 11-2B.

The peak elevation of the East-Central Hide Pile is around 98 feet. Individual soil piles form several peaks along the eastern and southern sections of the East-Central Hide Pile. The hide pile is bounded on the north by Wetland 2A. The eastern side of the hide pile coincides with the Aberjona River and a pond known as Wetland 3C. The western section of the hide pile grades into areas of low relief.

The proposed grading plan for the East-Central Hide Pile is presented on Sheets 11-7A and 11-7B, and cross-sections are shown on Sheets 11-9 through 11-11. The grading plan was largely controlled by requirements to flatten steep slopes on the north side of the hide pile, covering of isolated soil piles and making allowances to facilitate the future extension of Commerce Way.

The northern limit of the East-Central Hide Pile is formed by the southern bank of Wetland 2A. The existing grade at this location is on the order of 60 percent as illustrated by cross-section G-G'. Flattening of this slope while avoiding major excavation of hide residues, leads inevitably to encroachment into the wetland channel. It was therefore necessary to route a section of the channel via culvert to accommodate the remediated slope. The extent of the proposed culvert is shown on Sheets 11-7A and 11-7B and details are presented in Chapter 13.

Several individual soil piles exist within the limits of the East-Central Hide Pile. To minimize excavations in areas potentially containing hide residue, the grading plan provides for placement of cover above existing grade on most soil piles. Cross-section F-F' illustrates the proposed grading in the vicinity of such a soil pile along the southern limit of the East-Central Hide Pile. A 2-foot high gabion retaining wall is used along the southern edge to avoid land take within the adjacent developed area.

The grading on the eastern slope of the East-Central Hide Pile has taken into account the potential future extension of Commerce Way. Slopes are generally maintained flatter than 33 percent. Slopes are increased locally to 40 percent as illustrated on cross-section H-H'.

11.4.1.4 West Hide Pile Grading

The West Hide Pile is located in the northwest portion of the Site close to the Site boundary. The eastern and southern sides of the hide pile are bounded by Wetland 1C. Boston Edison Right-of-Way No. 14 runs east-west across the center of the hide pile. The location and extent of the West Hide Pile, as defined by the Consent Decree, is shown on Sheet 11-2A.

The top of the West Hide Pile is saddled with peak elevations around 92 and 94 feet. The saddle produces a depression in the center of the hide pile and swales on the east and west slopes. The existing side slopes of the West Hide Pile vary between 1:1 to 1.5H:1V along the eastern side and southeast corner while flattening along the northern and western sides.

The proposed grading plan for the West Hide Pile is presented on Sheet 11-7A and cross-sections are shown on Sheets 11-12 and 11-13. The most significant design consideration for the West Hide Pile is the regrading of the slopes along the eastern side, to improve stability and reduce erosion while minimizing impact to the wetland. A maximum slope of 40 percent is designed.

Existing power poles on the West Hide Pile are to be removed and new power poles constructed on the hide pile by Boston Edison Company and/or contractors to the Boston Edison Company. It is understood that the new poles will be placed on piled foundation.

11.4.1.5 Cover in Developed Areas

The RDAP defines roads, buildings, parking lots, concrete slabs, and railroad lines as "cover equivalents". Such areas are defined on Sheet 11-4 and do not require construction of a cover. A detailed inspection of the existing parking lots has been undertaken to document their current condition. The results of this inspection are presented in Table 11-2. Pavement ratings completed for these areas are included in Appendix 11-I. The proposed Groundwater Treatment Plant and surrounding paved area will also constitute a cover equivalent when constructed.

The RDAP and the ROD recognize that the placement of an above grade cover in developed areas, for example against existing structures, may be inappropriate. The RDAP states that for small areas, such as the landscaped areas between buildings and parking lots, excavation and replacement with clean soil or asphalt may be proposed. The cover design in developed areas presented in Sheets 11-3A to 11-3D and 11-7A to 11-7D is consistent with these requirements. At grade covers are limited to small areas, and elsewhere excavations are only made immediately adjacent to buildings, with a suitable transition to above grade cover. The transition adjacent to buildings also incorporates a swale to control surface water (see Detail 7 on Sheet 11-24). Minor excavation is also required for the transition between above grade cover and existing roads, railway lines, and parking lots as presented in Detail 8 on Sheet 11-24.

The topography within the developed areas of the Site has little relief and the above grade cover is represented by shading on Sheets 11-7A to 11-7D. Also presented are transitions from at grade cover to above grade cover. At grade cover locations, where the existing soil will be excavated and replaced with cover, are presented on Sheets 11-3A to 11-3D.

Asphalt cover equivalents are proposed at some locations within the developed areas of the Site, at the request of particular Landowners, as shown on Sheets 11-3A to 11-3D and 11-7A to 11-7D.

11.4.2 Permeable Cover Section

The cover section consists of a nonwoven geotextile overlain by 16 inches of clean soil. The Alternate Cover Design Report (Golder, 1989) proposed the use of a 4 oz/yd^2 nonwoven geotextile. The weight of the geotextile has been upgraded to improve its survivability and the durability of the cover. The different weights of geotextile selected for specific locations are described below.

In areas other than the hide piles where the remediated slope is flatter than 25 percent, a 6 oz/yd^2 nonwoven geotextile is used. In areas of hide piles where slopes are flatter than 25 percent, a 16 oz/yd^2 nonwoven geotextile is used to provide a drainage/capillary break medium. This drain will function to help prevent sloughing during thawing in case frost penetration in excess of the average occurs. In areas where the remediated slope is between 25 percent and 33 percent, other than on hide piles, a 16 oz/yd^2 nonwoven geotextile is also used. For slopes in this gradient range on hide piles, a geocomposite drain (Tex-Net TN3002CN or similar) comprising a geonet with geotextile factory bonded on both sides is utilized. This drain is extended to an elevation 10 feet above the toe of slope to intercept any seepage which may occur as a result of groundwater mounding within the hide piles. Details 2 and 3 on Sheet 11-24 show the permeable cover sections which include the drain. Where a geocomposite drain is present, the 16 oz/yd^2 nonwoven geotextile will be sewn to the lower of the geotextile layers of the geocomposite.

Cover soils placed over the geotextile will be granular materials with inherently low potential to clog the geotextile. An analysis of clogging potential (Appendix 11-J), indicates that typical off-Site borrow soils easily meet published criteria to prevent geotextile clogging due to migration of soil particles.

A rip-rap toe drain is provided around the base of the West Hide Pile. The toe drain functions to provide drainage and a transition to Wetland 1C, mitigation area wetlands or to the permeable cover. The transition to Wetland 1C is illustrated in Details 1 and 2 on Sheet 11-25, Detail 6 on 11-26 and in cross-sections on Sheets 11-12 and 11-13. The transition of the toe drain to permeable cover is indicated in Detail 3 on Sheet 11-25.

A rip-rap toe drain also provided along the north side of the West Hide Pile as illustrated in Detail 4 on Sheet 11-25 and cross section M-M′ on Sheet 11-13.

On the South Hide Pile, a rip-rap toe drain is provided along the toe of the slope on the east and south sides. The toe protection is shown on Detail 5 on Sheet 11-25, and on cross-sections A-A′ and D-D′ on Sheet 11-8. The toe drain provides a transition to an existing parking lot on the east side and to the Site boundary on the south side of the hide pile. A gabion retaining wall is used on the north side of the South Hide Pile as shown in Detail 6 on Sheet 11-25.

The drainage layer on the East-Central Hide Pile extends into a drainage channel on the east side. Where a suitable discharge point does not exist, the drainage layer is extended 10 feet beyond the toe of slope and sewn with the geotextile of the surrounding cover (see Detail 7 on Sheet 11-25).

In all areas where the remediated slope is steeper than 33 percent, a geogrid reinforcement layer is included at the base of the cover soil immediately above the geosynthetic or a rip-rap facing is used for localized areas.

11.4.2.1 Gravel Surface Cover

A gravel surface will be used instead of vegetation in certain areas of the soil cover (see Sheets 11-3A to 11-3D and 11-7A to 11-7D). This feature was requested by particular Landowners to simplify cover maintenance. A 3-inch thick layer of open graded gravel is proposed overlying 13-inches of cover soil as shown on Details 1 and 2 on Sheet 11-26. A gravel surface is only used in flat, limited areas of the Site to prevent cover erosion. The gravel surface is intended solely to simplify maintenance and not to provide a surface to support vehicular traffic.

11.4.2.2 Asphalt Cover

The RDAP permits the use of asphalt cover in developed portions of the Site adjacent to existing buildings. Asphalt cover is incorporated in the design where requested by individual Landowners as indicated in Sheet 11-3A to 11-3D. The asphalt cross-section is shown in Details 5 and 6 on Sheet 11-24 and comprises, from bottom to top:

- 6 oz/yd^2 nonwoven geotextile
- 6-inch granular subbase
- 4-inch asphalt binding course
- 2-inch asphalt wearing surface

An asphalt access roadway of essentially the same cross-section is proposed in several locations along the Boston Edison Right-of-Way No. 9 and in Right-of-Way No. 14 on the West Hide Pile (see Sheets 11-3A to 11-3D and 11-7A to 11-7D). These roadways have been designed at the request of Boston Edison to provide access to power distribution poles. In these instances, a 10 inch subbase is used so that the total cover thickness is the same as the adjacent soil cover.

11.4.3 Drainage

The surface water drainage plan for the proposed grading of the Site is presented in Sheet 11-21. The surface water drainage patterns existing prior to construction are generally maintained in the developed areas of the Site. As discussed in Section 11.4.1.1, the hide piles are remediated with smooth, constant slopes. A minimum grade of 3 percent is generally maintained on the tops of the hide piles to facilitate drainage. In small isolated areas the gradient is slightly less than 3 percent in order to avoid excess fill surcharging at the crest of slopes.

Surface water management structures have been designed along the east side of the East Central Hide Pile. These structures consist of a drainage channel, culvert, and outlet structure. Details are provided on Sheet 11-23. The water is discharged to the Aberjona River via a gabion splash pad. Hydrological design calculations are presented in Appendix 11-K.

Conceptual designs of detention basins, inlet and outlet structures, and drainage patterns in areas of the Site where an asphalt cover option has been requested by a Landowner are provided in Chapter 13.

11.4.4 Consolidation Area

As required by the Consent Decree, a consolidation area has been designed to dispose of soils excavated on Site after remediation, containing metals below action levels.

The consolidation area is located on the southern slope of the East-Central Hide Pile and has a capacity of approximately 14,000 cubic yards, with a maximum height that does not exceed the permeable cover contours. Side slopes should not exceed 8H:1V during disposal periods and final slopes are designed at a maximum of 10H:1V.

The area selected for the consolidation area will be capped with the permeable cover as part of the remedial work. After remediation, the soils to be disposed of in this area will be placed on the cover in a manner that does not exceed the limits and slopes of the closure plan. Temporary seeding shall be used as necessary between filling stages. Appropriate erosion and sedimentation measures such as silt fences and hay bales should be used to prevent sediment transport off-Site or into waterbodies. Closure will comprise capping with a 4 inch topsoil cover after a period of seven years or when the consolidation area is full, whichever is sooner as specified by the Consent Decree.

A preliminary proposed final closure plan is presented in Sheet 11-22. However, this plan should be modified at the time of closure to adjust it to the final dimensions of the soil pile existing at that time.

The consolidation area will be secured with a fence as shown on Sheet 11-22 and access will be provided from Atlantic Avenue.

11.4.5 Utility Corridor

A clean utility corridor has been designed in the areas of a potential future road alignment which passes through areas containing soil with metals above action levels. The utility corridor will be excavated and backfilled with clean soil. A single corridor has been designed, located to the west of the Aberjona River as an extension of Commerce Way. The corridor is 20 feet wide at its base and has a depth of approximately 8 feet from a conceptual final road grade. The utility corridor is shown in plan on Sheet 11-7B, in profile on Sheet 11-20 and in Detail 5 on Sheet 11-23.

11.4.6 Gravel Access Road

Gravel roads are designed to facilitate access on-Site. A gravel access road is provided from the termination of Commerce Way to the gas treatment system to facilitate operation and maintenance. The road is 20 feet wide becoming 15 feet wide as it turns north to the gas treatment system. The road design is shown Detail 6 on Sheet 11-23. Design calculations are included as Appendix 12-H.

11.4.7 Erosion and Sedimentation Control

Proper erosion and sedimentation control measures will be maintained during the construction of the remedy. Straw bales and silt fences (see Details 3 and 4, Sheet 11-28) will be placed along the perimeter of the hide piles to prevent the transport of sediment into the adjacent wetlands and streams. The straw bales and silt fences will be properly maintained and replaced as needed until the construction of the cover system has been completed and vegetation has developed. If necessary, temporary erosion control measures such as erosion mat and diversion swales at the crest of slope should be utilized prior to permanent vegetation. Additional erosion and sedimentation details are provided in the Specifications.

11.4.8 Construction Considerations

Prior to placement of the permeable cover in areas other than the hide piles, all existing vegetation is to be cleared and root matter grubbed. In areas of the hide piles, all existing above ground vegetation is to be cleared, tree trunks cut to ground surface, and the root mat left in place. Woody material from above ground, roots, and other vegetation will be chipped and composted for later placement as fill under permeable cover. In order to prevent certain species (notably phragmities) from re-establishing growth, a herbicide may also be employed. Composted material will be spread in thin lifts on the flatter areas of the hide piles and tilled in prior to subgrade proof rolling.

The existing grade shall be proof rolled prior to placement of any fill to minimize the potential for surface sloughing. The maximum slope required to be proof rolled is 2.5H:1V. Where existing slopes are steeper than

2.5H:1V, the thickness of fill is significant and will function to prevent surface sloughing.

The hide piles will be proof rolled to compact the Surficial Materials. Fill material placed to stabilize the hide pile slopes will be placed and compacted in horizontal layers not thicker than a 12 inch loose thickness to a density equivalent to 95 percent of Standard Proctor. All fill placement and cover construction will be carried out from toe to crest of the slopes, so that the slope stability is not temporarily reduced by construction operations. Existing vegetation shall be cleared in stages just in advance of filling operations. The final prepared grade over the entire cover area will be proof rolled with a smooth-wheeled roller prior to placement of the geotextile. A smooth wheeled compactor of 10 ton minimum weight will be utilized for proof rolling.

In Site areas other than the hide piles, cleared and grubbed subgrades will also be proof rolled with a similar compactor.

Abandoned underground facilities are known to exist in parts of the Site as described in Chapter 5. Known, abandoned, below grade tanks will be cleaned and backfilled with lean concrete. The locations and decommissioning methods for other known features are shown on Sheet 11-5. Other abandoned below grade features which may be discovered during construction of the cover will be similarly backfilled if it is judged that they could otherwise impair the long-term effectiveness of the remedy.

Additional above grade features also exist at the Site which will require removal or demolition, Sheet 11-5. Disposition of debris from these features will vary depending on specific material types. Steel tanks or other metal debris may be decontaminated and removed from the Site following appropriate protocols for disposition of the material. Concrete from demolition operations may be crushed and reinforcing steel removed. The crushed concrete may then be used as fill on-Site subject to adherence to specific material specifications. Decontaminated reinforcing steel may be removed from the Site following appropriate protocols.

Approximately 60 monitoring wells will require decommissioning following the procedures outlined in the Specifications. Locations of these wells are shown on Sheet 11-6. Additionally, approximately 60 piezometers exist at locations throughout the Site which also have to be properly abandoned. New monitoring wells will be installed on completion of the cover to monitor the performance of the groundwater extraction system. Locations will be specified in the 100% Design Report Part II.

Material excavated elsewhere on the Site will be the primary source of fill to regrade the slopes of the hide piles and flat areas of the Site. Soils excavated from wetland areas will require dewatering to bring them to a suitable moisture content to allow placement as fill. As wetlands sediments may contain metals above action levels, it will be necessary to carefully control the dewatering process to ensure that water removed from the soils is contained and undergoes proper disposal. Depending upon water quality and schedule it may be possible to treat this decanted water in the on-Site groundwater treatment plant.

In general on-Site materials used for fill on the hide piles must be granular material relatively free of fines and other deleterious materials as outlined in the Specifications. Excavated soils not meeting this requirement may be placed as fill in permeable cover areas of the Site flatter than 8H:1V. Organic material shall be placed at a maximum thickness of 3 feet in any location to minimize the potential for settlement. Depending upon specific quantities and quality of excavated materials, fill placement on the East-Central Hide Pile may deviate from the contours shown on the grading plans. However, no slopes may exceed 4H:1V unless currently shown on plan, and drainage directions may not deviate significantly from those on the grading plan.

After proof rolling and filling to reach subgrade elevation the geotextile will be placed. All seams will be carefully sewn and inspected for quality control by the Q.A. Inspector. In hide pile areas where geocomposite is used instead of geotextile, adjacent sheets of geocomposite will be seamed by tying the geonets together and sewing the upper geotextile sheets. All seaming will be inspected by the Q.A. Inspector.

In those areas with slopes steeper than 33 percent, geogrid will be placed on top of the geocomposite. Adjacent sheets will be tied, and roll ends joined together using a HDPE bar as outlined in the Specifications.

The soil cover will then be placed over the geosynthetics. The soil will be placed in a manner that minimizes imposed stress on the underlying geosynthetics by using low ground pressure earth moving equipment and maintaining a minimum thickness of 12 inches of soil between placing equipment and the geosynthetic at all times. So as to enhance the stability of the hide pile cover, soil shall be placed from the base of the pile toward the top. Cover soil in areas of permeable cover that will not be paved will be nominally compacted by the action of the placing equipment only.

In areas where the permeable cover ties into wetlands or drainage facilities, careful construction will be required to properly build the transition details. In these areas a number of geosynthetics will have to be joined with multiple sewn seams, all of which will be inspected. In areas of the wetlands and drainage channels, subgrades are likely to be easily disturbed, so it is important that the contractor minimize potential disturbance by the use of properly sized and operated equipment.

Throughout the permeable cover areas of the Site, changes in existing grade will require adjustment of a number of utility facilities such a manholes, drop structures and pole guy wires. Modification of these facilities will need to be coordinated with a variety of utility and public works agencies and will be controlled by the Trust's Representative.

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TABLE 11-1 WEST HIDE PILE STABILITY				
SECTION	GROUNDWATER CONDITION	FAILURE MODE	FACTOR OF SAFETY	
			EXISTING	REMEDIATED
L-L'	Long Term	Surface Sloughing	0.6	N/A
		Circular	1.3	1.6
		Non-Circular	1.4	--
	Perched Water Table	Surface Sloughing		
		- seepage emerging from slope	0.0	N/A
		- seepage parallel to slope	0.3	N/A
		Circular	1.3	1.6
J-J'	Long Term	Surface Sloughing	0.8	N/A
		Circular	1.6	--
I-I'	Long Term	Circular	--	1.7

TABLE 11-2 ASPHALT RATING SUMMARY

7/18/91

Address	Owner	Area within Cap (SF) [1]	Defect Ratings: Linear Cracks 0-10 [2]	Alli-gator Cracks 0-20	Upheaval 0-20	Pot Holes 0-10	Raveling 0-20	Grade Depres-sions 0-20	Total Defect Ratings	Condition Rating [3]	General Condition Rating [4]	Typical Type of Repair or Maintenance
10 Atlantic Avenue	Atlantic Ave. Trust	14,764	5	5	2	0	7	3	22	78	Fair	Fill Cracks & Overlay
15 Atlantic Avenue	Atlantic Ave. Assoc.	150	0	0	0	0	3	1	4	96	Good	Seal Coat or Slurry Seal
20 Atlantic Avenue	Winter Hill Store House	78,430	4	8	2	2	6	4	26	74	Poor	Patch & Overlay/Coat
130 Commerce Way	Sunder & Hiro Ganglani	27,203	3	6	1	3	4	7	24	76	Poor	Patch & Overlay/Coat
210 New Boston St.	Pebco	30,866	4	4	2	2	2	3	17	83	Fair	Fill Cracks & Overlay
211 New Boston St.	Dagata	1,104	5	5	1	5	5	2	23	77	Poor	Patch & Overlay/Coat
216 New Boston St.	PX Realty	88,757	3	3	1	2	2	2	13	87	Fair	Fill Cracks & Overlay
217 New Boston St.	J. Koster	18,406	1	1	4	1	2	2	11	89	Fair	Fill Cracks & Overlay
219 New Boston St.	J. Koster	5,134	2	1	2	1	1	3	10	90	Good	Seal Coat or Slurry Seal
223 New Boston St.	Aero Realty	10,230	4	6	2	3	3	1	19	81	Poor	Patch & Overlay/Coat
225 & 227 New Boston St.	J. Koster	18,194	2	3	1	1	3	2	12	88	Fair	Fill Cracks & Overlay
229 & 231 New Boston St.	J. Koster	14,183	2	1	2	1	3	3	12	88	Fair	Fill Cracks & Overlay
204 Merrimac St.	Positive Start	43,400	2	1	1	0	1	1	6	94	Good	Seal Coat or Slurry Seal
225 Merrimac St.	PX Realty	45,101	4	5	5	1	6	4	25	75	Poor	Patch & Overlay/Coat
Atlantic Ave.	Remedial Trust	2,312	4	4	0	1	1	0	10	90	Good	Seal Coat or Slurry Seal

NOTES:

[1] Area – The area evaluated is the portion of the parking lot or street within the permeable cover limits.

[2] Defect Rating – The Defect Rating indicates the relative frequency of defects within the scale range for each defect catagory. For example a zero defect rating indicates there is no problem. Defect Ratings at the mid-range of the scale indicates the defect is significant and characterizes approximately half the area. Likewise, a defect rating that equals the upper end of the scale would signify the defect is predominant over the entire pavement area.

[3] Condition Rating – The condition rating value is the sum of Defect Ratings subtracted from 100. The Condition Rating provides a general indicator of the type and degree of repair work necessary.

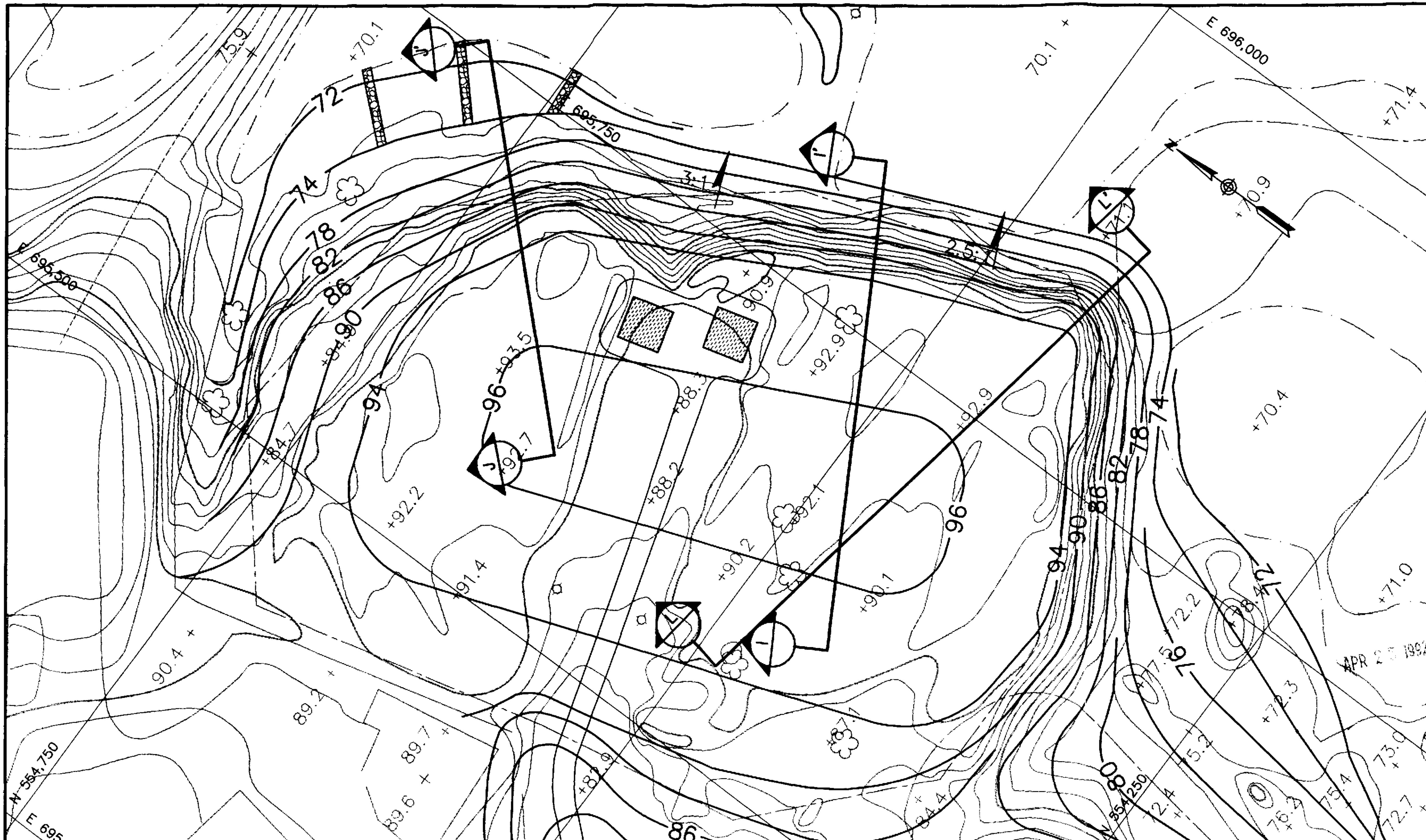
[4] General Condition Rating:

Good – Exhibits fine cracking and some raveling of fine aggregate; the ordinary effects of wear and tear.

Fair – Characterized by random cracks of up to (1/2 in.) in width, and raveled aggregate.

Poor – Displays random cracks, raveled aggregate, depressions, local alligatored areas, pot holes and upheaval.

C:TB11-1.WK1



JOB No.: 903-6400	SCALE: 1"=50'
DRAWN: RDH	DATE: 05/18/92
CHECKED: PSf	FILE No.: 1MA01-944

Golder Associates

WEST HIDE PILE SLOPE STABILITY CROSS SECTION PLAN

INDUSTRI-PLEX SITE REMEDIAL TRUST

FIGURE **11-1**

LEGEND

PRELIMINARY EXTENT OF COVER COMPRISED OF:

AREA WITH As, Pb AND/OR Cr CONCENTRATIONS AT OR ABOVE CONSENT DECREE ACTION LEVELS AND/OR WETLANDS CONTAINING HIDE RESIDUE

HIDE PILES BASED ON CONSENT DECREE

STREAMS AND LIMIT OF WETLANDS TO BE REMEDIATED CONTAINING As, Pb AND/OR Cr CONCENTRATIONS AT OR ABOVE CONSENT DECREE ACTION LEVELS AND/OR HIDE RESIDUE

SITE BOUNDARY

PROPERTY BOUNDARIES

STREAMS AND WATERCOURSES

SOIL PILES

MARSH

NOTES

1.) PROPERTY SURVEY PERFORMED BY SAIC ENGINEERING INC., APRIL 1990.

2.) COVER EXTENT INCLUDES AREAS WITH EXISTING COVER IN THE FORM OF BUILDINGS AND PAVED AREAS.

APR 25 1992

REV	DATE	DESCRIPTION	DR BY	RVW BY

SCALE: AS SHOWN

PROJECT No. 903-6400

DES BY	ALK	12/04/90
DR BY	JSG	12/04/90
CHK BY	RPT	1-31-91
RVW BY	RCR	1-31-91

PROJECT: INDUSTRI-PLEX SITE REMEDIAL TRUST WOBURN, MASSACHUSETTS

SHEET TITLE: PRELIMINARY EXTENT OF COVER

Golder Associates
Mt. Laurel, New Jersey

SHEET 1 OF 1

DRAWING No. MA01-282

FIGURE 1

APPENDIX 11-A

Alternate Permeable Cover Design

1989 APPROVED ALTERNATIVE PERMEABLE COVER DESIGN

This section is reproduced from the PDI Task S-3 Interim Final Report (Golder Associates, 1990b)

2.2 Permeable Cap Requirements

A cost effective permeable cover is discussed in the Alternative Cover Design Report (ACDR) prepared by Golder Associates (1989). This alternate cover design was subsequently approved by the USEPA and MDEP. Specifically, the permeable cap components as approved by USEPA and MDEP are (from bottom to top):

1. A geotextile; and
2. A 16-inch thick imported soil fill.

The factors that were considered in the selection of the alternate cap included:

- Elimination of direct contact of contaminated soils with the public;
- Effect of freeze/thaw cycle;
- Effect of erosion;
- Durability and long-term reliability, and
- Quality control during installation.

2.2.1 Geotextile

The geotextile will serve several functions. First, it will provide a visual definition of the top of the contaminated soils and provide separation between the contaminated soils and the imported borrow soil. The geotextile can be specifically included in the institutional controls for the site as a further means of reducing the chance of incidental contact through land use. Secondly, the geotextile will inhibit the upward migration of stones and construction debris from the existing soil matrix as a result of freeze/thaw. The geotextile, itself, is not subject to freeze/thaw effects and will allow water to freely move upward or downward. In addition, the geotextile can have sufficient mechanical strength and modulus to resist uplifting objects from the contaminated soils. Thirdly, it provides a continuous barrier in the event the soil cover is eroded or locally disturbed. Lastly, the geotextile discourages root penetration into contaminated soils.

The ACDR indicates several properties of the geotextile that will meet or exceed the engineering requirements and functions at the site. The geotextile shall be made of polypropylene or polyester. These materials are considered to have a high degree of biological and chemical stability as described in the ACDR. The effective opening size shall be approximately 0.2 mm (No. 70 sieve size) to minimize the potential of fine grained particles migrating between the contaminated soil and the cover soil. Puncture strength is an important property of a geotextile, particularly in relation to the vertical displacement of objects due to freeze/thaw action. The ACDR indicates that a puncture strength of 40 pounds is adequate to resist upward migration of objects due to freeze/thaw. The ACDR recommends that a non-woven geotextile with a unit weight of 4 ounces per square yard is suitable to meet the functions required at the site.

In addition, several measures should be taken to ensure a stable foundation for the geotextile. These steps include clearing and grubbing, proof rolling, excavation of, or placement of, additional fill over areas that may puncture the geotextile or cause substantial settlements.

2.2.2 Cover Soil

The permeable cap cross-section approved by USEPA and MDEP requires a 16-inch thick cover soil overlying the geotextile. The cover soil has been designed to serve several functions.

First, the soil cover will function in conjunction with the geotextile as a physical barrier to prevent direct contact with contaminated soils. Secondly, it will help mitigate the impact of freeze/thaw and erosion. The depth of frost during an average winter was calculated to remain within a 16-inch cover. Regarding erosion, it was demonstrated in the ACDR that the amount of erosion in locally damaged areas of the cover is not expected to be greater than 4 inches per year, therefore any damaged areas can be repaired as part of the maintenance program.

Thirdly, the soil cover must sustain vegetation growth. This is an important factor in evaluating its durability. A vegetated surface will greatly reduce erosion and also control the effects of freeze/thaw. Lastly, the ACDR demonstrated that 12 inches of soil over the geotextile is the upper bound for root penetration and protection of the geotextile during construction. The likelihood of phytotoxicity is reduced since roots are not likely to encounter contaminated soils. The potential for geotextile damage during construction is also minimized by placing a 16 inch layer of cover soil.

The ACDR does not specify or suggest a particular soil type or gradation for the cover. It does reference certain cover soil properties necessary to achieve the desired functions. The report specifies the cover soil shall be a mineral soil which will not breakdown or degrade in the natural environment. The cover soil shall also have the ability to support vegetative growth. The report indicates that materials suitable for growth of a vegetative cover will either have sufficient fines or would be blended with fine-grained soils. The ACDR states that it is expected the cover soil will generally have a fines content greater than 20 percent which is equal to or greater than that for the majority of the site. The use of mulch and fertilizer can also be used to enhance vegetative growth.

Strength and compressibility are not significant properties for the 16 inch cover soil, since it will not be required to withstand significant loading. In fact, it is suggested that the soil cover be placed in a single lift and spread with low ground pressure equipment in order to minimize disturbance to the underlying geotextile. It would also be difficult for rapid and persistent vegetative growth to take place on a compacted surface.

Strength, compressibility and compaction are of importance in areas where a significant thickness of fill will be required during regrading operations. Strength and compressibility requirements are dependeant on the type of land use (i.e., roads, parking lots, open areas). In these areas, all fill layers, except the uppermost, shall be placed and compacted in controlled engineered lifts consistent with the future land use of a particular area.

Reference:

Golder Associates Inc., May 1989, Alternative Cover Design, Woburn, Massachusetts, prepared for Monsanto Chemical Company.

APPENDIX 11-B

Typical Landowner Package

INDUSTRI-PLEX SITE

REMEDIAL TRUST

36 COMMERCE WAY
WOBURN, MA 01801

IMPORTANT - IMMEDIATE ACTION REQUIRED

CERTIFIED MAIL - RETURN RECEIPT REQUESTED

Dear

Pursuant to Section XVI.B.(2 & 3) of the Industri-Plex Consent Decree entered in United States District Court for the District of Massachusetts on April 24. 1989, you are hereby notified as a Landowner on the Industri-Plex Superfund Site, by the Settlers under the Consent Decree, of certain important information concerning the proposed remedy as it affects your property, and of your opportunity to make a decision concerning the type of cover you would select for your property. As explained below, however, you have only a limited period of time in which to make this selection. Enclosed are sampling maps showing known locations and concentrations of Hazardous Substance on your property and reasonable interpolations of such data delineating the areas on your property containing hazardous substances above action levels that will receive cover. Also enclosed is information on the basic type of cover selected for your property by the Remedial Trust, other types of cover consistent with the Remedy that we anticipate will be feasible to place on each area and the incremental cost of the various options over the basic cover. You may select one of the various alternative cover designs described herein, but you will be responsible for the difference in cost between the base cover and the options you select.

Please be advised that it is your responsibility to select a cover option appropriate for your intended use of the property. You will be responsible for the cost of any repairs to the cover resulting from surface use. Moreover certain cover options simply may not be appropriate for your intended use. Certain cover options may be more expensive initially than other options, but may require less maintenance and therefore may be more

TEL. 617-932-9599
FAX 617-932-9589

economical in the long run. You will have to make this choice in view of your intended use of the property.

EPA in consultation with the Commonwealth must approve the specific cover design. This means that your preferred option for an alternative cover on your site may not be acceptable to the Agencies. There may also be circumstances in which the cover you prefer will prove to be infeasible, because of overall design considerations such as integrating the individual landowner covers.

Please note that <u>you must notify us in writing within 30 days</u> of receipt of this letter of your preferences as to the type(s) of cap or cover to be placed on the specified locations or of your preferences for excavation and backfilling of designated areas on your property. If you do not notify us of a cover selection within this period, we will have no choice but to assume that you will be satisfied with our selection(s) of cover.

The Remedial Trust plans to hold a group meeting with interested landowners shortly after you receive this notice. You will be advised of the meeting location and time via a separate communication. Should you have wish to discuss any aspects of this transmittal we would be pleased to meet with you or your authorized representative by appointment in our 36 Commerce Way office on February 26 & 27 or at another mutually convenient time. Please call me at (617) 932-9599 or (314) 694-1617 collect to arrange an appointment .

Sincerely,

Warren L. Smull
Coordinator

LANDOWNER INFORMATION PACKAGE

The contents of this package are as follows:

1. Figure 5-1, Preliminary Extent Of Permeable Cover, Industri-Plex Site Remedial Trust Project, Woburn, Massachusetts.

This drawing provides you with an overview of the Site boundaries and the approximate extent of the areas of the Site which will receive permeable cover.

2. Figure 2, Extent Of Hazardous Substances.

This drawing shows your specific property, your property boundaries and the sampling points that were used during the RI/FS and/or Predesign Investigation studies.

3. Figure 3, Landowner Cover Options.

This drawing shows your property and defines the various areas of the property that are to receive cover, cover equivalents or that currently have cover equivalents in place.

4. Figure 4 Thru Figure 9, Cover Cross-sections.

These drawings show typical cross-sections of the cover at various locations on your property.

5. Table 1, Summary Of Soil Sample Analyses

This table displays the arsenic, lead, chromium and hide soil sample analyses for your property. The borehole numbers allow you to determine the sample location on Figure 2. Please note the action levels for the three metals on the cover sheet. Exceeding one or more of these levels is the basis for cover being required.

6. Table 2, Landowner Cover Options.

This table details for each area of your property shown on Figure 3: (a) the current condition of the area; the cover proposed for that area by the Remedial Trust, which will be provided as part of the basic remedy and at no additional cost to the landowner; (b) alternate cover No.1, this is a possible option to the Proposed ISRT Cover and the additional cost to the landowner is stated; and (c) alternate cover No. 2, this is another possible option and the additional cost over the Proposed ISRT Cover is stated.

The costs identified herein are preliminary estimates. Prior to construction, the Remedial Trust will prepare a final estimate of the additional cost of any cover options you have chosen. The additional cost for construction of the options is payable by the owner prior to construction. If during construction we encounter conditions previously unknown to the Remedial Trust that will result in significantly higher costs than anticipated to install an option we will suspend work and notify you. Should you chose to proceed with the option you will be obligated to reimburse the Remedial Trust for such additional costs. Conditions previously unknown to the Remedial Trust include undisclosed utilities, underground obstacles, unexpected contamination or similar occurrences.

In considering cover options and the Proposed ISRT Cover the landowner should include the cost of surface maintenance and repair, which will be a landowner responsibility, as part of the cost of the cover. For example, grassed surfaces will require mowing, reseeding and erosion control; asphalt surfaces will require periodic resealing; and concrete will require periodic recaulking of expansion joints. These maintenance costs will be in addition to repair and replacement costs which will vary with the surface usage and type of surface. Thus the landowner should take into consideration the total cost, including initial construction cost and continuing maintenance and repair costs, when selecting a cover for an expected surface use.

7. Other Information

Various properties on the Site have materials and rubble stored on areas that are to receive cover. We would encourage you to dispose of any materials no longer of value to you prior to the initiation of cover construction on your specific site, and we would be pleased to work with you such that you can move materials of value prior to cover construction on that portion of the site.

In the alternative the Remedial Trust will move any materials left on areas to receive cover without assuming liability for any damage resulting from such movement and replace them on the top of the completed cover. If you wish, we will attempt to arrange, on your behalf, for disposal of any unwanted materials on your site and dispose of them at your additional cost. These arrangements must be made and any applicable fees paid prior to initiation of construction.

Please note that the EPA in consultation with the Commonwealth must approve the specific cover design. This means that your preferred option for an alternative cover on your property may not be acceptable to the Agencies. Please also note that there may be circumstances in which your preferred cover will prove to be infeasible, because of overall design considerations such as integrating all the individual covers. In some cases, combinations of the base case and any options available may be possible. The Remedial Trust will be pleased to discuss these possibilities with you.

TABLE 1

SUMMARY OF SOIL SAMPLE ANALYSES

NOTES:

1. The Consent Decree Action Levels are 300 ppm for Arsenic, 600 ppm for Lead and 1000 ppm for Chromium (ppm = parts per million).

TABLE 1: SUMMARY OF SOIL SAMPLE ANALYSES

Soil Samples

Borehole Number	Sample Depth Inches	Arsenic (ppm)	Lead (ppm)	Chromium (ppm)	Was Hide Residue Detected?
23	006	256	646	76.6	No
23	018	1670	2830	64.9	No
23	022	3460	1720	74.6	Yes
2733	0-096	Not Tested	Not Tested	Not Tested	No
2733	012	255.00	7050.00	37.30	No
2733	036	189.00	8430.00	122.00	No
2733	060	726.00	5000.00	637.00	No
2830	0-096	Not Tested	Not Tested	Not Tested	No
2830	012	27.00	130.00	15.60	No
2830	036	23.00	20.00	2.60	
2830	060	1600.00	1070.00	7.80	No
2830	084	7.00	Not Detected	1.60	No
2832	0-048	Not Tested	Not Tested	Not Tested	No
2832	012	98.00	749.00	33.90	No
2832	036	212.00	5600.00	32.20	No
2832	060	122.00	430.00	149.00	No
2832	072-096	Not Tested	Not Tested	Not Tested	No
2929	012	116.00	590.00	150.00	No
2929	036	606.00	340.00	7.60	No
2929	060	561.00	941.00	20.10	No
2929	084	311.00	1060.00	24.80	No
2931	0-096	Not Tested	Not Tested	Not Tested	No
2931	012	162.00	3200.00	79.70	No
2931	036	361.00	7000.00	51.10	

TABLE 1: SUMMARY OF SOIL SAMPLE ANALYSES

Soil Samples

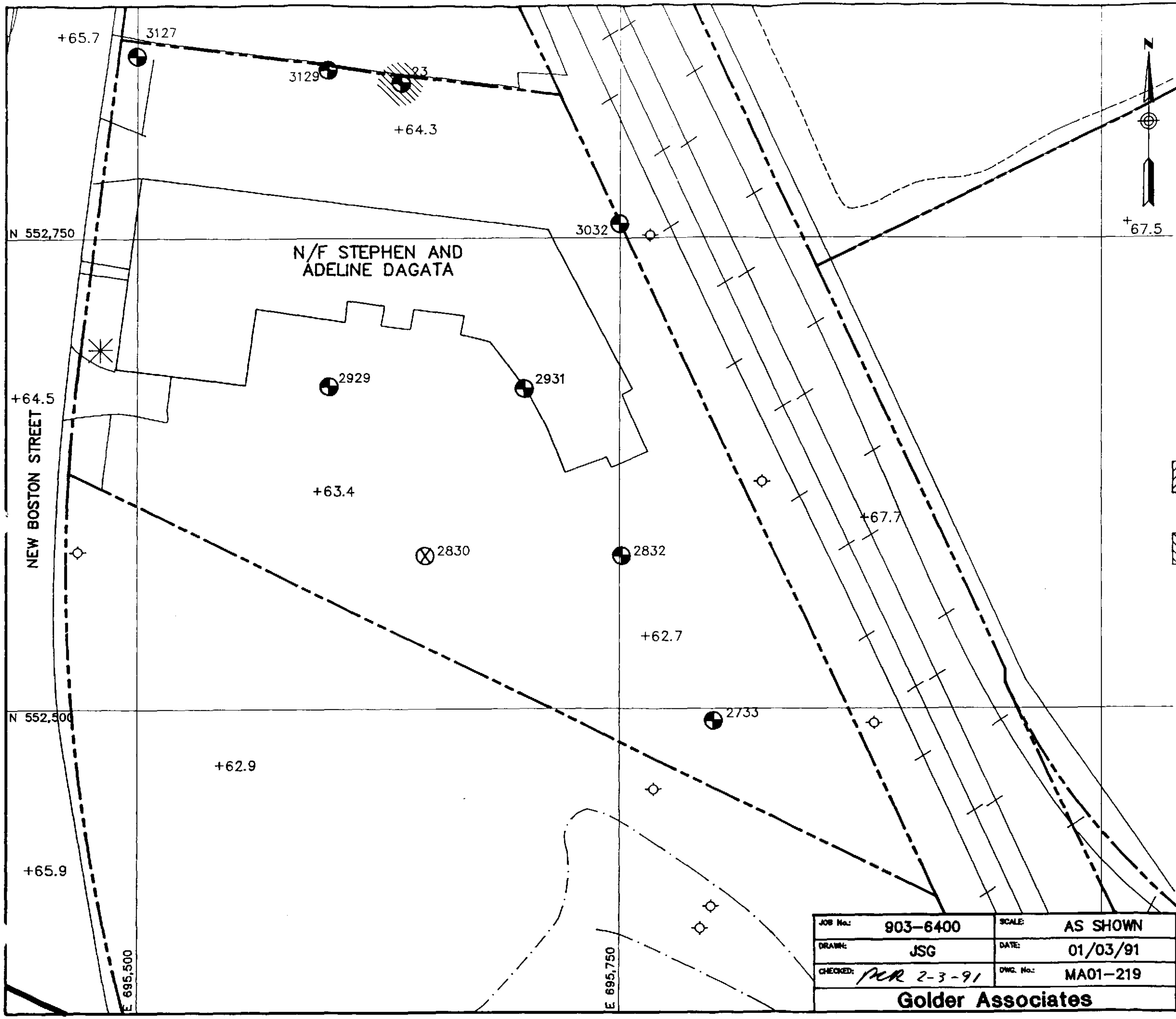
Borehole Number	Sample Depth Inches	Arsenic (ppm)	Lead (ppm)	Chromium (ppm)	Was Hide Residue Detected?
2931	060	990.00	8740.00	25.40	No
3032	0-096	Not Tested	Not Tested	Not Tested	No
3032	012	123.00	1310.00	38.50	No
3032	036	277.00	2600.00	13.70	No
3032	060	197.00	1210.00	8.20	No
3127	0-072	Not Tested	Not Tested	Not Tested	No
3127	012	1050.00	819.00	101.00	No
3127	036	1210.00	1120.00	8.50	No
3129	0-096	Not Tested	Not Tested	Not Tested	No
3129	012	12.00	540.00	67.70	No
3129	036	879.00	2700.00	96.00	No
3129	060	990.00	3900.00	34.10	No

TABLE 2

LANDOWNER COVER OPTIONS

AREA	CURRENT CONDITION	PROPOSED ISRT COVER TYPE	PROPOSED ISRT COVER FIGURE NUMBER	ALTERNATIVE COVER TYPE	ALTERNATIVE COVER FIGURE NUMBER
1	planter w\ grass, shrubs, small trees	at grade permeable cover	5	at grade asphalt cover	6
2	bare ground, some vegetation	at grade permeable cover	5	at grade asphalt cover	6
3	bare ground, some vegetation	above grade permeable cover	4	above grade asphalt cover	7
4	paved	no action	-	no action	-
5	building	no action	-	no action	-

NOTES
1. For area locations, see Figure 3.
2. Permeable cover grade will be locally adjusted to suit tie-in to existing buildings, paved areas etc.
3. Costs do not include the removal of obstructions, stored materials etc. (e.g. drums, construction equipment, vehicles) prior to covering.
4. Costing based on MEANS Heavy Construction Cost Data (1990) adjusted for construction in 1994.



LEGEND

- ○ BORING OR TEST PIT WITH As, Pb AND/OR Cr CONCENTRATION LEVELS BELOW CONSENT DECREE ACTION LEVELS. DATA AVAILABLE FOR ONLY ONE SOIL HORIZON WITHIN 36 INCHES OF GROUND SURFACE. BORING OR TEST PIT LOG CHECKED FOR HIDE RESIDUE.
- ⊗ BORING OR TEST PIT WITH As, Pb AND/OR Cr CONCENTRATION LEVELS BELOW CONSENT DECREE ACTION LEVELS. DATA AVAILABLE FOR MULTIPLE SOIL HORIZONS WITHIN 36 INCHES OF GROUND SURFACE. BORING OR TEST PIT LOG CHECKED FOR HIDE RESIDUE.
- BORING OR TEST PIT WITH As, Pb AND/OR Cr CONCENTRATION LEVELS AT OR ABOVE CONSENT DECREE ACTION LEVELS. DATA AVAILABLE AT ONE OR MORE SOIL HORIZONS WITHIN 36 INCHES OF GROUND SURFACE. BORING OR TEST PIT LOG CHECKED FOR HIDE RESIDUE.
- □ BORING OR TEST PIT LOG CHECKED FOR HIDE RESIDUE. As, Pb, AND/OR Cr DATA AVAILABLE FOR GREATER THAN 36 INCHES ONLY.
- ⊠ BORING OR TEST PIT LOG CHECKED FOR HIDE RESIDUE. BORING OR TEST PIT NOT SAMPLED FOR As, Pb OR Cr.
- HIDE PILES BASED ON CONSENT DECREE.
- HIDE RESIDUE PRESENT WITHIN 36 INCHES OF GROUND SURFACE BASED ON REMEDIAL INVESTIGATION (RI) DATA OR PRE-DESIGN INVESTIGATION (PDI) DATA.

NOTES

1.) SEE FIGURE 3 FOR CONTINUATION OF LEGEND.

2.) BORING AND TEST PIT DATA FROM: DIGITAL FILE PROVIDED BY U.S. EPA REGION I, PHASE I RI REPORT AND PHASE II RI REPORT (STAUFFER, 1983 AND 1984), PRE-DESIGN INVESTIGATION TASKS S-1, S-2, S-4, SW-1 AND GW-1 INTERIM FINAL REPORTS (GOLDER ASSOCIATES, 1990), RI GROUNDWATER/ SURFACE-WATER INVESTIGATION PLAN (ROUX ASSOCIATES, 1991).

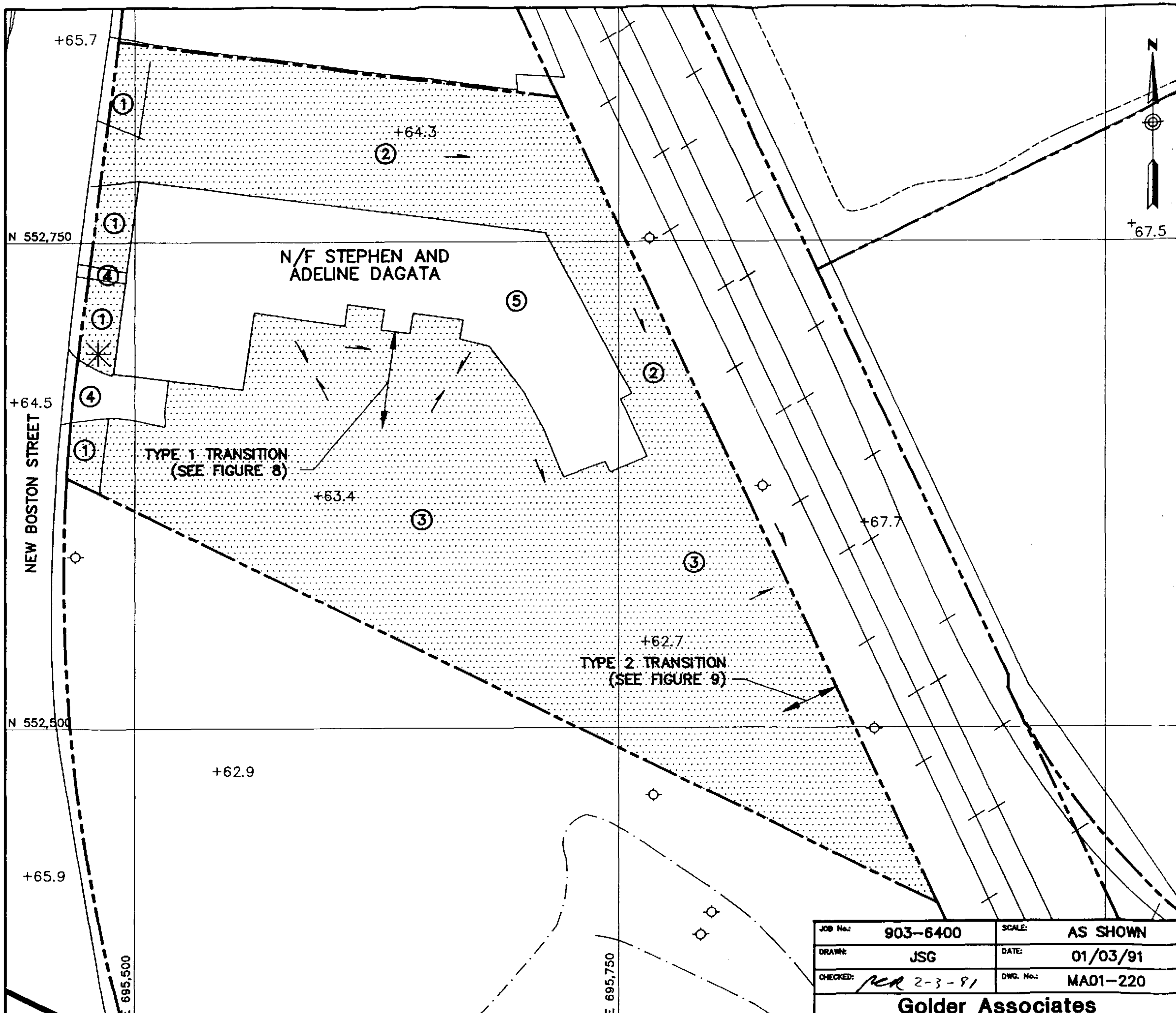
3.) PROPERTY SURVEY PERFORMED BY SAIC ENGINEERING INC., APRIL 1990.

APR 2 5 1992

4.) SOIL SAMPLE AND/OR SEDIMENT SAMPLE ANALYSES ARE INCLUDED IN THE ATTACHED TABLE 1.

50 0 50
scale feet

JOB No.: 903-6400	SCALE: AS SHOWN	EXTENT OF HAZARDOUS SUBSTANCES	
DRAWN: JSG	DATE: 01/03/91		
CHECKED: PCR 2-3-91	DWG. No.: MA01-219		
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST	FIGURE 2



LEGEND

- SITE BOUNDARY
- PROPERTY BOUNDARY
- N 553,250 GRID LINE
- PRELIMINARY EXTENT OF COVER FROM FIGURE 1
- EDGE OF WOODS
- TRAIL
- SOIL PILES
- STREAMS AND WATERCOURSES
- STREAMS AND LIMIT OF WETLANDS TO BE REMEDIATED
- RAILROAD TRACKS
- BUSH
- TREE
- SIGN
- UTILITY POLE
- CATCH BASIN
- MANHOLE
- FIRE HYDRANT
- +71.8 TYPICAL SPOT ELEVATION
- EXTENT OF PROPOSED COVER FOR LANDOWNER PROPERTY
- GENERAL DIRECTION OF WATER DRAINAGE AFTER REMEDIATION

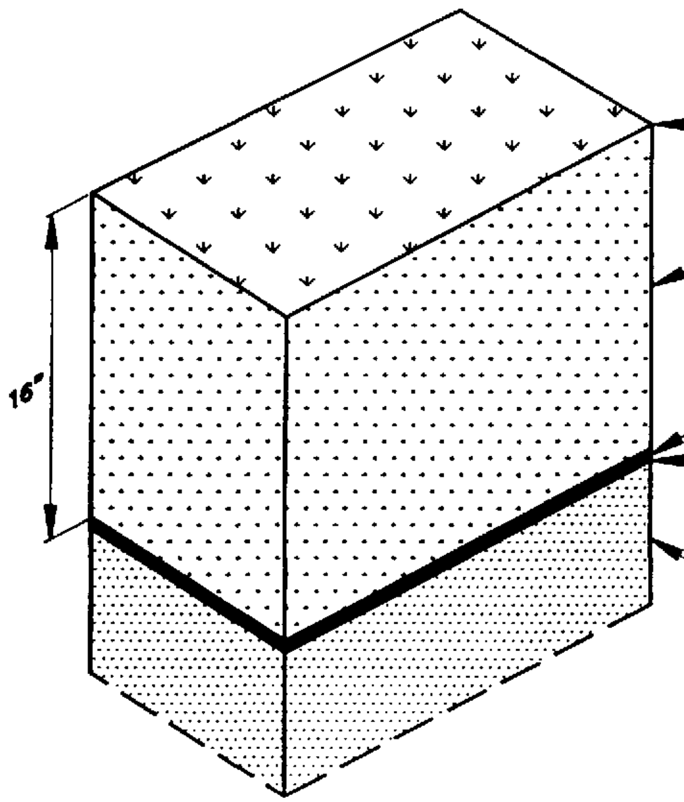
NOTES

1.) NUMBERED AREAS REFER TO PRESENT GROUND CONDITIONS AND COVER OPTIONS AS DESCRIBED IN THE ATTACHED TABLE 2.

2.) PROPERTY SURVEY PERFORMED BY SAIC ENGINEERING INC., APRIL 1990.

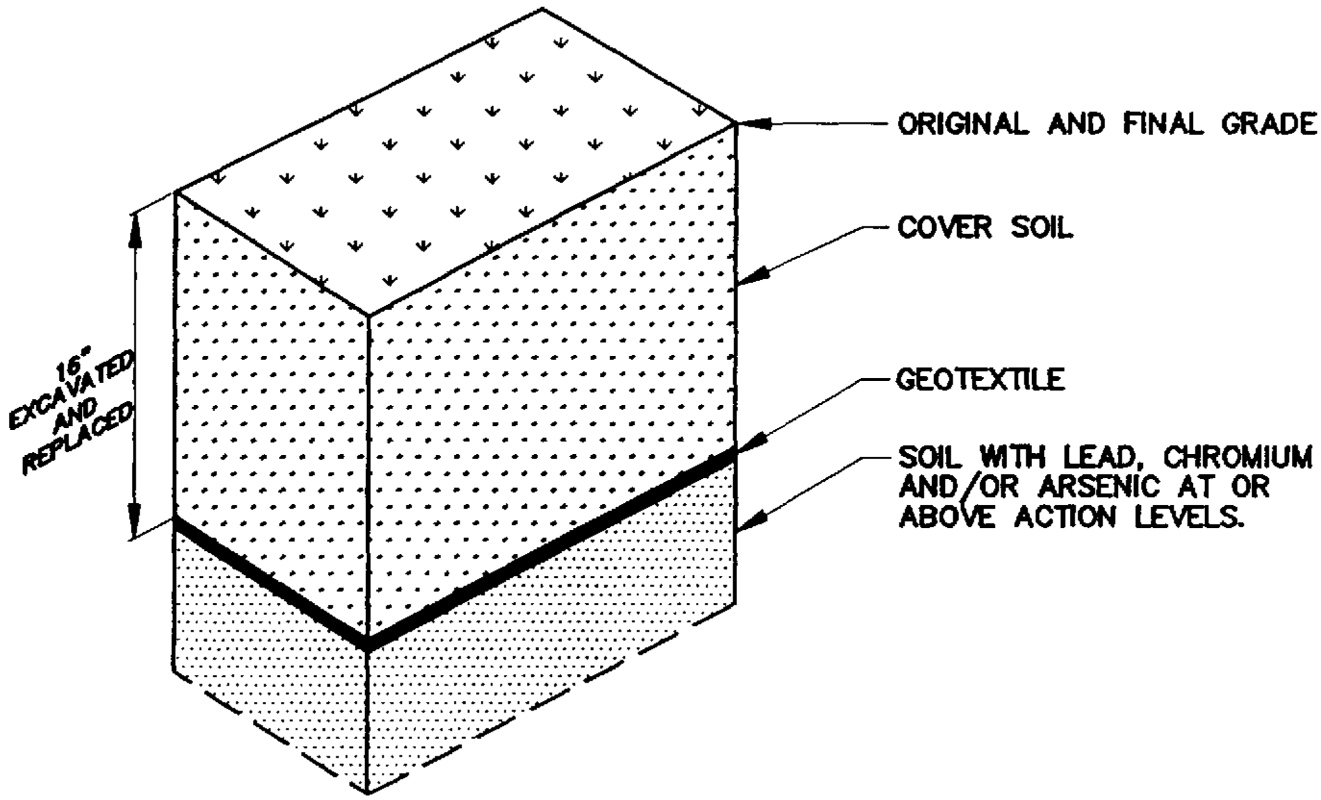
50 0 50
scale feet

JOB No.: 903-6400	SCALE: AS SHOWN	LANDOWNER COVER OPTIONS	
DRAWN: JSG	DATE: 01/03/91	APR 25 1992	
CHECKED: [signature] 2-3-91	DWG. No.: MA01-220		
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST	FIGURE 3



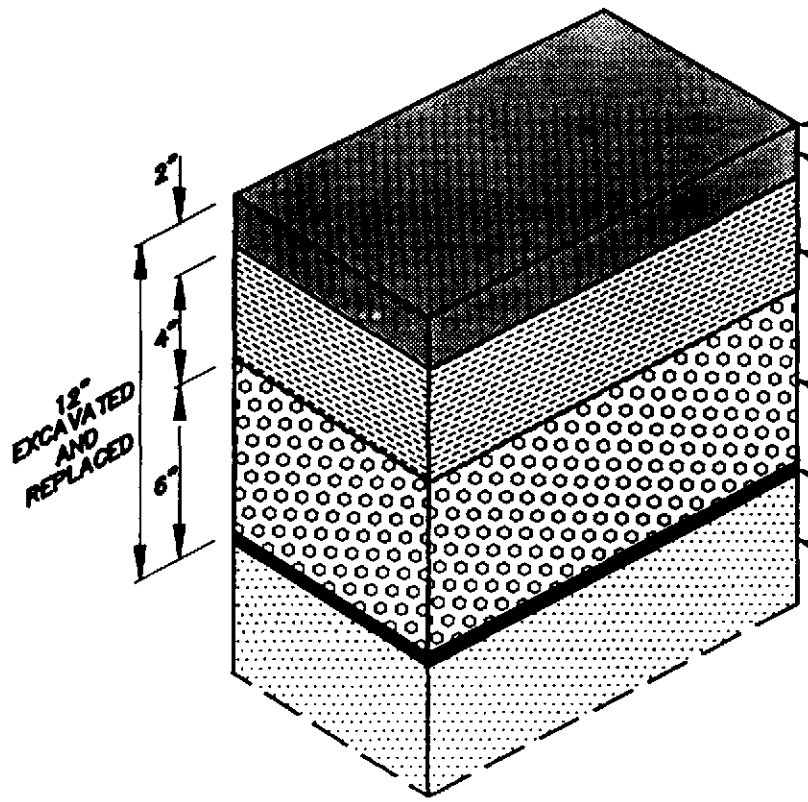
APR 25 1992

JOB No.: 903-6400	SCALE: N.T.S.	**ABOVE GRADE PERMEABLE COVER**		
DRAWN: JSG	DATE: 11/20/90			
CHECKED: MR	DWG. No.: MA01-218			
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST	FIGURE	4

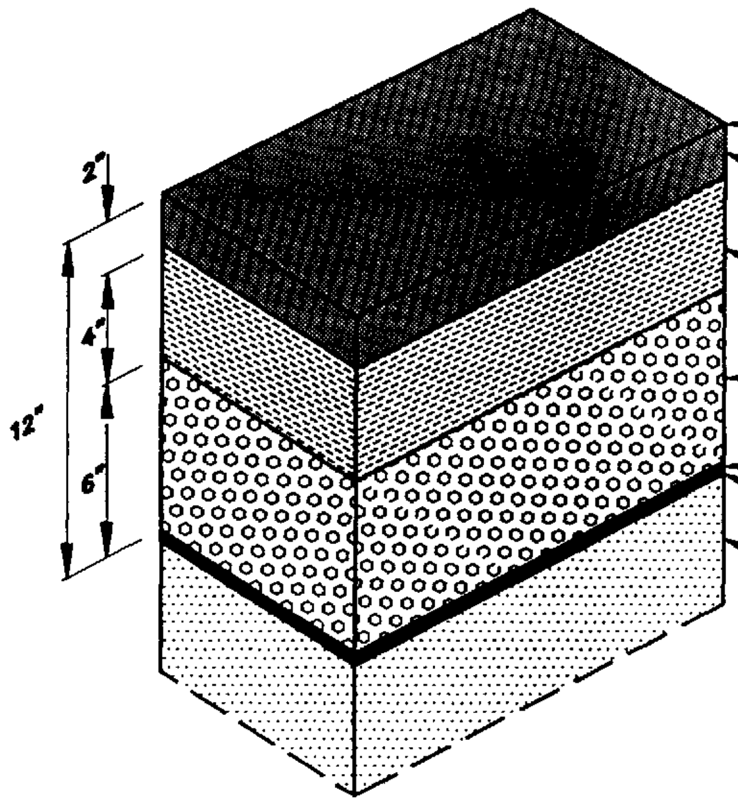


APR 25 1992

JOB No.: 903-6400	SCALE: N.T.S.	AT GRADE PERMEABLE COVER		
DRAWN: JSG	DATE: 12/31/90			
CHECKED: PCR	DWG. No.: MA01-222			
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST	FIGURE	5

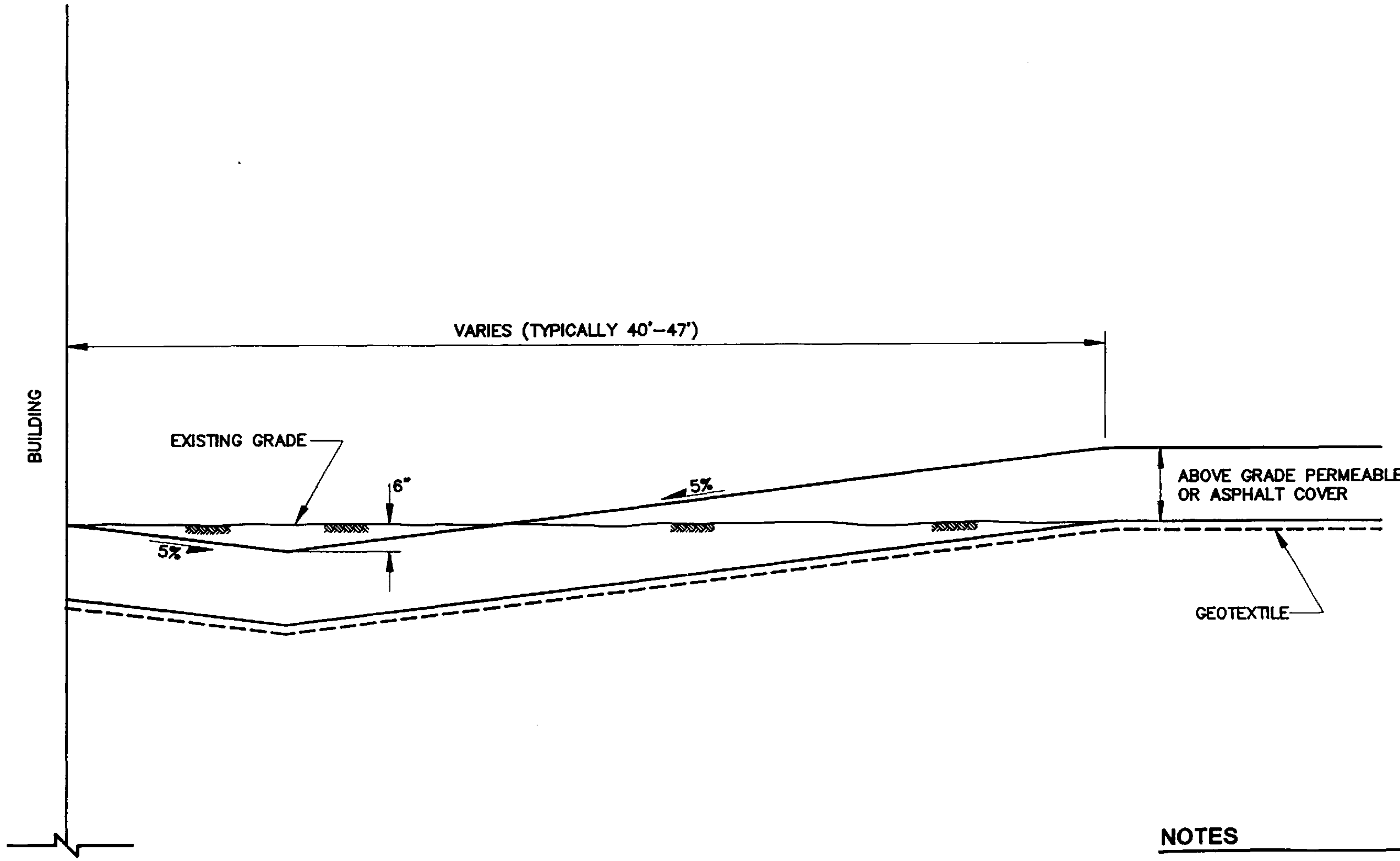


JOB No.: 903-6400	SCALE: N.T.S.	AT GRADE ASPHALT COVER		
DRAWN: JSG	DATE: 12/31/90			
CHECKED:	DWG. No.: MA01-221			
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST		FIGURE 6



APR 25 1992

JOB No.: 903-6400	SCALE: N.T.S.	ABOVE GRADE ASPHALT COVER		
DRAWN: JSG	DATE: 12/31/90			
CHECKED: [signature]	DWG. No.: MA01-281			
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST	FIGURE:	7



NOTES

1.) SEE FIGURE 3 FOR LOCATION OF TRANSITION ZONE.

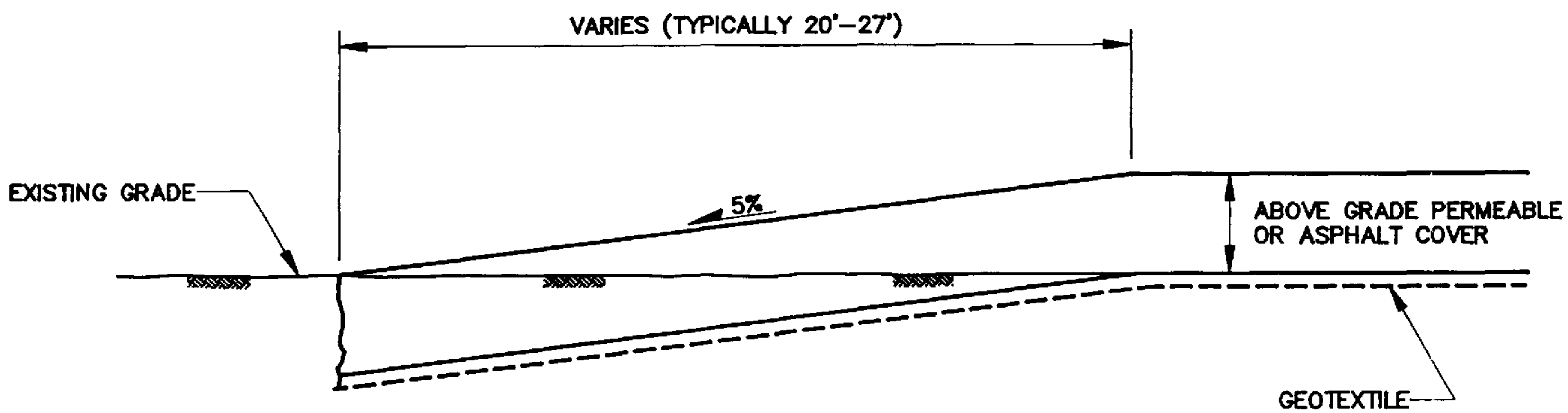
APR 25 1992

JOB No.:	903-6400	SCALE:	N.T.S.
DRAWN:	JSG	DATE:	12/31/90
CHECKED:	PCR	DWG. No.:	MA01-283

Golder Associates

TYPE 1 TRANSITION ZONE SECTION

INDUSTRI-PLEX SITE REMEDIAL TRUST | FIGURE 8



NOTES

1.) SEE FIGURE 3 FOR LOCATION OF TRANSITION ZONE.

APR 2 3 1992

JOB No.: 903–6400	SCALE: N.T.S.	TYPE 2 TRANSITION ZONE SECTION	
DRAWN: JSG	DATE: 12/31/90		
CHECKED: PER	DWG. No.: MA01–280		
Golder Associates		INDUSTRI–PLEX SITE REMEDIAL TRUST	FIGURE 9

APPENDIX 11-C
Existing Slope Stability Calculations

Shear Strength Parameters for Fill and Hide Residue

Golder Associates	SUBJECT Estimation of ϕ' and c' from p'-q plot		
	Job No. 903 6900	Made by AD	Date 12-18-91
	Ref. ISRT	Checked [illegible]	Sheet 1 of 3
		Reviewed [illegible]	

Objective : To estimate ϕ' and c' for the Fill and Hide Residue from the results of triaxial tests plotted on a p'-q plot, where:

ϕ' = effective angle of shearing resistance
c' = effective cohesive strength.

Reference : Lambe-Whitman, Soil Mechanics, 1969, "Use of a p-q Diagram," p. 141-2.

Method : Four types of triaxial test were performed on the Fill and Hide Residue :

1. Undisturbed Consolidated-Drained
2. Undisturbed Consolidated-Undrained with pore pressure measurement
3. Remolded C-D
4. Remolded C-U w/o p.p.m.

The failure points were plotted on a p'-q plot, where:

$$p' = \frac{\sigma_1' + \sigma_3'}{2} \qquad q = \frac{\sigma_1' - \sigma_3'}{2}$$

with:

p' = effective normal stress at failure
q = shear stress
σ_1' = major principal effective stress at failure
σ_3' = minor " " " "

→ From the plotted data, a "best fit" line was drawn through the data to determine the slope (ψ) and intercept (d) of the p'-q plot.

ϕ' and c' were determined from (ψ) and (d) using the equations:

$$\sin\phi' = \tan\psi \quad \text{and} \quad c' = \frac{d}{\cos\phi'}.$$

A lower bound line was drawn through the data to determine the lower bound ϕ' and c' values using the same relationships as above.

Calculations :

1) "Best Fit" line : $\psi = 31°$ and $d = 1.6$ psi

$$\sin\phi' = \tan\psi$$
$$\sin\phi' = \tan(31°)$$
$$\phi' = 36.9° \approx \underline{\underline{37°}}$$

$$c' = \frac{d}{\cos\phi}$$

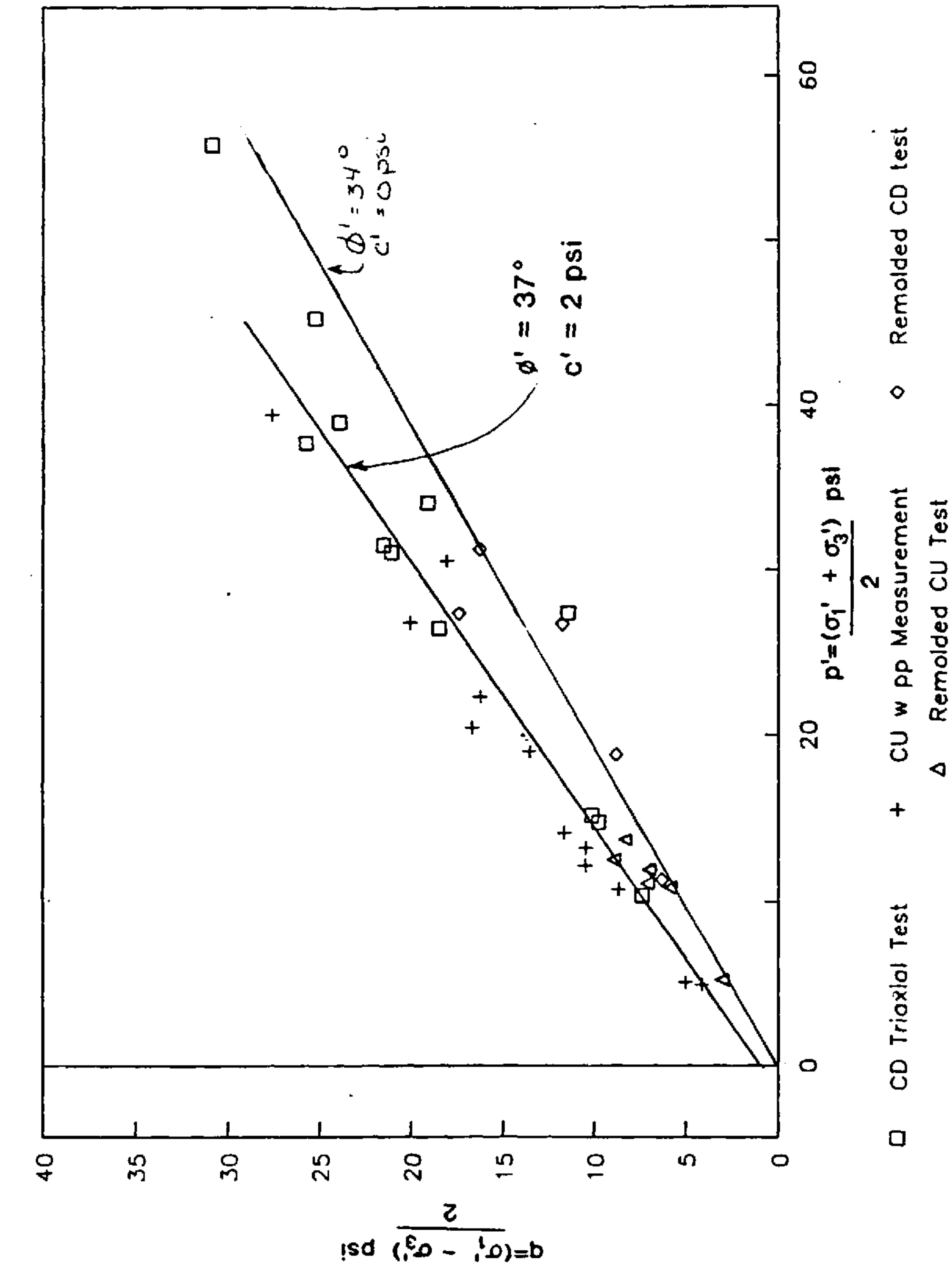
$$c = \frac{1.6}{\cos(37)} = \underline{\underline{2}}\text{ psi}$$

2) Lower Bound line : $\psi = 29°$ and $d = 0$

$$\sin\phi' = \tan\psi$$
$$\sin\phi' = \tan(29)$$
$$\phi' = 33.7° \approx \underline{\underline{34°}}$$

$$c' = \frac{d}{\cos\phi}$$

$$c' = \frac{0}{\cos(34)} = \underline{\underline{0}}\text{ psi}$$



JOB No.: 893-6255	SCALE: N/A	SHEAR STRENGTH p'-q PLOT FILL AND HIDE RESIDUE	
DRAWN: MRM	DATE: 09/14/90		
CHECKED: GAC	DWG. No.: MA01-156		
Golder Associates		ISRT/WOBURN/MA	FIGURE

Golder Associates	SUBJECT ESTIMATE Ø' FOR FILL & LIME RESIDUE (WHP)		
	Job No. 903-6700	Made by RAC	Date 11/1/91
	Ref.	Checked	Sheet 1 of 4
		Reviewed	

OBJECTIVE: TO ESTIMATE Ø' FOR THE FILL & LIME RESIDUE USING N-VALUES BASED ON SCHMERTMANN 1975

REFERENCE: 1) MEASUREMENT OF IN-SITU SHEAR STRENGTH JOHN H. SCHMERTMANN, 1975

ASSUMPTIONS:

1) METHOD FOR ESTIMATING Ø' FROM SPT-VALUES DOES NOT APPLY FOR DEPTHS LESS THAN 2m

2) USE SPT VALUES FROM WEST HIDE PILE

3) Thickness OF SURFICIAL MATERIAL = 6 ft
DEPTH TO GROUNDWATER = 24 ft

CALCULATIONS

1) CALCULATE OVERBURDEN PRESSURE σ_o' at 10', 15', 20'

σ_o' @ 10' = (100 pcf)(6 ft) + (125 pcf)(4) = 1100 psf = 0.55 kg/cm²

σ_o' @ 15' = (100 pcf)(6 ft) + (125 pcf)(9 ft) = 1725 psf = 0.86 kg/cm²

σ_o' @ 20' = (100 pcf)(6 ft) + (125 pcf)(14 ft) = 2350 psf = 1.175 kg/cm²

2) DETERMINE N FROM [illegible]

10' 8.75

15' = 8.75

20' = 8.75

Golder Associates

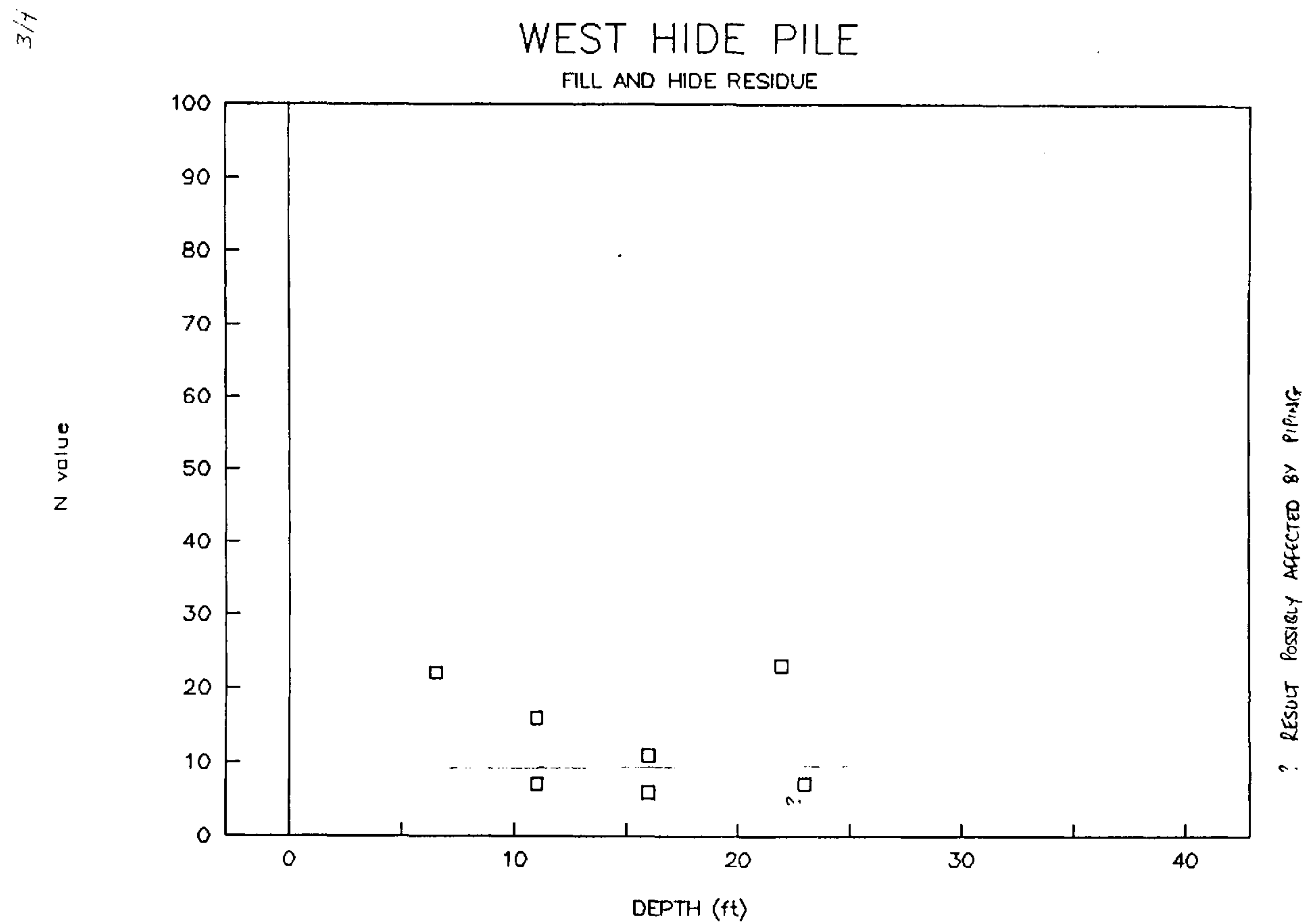
SUBJECT Estimat ϕ' for [illegible] WHP		
Job No. 903-6[illegible]0	Made by [illegible]	Date [illegible]
Ref.	Checked [illegible]	Sheet 2 of 4
	Reviewed	

Calculations (cont'd)

3) Determine ϕ' from attached chart

Depth	N	ϕ'
10	[illegible]	36.5°
15	8.75	35°
20	8.75	33°

For overall stability calculation
Take $\phi' = 35°$



3/7
WEST HIDE PILE
FILL AND HIDE RESIDUE
N value
DEPTH (ft)
0
10
20
30
40
50
60
70
80
90
100
? RESULT POSSIBLY AFFECTED BY PIPING

shear strength from the SPT must be treated as just that -- an estimate. However, the pressure of circumstances often force engineers to make such estimates and the writer offers the following as the state-of-the-art on this subject.

2.21 ϕ' in Sands: As part of his very thorough review of the state-of-the-art of the SPT, deMello (1971, pp. 26-31) studied the published N-ϕ correlation data -- which he found meager. He reviewed the data of Gibbs and Holtz (1957), assumed an equation based on Prandtl-Caquot-Buisman idealized theory, and found by statistical evaluation that the G & H data fit the equation with good confidence. Figure 1 herein presents this correlation, as adapted from deMello's Figure 11. He then checked this correlation prediction method against mostly unpublished data from private sources and found reasonable agreement provided one considered Figure 1 as usually conservative and does not apply it to SPTs at "very shallow depths" (writer's interpretation = less than 2 m.).

The above can produce only rough estimates of ϕ' in sands. An experienced engineer with a sample in hand can perhaps estimate ϕ' triaxial with better accuracy. However, this SPT method does provide the engineer with a rough check and some documentation. But, variables other than ϕ' can have a great influence on the N-value. For example Barthélamy (1974) found in the Duke University chamber that N reduced about 55% when he added 10% mica to a sand, but kept the same triaxial ϕ' of about 39 degrees. Adding the mica reduced the triaxial initial tangent modulus, E_i, by about 60%. Obviously, a soil's compressibility has great importance.

One can also estimate ϕ' using relative density as an intermediate parameter, but deMello recommended against this common practice. The correlation most used for the estimate of relative density is that based on the research of Gibbs and Holtz (1957). To reduce conservatism at high D_r many engineers now include the modifications proposed by Bazaraa (1967). Figure 2 presents these correlations in a form convenient for easily estimating relative density from the N-values. Because of general agreement with the philosophy expressed by Meyerhof (1965), most engineers no longer modify the N-value when testing below

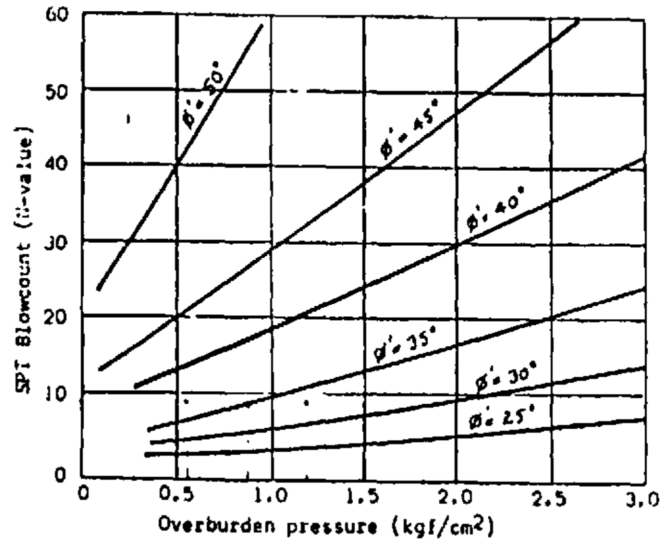


FIGURE 1 - METHOD FOR ESTIMATING ø' FROM SPT
(based on deMello's 1971 analysis USBR data)

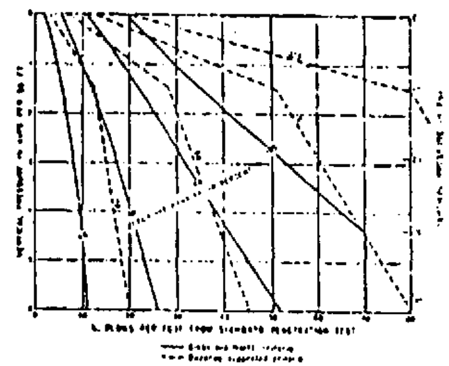


FIGURE 2 - COMPARISON OF THE GIBBS & HOLTZ (1957) D_r CRITERIA AND THE BAZARAA (1967) SUGGESTED CRITERIA
(as presented by Holtz and Gibbs, 1969, Fig. 26)

Section L-L' - Existing Conditions
Surface Sloughing - Long-Term and Perched Water Tables

Golder Associates	SUBJECT ISRT - West Hide Pile Stability		
	Job No. 903-6400 Ref. INDUSTRI-PLEX /WOBURN/MA	Made by DOKL Checked RD Reviewed [illegible]	Date 5/14/92 Sheet 1 of 4

Objective : Evaluate the factor of safety of the existing slopes of the West Hide Pile against shallow sloughing

Method : 1) Determine the steepest inclination of existing slope

2) Select the most representative soil parameters

3) Perform infinite slope analyses outlined in Ref. 3 for the steepest existing slope.

References :

1) Golder, 1992, "Final Grading Plan," Drawing # 11-7. Proj # 903-6400

2) Golder, 1992, "Slope Stability Analyses," by DOKL, Proj # 903-6400

3) Duncan & Buchignani, 1975, "An Engineering Manual for Slope Stability Studies" University of Berkeley.

4) Golder, 1990 "Pre-Design Investigation Task S-2 Stability of Hide Piles, Interim Final Report," Proj # 893-6253

Golder Associates	SUBJECT ISRT - West Hide Pile Stability		
	Job No. 903-6400 Ref. INDUSTRI-PLEX / WOBURN / MA	Made by DOKL Checked RD Reviewed [signature]	Date 5/14/92 Sheet 2 of 4

Slope Angle (β) : The inclination of the existing slope was obtained from Refs 1 and 2.

For section L-L $\beta = 40$
section I-I $\beta = 31$
section J-J $\beta = 31$

$\therefore$ the steepest slope is 40°

Material Properties: The properties of the surficial material are obtained from Ref. 4 and are summarized as follows:

$\gamma = 100$ pcf
$c' = 0$
$\phi' = 25°$

Calculations:
Section L-L
1) Dry slope

$$FS = \frac{\tan\phi'}{\tan\beta} = \frac{\tan 25}{\tan 40} = 0.56$$

2) Perched Water Table leading to Seepage Emerging from slope:

Assume seepage angle, θ, is 5° from the horizontal

According to Ref. 3

$$r_u = \frac{\gamma_w}{\gamma} \frac{1}{(1 + \tan\beta \tan\theta)} = \frac{62.4}{100}\left(\frac{1}{1+\tan 40 \tan 5}\right)$$

$$= 0.58$$

Golder Associates	SUBJECT ISRT - Wet Hide Pile Stability		
	Job No. 903-6400 Ref. INDUSTRI-PLEX / WOBURN / MA	Made by DOKL Checked RD Reviewed PCR	Date 5/14/92 Sheet 3 of 4

$\cot\beta = \cot 40 = 1.19$ ✓

From the attached figure, $A = 0.08$ for $r_u = 0.58$ ✓

$\therefore$ F.S. $= 0.56 \times 0.08 = 0.04$ ✓

3) Perched Water Table, Seepage Parallel to Slope

Assume seepage surface is 80% of the depth of sloughing.

$$r_u = \frac{X}{T}\,\frac{\gamma_w}{\gamma}\cos^2\beta$$

$$= 0.8 \times \frac{62.4}{100}\cos^2(40) = 0.29$$ ✓

From the attached figure, $A = 0.52$ for $r_u = 0.29$ and $\cot\beta = 1.19$

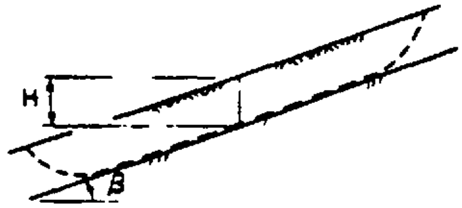
$\therefore$ F.S. $= 0.56 \times 0.52 = 0.29$ ✓

Section J-J

Since the steepest slope is at section L-L, and is the worst scenario, therefore, Section J-J is analyzed only for the dry slope condition

$$F.S = \frac{\tan\phi}{\tan\beta} = \frac{\tan 25}{\tan 31} = 0.8$$

4/4



γ = total unit weight of soil

γ_w = unit weight of water

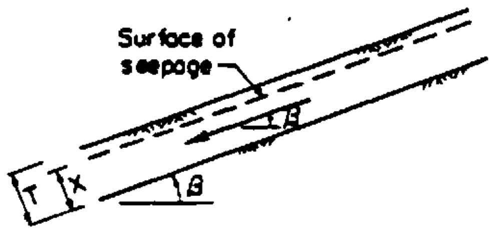
C' = cohesion intercept } Effective Stress
ϕ' = friction angle }

r_u = pore pressure ratio = $\frac{u}{\gamma H}$

u = pore pressure at depth H

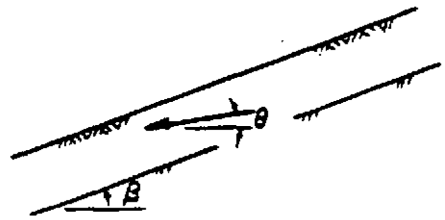
Steps:

1. Determine r_u from measured pore pressures or formulas at right
2. Determine A and B from charts below
3. Calculate $F = A \frac{\tan\phi'}{\tan\beta} + B \frac{C'}{\gamma H}$



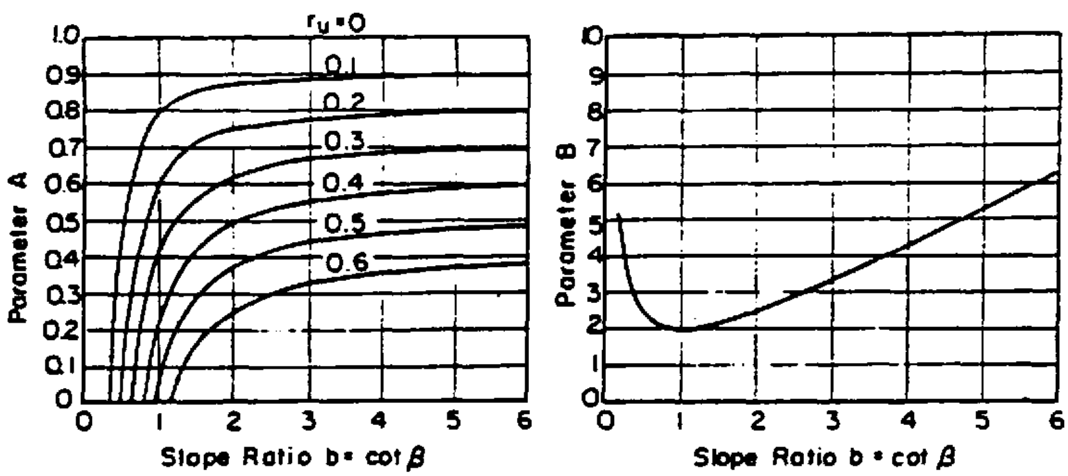
Seepage parallel to slope

$$r_u = \frac{X}{T} \frac{\gamma_w}{\gamma} \cos^2\beta$$



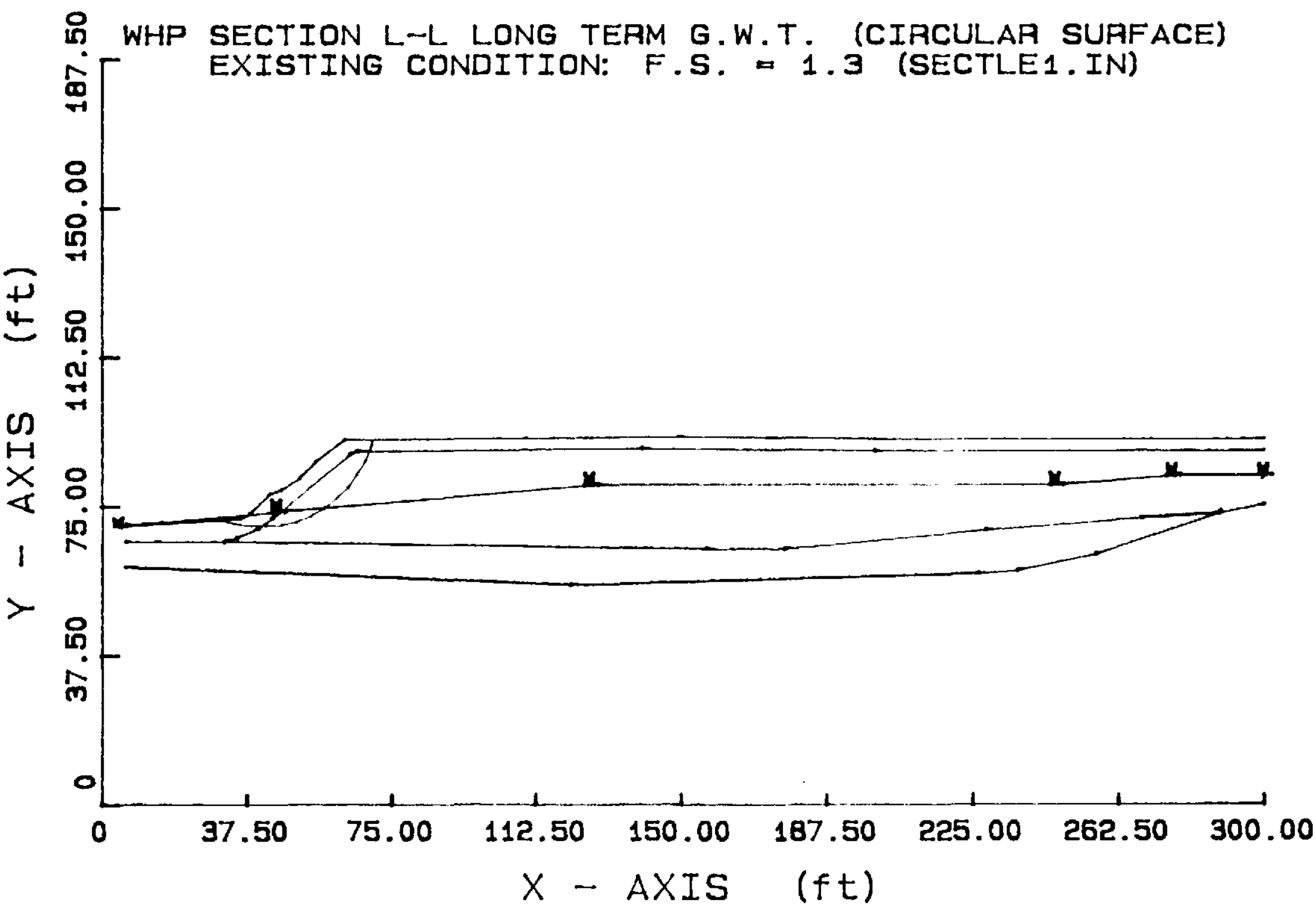
Seepage emerging from slope

$$r_u = \frac{\gamma_w}{\gamma} \frac{1}{1 + \tan\beta \tan\theta}$$



STABILITY CHARTS FOR INFINITE SLOPES.

Section L-L' - Existing Conditions
Circular - Long-Term and Perched Tables


WHP SECTION L-L LONG TERM G.W.T. (CIRCULAR SURFACE)
EXISTING CONDITION: F.S. = 1.3 (SECTLE1.IN)
Y - AXIS (ft)
X - AXIS (ft)
0
37.50
75.00
112.50
150.00
187.50
225.00
262.50
300.00

1/8

2/8

** PCSTABL5M **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

Run Date:	5/11/92
Time of Run:	
Run By:	DOKL
Input Data Filename:	SECTLE1.IN
Output Filename:	SECTLE1.OUT
Plotted Output Filename:	SECTLE1.PLT

PROBLEM DESCRIPTION ISRT: SECTION L-L, EXISTING SLOPE FILE S
ECTLE1.IN

BOUNDARY COORDINATES

11 Top Boundaries
31 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	6.00	70.50	10.00	70.50	1
2	10.00	70.50	36.00	72.00	1
3	36.00	72.00	38.00	73.00	1
4	38.00	73.00	43.00	78.00	1
5	43.00	78.00	46.00	79.00	1
6	46.00	79.00	51.00	82.00	1
7	51.00	82.00	55.00	86.20	1
8	55.00	86.20	62.50	91.70	1
9	62.50	91.70	150.00	92.80	1
10	150.00	92.80	200.00	92.00	1
11	200.00	92.00	300.00	92.00	1
12	6.00	66.00	31.50	66.00	3
13	31.50	66.00	35.00	67.00	2
14	35.00	67.00	40.00	69.30	2
15	40.00	69.30	44.00	72.00	2
16	44.00	72.00	64.00	88.30	2
17	64.00	88.30	66.00	89.00	2
18	66.00	89.00	140.00	89.50	2
19	140.00	89.50	200.00	89.00	2
20	200.00	89.00	300.00	89.00	2
21	31.50	66.00	156.50	64.00	3
22	156.50	64.00	176.50	64.00	3
23	176.50	64.00	228.50	69.00	3
24	228.50	69.00	268.50	72.00	3
25	268.50	72.00	288.50	73.00	3
26	288.50	73.00	300.00	75.40	4
27	6.00	59.50	121.50	55.00	4
28	121.50	55.00	226.50	58.00	4
29	226.50	58.00	236.50	59.00	4
30	236.50	59.00	257.00	63.00	4

31	257.00	63.00	288.50	73.00	4

3/8

1

ISOTROPIC SOIL PARAMETERS

4 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	90.0	100.0	.0	25.0	.00	.0	1
2	100.0	125.0	.0	34.0	.00	.0	1
3	120.0	120.0	.0	36.0	.00	.0	1
4	125.0	125.0	.0	37.0	.00	.0	1

1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 6 Coordinate Points

Point No.	X-Water (ft)	Y-Water (ft)
1	4.00	69.50
2	45.00	73.00
3	126.50	80.00
4	246.50	80.00
5	276.50	82.00
6	300.00	82.20

1

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.

900 Trial Surfaces Have Been Generated.

30 Surfaces Initiate From Each Of 30 Points Equally Spaced
Along The Ground Surface Between X = 30.00 ft.
and X = 50.00 ft.

Each Surface Terminates Between X = 70.00 ft.
and X = 80.00 ft.

Unless Further Limitations Were Imposed, The Minimum Elevation At Which A Surface Extends Is Y = .00 ft.

4.00 ft. Line Segments Define Each Trial Failure Surface.

4/8

Restrictions Have Been Imposed Upon The Angle Of Initiation.
The Angle Has Been Restricted Between The Angles Of -45.0
And -15.0 deg.

1

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Examined. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Bishop Method * *

Failure Surface Specified By 14 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	30.69	71.69
2	34.54	70.59
3	38.49	70.01
4	42.49	69.94
5	46.47	70.40
6	50.34	71.38
7	54.06	72.86
8	57.56	74.80
9	60.76	77.19
10	63.63	79.98
11	66.11	83.12
12	68.15	86.56
13	69.73	90.24
14	70.17	91.80

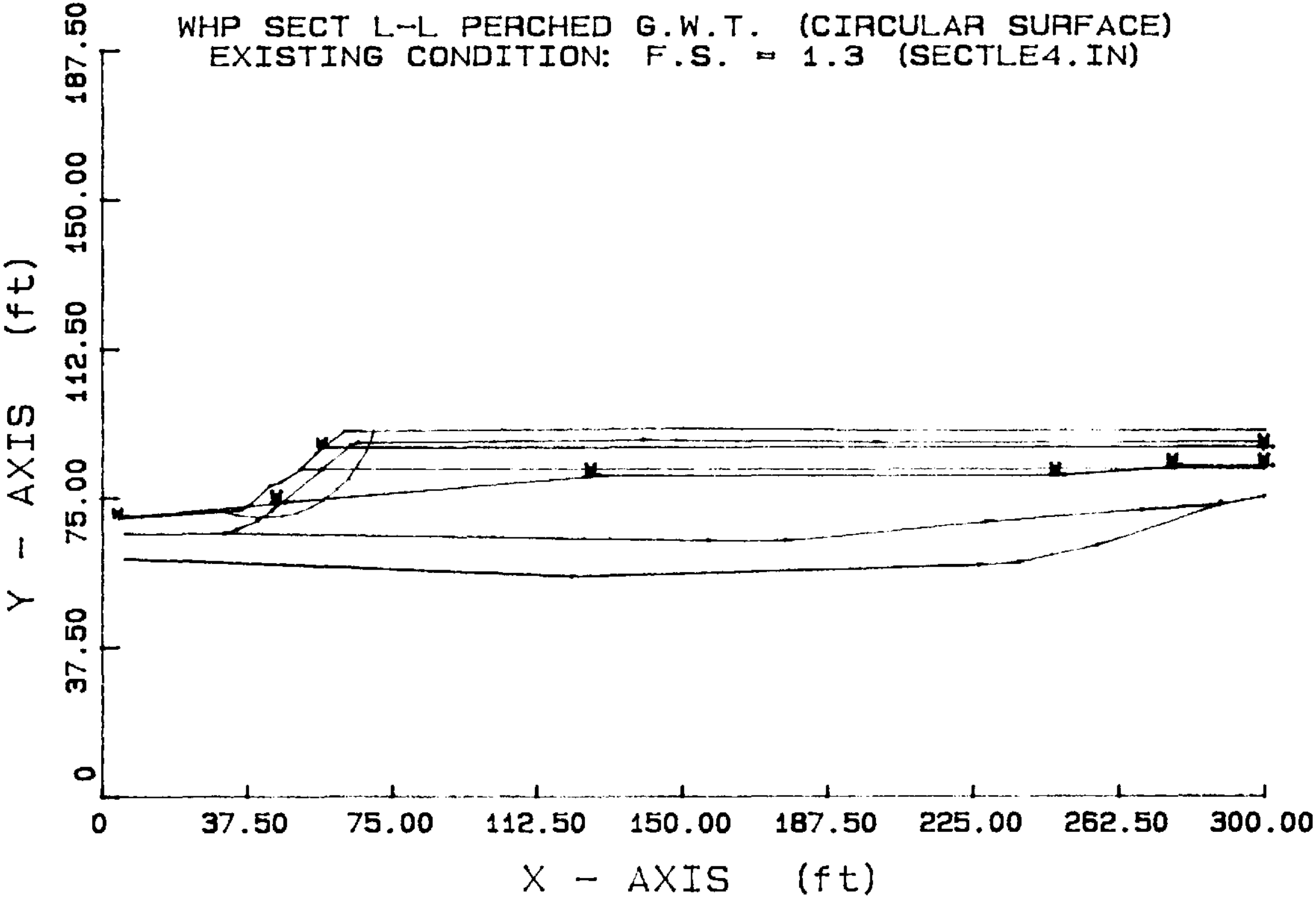
Circle Center At X = 41.0 ; Y = 100.3 and Radius, 30.4

*** 1.305 ***

Individual data on the 28 slices

Slice No.	Width Ft(m)	Weight Lbs(kg)	Water Force Top Lbs(kg)	Water Force Bot Lbs(kg)	Tie Force Norm Lbs(kg)	Tie Force Tan Lbs(kg)	Earthquake Force Hor Lbs(kg)	Earthquake Force Ver Lbs(kg)	Surcharge Load Lbs(kg)
1	3.8	254.1	32.5	198.6	.0	.0	.0	.0	.0
2	1.5	215.5	18.8	154.9	.0	.0	.0	.0	.0
3	2.0	450.5	.0	262.5	.0	.0	.0	.0	.0
4	.5	153.7	.0	73.7	.0	.0	.0	.0	.0
5	2.5	1131.5	.0	397.9	.0	.0	.0	.0	.0
6	1.5	979.2	.0	258.7	.0	.0	.0	.0	.0
7	.5	384.6	.0	90.1	.0	.0	.0	.0	.0
8	1.0	797.7	.0	176.0	.0	.0	.0	.0	.0
9	1.0	832.7	.0	174.1	.0	.0	.0	.0	
10	.3	217.1	.0	43.9	.0	.0	.0	.0	
11	.7	649.1	.0	128.3	.0	.0	.0	.0	.0
12	.5	415.0	.0	79.6	.0	.0	.0	.0	.0
13	3.9	3748.8	.0	597.1	.0	.0	.0	.0	.0
14	.7	678.2	.0	86.7	.0	.0	.0	.0	.0

WHP SECT L-L PERCHED G.W.T. (CIRCULAR SURFACE)
EXISTING CONDITION: F.S. = 1.3 (SECTLE4.IN)



8/5

0/8

** PCSTABL5M **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

Run Date: 5/11/92
Time of Run:
Run By: DOKL
Input Data Filename: SECTLE4.IN
Output Filename: SECTLE4.OUT
Plotted Output Filename: SECTLE4.PLT

PROBLEM DESCRIPTION ISRT: SECTION L-L, EXISTING SLOPE FILE S
ECTLE4.IN

BOUNDARY COORDINATES

11 Top Boundaries
34 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	6.00	70.50	10.00	70.50	1
2	10.00	70.50	36.00	72.00	1
3	36.00	72.00	38.00	73.00	1
4	38.00	73.00	43.00	78.00	1
5	43.00	78.00	46.00	79.00	1
6	46.00	79.00	51.00	82.00	1
7	51.00	82.00	55.00	86.20	2
8	55.00	86.20	62.50	91.70	2
9	62.50	91.70	150.00	92.80	2
10	150.00	92.80	200.00	92.00	2
11	200.00	92.00	300.00	92.00	2
12	51.00	82.00	56.50	82.00	1
13	6.00	66.00	31.50	66.00	5
14	31.50	66.00	35.00	67.00	4
15	35.00	67.00	40.00	69.30	4
16	40.00	69.30	44.00	72.00	4
17	44.00	72.00	56.50	82.00	4
18	56.50	82.00	64.00	88.30	3
19	64.00	88.30	66.00	89.00	3
20	66.00	89.00	140.00	89.50	3
21	140.00	89.50	200.00	89.00	3
22	200.00	89.00	300.00	89.00	3
23	56.50	82.00	300.00	82.00	4
24	31.50	66.00	156.50	64.00	5
25	156.50	64.00	176.50	64.00	5
26	176.50	64.00	228.50	69.00	5
27	228.50	69.00	268.50	72.00	5
28	268.50	72.00	288.50	73.00	5
29	288.50	73.00	300.00	75.40	6
30	6.00	59.50	121.50	55.00	6

7/8

31	121.50	55.00	226.50	58.00	6
32	226.50	58.00	236.50	59.00	6
33	236.50	59.00	257.00	63.00	6
34	257.00	63.00	288.50	73.00	6

1

ISOTROPIC SOIL PARAMETERS

6 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	90.0	100.0	.0	25.0	.00	.0	1
2	90.0	100.0	.0	25.0	.00	.0	2
3	100.0	125.0	.0	34.0	.00	.0	2
4	100.0	125.0	.0	34.0	.00	.0	1
5	120.0	120.0	.0	36.0	.00	.0	1
6	125.0	125.0	.0	37.0	.00	.0	1

1

2 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 6 Coordinate Points

Point No.	X-Water (ft)	Y-Water (ft)
1	4.00	69.50
2	45.00	73.00
3	126.50	80.00
4	246.50	80.00
5	276.50	82.00
6	300.00	82.20

Piezometric Surface No. 2 Specified by 2 Coordinate Points

Point No.	X-Water (ft)	Y-Water (ft)
1	56.50	87.00
2	300.00	87.00

1

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.

900 Trial Surfaces Have Been Generated.

8/8

30 Surfaces Initiate From Each Of 30 Points Equally Spaced
Along The Ground Surface Between X = 30.00 ft.
and X = 50.00 ft.

Each Surface Terminates Between X = 70.00 ft.
and X = 80.00 ft.

Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = .00 ft.

4.00 ft. Line Segments Define Each Trial Failure Surface.

Restrictions Have Been Imposed Upon The Angle Of Initiation.
The Angle Has Been Restricted Between The Angles Of -45.0
And -15.0 deg.

1

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Examined. They Are Ordered - Most Critical
First.

* * Safety Factors Are Calculated By The Modified Bishop Method * *

Failure Surface Specified By 14 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	30.69	71.69
2	34.54	70.59
3	38.49	70.01
4	42.49	69.94
5	46.47	70.40
6	50.34	71.38
7	54.06	72.86
8	57.56	74.80
9	60.76	77.19
10	63.63	79.98
11	66.11	83.12
12	68.15	86.56
13	69.73	90.24
14	70.17	91.80

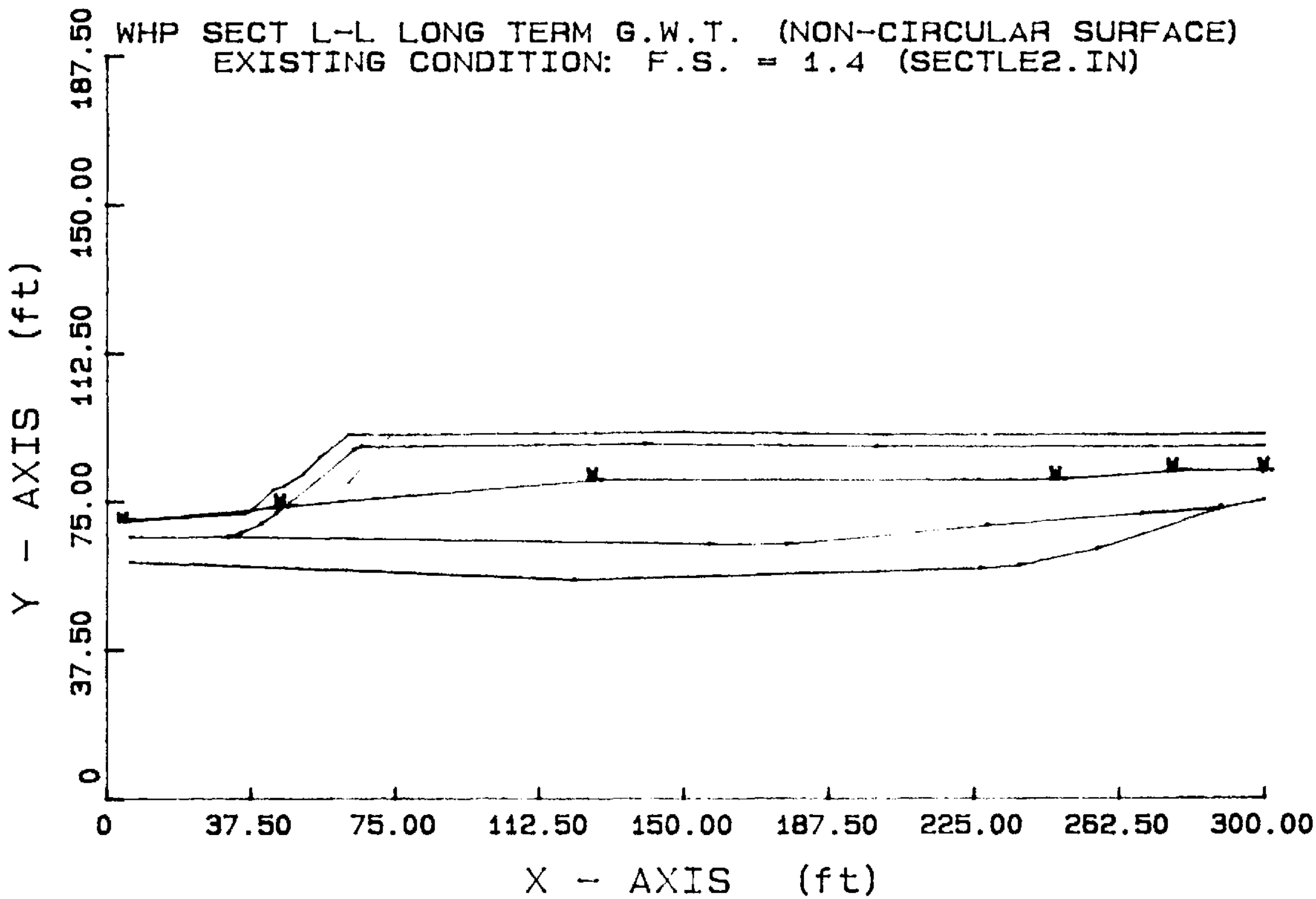
Circle Center At X = 41.0 ; Y = 100.3 and Radius, 30.4

*** 1.303 ***

Individual data on the 32 slices

Water Force	Water Force	Tie Force	Tie Force	Earthquake Force	Surcharge

Section L-L' - Existing Conditions
Non-Circular - Long-Term Water Table


WHP SECT L-L LONG TERM G.W.T. (NON-CIRCULAR SURFACE)
EXISTING CONDITION: F.S. = 1.4 (SECTLE2.IN)
Y - AXIS (ft)
0
37.50
75.00
112.50
150.00
187.50
X - AXIS (ft)
0
37.50
75.00
112.50
150.00
187.50
225.00
262.50
300.00

2/10

** PCSTABL5M **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

Run Date:	5/11/92
Time of Run:	
Run By:	DOKL
Input Data Filename:	SECTLE2.IN
Output Filename:	SECTLE2.OUT
Plotted Output Filename:	SECTLE2.PLT

PROBLEM DESCRIPTION ISRT: SECTION L-L, EXISTING SLOPE FILE S
ECTLE2.IN WEDGE FAILURE

BOUNDARY COORDINATES

11 Top Boundaries
31 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	6.00	70.50	10.00	70.50	1
2	10.00	70.50	36.00	72.00	1
3	36.00	72.00	38.00	73.00	1
4	38.00	73.00	43.00	78.00	1
5	43.00	78.00	46.00	79.00	1
6	46.00	79.00	51.00	82.00	1
7	51.00	82.00	55.00	86.20	1
8	55.00	86.20	62.50	91.70	1
9	62.50	91.70	150.00	92.80	1
10	150.00	92.80	200.00	92.00	1
11	200.00	92.00	300.00	92.00	1
12	6.00	66.00	31.50	66.00	3
13	31.50	66.00	35.00	67.00	2
14	35.00	67.00	40.00	69.30	2
15	40.00	69.30	44.00	72.00	2
16	44.00	72.00	64.00	88.30	2
17	64.00	88.30	66.00	89.00	2
18	66.00	89.00	140.00	89.50	2
19	140.00	89.50	200.00	89.00	2
20	200.00	89.00	300.00	89.00	2
21	31.50	66.00	156.50	64.00	3
22	156.50	64.00	176.50	64.00	3
23	176.50	64.00	228.50	69.00	3
24	228.50	69.00	268.50	72.00	3
25	268.50	72.00	288.50	73.00	3
26	288.50	73.00	300.00	75.40	4
27	6.00	59.50	121.50	55.00	4
28	121.50	55.00	226.50	58.00	4
29	226.50	58.00	236.50	59.00	4
30	236.50	59.00	257.00	63.00	4

31	257.00	63.00	288.50	73.00	4

3/10

1

ISOTROPIC SOIL PARAMETERS

4 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	90.0	100.0	.0	25.0	.00	.0	1
2	100.0	125.0	.0	34.0	.00	.0	1
3	120.0	120.0	.0	36.0	.00	.0	1
4	125.0	125.0	.0	37.0	.00	.0	1

1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 6 Coordinate Points

Point No.	X-Water (ft)	Y-Water (ft)
1	4.00	69.50
2	45.00	73.00
3	126.50	80.00
4	246.50	80.00
5	276.50	82.00
6	300.00	82.20

1

Trial Failure Surface Specified By 3 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	36.00	72.00
2	63.00	80.00
3	74.00	91.84

Spencer's Theta (deg)	FOS (Moment) (Equil.)	FOS (Force) (Equil.)
16.00	1.548	1.379
24.00	1.518	1.408
43.79	.000	1.494
25.81	1.507	1.415
21.20	1.531	1.398
40.95	1.081	1.480
27.81	1.492	1.423
26.02	1.505	1.416

4/10

36.46	1.350	1.459
32.32	1.441	1.441

Factor Of Safety For The Preceding Specified Surface = 1.441
Spencer's Theta = 32.32

Factor Of Safety Is Calculated By Spencer's Method of Slices

*** Line of Thrust ***

Slice No.	X Coord.	Y Coord.	L/H	Side Force (lbs)
1	37.10	72.51	.826	-3.
2	38.00	73.23	1.557	-2.
3	43.00	74.82	.190	27.
4	46.00	76.04	.268	60.
5	48.57	77.16	.297	90.
6	51.00	77.38	.168	318.
7	55.00	78.91	.150	838.
8	62.50	82.05	.185	2277.
9	63.00	82.26	.193	2389.
10	64.00	83.32	.210	1977.
11	66.00	85.30	.243	1253.
12	71.39	90.15	.400	166.
13	74.00	93.71	.000	1.

1

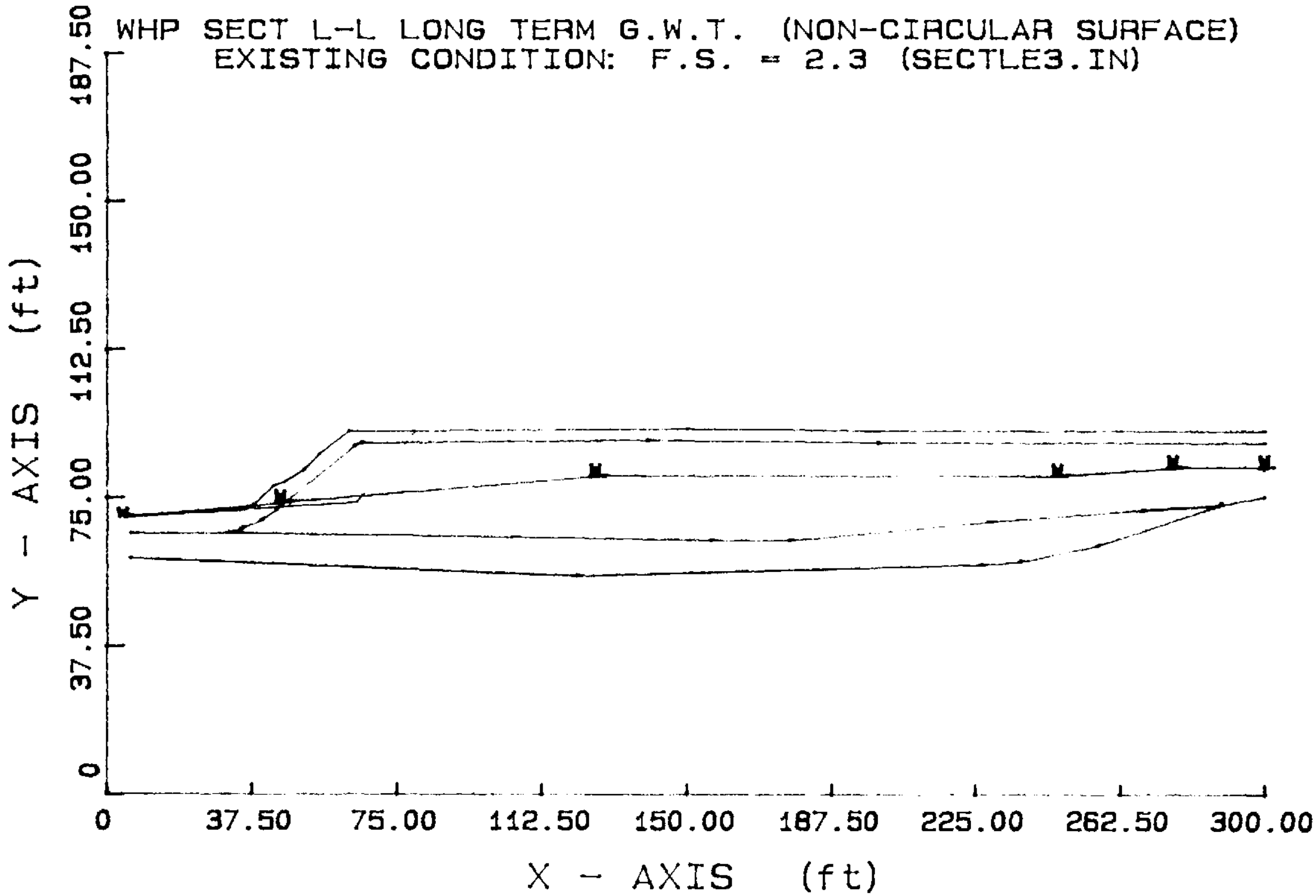
```
           Y            A     X     I     S            F     T

            .00    37.50     75.00    112.50    150.00    187.50

X        .00 +---------+---------+---------+---------+---------+
            -               * **
            -                  *
            -
            -
            -                 *
      37.50 +                 **
            -                  * *
            -                     *
            -                      *
            -                    S  *
            -                       *
A     75.00 +                       S
            -
            -
            -
            -
            -
X    112.50 +
            -              *
            -                    W
            -
            -                       *
            -
I    150.00 +                        *
            -                *
            -
            -
```

5/.0

```
            -                *
            -
S    187.50 +
            -
            -                       **
            -
            -
            -
     225.00 +              *
            -                 *
            -               *
            -                    W
            -
            -                *
F    262.50 +
            -                  *
            -                     W
            -
            -                  *
            -
T    300.00 +                   * W **
```



WHP SECT L-L LONG TERM G.W.T. (NON-CIRCULAR SURFACE)
EXISTING CONDITION: F.S. = 2.3 (SECTLE3.IN)
Y - AXIS (ft)
0
37.50
75.00
112.50
150.00
187.50
X - AXIS (ft)
0
37.50
75.00
112.50
150.00
187.50
225.00
262.50
300.00

6/10

7/10

** PCSTABL5M **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

Run Date:	5/11/92
Time of Run:	
Run By:	DOKL
Input Data Filename:	SECTLE3.IN
Output Filename:	SECTLE3.OUT
Plotted Output Filename:	SECTLE3.PLT

PROBLEM DESCRIPTION ISRT: SECTION L-L, EXISTING SLOPE FILE S
ECTLE3.IN WEDGE FAILURE

BOUNDARY COORDINATES

11 Top Boundaries
31 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	6.00	70.50	10.00	70.50	1
2	10.00	70.50	36.00	72.00	1
3	36.00	72.00	38.00	73.00	1
4	38.00	73.00	43.00	78.00	1
5	43.00	78.00	46.00	79.00	1
6	46.00	79.00	51.00	82.00	1
7	51.00	82.00	55.00	86.20	1
8	55.00	86.20	62.50	91.70	1
9	62.50	91.70	150.00	92.80	1
10	150.00	92.80	200.00	92.00	1
11	200.00	92.00	300.00	92.00	1
12	6.00	66.00	31.50	66.00	3
13	31.50	66.00	35.00	67.00	2
14	35.00	67.00	40.00	69.30	2
15	40.00	69.30	44.00	72.00	2
16	44.00	72.00	64.00	88.30	2
17	64.00	88.30	66.00	89.00	2
18	66.00	89.00	140.00	89.50	2
19	140.00	89.50	200.00	89.00	2
20	200.00	89.00	300.00	89.00	2
21	31.50	66.00	156.50	64.00	3
22	156.50	64.00	176.50	64.00	3
23	176.50	64.00	228.50	69.00	3
24	228.50	69.00	268.50	72.00	3
25	268.50	72.00	288.50	73.00	3
26	288.50	73.00	300.00	75.40	4
27	6.00	59.50	121.50	55.00	4
28	121.50	55.00	226.50	58.00	4
29	226.50	58.00	236.50	59.00	4
30	236.50	59.00	257.00	63.00	4

31	257.00	63.00	288.50	73.00	4	8/10

1

ISOTROPIC SOIL PARAMETERS

4 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	90.0	100.0	.0	25.0	.00	.0	1
2	100.0	125.0	.0	34.0	.00	.0	1
3	120.0	120.0	.0	36.0	.00	.0	1
4	125.0	125.0	.0	37.0	.00	.0	1

1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 6 Coordinate Points

Point No.	X-Water (ft)	Y-Water (ft)
1	4.00	69.50
2	45.00	73.00
3	126.50	80.00
4	246.50	80.00
5	276.50	82.00
6	300.00	82.20

1

Trial Failure Surface Specified By 3 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	36.00	72.00
2	65.00	74.00
3	80.00	91.92

Spencer's Theta (deg)	FOS (Moment) (Equil.)	FOS (Force) (Equil.)
16.00	2.512	2.037
24.00	2.466	2.173
41.20	2.020	2.548
32.18	2.349	2.333
29.51	2.398	2.278
33.59	2.317	2.364
32.05	2.351	2.331
32.16	2.349	2.333

32.54 2.341 2.341

9/10

Factor Of Safety For The Preceding Specified Surface = 2.341
Spencer's Theta = 32.54

Factor Of Safety Is Calculated By Spencer's Method of Slices

*** Line of Thrust ***

Slice No.	X Coord.	Y Coord.	L/H	Side Force (lbs)
1	38.00	72.71	.660	6.
2	43.00	73.96	.267	222.
3	44.74	74.31	.285	363.
4	45.00	74.30	.277	402.
5	45.25	74.30	.272	442.
6	46.00	74.38	.268	562.
7	51.00	75.56	.282	1581.
8	55.00	76.53	.250	2781.
9	62.50	78.44	.258	6002.
10	64.00	78.84	.276	6757.
11	65.00	79.12	.289	7260.
12	65.65	80.00	.308	6633.
13	66.00	79.91	.285	6423.
14	77.62	90.30	.433	211.
15	80.00	129.64	.000	0.

1

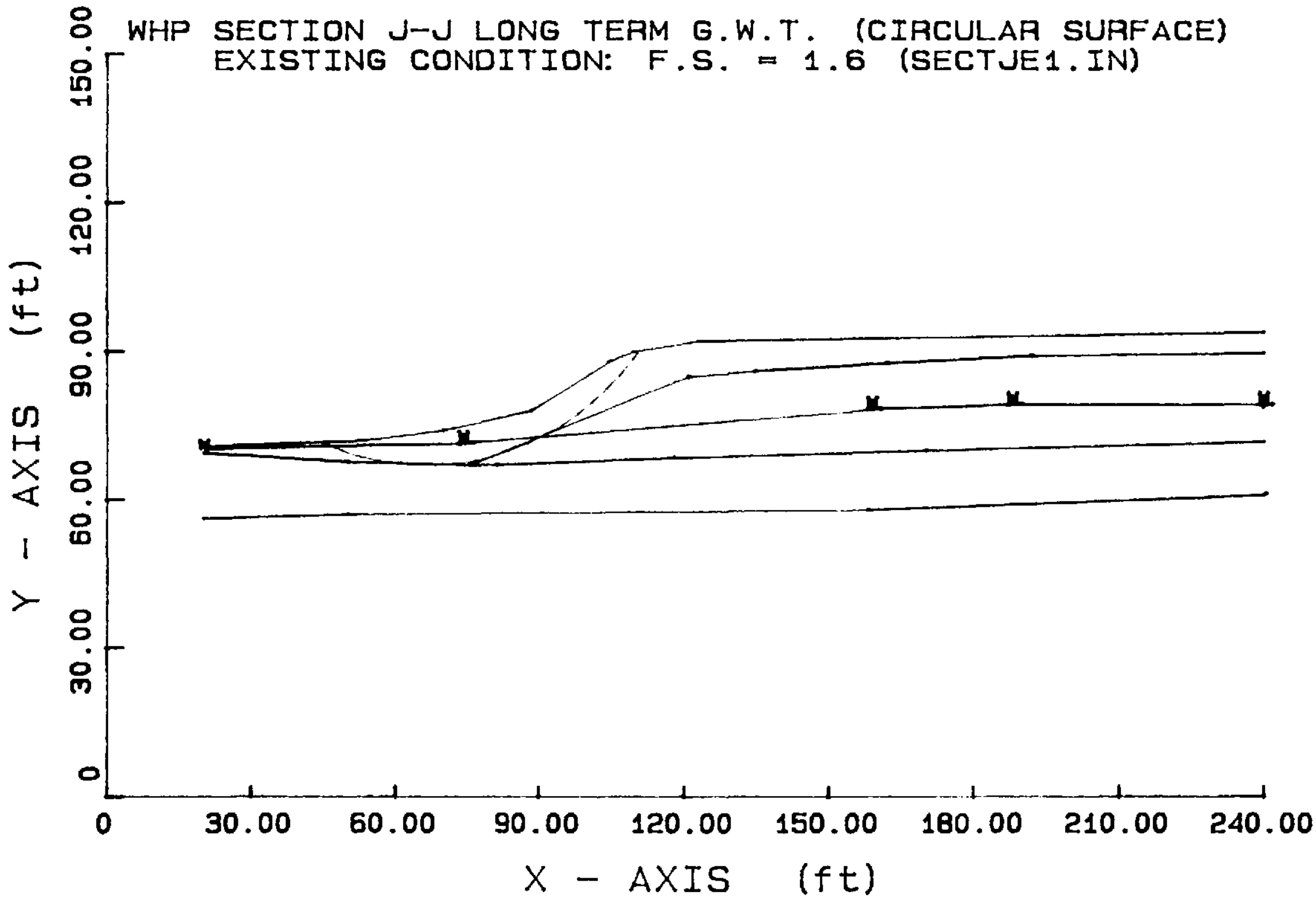
```
            Y           A     X     I     S            F     T

            .00     37.50     75.00    112.50    150.00    187.50

X       .00 +---------+---------+---------+---------+---------+
            -               * **
            -                  *
            -
            -
            -                 *
      37.50 +                 **
            -                  * *
            -                     *
            -                      *
            -                   S   *
            -                       *
A     75.00 +
            -                        S
            -
            -
            -
            -
X    112.50 +
            -              *
            -                    W
            -
            -                       *
            -
I    150.00 +                        *
            -                *
            -
```

```
            -
            -                *
            -
S    187.50 +
            -
            -                       **
            -
            -
            -
     225.00 +              *
            -                 *
            -               *
            -                    W
            -
            -                *
F    262.50 +
            -                  *
            -                     W
            -
            -                  *
            -
T    300.00 +                   * W **
```

Section J-J' - Existing Conditions
Circular - Long-Term Water Table


WHP SECTION J-J LONG TERM G.W.T. (CIRCULAR SURFACE)
EXISTING CONDITION: F.S. = 1.6 (SECTJE1.IN)
Y - AXIS (ft)
X - AXIS (ft)
0
30.00
60.00
90.00
120.00
150.00
180.00
210.00
240.00

2/4

** PCSTABL5M **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

Run Date: 5/8/92
Time of Run:
Run By: DOKL
Input Data Filename: SECTJE1.IN
Output Filename: SECTJE1.OUT
Plotted Output Filename: SECTJE1.PLT

PROBLEM DESCRIPTION ISRT: SECTION J-J, EXISTING SLOPE FILE S
ECTJE1.IN

BOUNDARY COORDINATES

7 Top Boundaries
21 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	20.00	70.70	53.00	72.00	1
2	53.00	72.00	70.00	74.00	1
3	70.00	74.00	88.50	78.00	1
4	88.50	78.00	104.50	88.00	1
5	104.50	88.00	109.50	89.90	1
6	109.50	89.90	123.00	92.00	1
7	123.00	92.00	240.00	94.00	1
8	20.00	69.30	50.00	67.60	3
9	50.00	67.60	75.50	67.00	3
10	75.50	67.00	121.00	85.00	2
11	121.00	85.00	135.00	86.00	2
12	135.00	86.00	162.00	87.50	2
13	162.00	87.50	192.00	89.00	2
14	192.00	89.00	240.00	89.80	2
15	75.50	67.00	81.00	67.00	3
16	81.00	67.00	118.00	68.50	3
17	118.00	68.50	170.00	70.00	3
18	170.00	70.00	240.00	71.80	3
19	20.00	56.20	50.00	57.00	4
20	50.00	57.00	158.00	58.00	4
21	158.00	58.00	240.00	61.00	4

1

ISOTROPIC SOIL PARAMETERS

4 Type(s) of Soil

3/4

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	90.0	100.0	.0	25.0	.00	.0	1
2	100.0	125.0	.0	34.0	.00	.0	1
3	120.0	120.0	.0	36.0	.00	.0	1
4	125.0	125.0	.0	37.0	.00	.0	1

1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 5 Coordinate Points

Point No.	X-Water (ft)	Y-Water (ft)
1	20.00	70.00
2	74.00	71.00
3	159.00	78.00
4	188.00	79.00
5	240.00	79.00

1

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.

400 Trial Surfaces Have Been Generated.

20 Surfaces Initiate From Each Of 20 Points Equally Spaced
Along The Ground Surface Between X = 40.00 ft.
and X = 70.00 ft.

Each Surface Terminates Between X = 110.00 ft.
and X = 130.00 ft.

Unless Further Limitations Were Imposed, The Minimum Elevation At Which A Surface Extends Is Y = .00 ft.

4.00 ft. Line Segments Define Each Trial Failure Surface.

Restrictions Have Been Imposed Upon The Angle Of Initiation. The Angle Has Been Restricted Between The Angles Of -45.0 And -20.0 deg.

1

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Examined. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Bishop Method * *

Failure Surface Specified By 20 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	44.74	71.67
2	48.47	70.24
3	52.30	69.08
4	56.20	68.21
5	60.16	67.62
6	64.15	67.33
7	68.15	67.33
8	72.14	67.62
9	76.10	68.21
10	80.00	69.08
11	83.83	70.24
12	87.56	71.67
13	91.18	73.38
14	94.66	75.35
15	97.99	77.56
16	101.15	80.02
17	104.12	82.70
18	106.89	85.59
19	109.43	88.67
20	110.40	90.04

Circle Center At X = 66.1 ; Y = 121.7 and Radius, 54.4

*** 1.632 ***

Individual data on the 30 slices

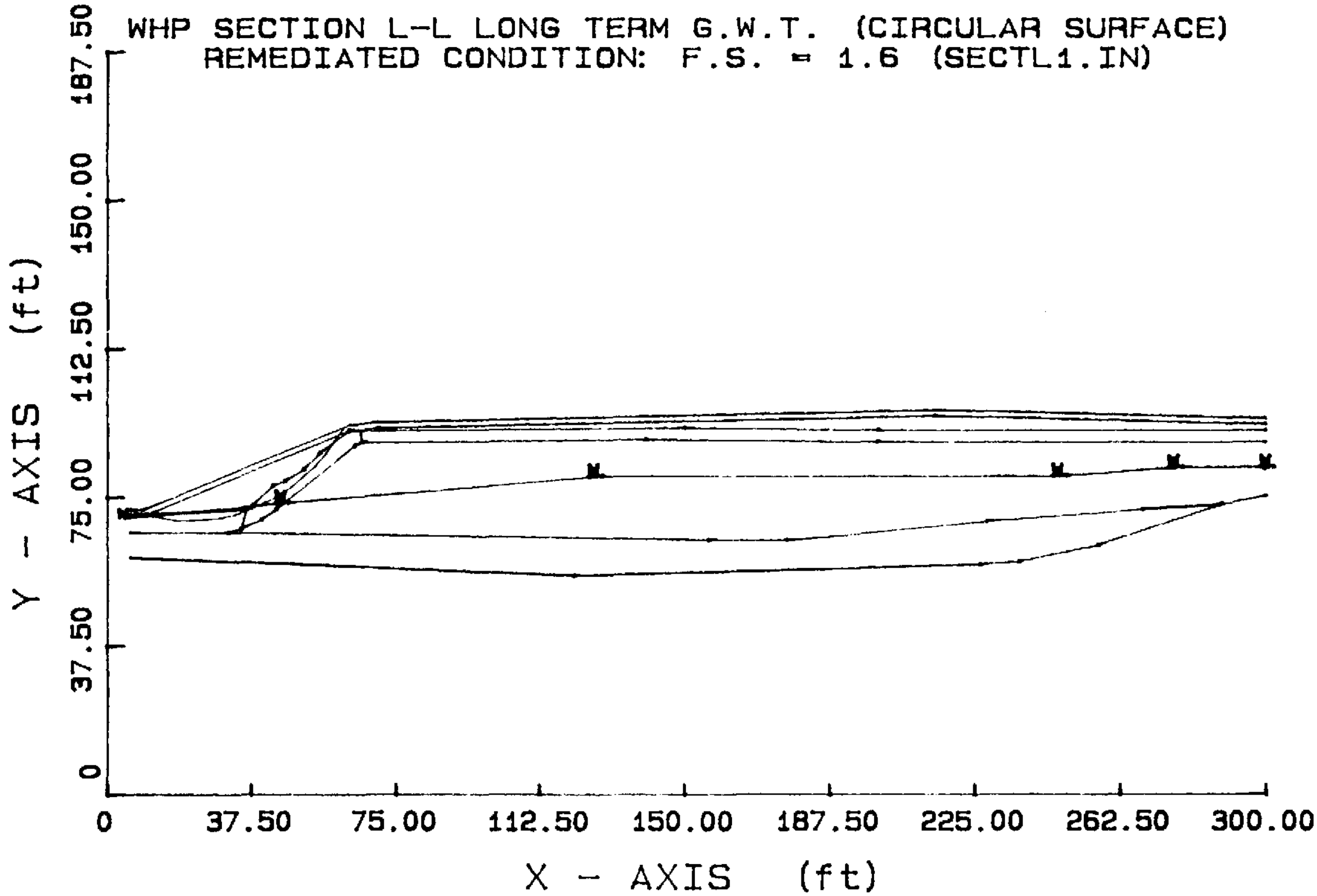
Slice No.	Width Ft(m)	Weight Lbs(kg)	Water Force Top Lbs(kg)	Water Force Bot Lbs(kg)	Tie Force Norm Lbs(kg)	Tie Force Tan Lbs(kg)	Earthquake Force Hor Lbs(kg)	Earthquake Force Ver Lbs(kg)	Surcharge Load Lbs(kg)
1	3.0	173.8	.0	.0	.0	.0	.0	.0	.0
2	.7	93.1	.0	6.9	.0	.0	.0	.0	.0
3	3.8	805.3	.0	225.2	.0	.0	.0	.0	.0
4	.7	199.2	.0	71.7	.0	.0	.0	.0	.0
5	3.2	1110.7	.0	424.9	.0	.0	.0	.0	.0
6	4.0	1782.3	.0	696.8	.0	.0	.0	.0	.0
7	4.0	2142.9	.0	824.8	.0	.0	.0	.0	.0
8	4.0	2379.5	.0	879.8	.0	.0	.0	.0	.0
9	1.9	1146.7	.0	406.6	.0	.0	.0	.0	.0
10	2.1	1359.9	.0	455.1	.0	.0	.0	.0	.0
11	1.9	1216.6	.0	378.4	.0	.0	.0	.0	.0
12	2.1	1390.9	.0	399.7	.0	.0	.0	.0	.0
13	3.9	2597.3	.0	668.9	.0	.0	.0	.0	.0
14	3.2	2085.8	.0	432.1	.0	.0	.0	.0	
15	.6	373.7	.0	63.4	.0	.0	.0	.0	
16	3.7	2207.5	.0	250.4	.0	.0	.0	.0	.0
17	.9	511.0	.0	16.8	.0	.0	.0	.0	.0
18	.2	87.0	.0	.5	.0	.0	.0	.0	.0
19	.0	18.9	.0	.0	.0	.0	.0	.0	.0

APPENDIX 11-D

Remediated Slope Stability Calculations

Section L-L' - Remediated Conditions
Circular - Long-Term Water Table
Unit Weights of Fill and Hide Residue = 115 and 125 pcf

Unit Weights of Fill and Hide Residue = 125 pcf


WHP SECTION L-L LONG TERM G.W.T. (CIRCULAR SURFACE)
REMEDIATED CONDITION: F.S. = 1.6 (SECTL1.IN)
Y - AXIS (ft)
0
37.50
75.00
112.50
150.00
187.50
0
37.50
75.00
112.50
150.00
187.50
225.00
262.50
300.00
X - AXIS (ft)

2/4

** PCSTABL5M **

by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

Run Date: 5/7/92
Time of Run:
Run By: DOKL
Input Data Filename: SECTL1.IN
Output Filename: SECTL1.OUT
Plotted Output Filename: SECTL1.PLT

PROBLEM DESCRIPTION ISRT: SECTION L-L, REMEDIATED SLOPE FILE SECTL1.IN

BOUNDARY COORDINATES

5 Top Boundaries
43 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	6.00	72.00	9.50	72.00	1
2	9.50	72.00	62.50	93.30	1
3	62.50	93.30	70.00	94.00	1
4	70.00	94.00	215.00	97.00	1
5	215.00	97.00	300.00	95.00	1
6	6.00	70.50	10.00	70.50	3
7	10.00	70.50	62.50	91.80	2
8	62.50	91.80	70.00	92.50	2
9	70.00	92.50	215.00	95.50	2
10	215.00	95.50	300.00	93.50	2
11	10.00	70.50	36.00	72.00	3
12	36.00	72.00	38.00	73.00	4
13	38.00	73.00	43.00	78.00	4
14	43.00	78.00	46.00	79.00	4
15	46.00	79.00	51.00	82.00	4
16	51.00	82.00	55.00	86.20	4
17	55.00	86.20	62.50	91.70	4
18	62.50	91.70	66.10	92.00	8
19	66.10	92.00	150.00	92.80	8
20	150.00	92.80	200.00	92.00	8
21	200.00	92.00	300.00	92.00	8
22	35.00	67.00	36.00	72.00	8
23	66.00	89.00	66.10	92.00	8
24	6.00	66.00	31.50	66.00	6
25	31.50	66.00	35.00	67.00	5
26	35.00	67.00	40.00	69.30	5
27	40.00	69.30	44.00	72.00	5
28	44.00	72.00	64.00	88.30	5
29	64.00	88.30	66.00	89.00	5
30	66.00	89.00	140.00	89.50	5

3,4

31	140.00	89.50	200.00	89.00	5
32	200.00	89.00	300.00	89.00	5
33	31.50	66.00	156.50	64.00	6
34	156.50	64.00	176.50	64.00	6
35	176.50	64.00	228.50	69.00	6
36	228.50	69.00	268.50	72.00	6
37	268.50	72.00	288.50	73.00	6
38	288.50	73.00	300.00	75.40	7
39	6.00	59.50	121.50	55.00	7
40	121.50	55.00	226.50	58.00	7
41	226.50	58.00	236.50	59.00	7
42	236.50	59.00	257.00	63.00	7
43	257.00	63.00	288.50	73.00	7

1

ISOTROPIC SOIL PARAMETERS

8 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	120.0	.0	.0	.00	.0	1
2	125.0	125.0	.0	33.0	.00	.0	1
3	90.0	100.0	.0	32.0	.00	.0	1
4	90.0	100.0	.0	25.0	.00	.0	1
5	100.0	125.0	.0	34.0	.00	.0	1
6	120.0	120.0	.0	36.0	.00	.0	1
7	125.0	125.0	.0	37.0	.00	.0	1
8	90.0	100.0	.0	32.0	.00	.0	0

1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 6 Coordinate Points

Point No.	X-Water (ft)	Y-Water (ft)
1	4.00	69.50
2	45.00	73.00
3	126.50	80.00
4	246.50	80.00
5	276.50	82.00
6	300.00	82.20

1

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.

400 Trial Surfaces Have Been Generated.

20 Surfaces Initiate From Each Of 20 Points Equally Spaced
Along The Ground Surface Between X = 6.00 ft.
and X = 25.00 ft.

Each Surface Terminates Between X = 50.00 ft.
and X = 70.00 ft.

Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = .00 ft.

4.00 ft. Line Segments Define Each Trial Failure Surface.

Restrictions Have Been Imposed Upon The Angle Of Initiation.
The Angle Has Been Restricted Between The Angles Of -45.0
And -5.0 deg.

1

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Examined. They Are Ordered - Most Critical
First.

* * Safety Factors Are Calculated By The Modified Bishop Method * *

Failure Surface Specified By 17 Coordinate Points

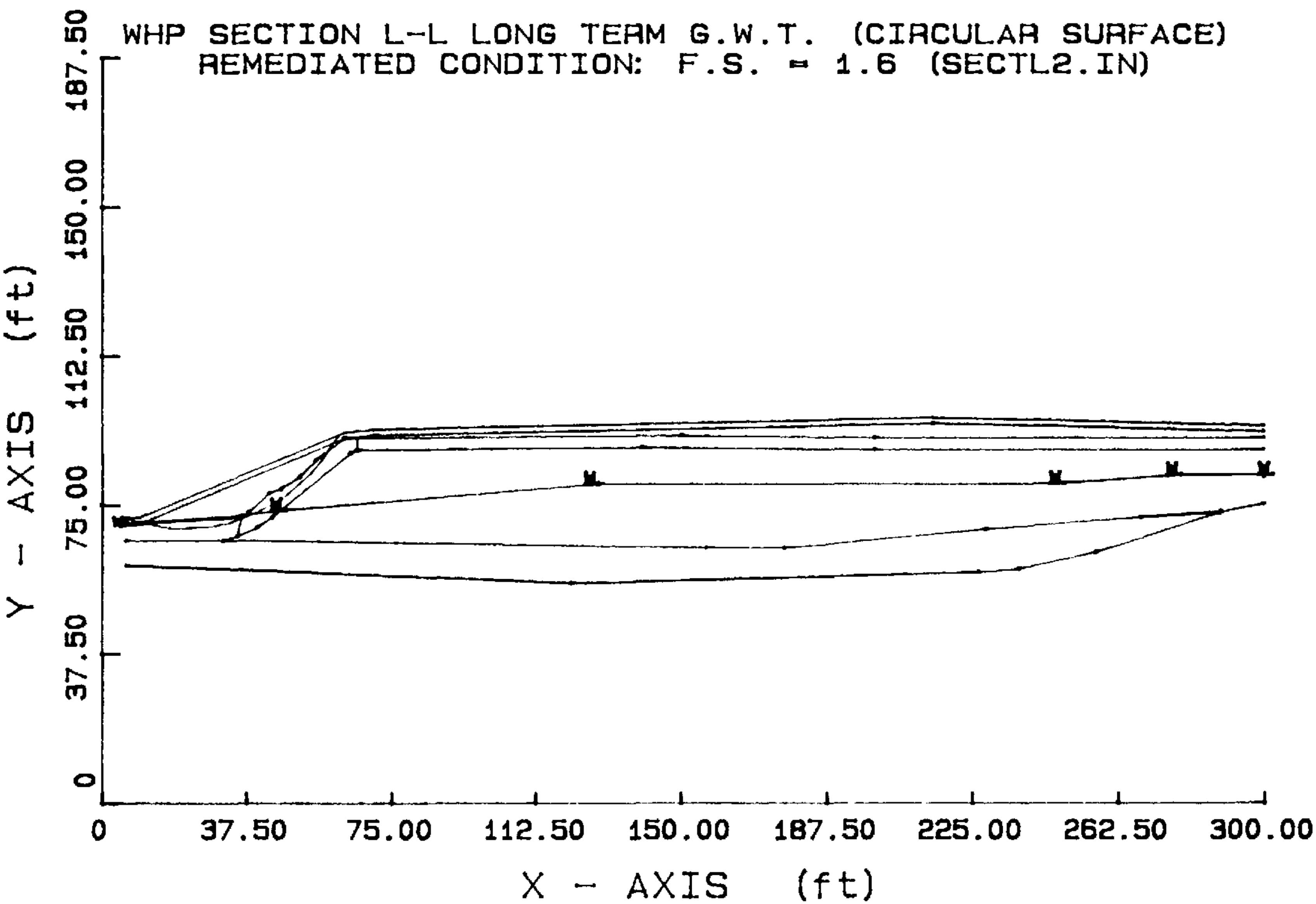
Point No.	X-Surf (ft)	Y-Surf (ft)
1	6.00	72.00
2	9.82	70.82
3	13.74	69.99
4	17.71	69.50
5	21.70	69.36
6	25.70	69.58
7	29.66	70.14
8	33.55	71.05
9	37.35	72.30
10	41.03	73.88
11	44.55	75.78
12	47.89	77.98
13	51.03	80.46
14	53.93	83.21
15	56.58	86.20
16	58.96	89.42
17	60.95	92.68

Circle Center At X = 21.3 ; Y = 114.8 and Radius, 45.4

*** 1.613 ***

Individual data on the 32 slices

Unit Weights of Fill and Hide Residue = 115 pcf


WHP SECTION L-L LONG TERM G.W.T. (CIRCULAR SURFACE)
REMEDIATED CONDITION: F.S. = 1.6 (SECTL2.IN)
Y - AXIS (ft)
X - AXIS (ft)
0
37.50
75.00
112.50
150.00
187.50
225.00
262.50
300.00

2/4

** PCSTABL5M **

by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

Run Date: 5/7/92
Time of Run:
Run By: DOKL
Input Data Filename: SECTL2.IN
Output Filename: SECTL2.OUT
Plotted Output Filename: SECTL2.PLT

PROBLEM DESCRIPTION ISRT: SECTION L-L, REMEDIATED SLOPE FILE
SECTL1.IN

BOUNDARY COORDINATES

5 Top Boundaries
43 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	6.00	72.00	9.50	72.00	1
2	9.50	72.00	62.50	93.30	1
3	62.50	93.30	70.00	94.00	1
4	70.00	94.00	215.00	97.00	1
5	215.00	97.00	300.00	95.00	1
6	6.00	70.50	10.00	70.50	3
7	10.00	70.50	62.50	91.80	2
8	62.50	91.80	70.00	92.50	2
9	70.00	92.50	215.00	95.50	2
10	215.00	95.50	300.00	93.50	2
11	10.00	70.50	36.00	72.00	3
12	36.00	72.00	38.00	73.00	4
13	38.00	73.00	43.00	78.00	4
14	43.00	78.00	46.00	79.00	4
15	46.00	79.00	51.00	82.00	4
16	51.00	82.00	55.00	86.20	4
17	55.00	86.20	62.50	91.70	4
18	62.50	91.70	66.10	92.00	8
19	66.10	92.00	150.00	92.80	8
20	150.00	92.80	200.00	92.00	8
21	200.00	92.00	300.00	92.00	8
22	35.00	67.00	36.00	72.00	8
23	66.00	89.00	66.10	92.00	8
24	6.00	66.00	31.50	66.00	6
25	31.50	66.00	35.00	67.00	5
26	35.00	67.00	40.00	69.30	5
27	40.00	69.30	44.00	72.00	5
28	44.00	72.00	64.00	88.30	5
29	64.00	88.30	66.00	89.00	5
30	66.00	89.00	140.00	89.50	5

3/4

31	140.00	89.50	200.00	89.00	5
32	200.00	89.00	300.00	89.00	5
33	31.50	66.00	156.50	64.00	6
34	156.50	64.00	176.50	64.00	6
35	176.50	64.00	228.50	69.00	6
36	228.50	69.00	268.50	72.00	6
37	268.50	72.00	288.50	73.00	6
38	288.50	73.00	300.00	75.40	7
39	6.00	59.50	121.50	55.00	7
40	121.50	55.00	226.50	58.00	7
41	226.50	58.00	236.50	59.00	7
42	236.50	59.00	257.00	63.00	7
43	257.00	63.00	288.50	73.00	7

1

ISOTROPIC SOIL PARAMETERS

8 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	120.0	.0	.0	.00	.0	1
2	125.0	125.0	.0	33.0	.00	.0	1
3	90.0	100.0	.0	32.0	.00	.0	1
4	90.0	100.0	.0	25.0	.00	.0	1
5	92.0	115.0	.0	34.0	.00	.0	1
6	120.0	120.0	.0	36.0	.00	.0	1
7	125.0	125.0	.0	37.0	.00	.0	1
8	90.0	100.0	.0	32.0	.00	.0	0

1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 6 Coordinate Points

Point No.	X-Water (ft)	Y-Water (ft)
1	4.00	69.50
2	45.00	73.00
3	126.50	80.00
4	246.50	80.00
5	276.50	82.00
6	300.00	82.20

1

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.

400 Trial Surfaces Have Been Generated.

4/4

20 Surfaces Initiate From Each Of 20 Points Equally Spaced Along The Ground Surface Between X = 6.00 ft. and X = 25.00 ft.

Each Surface Terminates Between X = 50.00 ft. and X = 70.00 ft.

Unless Further Limitations Were Imposed, The Minimum Elevation At Which A Surface Extends Is Y = .00 ft.

4.00 ft. Line Segments Define Each Trial Failure Surface.

Restrictions Have Been Imposed Upon The Angle Of Initiation. The Angle Has Been Restricted Between The Angles Of -45.0 And -5.0 deg.

1

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Examined. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Bishop Method * *

Failure Surface Specified By 17 Coordinate Points

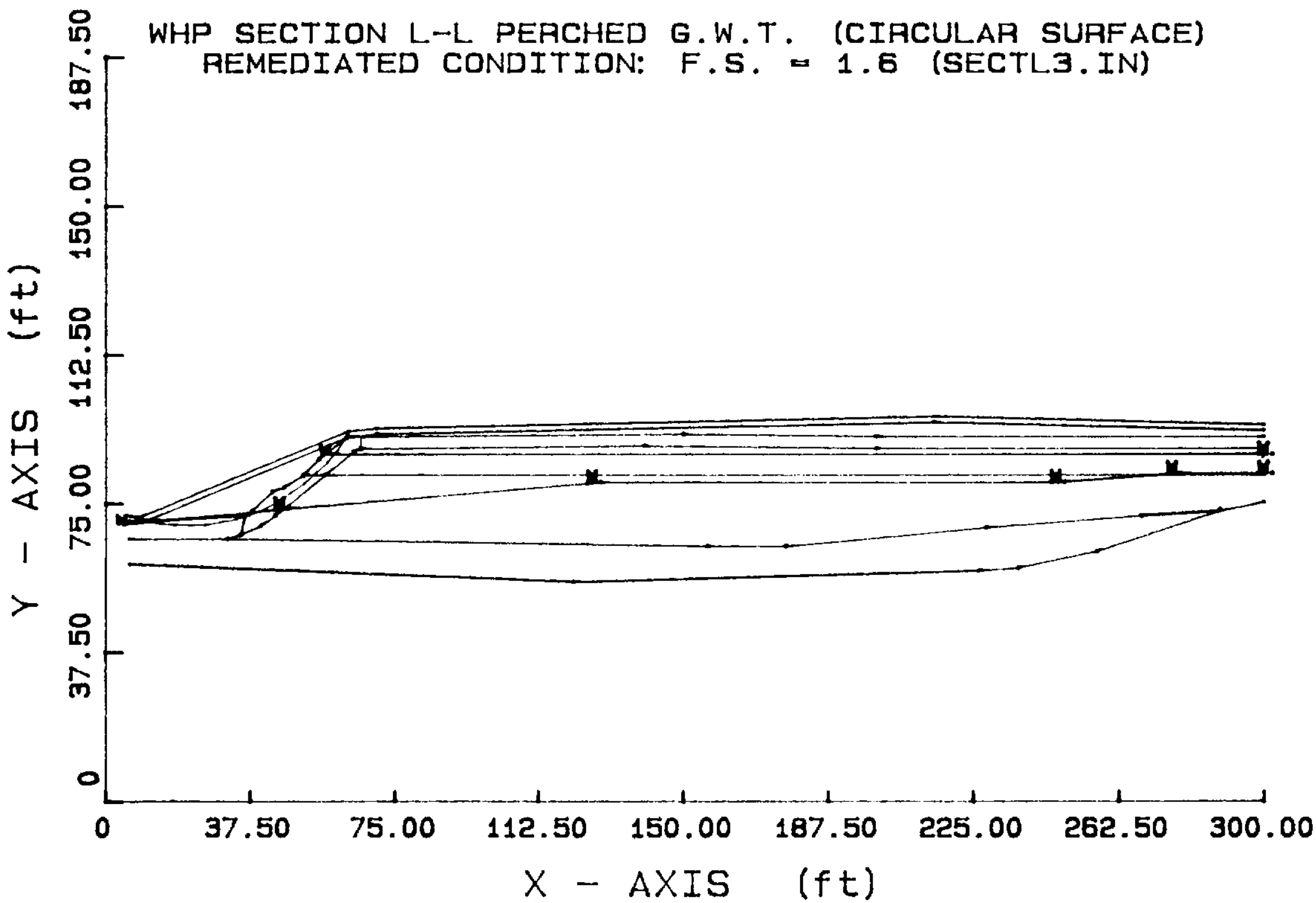
Point No.	X-Surf (ft)	Y-Surf (ft)
1	6.00	72.00
2	9.82	70.82
3	13.74	69.99
4	17.71	69.50
5	21.70	69.36
6	25.70	69.58
7	29.66	70.14
8	33.55	71.05
9	37.35	72.30
10	41.03	73.88
11	44.55	75.78
12	47.89	77.98
13	51.03	80.46
14	53.93	83.21
15	56.58	86.20
16	58.96	89.42
17	60.95	92.68

Circle Center At X = 21.3 ; Y = 114.8 and Radius, 45.4

*** 1.613 ***

Individual data on the 32 slices

Section L-L' - Remediated Condition
Circular - Perched Water Table


WHP SECTION L-L PERCHED G.W.T. (CIRCULAR SURFACE)
REMEDIATED CONDITION: F.S. = 1.6 (SECTL3.IN)
Y - AXIS (ft)
0
37.50
75.00
112.50
150.00
187.50
0
37.50
75.00
112.50
150.00
187.50
225.00
262.50
300.00
X - AXIS (ft)

2/6

** PCSTABL5M **

by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

Run Date:	5/7/92
Time of Run:	
Run By:	DOKL
Input Data Filename:	SECTL3.IN
Output Filename:	SECTL3.OUT
Plotted Output Filename:	SECTL3.PLT

PROBLEM DESCRIPTION ISRT: SECTION L-L, REMEDIATED SLOPE FILE SECTL3.IN PERCHED W.T.

BOUNDARY COORDINATES

5 Top Boundaries
46 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	6.00	72.00	9.50	72.00	1
2	9.50	72.00	62.50	93.30	1
3	62.50	93.30	70.00	94.00	1
4	70.00	94.00	215.00	97.00	1
5	215.00	97.00	300.00	95.00	1
6	6.00	70.50	10.00	70.50	3
7	10.00	70.50	62.50	91.80	2
8	62.50	91.80	70.00	92.50	2
9	70.00	92.50	215.00	95.50	2
10	215.00	95.50	300.00	93.50	2
11	10.00	70.50	36.00	72.00	3
12	36.00	72.00	38.00	73.00	4
13	38.00	73.00	43.00	78.00	4
14	43.00	78.00	46.00	79.00	4
15	46.00	79.00	51.00	82.00	4
16	51.00	82.00	55.00	86.20	5
17	55.00	86.20	62.50	91.70	5
18	62.50	91.70	66.10	92.00	10
19	66.10	92.00	150.00	92.80	10
20	150.00	92.80	200.00	92.00	10
21	200.00	92.00	300.00	92.00	10
22	66.00	89.00	66.10	92.00	10
23	51.00	82.00	56.50	82.00	4
24	35.00	67.00	36.00	72.00	4
25	6.00	66.00	31.50	66.00	8
26	31.50	66.00	35.00	67.00	7
27	35.00	67.00	40.00	69.30	7
28	40.00	69.30	44.00	72.00	7
29	44.00	72.00	56.50	82.00	7
30	56.50	82.00	64.00	88.30	6

3/6

31	64.00	88.30	66.00	89.00	6
32	66.00	89.00	140.00	89.50	6
33	140.00	89.50	200.00	89.00	6
34	200.00	89.00	300.00	89.00	6
35	56.50	82.00	300.00	82.00	7
36	31.50	66.00	156.50	64.00	8
37	156.50	64.00	176.50	64.00	8
38	176.50	64.00	228.50	69.00	8
39	228.50	69.00	268.50	72.00	8
40	268.50	72.00	288.50	73.00	8
41	288.50	73.00	300.00	75.40	9
42	6.00	59.50	121.50	55.00	9
43	121.50	55.00	226.50	58.00	9
44	226.50	58.00	236.50	59.00	9
45	236.50	59.00	257.00	63.00	9
46	257.00	63.00	288.50	73.00	9

1

ISOTROPIC SOIL PARAMETERS

10 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	120.0	.0	.0	.00	.0	1
2	125.0	125.0	.0	33.0	.00	.0	1
3	90.0	100.0	.0	32.0	.00	.0	1
4	90.0	100.0	.0	25.0	.00	.0	1
5	90.0	100.0	.0	25.0	.00	.0	2
6	100.0	125.0	.0	34.0	.00	.0	2
7	100.0	125.0	.0	34.0	.00	.0	1
8	120.0	120.0	.0	36.0	.00	.0	1
9	125.0	125.0	.0	37.0	.00	.0	1
10	90.0	100.0	.0	32.0	.00	.0	0

1

2 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 6 Coordinate Points

Point No.	X-Water (ft)	Y-Water (ft)
1	4.00	69.50
2	45.00	73.00
3	126.50	80.00
4	246.50	80.00
5	276.50	82.00
6	300.00	82.20

Piezometric Surface No. 2 Specified by 2 Coordinate Points

4/6

Point No.	X-Water (ft)	Y-Water (ft)
1	56.50	87.00
2	300.00	87.00

1

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.

400 Trial Surfaces Have Been Generated.

20 Surfaces Initiate From Each Of 20 Points Equally Spaced Along The Ground Surface Between X = 6.00 ft.
and X = 25.00 ft.

Each Surface Terminates Between X = 60.00 ft.
and X = 80.00 ft.

Unless Further Limitations Were Imposed, The Minimum Elevation At Which A Surface Extends Is Y = .00 ft.

4.00 ft. Line Segments Define Each Trial Failure Surface.

Restrictions Have Been Imposed Upon The Angle Of Initiation. The Angle Has Been Restricted Between The Angles Of -45.0 And -10.0 deg.

1

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Examined. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Bishop Method * *

Failure Surface Specified By 18 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	6.00	72.00
2	9.86	70.94
3	13.79	70.19
4	17.76	69.76
5	21.76	69.66
6	25.76	69.88
7	29.72	70.43
8	33.62	71.30
9	37.44	72.48
10	41.16	73.97
11	44.74	75.75
12	48.16	77.82

5/6

13	51.41	80.16
14	54.45	82.75
15	57.27	85.59
16	59.85	88.64
17	62.18	91.90
18	63.05	93.35

Circle Center At X = 21.0 ; Y = 118.8 and Radius, 49.1

*** 1.570 ***

Individual data on the 37 slices

Slice No.	Width Ft(m)	Weight Lbs(kg)	Water Force Top Lbs(kg)	Water Force Bot Lbs(kg)	Tie Force Norm Lbs(kg)	Tie Force Tan Lbs(kg)	Earthquake Force Hor Lbs(kg)	Earthquake Force Ver Lbs(kg)	Surcharge Load Lbs(kg)
1	3.5	202.8	.0	.0	.0	.0	.0	.0	.0
2	.4	46.5	.0	.0	.0	.0	.0	.0	.0
3	.8	144.5	.0	.0	.0	.0	.0	.0	.0
4	1.0	230.1	.0	.0	.0	.0	.0	.0	.0
5	1.6	525.3	.0	.0	.0	.0	.0	.0	.0
6	.5	213.0	.0	2.5	.0	.0	.0	.0	.0
7	4.0	2128.6	.0	131.9	.0	.0	.0	.0	.0
8	4.0	3028.4	.0	282.2	.0	.0	.0	.0	.0
9	4.0	3783.8	.0	352.0	.0	.0	.0	.0	.0
10	1.9	1997.7	.0	167.4	.0	.0	.0	.0	.0
	2.1	2377.2	.0	173.3	.0	.0	.0	.0	.0
[illegible]	3.9	4785.8	.0	248.4	.0	.0	.0	.0	.0
13	2.3	2922.9	.0	70.4	.0	.0	.0	.0	.0
14	.3	381.4	.0	3.7	.0	.0	.0	.0	.0
15	.4	521.8	.0	3.1	.0	.0	.0	.0	.0
16	.3	446.1	.0	.8	.0	.0	.0	.0	.0
17	.5	727.9	.0	.0	.0	.0	.0	.0	.0
18	.6	736.9	.0	.0	.0	.0	.0	.0	.0
19	3.2	4082.5	.0	.0	.0	.0	.0	.0	.0
20	1.8	2271.2	.0	.0	.0	.0	.0	.0	.0
21	1.7	2085.4	.0	.0	.0	.0	.0	.0	.0
22	1.3	1495.3	.0	.0	.0	.0	.0	.0	.0
23	2.2	2480.0	.0	.0	.0	.0	.0	.0	.0
24	2.8	3036.9	.0	.0	.0	.0	.0	.0	.0
25	.4	409.6	.0	.0	.0	.0	.0	.0	.0
26	2.2	2041.6	.0	.0	.0	.0	.0	.0	.0
27	.9	761.8	.0	335.3	.0	.0	.0	.0	.0
28	.6	448.6	.0	193.6	.0	.0	.0	.0	.0
29	1.5	1111.4	.0	389.8	.0	.0	.0	.0	.0
30	.8	506.8	.0	122.6	.0	.0	.0	.0	.0
31	1.2	683.0	.0	81.4	.0	.0	.0	.0	.0
32	1.4	638.3	.0	.0	.0	.0	.0	.0	.0
33	1.7	533.0	.0	.0	.0	.0	.0	.0	.0
34	.4	86.7	.0	.0	.0	.0	.0	.0	.0
35	.2	38.0	.0	.0	.0	.0	.0	.0	.0
36	.3	41.1	.0	.0	.0	.0	.0	.0	.0
37	.6	28.8	.0	.0	.0	.0	.0	.0	.0

Failure Surface Specified By 19 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)

6/6

1	6.00	72.00
2	9.88	71.02
3	13.82	70.33
4	17.80	69.92
5	21.80	69.81
6	25.79	69.99
7	29.76	70.46
8	33.69	71.22
9	37.55	72.27
10	41.33	73.59
11	45.00	75.18
12	48.54	77.04
13	51.94	79.15
14	55.17	81.51
15	58.22	84.09
16	61.08	86.89
17	63.72	89.89
18	66.14	93.08
19	66.53	93.68

Circle Center At X = 21.3 ; Y = 124.6 and Radius, 54.8

*** 1.586 ***

Failure Surface Specified By 17 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	9.00	72.00
2	12.85	70.92
3	16.78	70.18
4	20.76	69.78
5	24.76	69.72
6	28.75	70.01
7	32.70	70.64
8	36.58	71.62
9	40.36	72.92
10	44.02	74.54
11	47.52	76.48
12	50.84	78.70
13	53.96	81.21
14	56.85	83.98
15	59.49	86.98
16	61.86	90.20
17	63.82	93.42

Circle Center At X = 23.4 ; Y = 116.0 and Radius, 46.3

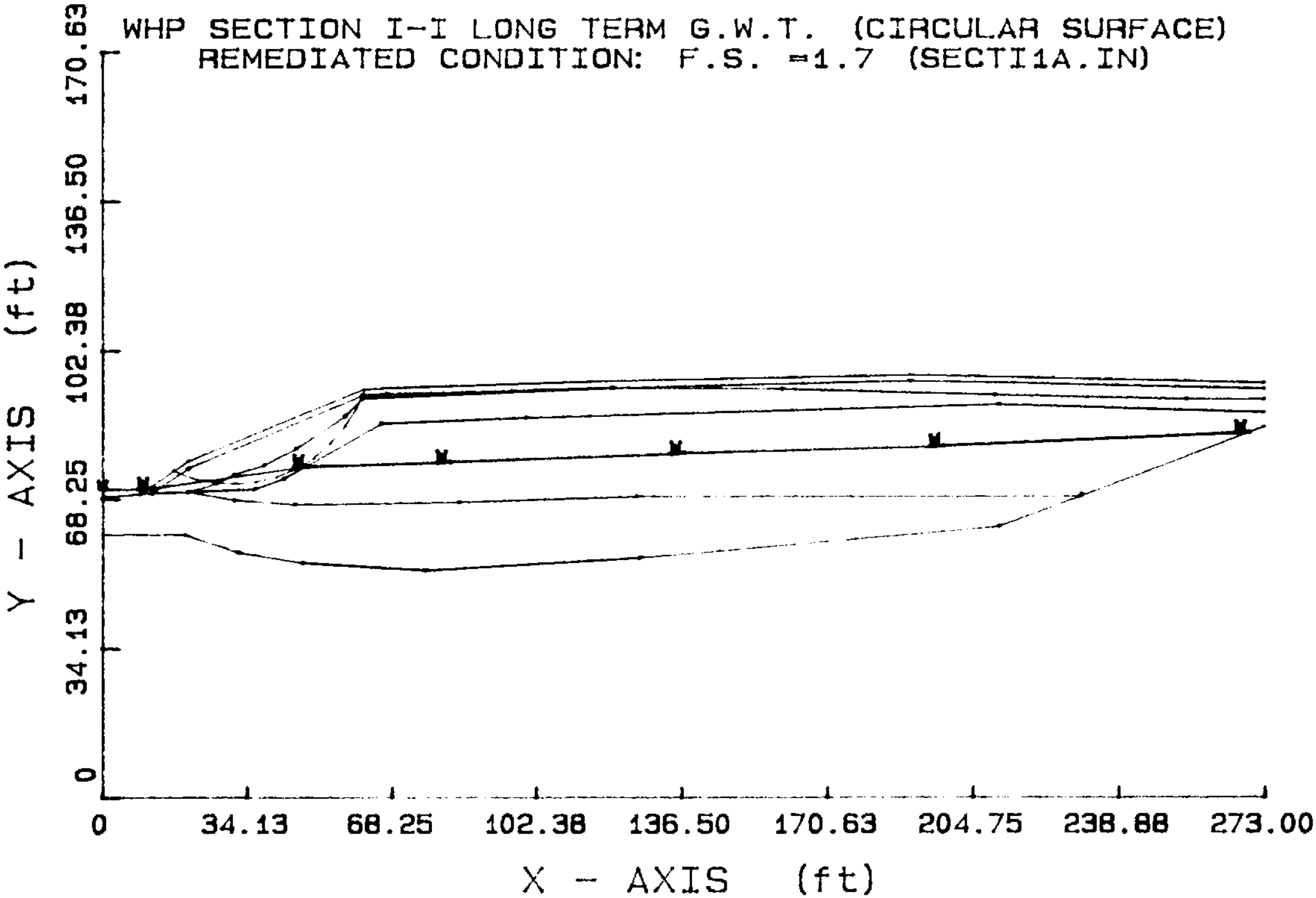
*** 1.618 ***

Failure Surface Specified By 17 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)

Section I-I' - Remediated Conditions
Circular - Long-Term Water Table
Unit Weights of Fill and Hide Residue = 115 and 125 pcf

Unit Weights of Fill and Hide Residue = 125 pcf


WHP SECTION I-I LONG TERM G.W.T. (CIRCULAR SURFACE)
REMEDIATED CONDITION: F.S. =1.7 (SECTI1A.IN)
Y - AXIS (ft)
0
34.13
68.25
102.38
136.50
170.63
X - AXIS (ft)
0
34.13
68.25
102.38
136.50
170.63
204.75
238.88
273.00

2/4

** PCSTABL5M **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

Run Date: 4/24/92
Time of Run:
Run By: DOKL
Input Data Filename: SECTI1A.IN
Output Filename: SECTI1A.OUT
Plotted Output Filename: SECTI1A.PLT

PROBLEM DESCRIPTION ISRT: SECTION I-I, REMEDIATED SLOPE FILE
SECTI5A.IN

BOUNDARY COORDINATES

7 Top Boundaries
45 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	.00	69.00	9.00	69.50	5
2	9.00	69.50	20.00	77.00	1
3	20.00	77.00	61.00	93.50	1
4	61.00	93.50	66.00	94.00	1
5	66.00	94.00	120.00	95.50	1
6	120.00	95.50	190.00	97.00	1
7	190.00	97.00	273.00	95.00	1
8	9.00	69.50	11.50	69.50	5
9	11.50	69.50	20.00	75.50	2
10	20.00	75.50	61.00	92.00	2
11	61.00	92.00	67.00	92.50	2
12	67.00	92.50	120.00	94.00	2
13	120.00	94.00	190.00	95.50	2
14	190.00	95.50	273.00	93.50	2
15	11.50	69.50	20.00	70.00	5
16	20.00	70.00	27.00	72.00	3
17	27.00	72.00	31.00	74.00	3
18	31.00	74.00	38.00	76.00	3
19	38.00	76.00	45.50	80.00	3
20	45.50	80.00	57.00	87.50	3
21	57.00	87.50	61.00	91.50	3
22	61.00	91.50	120.00	93.80	3
23	120.00	93.80	160.00	93.50	3
24	160.00	93.50	210.00	92.00	3
25	210.00	92.00	255.00	91.00	3
26	255.00	91.00	273.00	91.00	3
27	20.00	70.00	36.00	70.50	4
28	36.00	70.50	42.50	73.00	4
29	42.50	73.00	65.50	85.80	4
30	65.50	85.80	100.00	87.00	4

3/4

31	100.00	87.00	211.00	90.00	4
32	211.00	90.00	273.00	88.00	4
33	20.00	70.00	31.00	68.00	5
34	31.00	68.00	45.00	67.00	5
35	45.00	67.00	84.00	67.50	5
36	84.00	67.50	126.00	69.00	5
37	126.00	69.00	230.00	69.00	5
38	230.00	69.00	273.00	85.00	6
39	.00	60.00	19.50	60.00	6
40	19.50	60.00	32.00	56.00	6
41	32.00	56.00	47.00	53.50	6
42	47.00	53.50	76.00	52.00	6
43	76.00	52.00	126.50	55.00	6
44	126.50	55.00	211.00	62.00	6
45	211.00	62.00	230.00	69.00	6

1

ISOTROPIC SOIL PARAMETERS

6 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	120.0	.0	.0	.00	.0	1
2	125.0	125.0	.0	33.0	.00	.0	1
3	90.0	100.0	.0	25.0	.00	.0	1
4	100.0	125.0	.0	34.0	.00	.0	1
5	120.0	120.0	.0	36.0	.00	.0	1
6	125.0	125.0	.0	37.0	.00	.0	1

1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 7 Coordinate Points

Point No.	X-Water (ft)	Y-Water (ft)
1	.00	70.00
2	9.50	70.00
3	46.00	75.00
4	80.00	76.00
5	135.00	78.00
6	196.00	80.00
7	267.50	83.00

1

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.

400 Trial Surfaces Have Been Generated.

20 Surfaces Initiate From Each Of 20 Points Equally Spaced
Along The Ground Surface Between X = 9.00 ft.
and X = 25.00 ft.

Each Surface Terminates Between X = 60.00 ft.
and X = 80.00 ft.

Unless Further Limitations Were Imposed, The Minimum Elevation At Which A Surface Extends Is Y = 40.00 ft.

5.00 ft. Line Segments Define Each Trial Failure Surface.

Restrictions Have Been Imposed Upon The Angle Of Initiation. The Angle Has Been Restricted Between The Angles Of -45.0 And -20.0 deg.

1

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Examined. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Bishop Method * *

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	16.58	74.67
2	21.28	72.95
3	26.18	71.96
4	31.17	71.70
5	36.15	72.19
6	40.99	73.42
7	45.60	75.35
8	49.88	77.95
9	53.72	81.15
10	57.03	84.89
11	59.76	89.08
12	61.81	93.58

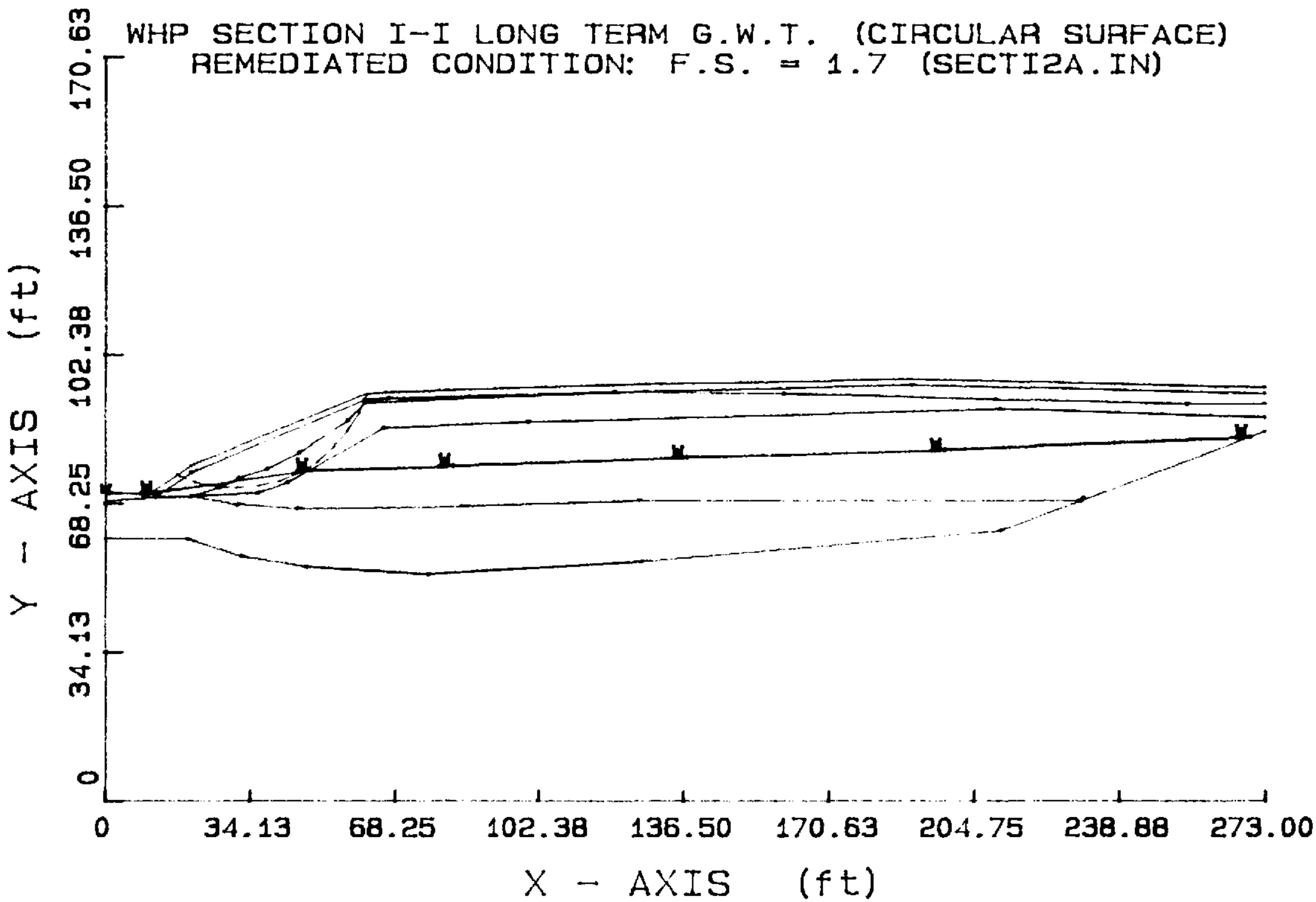
Circle Center At X = 30.4 ; Y = 105.2 and Radius, 33.5

*** 1.711 ***

Individual data on the 25 slices

Slice	Width	Weight	Water Force Top	Water Force Bot	Tie Force Norm	Tie Force Tan	Earthquake Force Hor	Earthquake Force Ver	Surcharge Load

Unit Weights of Fill and Hide Residue = 115 pcf



WHP SECTION I-I LONG TERM G.W.T. (CIRCULAR SURFACE)
REMEDIATED CONDITION: F.S. = 1.7 (SECTI2A.IN)
Y - AXIS (ft)
170.63
136.50
102.38
68.25
34.13
0
34.13
68.25
102.38
136.50
170.63
204.75
238.88
273.00
X - AXIS (ft)

2/4

** PCSTABL5M **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

Run Date:	4/24/92
Time of Run:	
Run By:	DOKL
Input Data Filename:	SECTI2A.IN
Output Filename:	SECTI2A.OUT
Plotted Output Filename:	SECTI2A.PLT

PROBLEM DESCRIPTION ISRT: SECTION I-I, REMEDIATED SLOPE FILE SECTI6A.IN

BOUNDARY COORDINATES

7 Top Boundaries
45 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	.00	69.00	9.00	69.50	5
2	9.00	69.50	20.00	77.00	1
3	20.00	77.00	61.00	93.50	1
4	61.00	93.50	66.00	94.00	1
5	66.00	94.00	120.00	95.50	1
6	120.00	95.50	190.00	97.00	1
7	190.00	97.00	273.00	95.00	1
8	9.00	69.50	11.50	69.50	5
9	11.50	69.50	20.00	75.50	2
10	20.00	75.50	61.00	92.00	2
11	61.00	92.00	67.00	92.50	2
12	67.00	92.50	120.00	94.00	2
13	120.00	94.00	190.00	95.50	2
14	190.00	95.50	273.00	93.50	2
15	11.50	69.50	20.00	70.00	5
16	20.00	70.00	27.00	72.00	3
17	27.00	72.00	31.00	74.00	3
18	31.00	74.00	38.00	76.00	3
19	38.00	76.00	45.50	80.00	3
20	45.50	80.00	57.00	87.50	3
21	57.00	87.50	61.00	91.50	3
22	61.00	91.50	120.00	93.80	3
23	120.00	93.80	160.00	93.50	3
24	160.00	93.50	210.00	92.00	3
25	210.00	92.00	255.00	91.00	3
26	255.00	91.00	273.00	91.00	3
27	20.00	70.00	36.00	70.50	4
28	36.00	70.50	42.50	73.00	4
29	42.50	73.00	65.50	85.80	4
30	65.50	85.80	100.00	87.00	4

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31	100.00	87.00	211.00	90.00	4
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35	45.00	67.00	84.00	67.50	5
36	84.00	67.50	126.00	69.00	5
37	126.00	69.00	230.00	69.00	5
38	230.00	69.00	273.00	85.00	6
39	.00	60.00	19.50	60.00	6
40	19.50	60.00	32.00	56.00	6
41	32.00	56.00	47.00	53.50	6
42	47.00	53.50	76.00	52.00	6
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44	126.50	55.00	211.00	62.00	6
45	211.00	62.00	230.00	69.00	6

1

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1	120.0	120.0	.0	.0	.00	.0	1
2	125.0	125.0	.0	33.0	.00	.0	1
3	90.0	100.0	.0	25.0	.00	.0	1
4	92.0	115.0	.0	34.0	.00	.0	1
5	120.0	120.0	.0	36.0	.00	.0	1
6	125.0	125.0	.0	37.0	.00	.0	1

1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 7 Coordinate Points

Point No.	X-Water (ft)	Y-Water (ft)
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5	135.00	78.00
6	196.00	80.00
7	267.50	83.00

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and X = 25.00 ft.

Each Surface Terminates Between X = 60.00 ft.
and X = 80.00 ft.

Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 40.00 ft.

5.00 ft. Line Segments Define Each Trial Failure Surface.

Restrictions Have Been Imposed Upon The Angle Of Initiation.
The Angle Has Been Restricted Between The Angles Of -45.0
And -20.0 deg.

1

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Examined. They Are Ordered - Most Critical
First.

* * Safety Factors Are Calculated By The Modified Bishop Method * *

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	16.58	74.67
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3	26.18	71.96
4	31.17	71.70
5	36.15	72.19
6	40.99	73.42
7	45.60	75.35
8	49.88	77.95
9	53.72	81.15
10	57.03	84.89
11	59.76	89.08
12	61.81	93.58

Circle Center At X = 30.4 ; Y = 105.2 and Radius, 33.5

*** 1.711 ***

Individual data on the 25 slices

Slice	Width	Weight	Water Force Top	Water Force Bot	Tie Force Norm	Tie Force Tan	Earthquake Force Hor	Earthquake Force Ver	Surcharge Load

APPENDIX 11-E

Interface Friction Testing

LIST OF TABLES

LIST OF FIGURES

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(continued)

LIST OF FIGURES (continued)

APPENDIX 11-E

1.0 INTRODUCTION

A laboratory testing program was completed to evaluate frictional interface resistance between several possible cover soil sources and geosynthetics representative of those which will be used during construction of the permeable and impermeable covers.

In order to select the critical interfaces to be tested, a variety of soil/geosynthetic and geosynthetic/geosynthetic interfaces were evaluated based upon a review of the published literature (Koutsourais, M.M., Sprague, C.J. and Pucetas, R.C., 1990; and Tensar, 1988). The following were determined to be the most critical:

1. Cover soil with geocomposite drainage layer or geotextile; and,
2. Geocomposite drainage layer with 60 mil textured HDPE.

Most of the on-Site material to be excavated and placed as compacted fill on cover slopes at the ISRT Site contains metals at or above Consent Decree Action Levels. The collection, shipping, and testing of this material would have been difficult owing to health and safety considerations. Based upon this limitation, it was decided to model the subgrade stiffness using clean borrow sources exclusively. Two soils were tested and compaction of the base layer soil in the direct shear tests varied to model the stiffness of in-situ subgrades and compacted fills. In-situ subgrade was simulated by placing and lightly compacting the soil to an approximate relative density of 60 percent. Compacted subgrades were simulated by compacting the soil to approximately 80 percent relative density.

2.0 MATERIALS TESTED

A number of possible borrow sources for compacted fill and cover material were evaluated in the PDI Cap Material Sources Report (Golder Associates, 1990). Of the material sources within reasonable proximity to the Site, the following two soil materials were identified as being suitable for the test program:

Soil A - Hubbardston Sand; and,
Soil B - Quinn Perkins Concrete Sand.

Both materials meet soil retention criteria for typical geotextiles or geocomposites such as may be used in construction, and exhibit grading characteristics and minimal fines contents to prevent geotextile clogging.

Samples of the two materials were collected at their sources by Golder Associates staff during the week of July 8, 1991. The samples were returned to the Golder Associates Mount Laurel office for classification and compaction testing and sent to the Golder Associates Calgary laboratory for use in the direct shear tests. Grain size distribution (ASTM D 422) and compaction tests (ASTM D 698) were completed on each soil. Results of these tests are presented in Figures 11-E1 through 11-E4.

Sufficient samples of geosynthetics were requested from the manufacturers and sent directly to the Calgary laboratory. The samples included:

1. Geotextile, 16 ounce/square yard nonwoven, Mirafi 160N, Mirafi Inc.
2. Geocomposite, 10 ounce/square yard nonwoven geotextile bonded both sides, Tex-Net TN3002CN, Fluid Systems Inc.

3. Geomembrane, Gundline HDT, 60 mil textured, Gundle Lining Systems.

The following seven test series, each consisting of three individual tests at varying normal pressures, were completed to evaluate interface friction characteristics:

1. Hubbardston, loosely placed on geotextile over loosely placed Hubbardston;

2. Quinn Perkins, loosely placed on geotextile over loosely placed Hubbardston;

3. Hubbardston, loosely placed on geotextile over compacted Hubbardston;

4. Quinn Perkins, loosely placed on geocomposite over compacted Hubbardston;

5. Quinn Perkins, loosely placed on geocomposite over loosely placed Hubbardston;

6. Geocomposite over 60 mil textured HDPE, Quinn Perkins, loosely placed above and below the geosynthetic interface; and,

7. Hubbardston loosely placed without geosynthetics. (Soil direct shear test.)

3.0 TESTS DESCRIPTION

The tests were conducted in a direct shear test apparatus on 16 inch by 11 inch specimens.

For the soil/geosynthetic tests, the cover soil was placed in the upper box and tested against the geosynthetic on top of the soil modelling the subgrade in the lower box. The geosynthetic was not fixed. For the geosynthetic/geosynthetic tests, the interface was placed between the upper box filled with cover soil, and the lower box filled with soil modelling the subgrade. Neither geosynthetic was fixed to the testing apparatus.

All tests were run under saturated conditions representing a worst case scenario. Each series included tests at differing normal pressures to define the failure envelope. In situ stresses acting upon the interfaces will be low, typically 150 psf or less. Therefore, the normal pressures used in the testing program ranged from 105 to 420 psf (0.73 to 2.9 psi) to bracket field stresses.

4.0 RESULTS

Results of the testing program are summarized on Table 11-E1. Stress-strain data and failure envelope plots are presented in Figures 11-E5 through 11-E32. As the data indicates, the peak friction angles measured between the cover soil and the geocomposite, or the geocomposite and textured HDPE range between 32 and 34 degrees; and the residual friction angles range between 30 and 33 degrees.

REFERENCES

Golder Associates Inc., 1990. Pre-Design Investigation Task S-3, Identify Sources of Cap Materials, Interim Final Report, Industri-Plex Site, Woburn, Massachusetts, September.

Koutsourais, M.M., Sprague, C.J. and Pucetas, R.C., 1990. Interfacial Friction Study of Cap and Liner Components for Landfill Design, The Fourth Geosynthetic Research Institute Seminar Proceedings, p. 149-169.

Tensar Technical Note: WM2, Draft, 1988. Tensar Geogrid Soil Reinforcement of Soil Veneer Covers.

TABLE 11-E1

RESULTS OF GEOSYNTHETICS INTERFACE FRICTION TESTS

TEST NUMBER	SPECIMEN CONFIGURATION	PEAK FRICTION ANGLE (degree)	RESIDUAL FRICTION ANGLE (degree)
1	Hubbardston Geotextile Loose Subgrade[1]	33°	31°
2	Quinn Perkins Geotextile Loose Subgrade	32°	30°
3	Hubbardston Geotextile Dense Subgrade	32°	31°
4	Quinn Perkins Geocomposite Dense Subgrade	33°	31°
5	Quinn Perkins Geocomposite Loose Subgrade	33°	33°
6	Geocomposite[2] Textured HDPE	33°	33°
7	Loose[3] Hubbardston	34°	32°

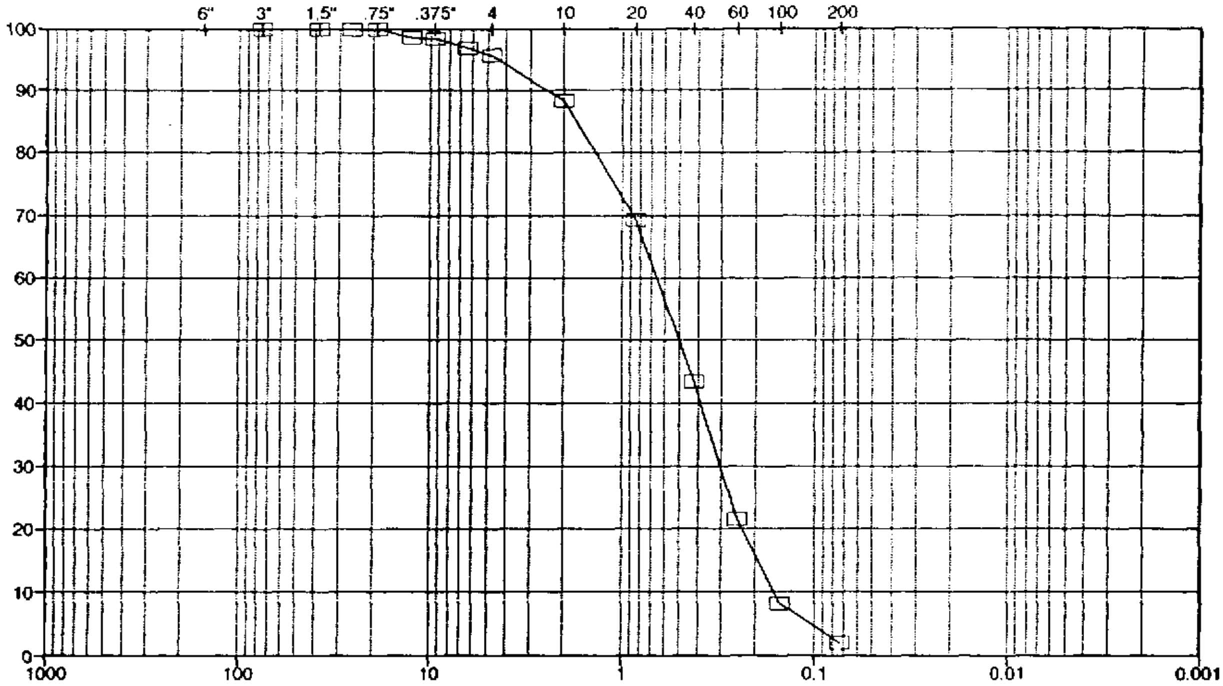
NOTES:

1 Subgrade composed of Hubbardston Sand.
2 Quinn Perkins loosely placed above and below geosynthetic interface. Failure at soil-geocomposite interface.
3 No geosynthetics, soil direct shear test. Peak strength envelope is non-linear. Possibly due to partial saturation of samples at 0.73 and 1.45 psi normal stresses.

Geotextile	16 ounce/sq.yd. nonwoven, Mirafi 160N
Geocomposite	10 ounce/sq.yd. nonwoven, bonded both sides TEX-NET TN3002CN
Geomembrane	Gundline, 60 mil textured HDPE

PARTICLE SIZE DISTRIBUTION ASTM D-421 AND 422

US STANDARD SIEVE OPENING SIZES



USCS

	Coarse	Fine	C	Med	Fine	
COBBLES	GRAVEL		SAND			FINES (Silt or Clay)

TECH: DL
DATE: 7/18/91
CHECKED: RMW
REVIEWED: RJC

SAMPLE ID	W%	LL	PL	PI	Other	DESCRIPTION
HUBBARDSTON SAND	1.69					Dark yellowish orange m-f SAND, trace f gravel trace fines
Sample Type: BULK	Date Tested: 7/17/91	USCS:				

JOB No.: 903-6400	SCALE: N/A	PARTICLE SIZE DISTRIBUTION HUBBARDSTON SAND	
DRAWN: RDT	DATE: 08/06/91		
CHECKED: RCD	DWG. No.: MA01-801		
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST	FIGURE 11-E1

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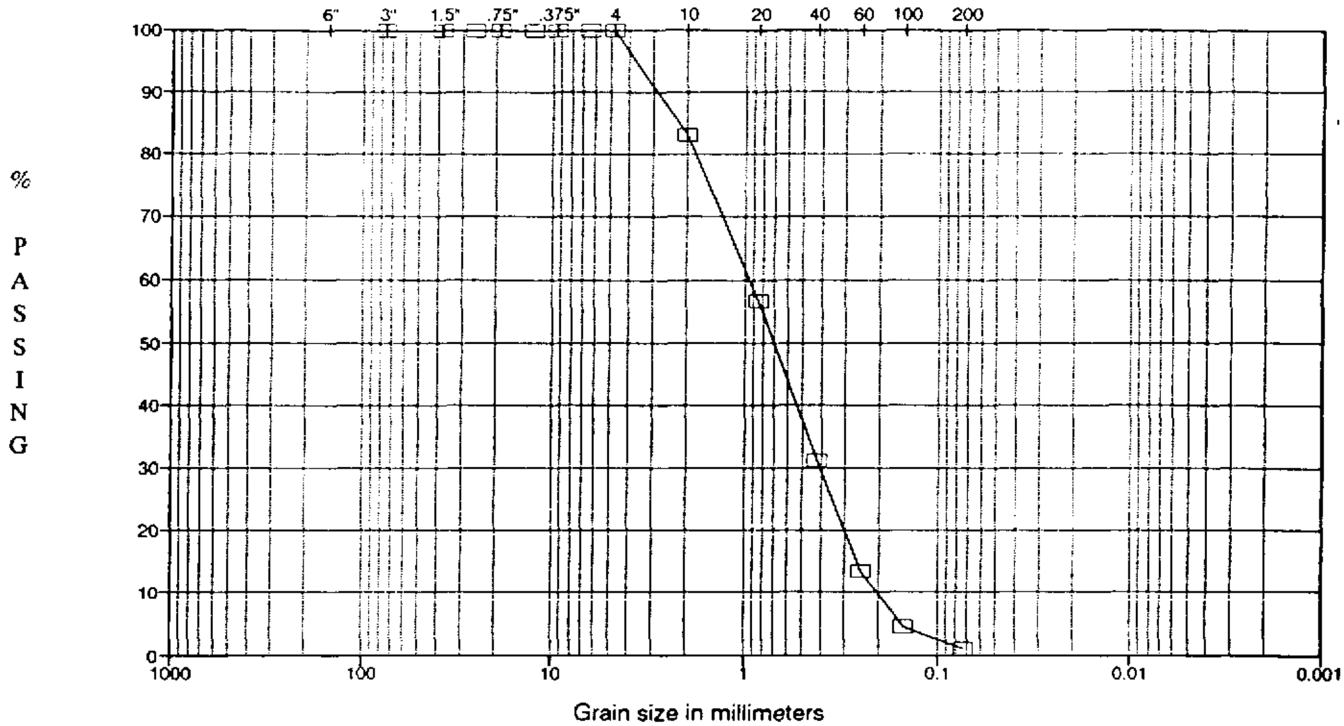
PARTICLE SIZE DISTRIBUTION ASTM D-421 AND 422

US STANDARD SIEVE OPENING SIZES



USCS

COBBLES	GRAVEL		SAND			FINES (Silt or Clay)
	Coarse	Fine	C	Med	Fine	

TECH: DL
DATE: 7/18/91
CHECKED: RMW
REVIEWED: RJF

SAMPLE ID	W%	LL	PL	PI	Other	DESCRIPTION
QUINN PERKINS	2.09					Greyish orange c-f SAND, trace fines trace f gravel

Sample Type: BULK Date Tested: 7/17/91 USCS:

JOB No.: 903-6400	SCALE: N/A	PARTICLE SIZE DISTRIBUTION QUINN PERKINS	
DRAWN: RDT	DATE: 08/06/91		
CHECKED: PCR	DWG. No.: MA01-802		
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST	FIGURE 11-E2

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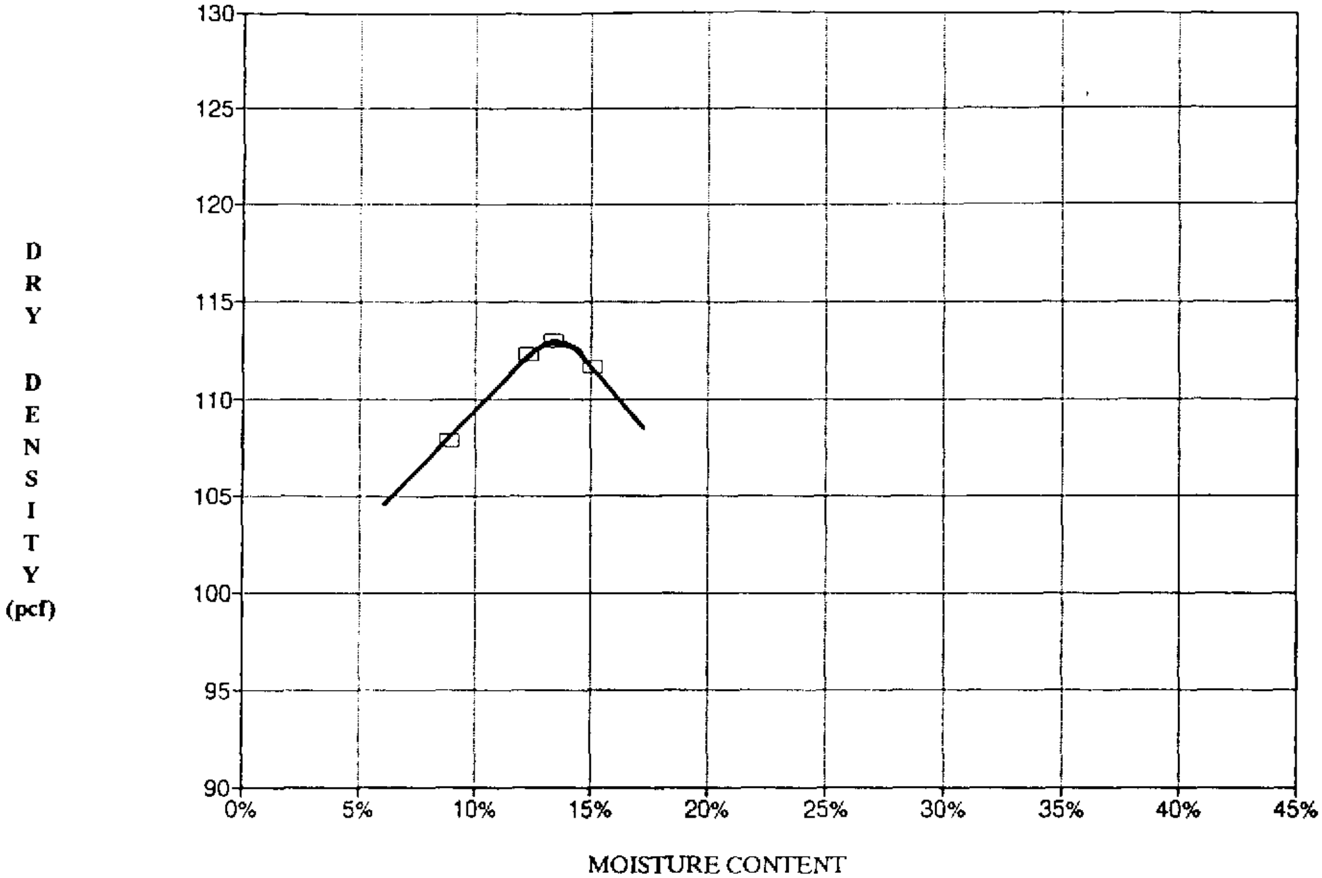

SAMPLE IDENTITY	Wn%	LL	PL	PI	DESCRIPTION
HUBBARDSTON SAND	1.7				Dark yellowish orange m-f SAND, trace f gravel trace fines
	MAXIMUM DRY DENSITY (pcf)			113.0	
	OPTIMUM MOISTURE (%)			13.4	
SAMPLE TYPE:	BULK		DATE TESTED:		7/18/91

LAB TECH: DL
DATE: 7/24/91
CHECKED: RDW
REVIEWED: RJD

JOB No.: 903-6400	SCALE: N/A	COMPACTION CHARACTERISTICS HUBBARDSTON SAND	
DRAWN: RDT	DATE: 08/06/91		
CHECKED: PCR	DWG. No.: MA01-803		
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST	FIGURE 11-E3

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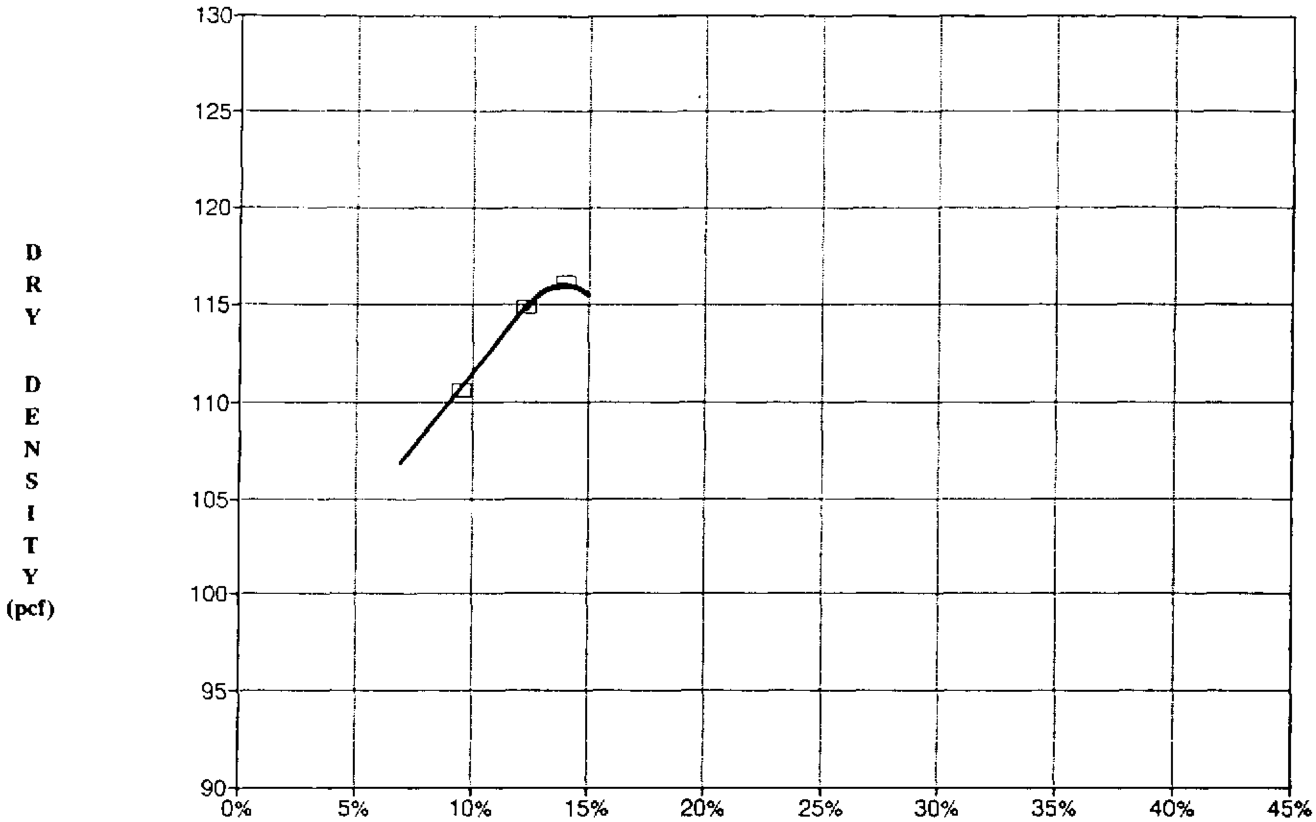
MOISTURE/DRY DENSITY CURVE
ASTM D-698



SAMPLE IDENTITY	Wn%	LL	PL	PI	DESCRIPTION
QUINN PERKINS	2.1				Greyish orange c-f SAND, trace fines trace f gravel
	MAXIMUM DRY DENSITY (pcf)	116.0			
	OPTIMUM MOISTURE (%)	14.8			
SAMPLE TYPE:	BULK		DATE TESTED:	7/18/91	

LAB TECH: DL
DATE: 7/24/91
CHECKED: [signature]
REVIEWED: RJD

JOB No.: 903-6400	SCALE: N/A	COMPACTION CHARACTERISTICS QUINN PERKINS
DRAWN: RDT	DATE: 08/06/91	
CHECKED: RCR	DWG. No.: MA01-804	
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST — FIGURE 11-E4

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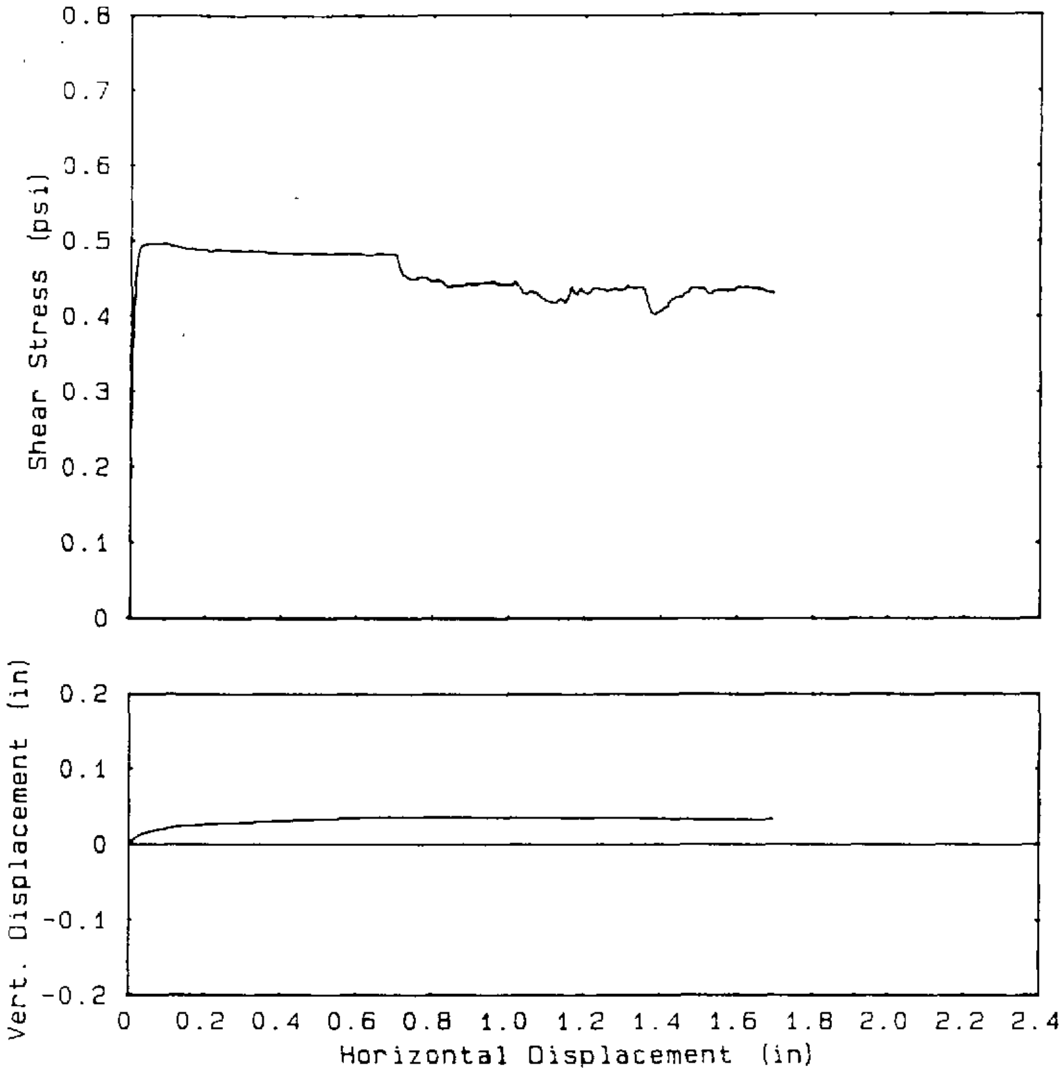
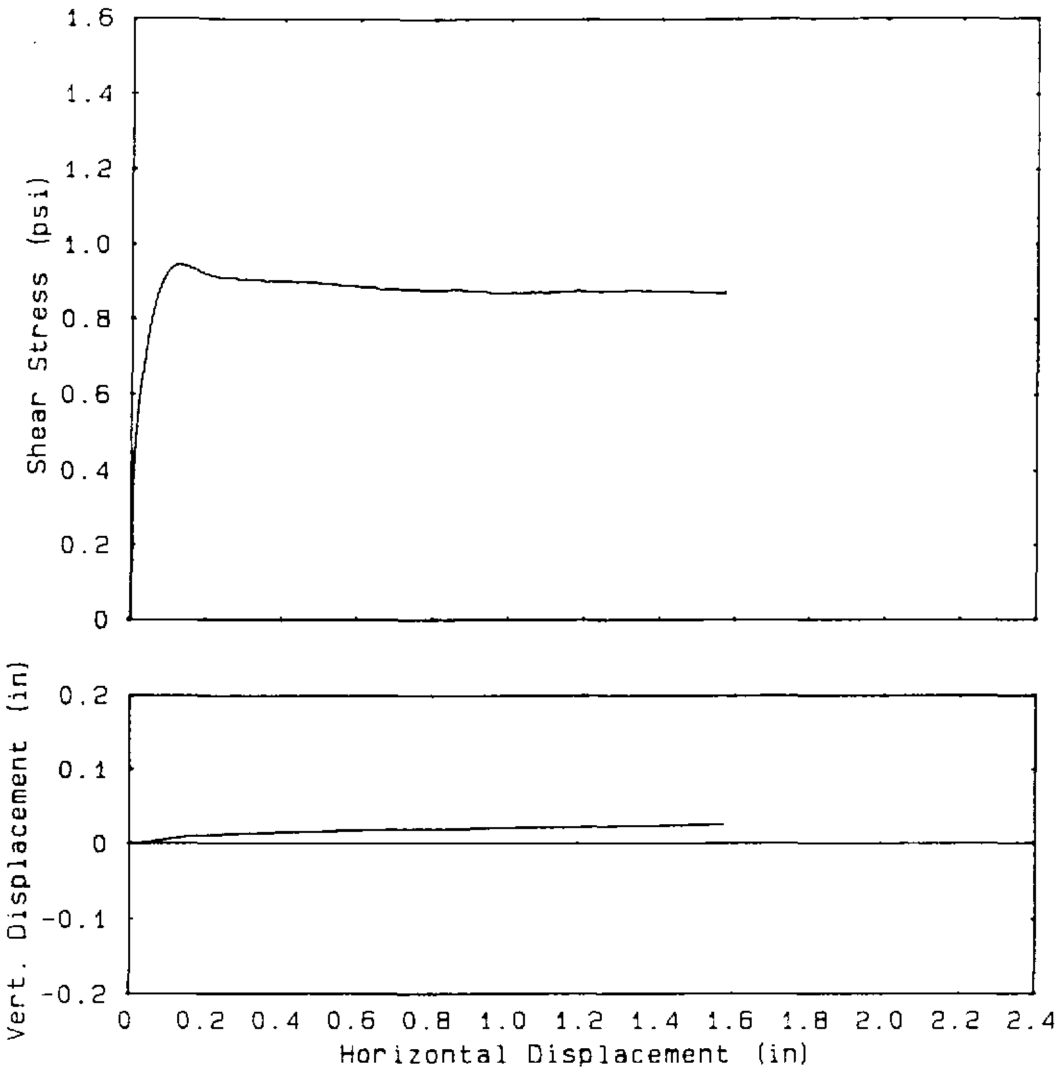


Large Direct Shear Test

Test Number	:	T1-05s
Test Date	:	25 July 1991
Shear Disp. Rate	:	2.47 in/hr
Sample Area	:	193.4 sq in
Sample Height	:	14.33 in
Normal Stress	:	.73 psi
Dry Density	:	103 pcf
Moisture Content	:	10 %
Sample Description	:	Loose Hubbardson Sand over
	:	Geotextile over Loose
Test Description	:	Hubbardson Sand
	:	Submerged.

No.: 903-6400	SCALE: N/A	INTERFACE FRICTION TESTING DIRECT SHEAR TEST T1-05s
DRAWN: RDT	DATE: 08/06/91	
CHECKED: [signature]	DWG. No.: MA01-805	
Golder Associates	INDUSTRI-PLEX SITE REMEDIAL TRUST	FIGURE 11-E5



Large Direct Shear Test

Test Number	:	T1-10s
Test Date	:	25 July 1991
Shear Disp. Rate	:	2.4 in/hr
Sample Area	:	193.4 sq in
Sample Height	:	11.81 in
Normal Stress	:	1.45 psi
Dry Density	:	103 pcf
Moisture Content	:	10 %
Sample Description	:	Loose Hubbardson Sand over
	:	Geotextile over Loose
Test Description	:	Hubbardson Sand
	:	Submerged.

JOB No.: 903-6400	SCALE: N/A	**INTERFACE FRICTION TESTING DIRECT SHEAR TEST T1-10s**	
DRAWN: RDT	DATE: 08/06/91		
CHECKED:	DWG. No.: MA01-806		
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST	FIGURE **11-E6**

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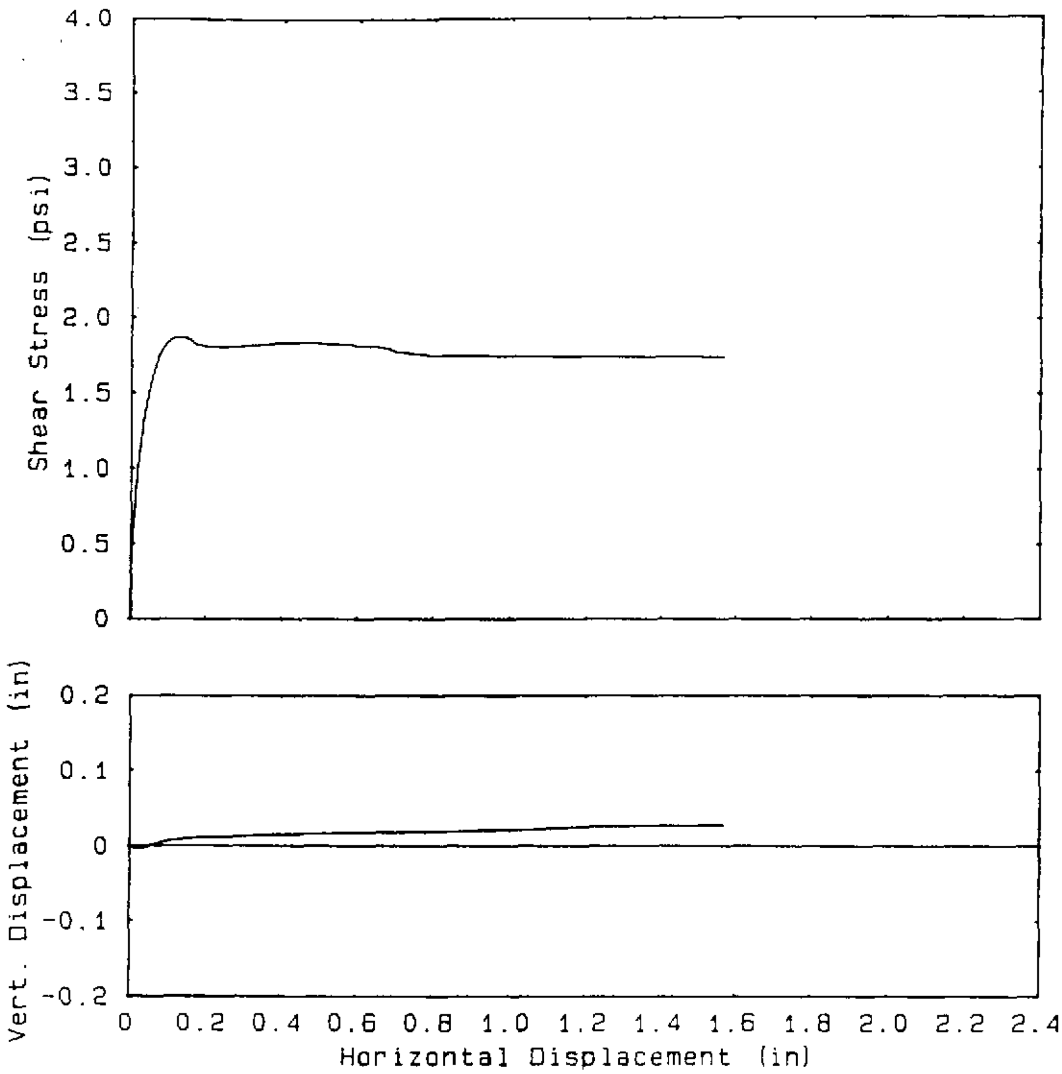

Large Direct Shear Test

Test Number	:	T1-20s
Test Date	:	26 July 1991
Shear Disp. Rate	:	2.42 in/hr
Sample Area	:	193.4 sq in
Sample Height	:	12.72 in
Normal Stress	:	2.9 psi
Dry Density	:	103 pcf
Moisture Content	:	10 %
Sample Description	:	Loose Hubbardson Sand over
	:	Geotextile over Loose
Test Description	:	Hubbardson Sand
	:	Submerged.

JOB No.: 903-6400	SCALE: N/A	INTERFACE FRICTION TESTING DIRECT SHEAR TEST T1-20s	
DRAWN: RDT	DATE: 08/06/91		
CHECKED:	DWG. No.: MA01-807		
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST	FIGURE 11-E7

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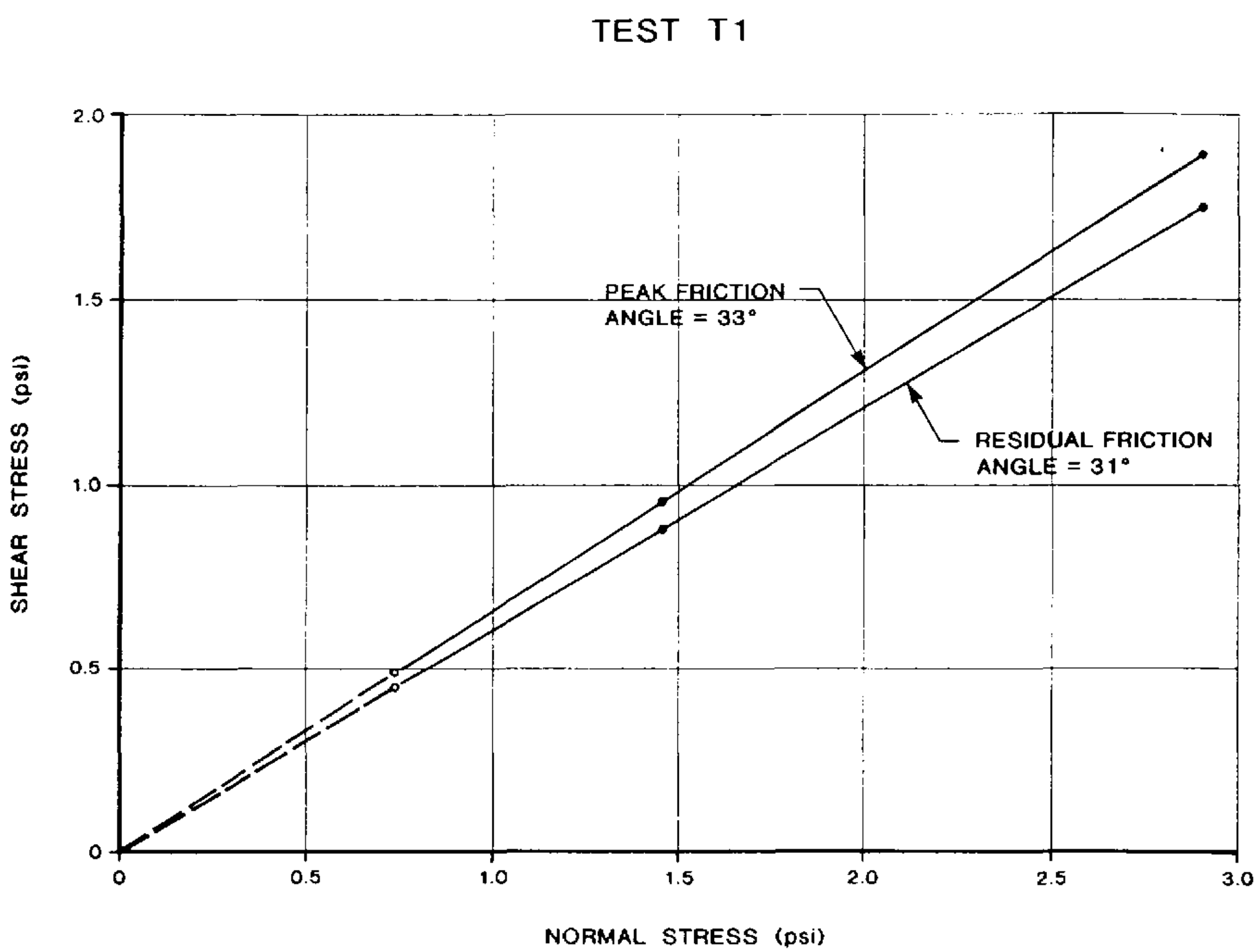
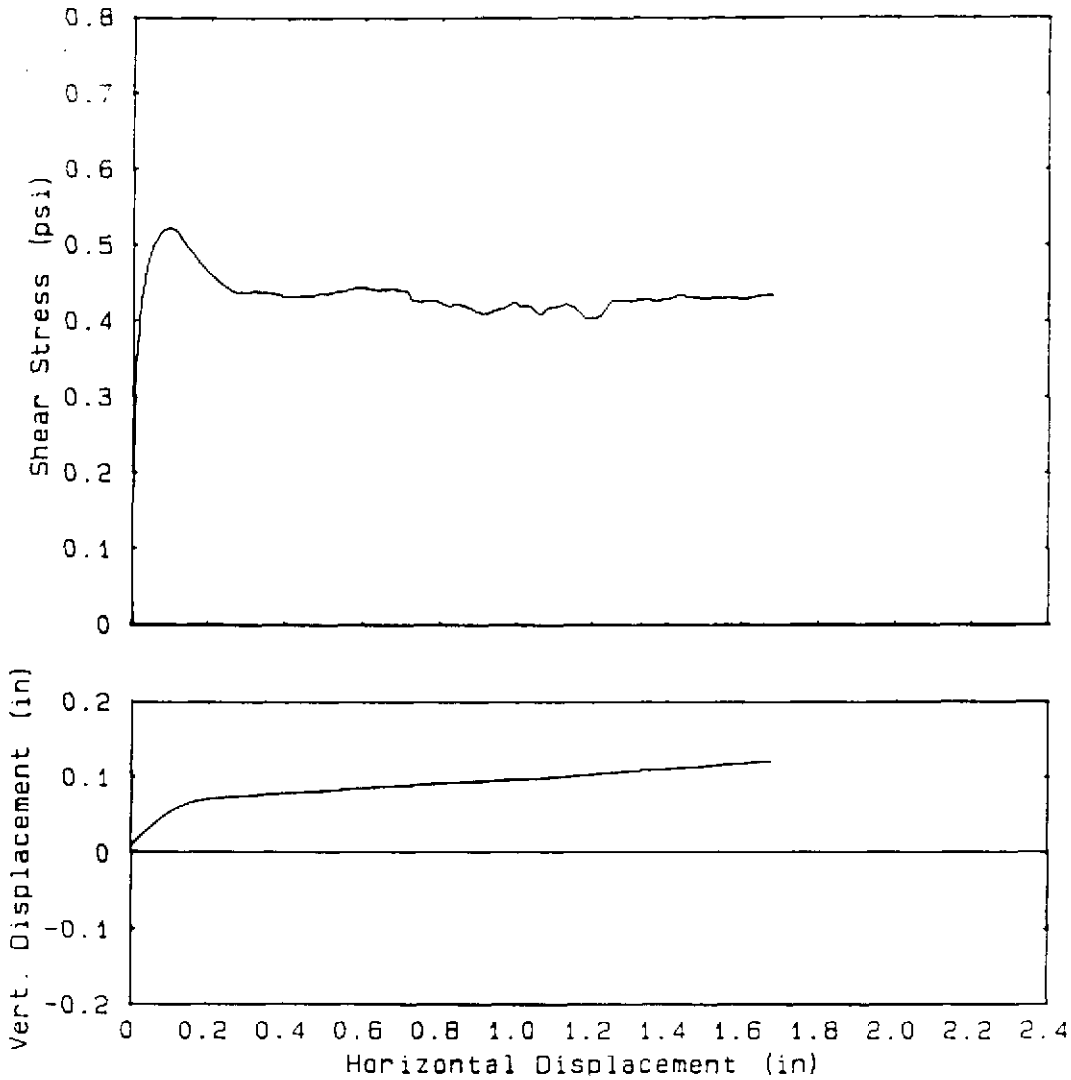


TESTS AT 0.73, 1.45 AND 2.90 psi
NORMAL STRESS

JOB No.: 903-6400	SCALE: N/A	**INTERFACE FRICTION ANGLES DIRECT SHEAR TEST T1**
DRAWN: RDT	DATE: 08/07/91	
CHECKED: PCR	DWG. No.: MA01-808	
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST — FIGURE **11-E8**

15696

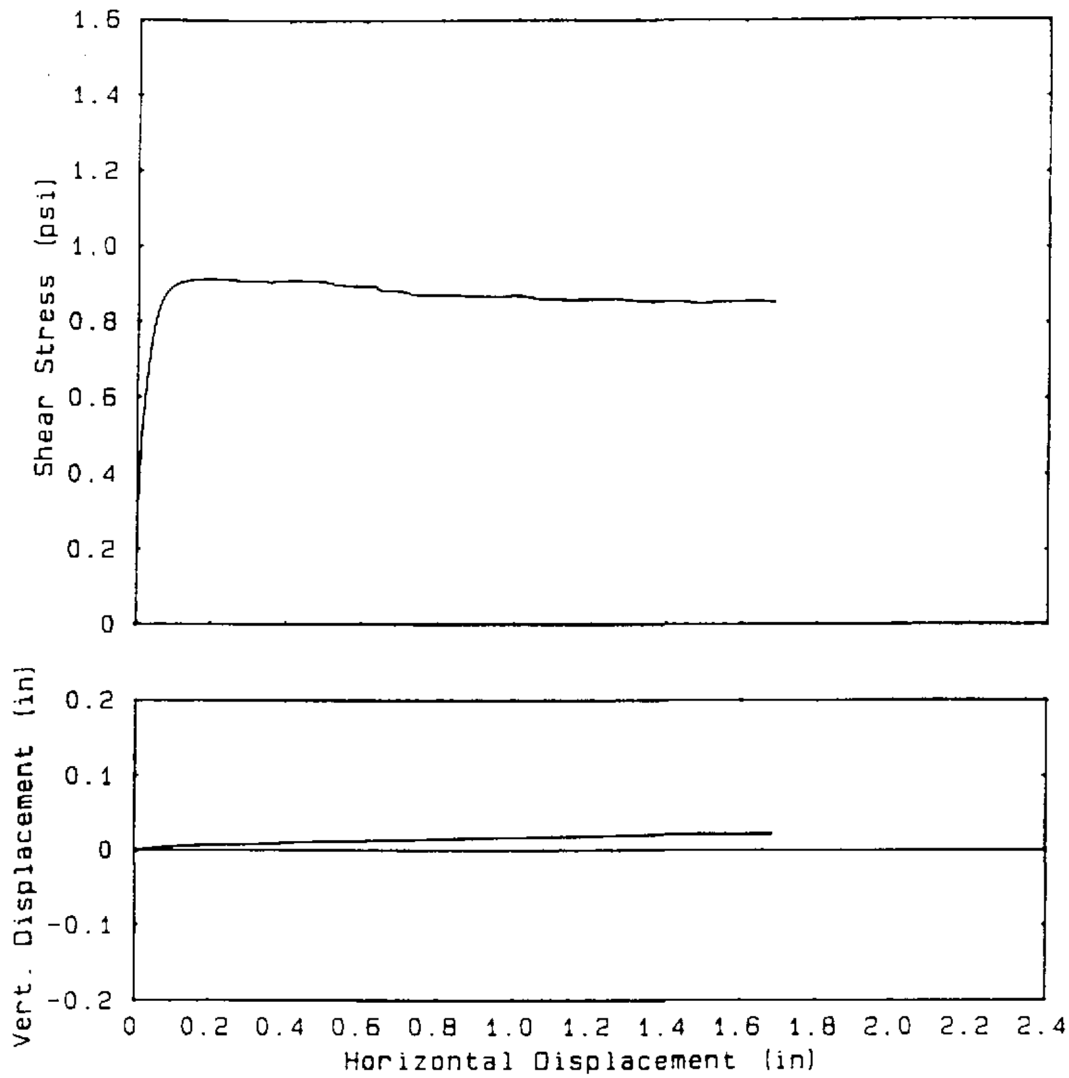


Large Direct Shear Test

Test Number	:	t2-05s
Test Date	:	26 July 1991
Shear Disp. Rate	:	2.37 in/hr
Sample Area	:	193.4 sq in
Sample Height	:	14.57 in
Normal Stress	:	.73 psi
Dry Density	:	103 pcf
Moisture Content	:	10 %
Sample Description	:	Loose Quinn Perkins Sand over
	:	Geotextile over Loose
Test Description	:	Hubbardson Sand
	:	Submerged.

JOB No.: 903-6400	SCALE: N/A	**INTERFACE FRICTION TESTING DIRECT SHEAR TEST T2-05s**
DRAWN: RDT	DATE: 08/06/91	
CHECKED: *[signature]*	DWG. No.: MA01-809	
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST — FIGURE **11-E9**

158696

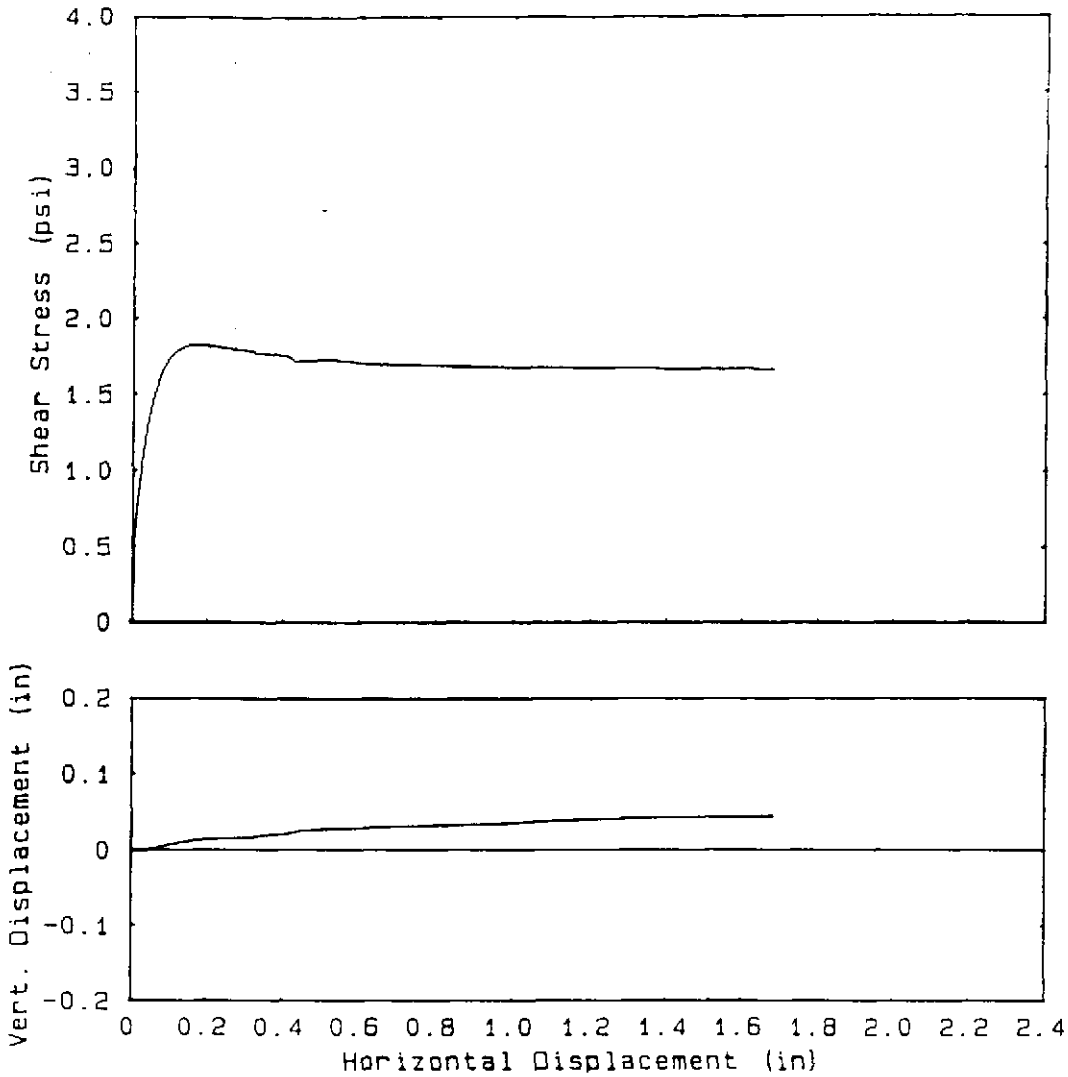


Large Direct Shear Test

Test Number	:	t2-10s
Test Date	:	26 July 1991
Shear Disp. Rate	:	2.33 in/hr
Sample Area	:	193.4 sq in
Sample Height	:	12.72 in
Normal Stress	:	1.45 psi
Dry Density	:	103 pcf
Moisture Content	:	10 %
Sample Description	:	Loose Quinn Perkins Sand over
	:	Geotextile over Loose
Test Description	:	Hubbardson Sand
	:	Submerged.

JOB No.: 903-6400	SCALE: N/A	INTERFACE FRICTION TESTING DIRECT SHEAR TEST T2-10s
DRAWN: RDT	DATE: 08/06/91	
CHECKED: *[signature]*	DWG. No.: MA01-810	
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST — FIGURE 11-E10

158896

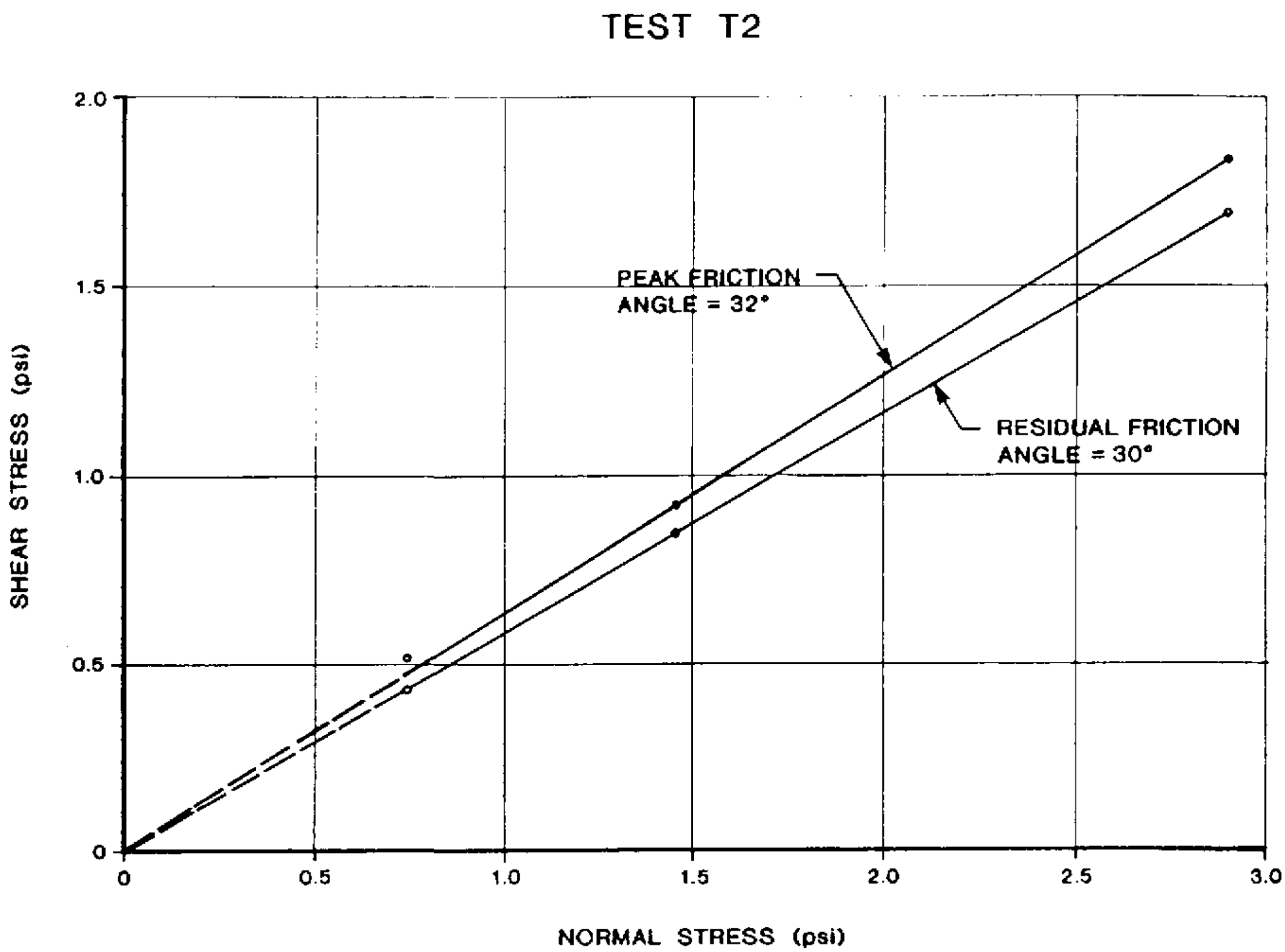


Large Direct Shear Test

Test Number	:	T2-20s
Test Date	:	27 July 1991
Shear Disp. Rate	:	2.41 in/hr
Sample Area	:	193.4 sq in
Sample Height	:	11.77 in
Normal Stress	:	2.9 psi
Dry Density	:	103 pcf
Moisture Content	:	10 %
Sample Description	:	Loose Quinn Perkins Sand over
	:	Geotextile over Loose
Test Description	:	Hubbardson Sand
	:	Submerged.

JOB No.: 903-6400	SCALE: N/A	**INTERFACE FRICTION TESTING DIRECT SHEAR TEST T2-20s**
DRAWN: RDT	DATE: 08/06/91	
CHECKED:	DWG. No.: MA01-811	
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST — FIGURE 11-E11

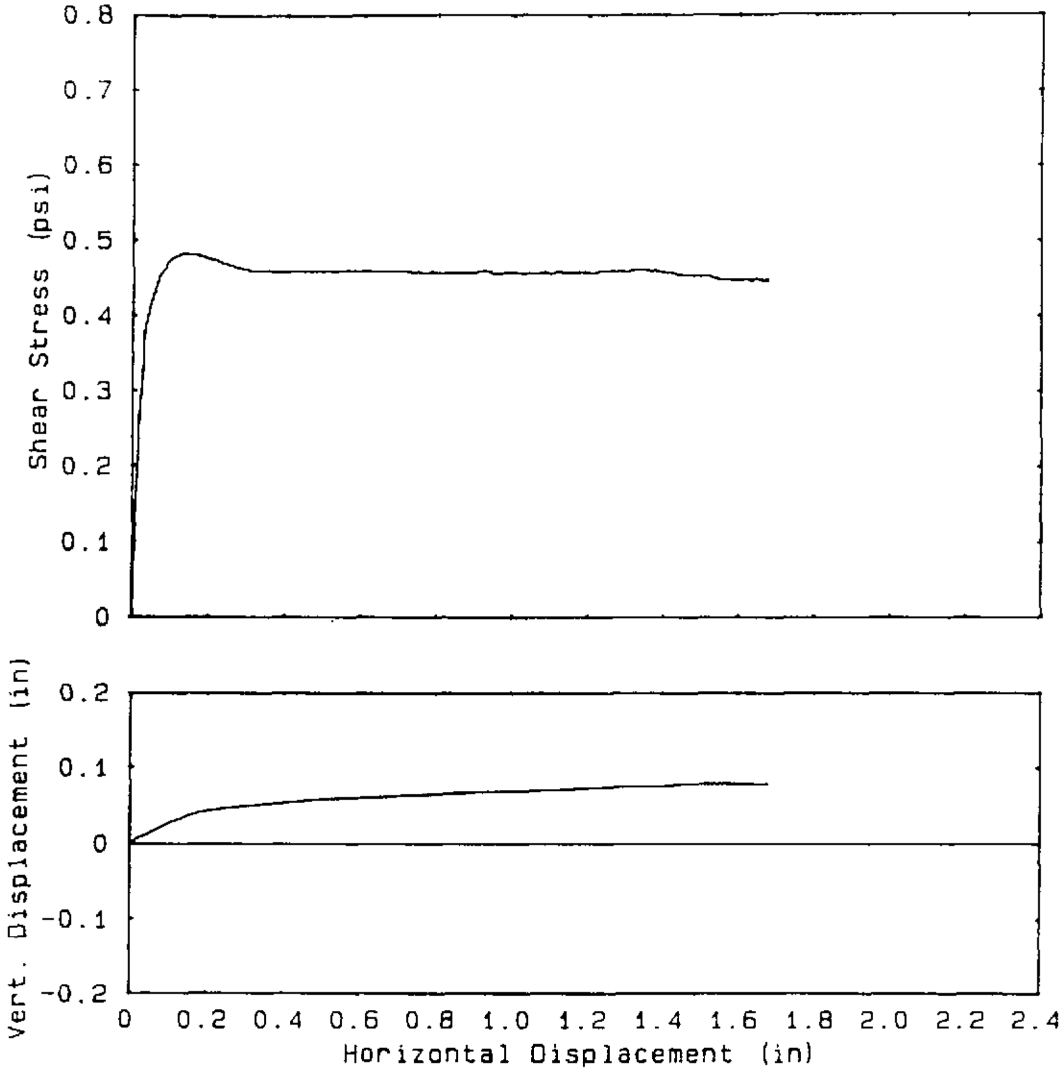
158696



TESTS AT 0.73, 1.45 AND 2.90 psi
NORMAL STRESS

JOB No.: 903-6400	SCALE: N/A	**INTERFACE FRICTION ANGLES DIRECT SHEAR TEST T2**	
DRAWN: RDT	DATE: 08/07/91		
CHECKED: RCR	DWG. No.: MA01-812		
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST	FIGURE **11-E12**

15896

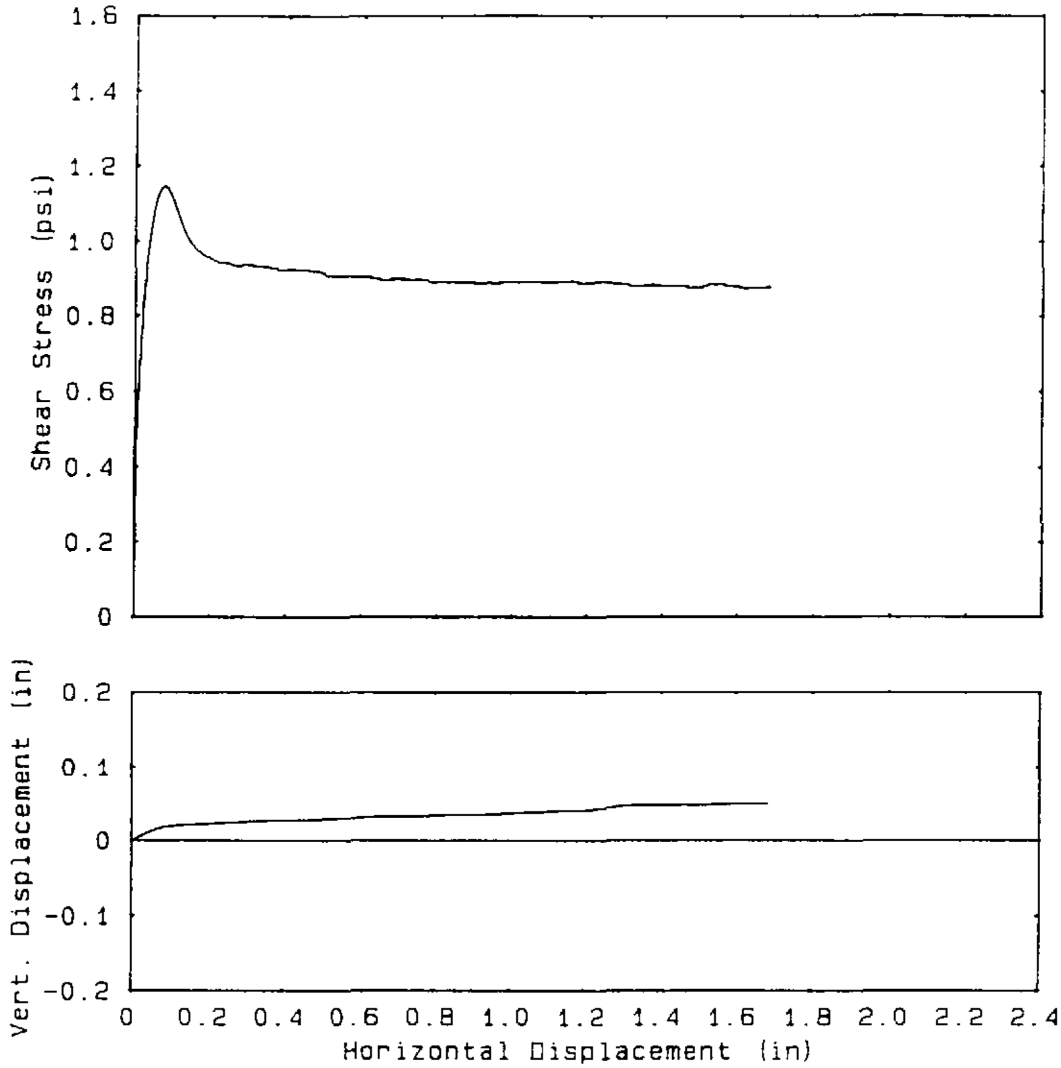


Large Direct Shear Test

Test Number	:	T3-05s
Test Date	:	27 July 1991
Shear Disp. Rate	:	2.38 in/hr
Sample Area	:	193.4 sq in
Sample Height	:	14.49 in
Normal Stress	:	.73 psi
Dry Density	:	103 pcf
Moisture Content	:	10 %
Sample Description	:	Loose Hubbardson Sand over
	:	Geotextile over Compacted
Test Description	:	Hubbardson Sand
	:	Submerged.

JOB No.: 903-6400	SCALE: N/A	**INTERFACE FRICTION TESTING DIRECT SHEAR TEST T3-05s**
DRAWN: RDT	DATE: 08/07/91	
CHECKED:	DWG. No.: MA01-813	
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST — FIGURE **11-E13**

15666

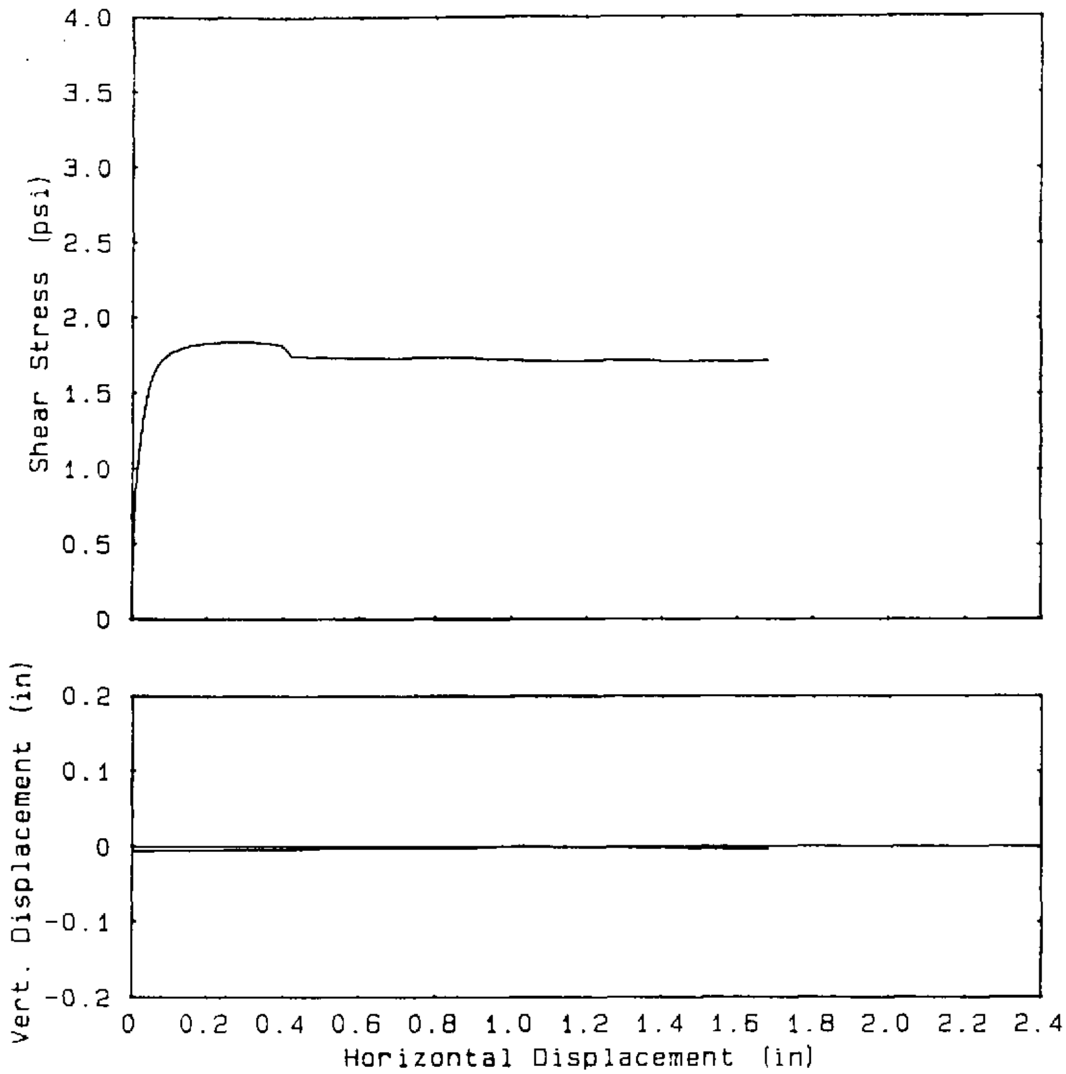


Large Direct Shear Test

Test Number	:	T3-10s
Test Date	:	27 July 1991
Shear Disp. Rate	:	2.21 in/hr
Sample Area	:	193.4 sq in
Sample Height	:	12.72 in
Normal Stress	:	1.45 psi
Dry Density	:	103 pcf
Moisture Content	:	10 %
Sample Description	:	Loose Hubbardson Sand over
	:	Geotextile over Compacted
Test Description	:	Hubbardson Sand
	:	Submerged.

JOB No.: 903-6400	SCALE: N/A	INTERFACE FRICTION TESTING DIRECT SHEAR TEST T3-10s
DRAWN: RDT	DATE: 08/07/91	
CHECKED: PCR	DWG. No.: MA01-814	
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST — FIGURE 11-E14

15696

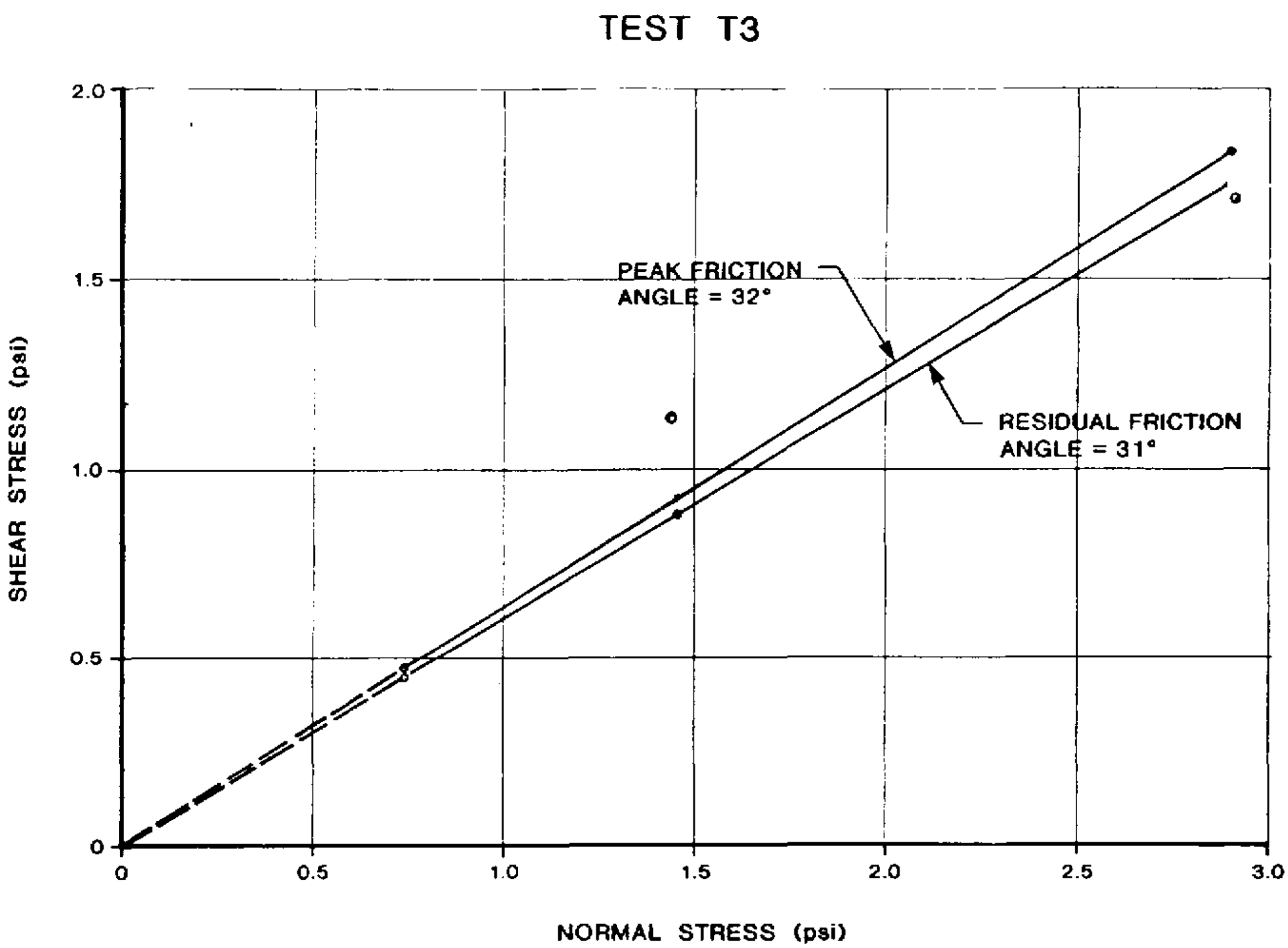


Large Direct Shear Test

Test Number	:	T3-20s
Test Date	:	29 July 1991
Shear Disp. Rate	:	2.52 in/hr
Sample Area	:	193.4 sq in
Sample Height	:	11.54 in
Normal Stress	:	2.9 psi
Dry Density	:	103 pcf
Moisture Content	:	10 %
Sample Description	:	Loose Hubbardson Sand over
	:	Geotextile over Compacted
Test Description	:	Hubbardson Sand
	:	Submerged.

JOB No.: 903-6400	SCALE: N/A	**INTERFACE FRICTION TESTING DIRECT SHEAR TEST T3-20s**
DRAWN: RDT	DATE: 08/07/91	
CHECKED:	DWG. No.: MA01-815	
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST — FIGURE 11-E15

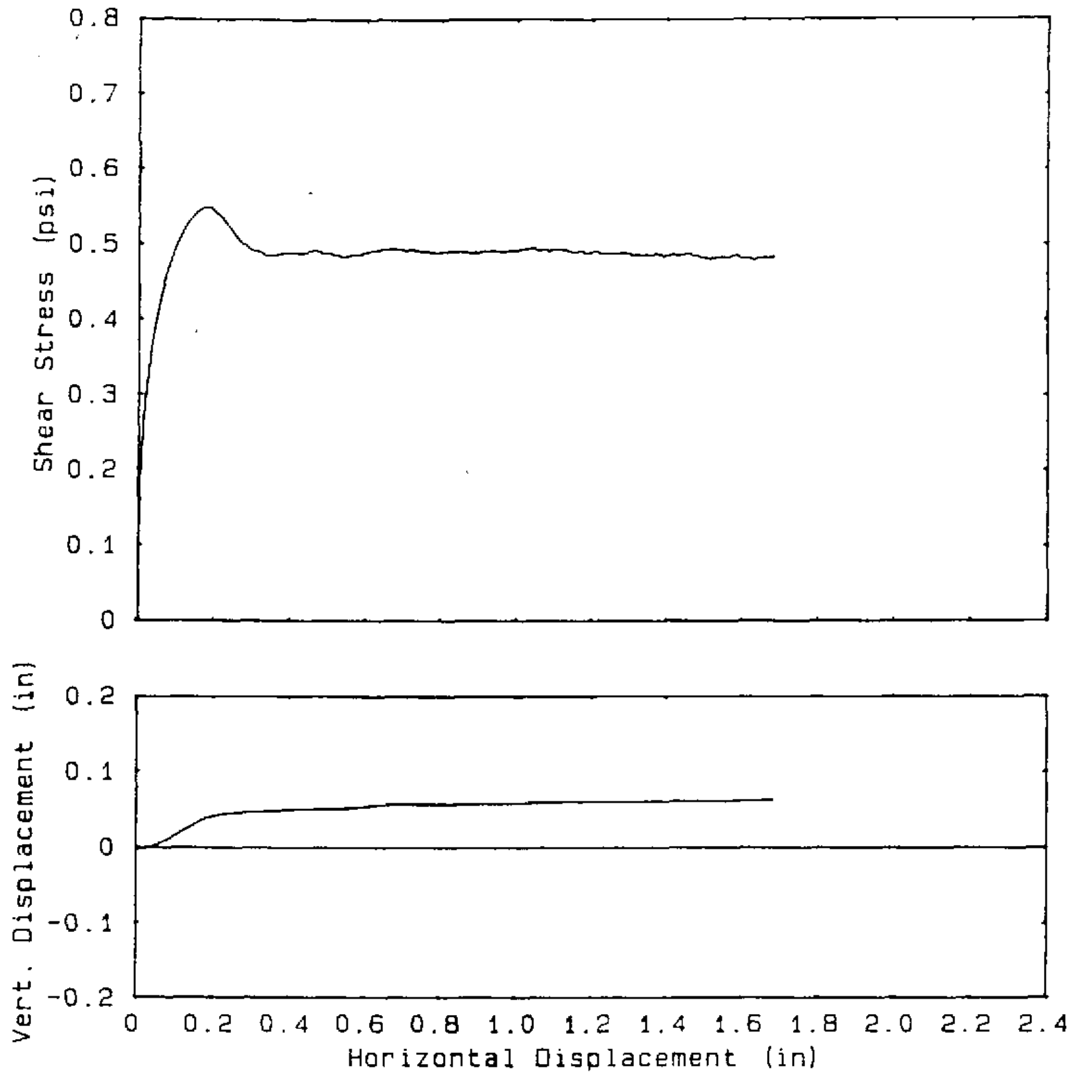
158696



TESTS AT 0.73, 1.45 AND 2.90 psi
NORMAL STRESS

JOB No.: 903-6400	SCALE: N/A	INTERFACE FRICTION ANGLES DIRECT SHEAR TEST T3
DRAWN: RDT	DATE: 08/07/91	
CHECKED: MR	DWG. No.: MA01-816	
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST — FIGURE 11-E16

158696

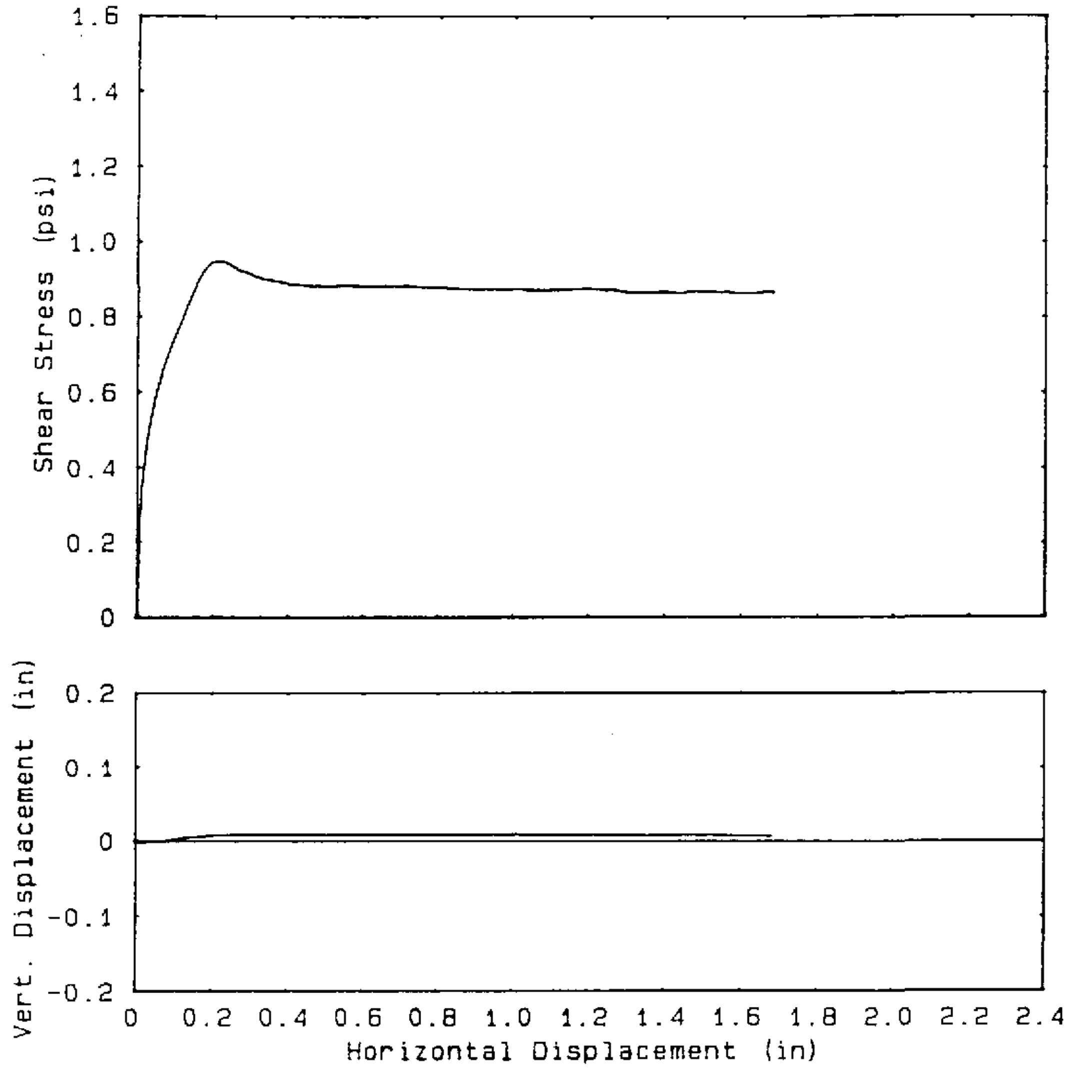


Large Direct Shear Test

Test Number	:	t4-05s
Test Date	:	29 July 1991
Shear Disp. Rate	:	2.56 in/hr
Sample Area	:	193.4 sq in
Sample Height	:	13.94 in
Normal Stress	:	.73 psi
Dry Density	:	108 pcf
Moisture Content	:	10 %
Sample Description	:	Loose Quinn Perkins Sand over
	:	Geocomposite over Compacted
Test Description	:	Hubbardson Sand
	:	Submerged.

JOB No.: 903-6400	SCALE: N/A	INTERFACE FRICTION TESTING DIRECT SHEAR TEST T4-05s
DRAWN: RDT	DATE: 08/07/91	
CHECKED: [signature]	DWG. No.: MA01-817	
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST — FIGURE 11-E17

158696

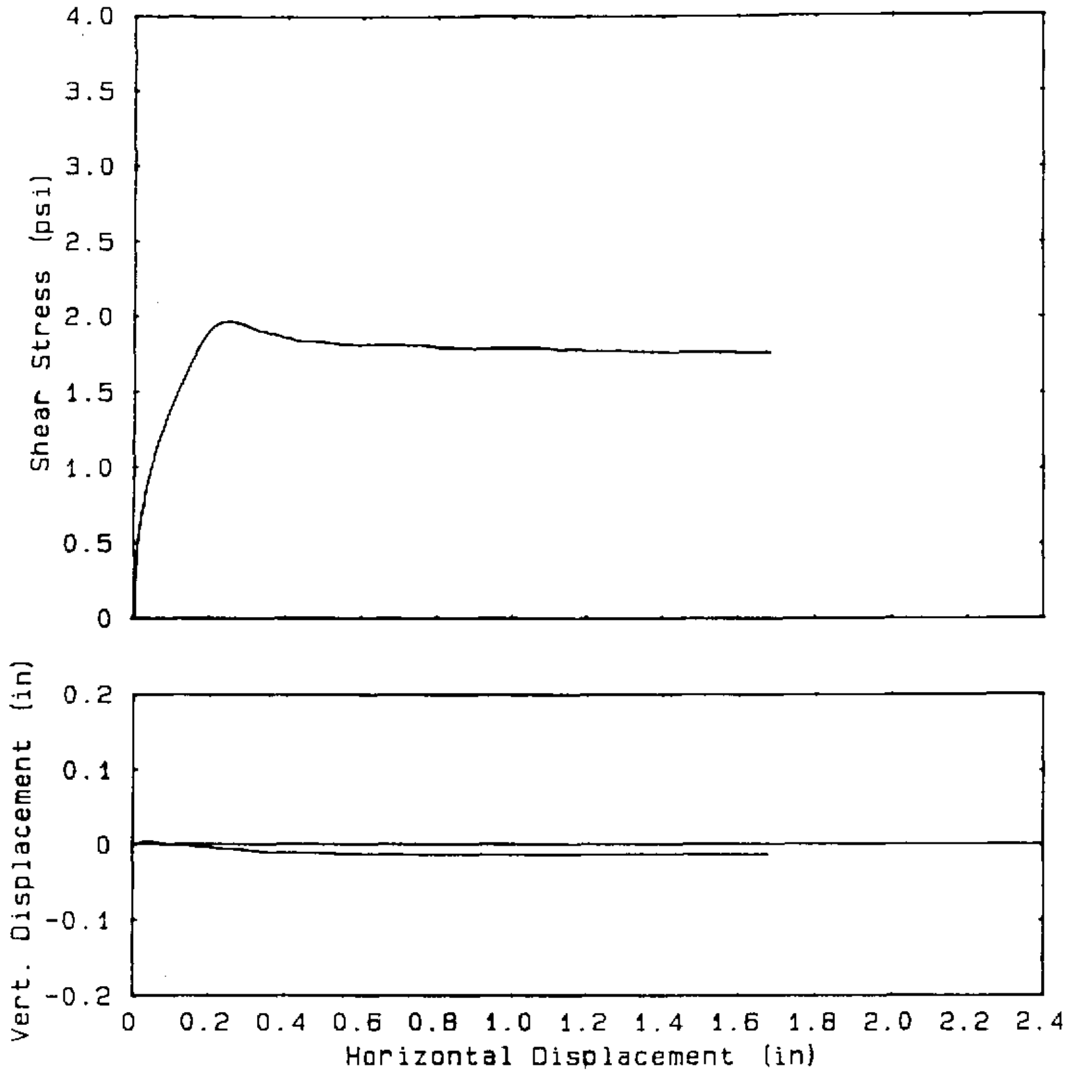


Large Direct Shear Test

Test Number	:	t4-10s
Test Date	:	29 July 1991
Shear Disp. Rate	:	2.48 in/hr
Sample Area	:	193.4 sq in
Sample Height	:	13.5 in
Normal Stress	:	1.45 psi
Dry Density	:	108 pcf
Moisture Content	:	10 %
Sample Description	:	Loose Quinn Perkins Sand over
	:	Geocomposite over Compacted
Test Description	:	Hubbardson Sand
	:	Submerged.

JOB No.: 903-6400	SCALE: N/A	INTERFACE FRICTION TESTING DIRECT SHEAR TEST T4-10s
DRAWN: RDT	DATE: 08/07/91	
CHECKED: MR	DWG. No.: MA01-818	
Golder Associates	INDUSTRI-PLEX SITE REMEDIAL TRUST	FIGURE 11-E18

158696



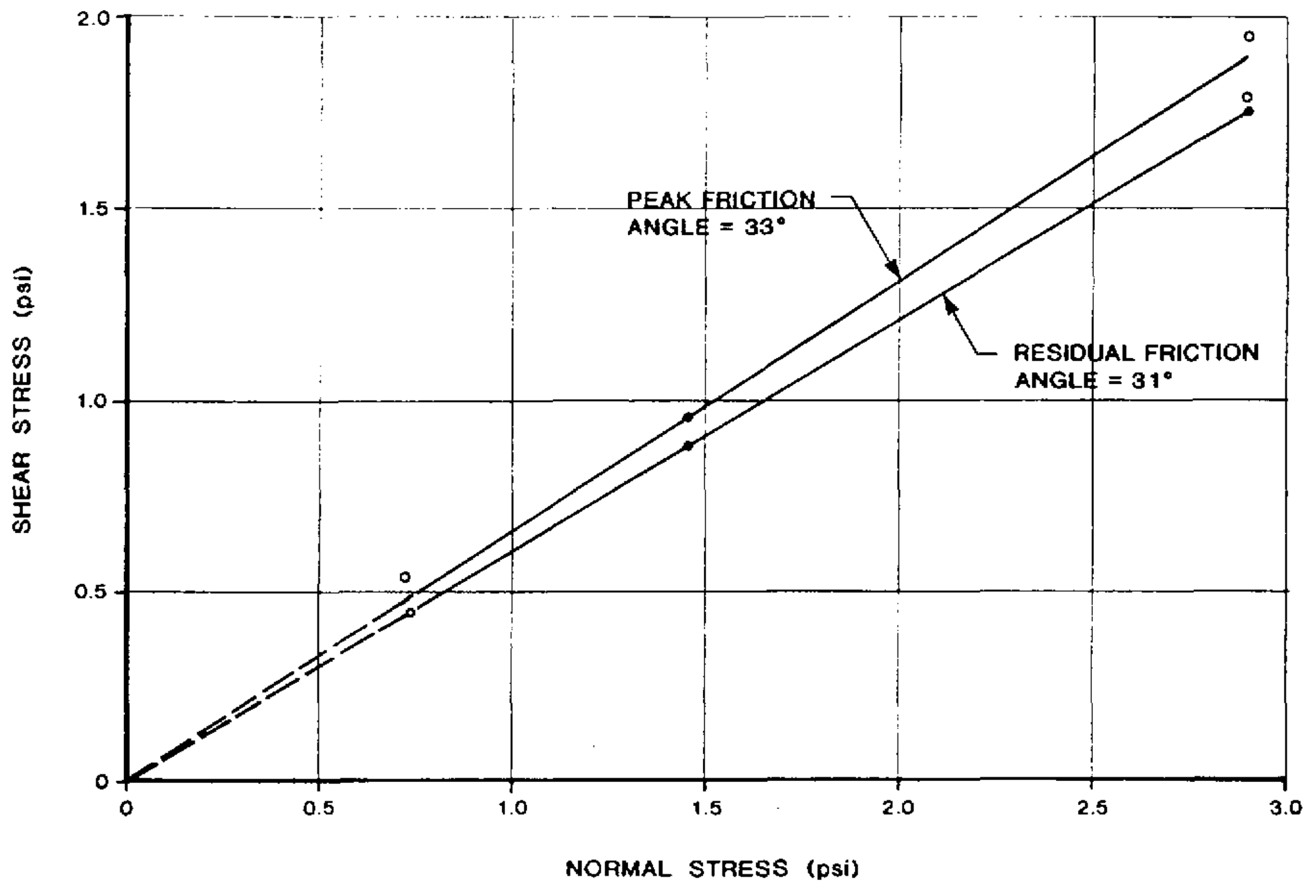
Large Direct Shear Test

Test Number	:	t4-20s
Test Date	:	30 July 1991
Shear Disp. Rate	:	2.39 in/hr
Sample Area	:	193.4 sq in
Sample Height	:	13.5 in
Normal Stress	:	2.9 psi
Dry Density	:	108 pcf
Moisture Content	:	10 %
Sample Description	:	Loose Quinn Perkins Sand over
	:	Geocomposite over Compacted
Test Description	:	Hubbardson Sand
	:	Submerged.

JOB No.: 903-6400	SCALE: N/A	INTERFACE FRICTION TESTING DIRECT SHEAR TEST T4-20s
DRAWN: RDT	DATE: 08/07/91	
CHECKED:	DWG. No.: MA01-819	
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST — FIGURE 11-E19

158698

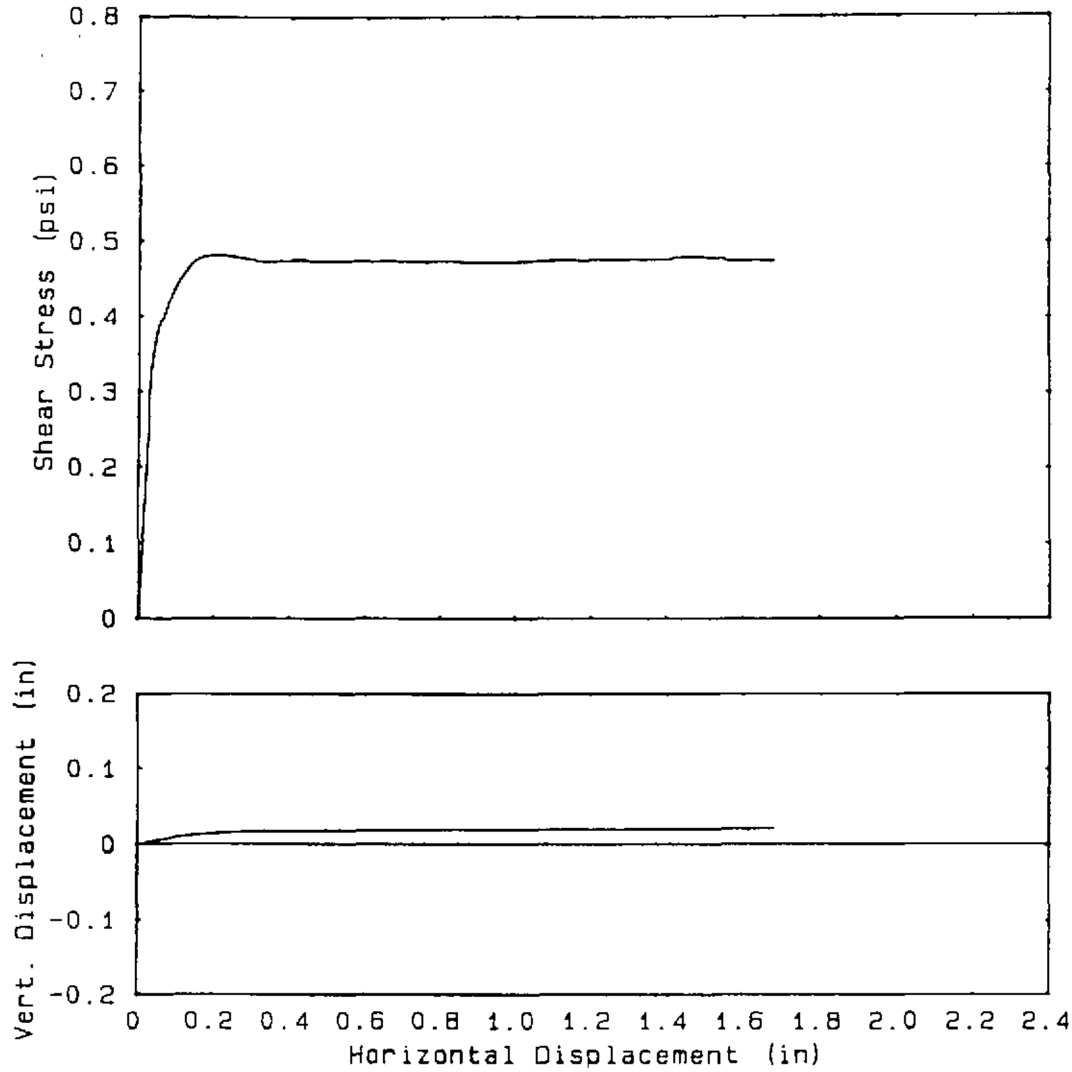
TEST T4



TESTS AT 0.73, 1.45 AND 2.90 psi NORMAL STRESS

JOB No.: 903-6400	SCALE: N/A	**INTERFACE FRICTION ANGLES DIRECT SHEAR TEST T4**
DRAWN: RDT	DATE: 08/07/91	
CHECKED: [signature]	DWG. No.: MA01-820	
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST — FIGURE 11-E20

15899

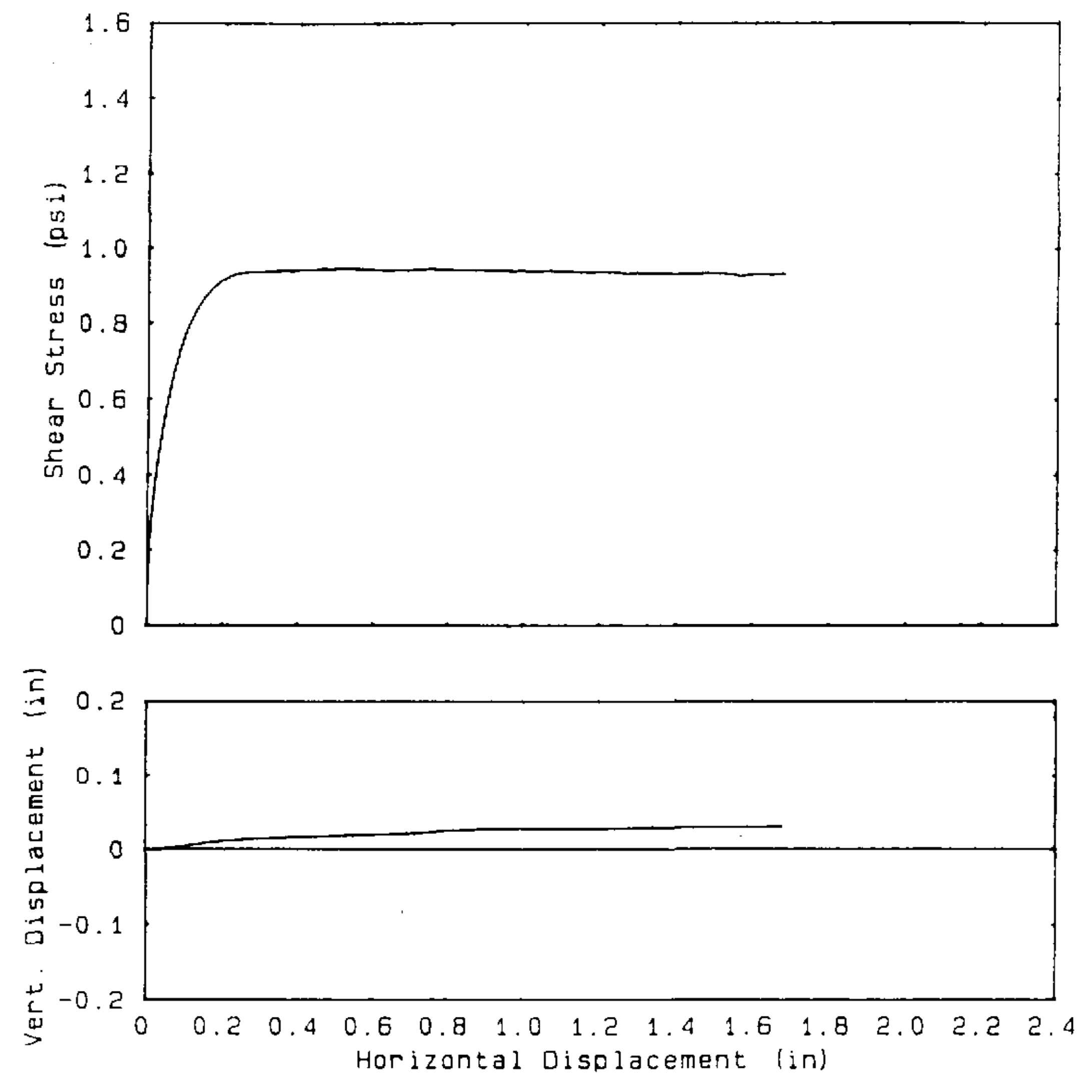


Large Direct Shear Test

Test Number	:	T5-05s
Test Date	:	30 July 1991
Shear Disp. Rate	:	2.42 in/hr
Sample Area	:	193.4 sq in
Sample Height	:	13.5 in
Normal Stress	:	.73 psi
Dry Density	:	108 pcf
Moisture Content	:	10 %
Sample Description	:	Loose Quinn Perkins Sand over
	:	Geocomposite over Loose
Test Description	:	Hubbardson Sand
	:	Submerged.

JOB No.: 903-6400	SCALE: N/A	**INTERFACE FRICTION TESTING DIRECT SHEAR TEST T5-05s**
DRAWN: RDT	DATE: 08/07/91	
CHECKED:	DWG. No.: MA01-821	
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST — FIGURE **11-E21**

15896

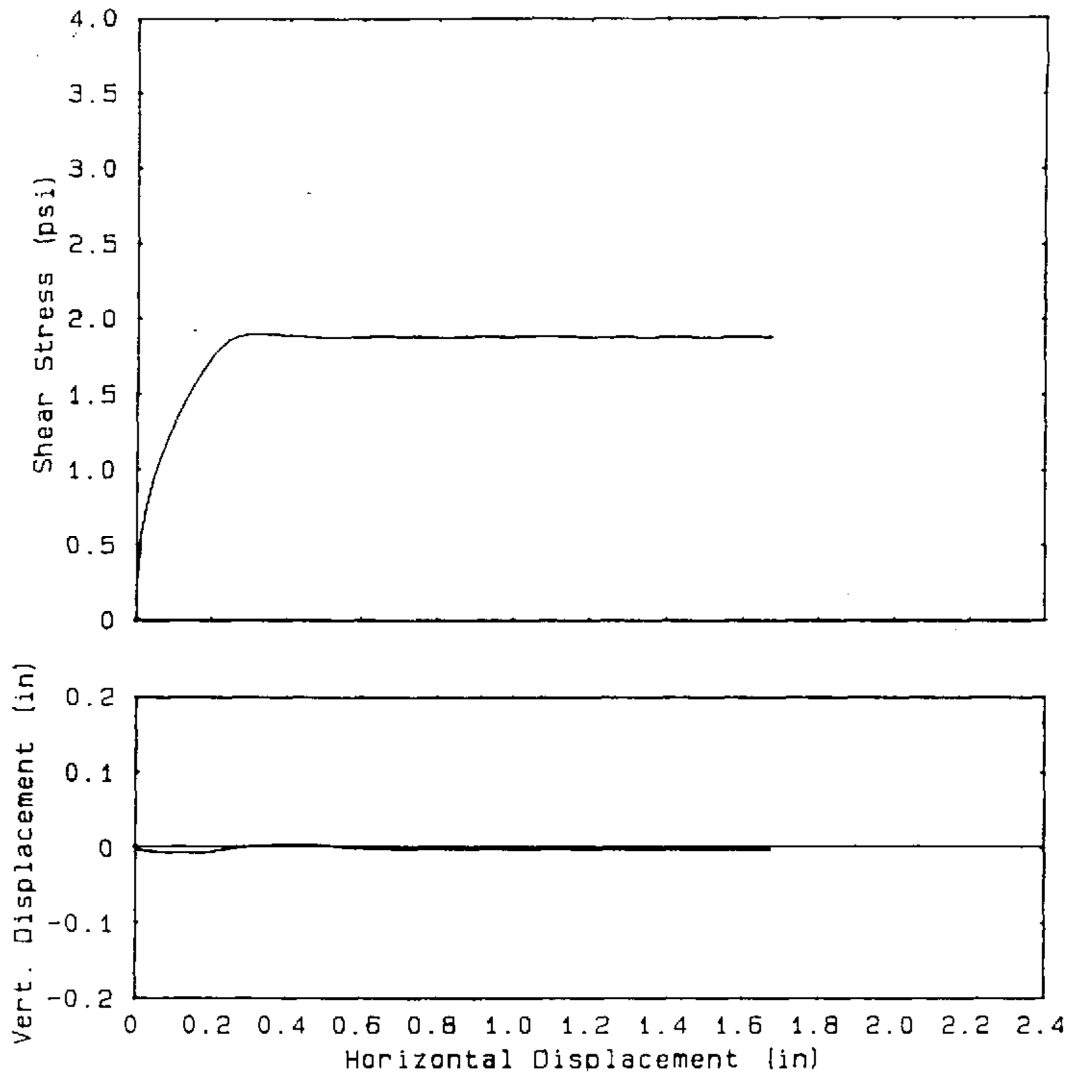


Large Direct Shear Test

Test Number	:	T5-10s
Test Date	:	30 July 1991
Shear Disp. Rate	:	2.36 in/hr
Sample Area	:	193.4 sq in
Sample Height	:	13.5 in
Normal Stress	:	1.45 psi
Dry Density	:	108 pcf
Moisture Content	:	10 %
Sample Description	:	Loose Quinn Perkins Sand over
	:	Geocomposite over Loose
Test Description	:	Hubbardson Sand
	:	Submerged.

JOB No.: 903-6400	SCALE: N/A	INTERFACE FRICTION TESTING DIRECT SHEAR TEST T5-10s
DRAWN: RDT	DATE: 08/07/91	
CHECKED: *[signature]*	DWG. No.: MA01-822	
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST — FIGURE 11-E22

158890



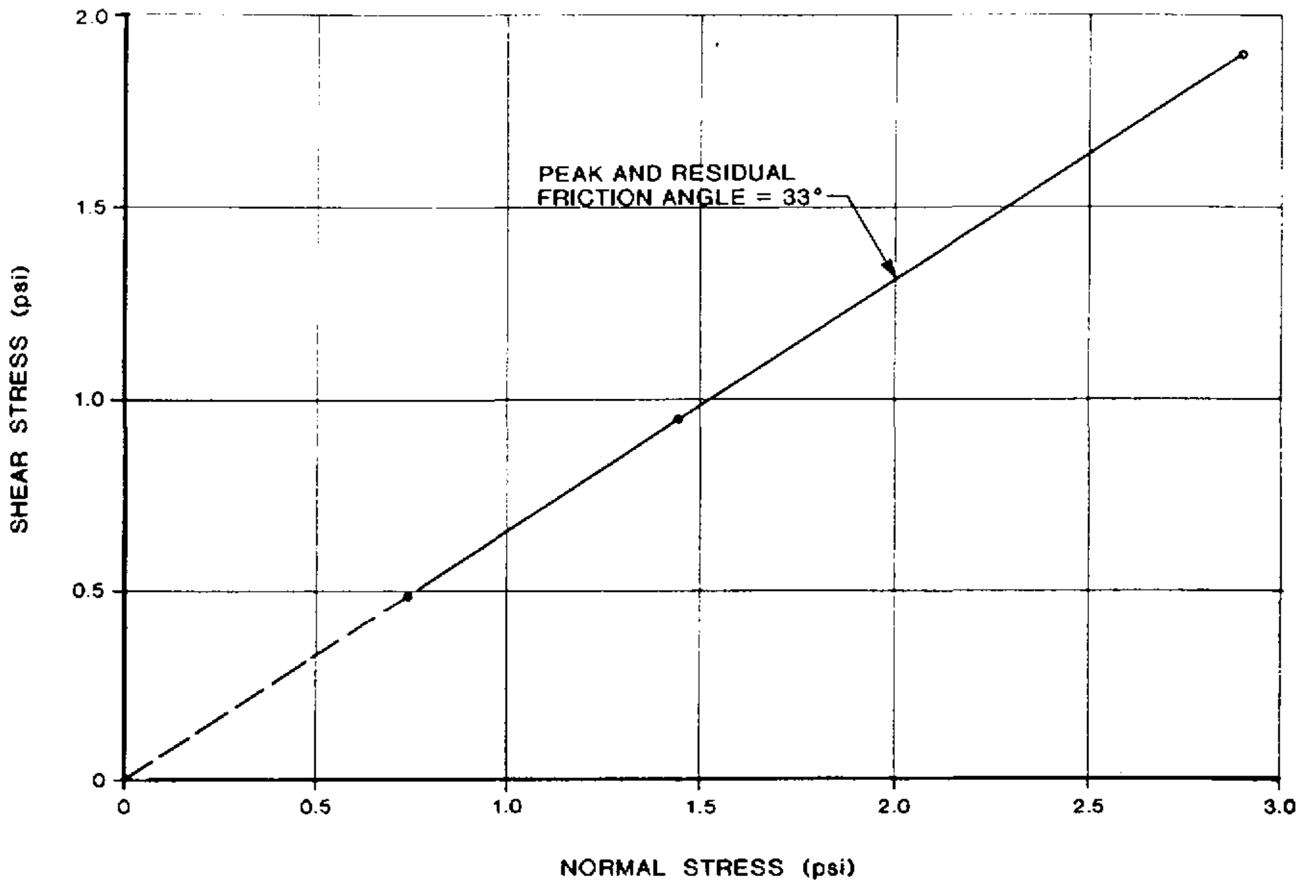
Large Direct Shear Test

Test Number	:	T5-20s
Test Date	:	31 July 1991
Shear Disp. Rate	:	2.34 in/hr
Sample Area	:	193.4 sq in
Sample Height	:	13.5 in
Normal Stress	:	2.9 psi
Dry Density	:	103 pcf
Moisture Content	:	10 %
Sample Description	:	Loose Quinn Perkins Sand over
	:	Geocomposite over Loose
Test Description	:	Hubbardson Sand
	:	Submerged.

JOB No.: 903-6400	SCALE: N/A	**INTERFACE FRICTION TESTING DIRECT SHEAR TEST T5-20s**
DRAWN: RDT	DATE: 08/07/91	
CHECKED:	DWG. No.: MA01-823	
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST — FIGURE **11-E23**

158696

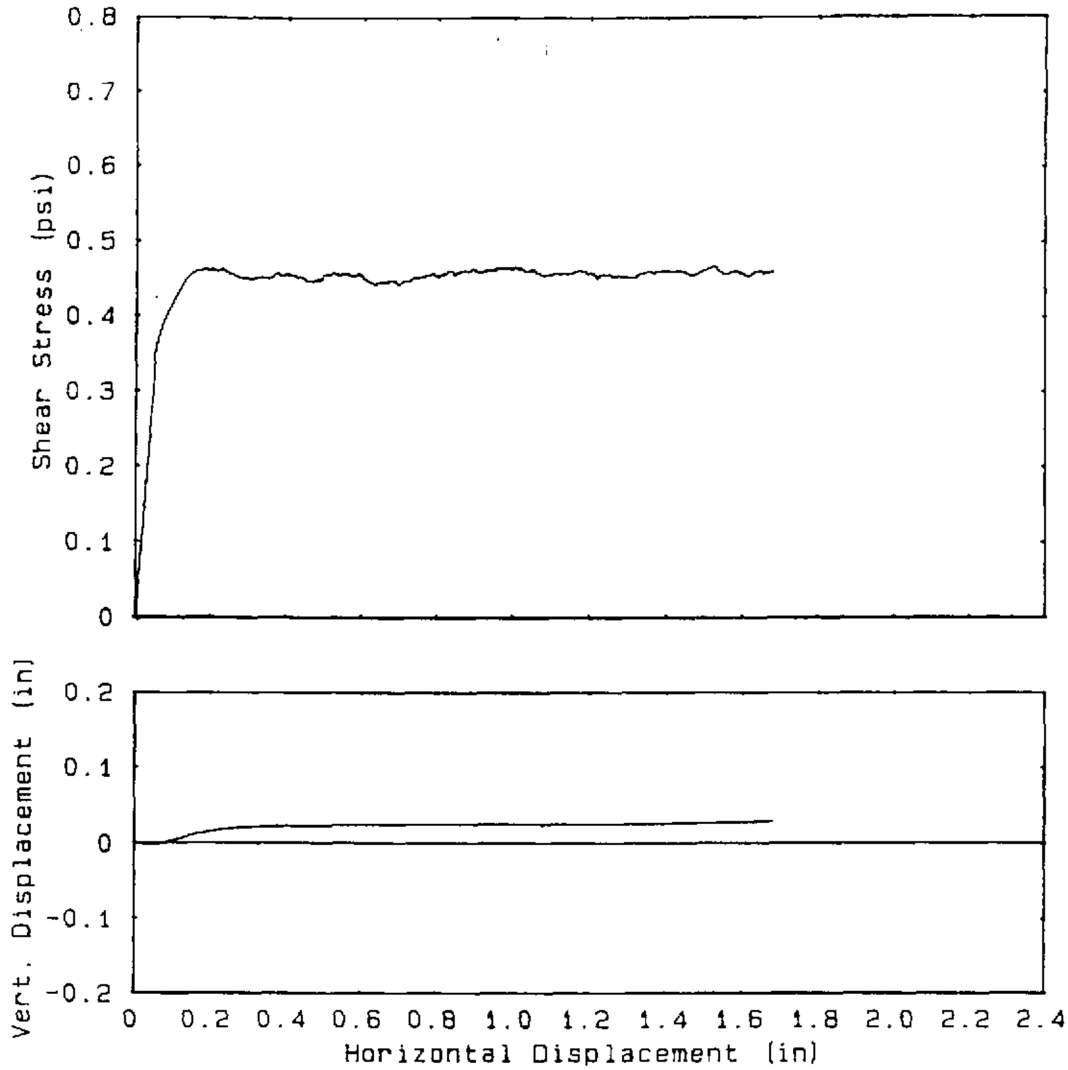
TEST T5



TESTS AT 0.73, 1.45 AND 2.90 psi
NORMAL STRESS

JOB No.: 903-6400	SCALE: N/A	INTERFACE FRICTION ANGLES DIRECT SHEAR TEST T5
DRAWN: RDT	DATE: 08/07/91	
CHECKED: RCR	DWG. No.: MA01-824	
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST — FIGURE 11-E24

158696

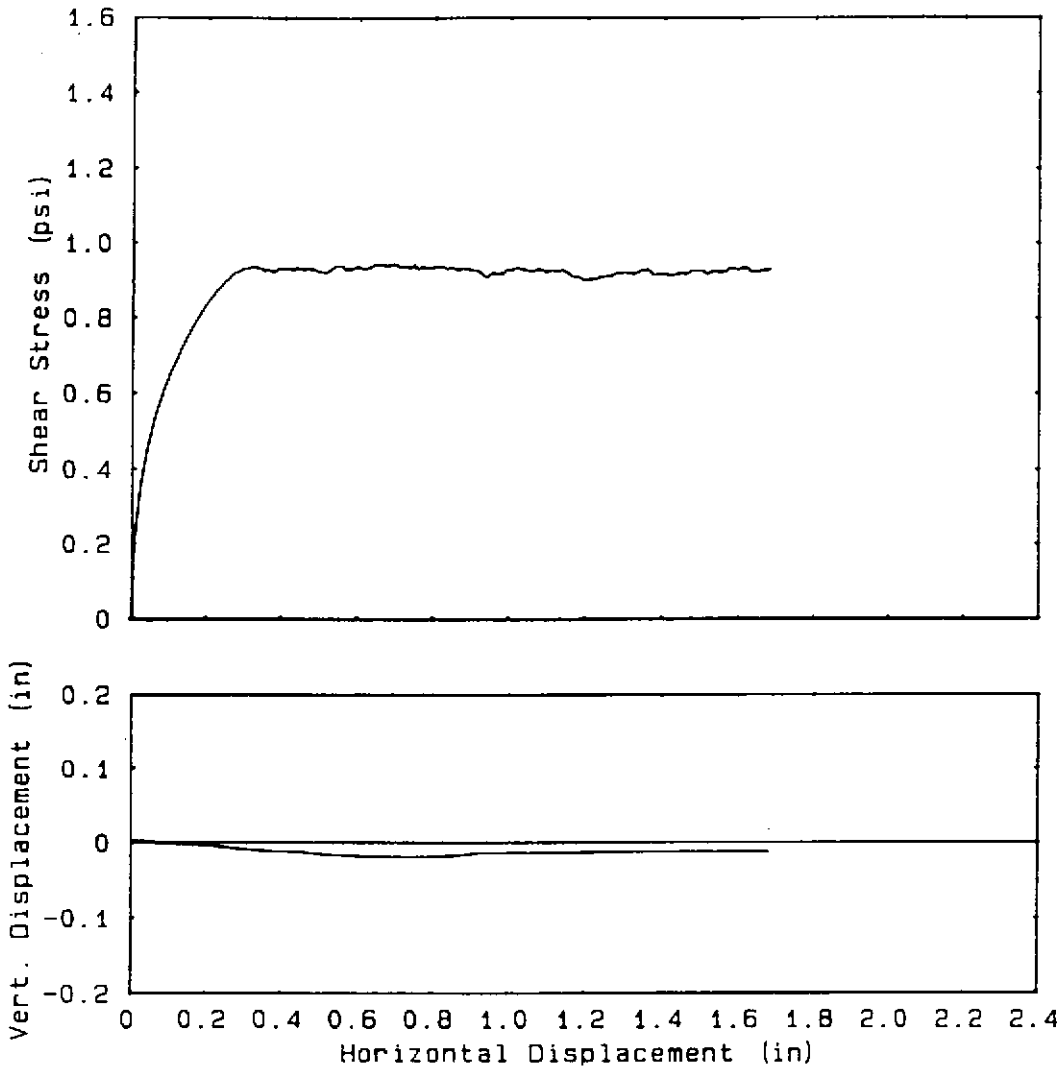


Large Direct Shear Test

Test Number	:	t6-05s
Test Date	:	31 July 1991
Shear Disp. Rate	:	2.38 in/hr
Sample Area	:	193.4 sq in
Sample Height	:	13.5 in
Normal Stress	:	.73 psi
Dry Density	:	105 pcf
Moisture Content	:	11 %
Sample Description	:	Loose Quinn Perkins Sand over
	:	Geocomposite over 60 mil Textured
Test Description	:	HDPE over Loose Quinn Perkinss Sand
	:	Submerged.

JOB No.: 903-6400	SCALE: N/A	**INTERFACE FRICTION TESTING DIRECT SHEAR TEST T6-05s**
DRAWN: RDT	DATE: 08/07/91	
CHECKED:	DWG. No.: MA01-825	
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST — FIGURE 11-E25

15886

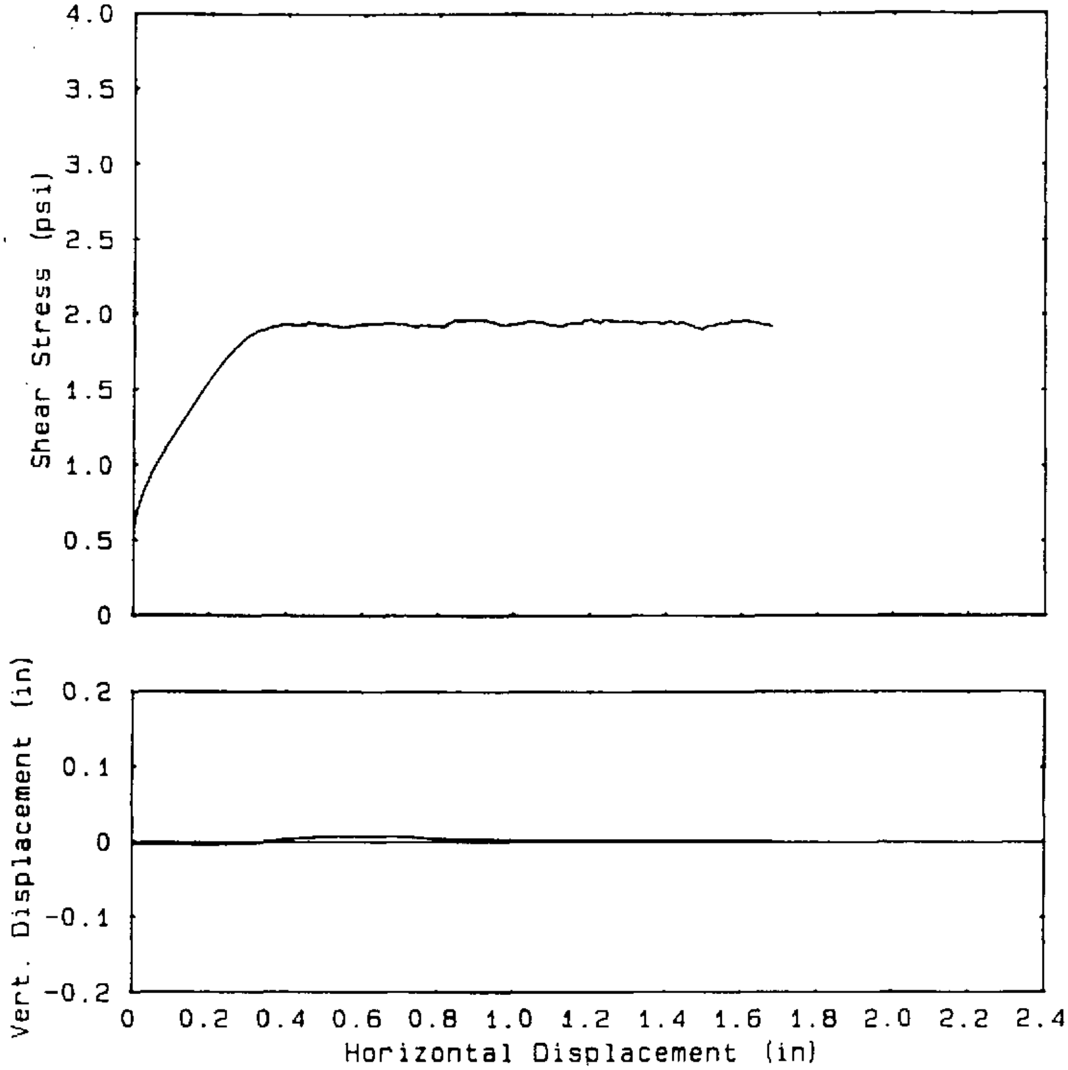


Large Direct Shear Test

Test Number	: T6-10s
Test Date	: 01 Aug 1991
Shear Disp. Rate	: 2.45 in/hr
Sample Area	: 193.4 sq in
Sample Height	: 13.5 in
Normal Stress	: 1.45 psi
Dry Density	: 105 pcf
Moisture Content	: 11 %
Sample Description	: Loose Quinn Perkins Sand over
	: Geocomposite over 60 mil Textured
Test Description	: HDPE over Loose Quinn Perkins Sand
	: Submerged.

JOB No.: 903-6400	SCALE: N/A	**INTERFACE FRICTION TESTING DIRECT SHEAR TEST T6-10s**
DRAWN: RDT	DATE: 08/07/91	
CHECKED:	DWG. No.: MA01-826	
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST — FIGURE 11-E26

158696



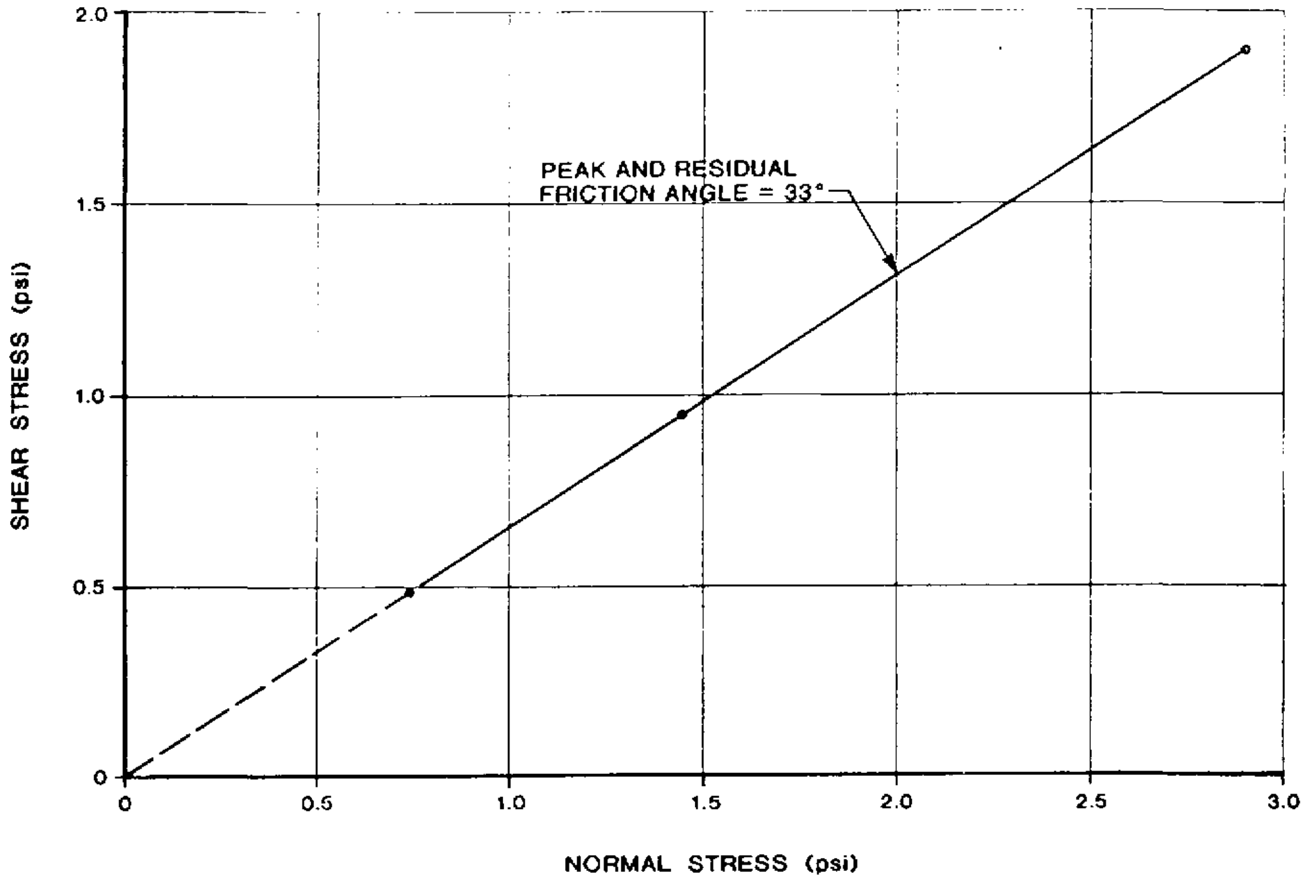
Large Direct Shear Test

Test Number	:	T6-20s
Test Date	:	01 Aug 1991
Shear Disp. Rate	:	2.4 in/hr
Sample Area	:	193.4 sq in
Sample Height	:	13.5 in
Normal Stress	:	2.9 psi
Dry Density	:	105 pcf
Moisture Content	:	11 %
Sample Description	:	Loose Quinn Perkins Sand over
	:	Geocomposite over 60 mil Textured
Test Description	:	HDPE over Loose Quinn Perkins Sand
	:	Submerged.

JOB No.: 903-6400	SCALE: N/A	INTERFACE FRICTION TESTING DIRECT SHEAR TEST T6-20s
DRAWN: RDT	DATE: 08/07/91	
CHECKED: MR	DWG. No.: MA01-827	
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST — FIGURE 11-E27

15696

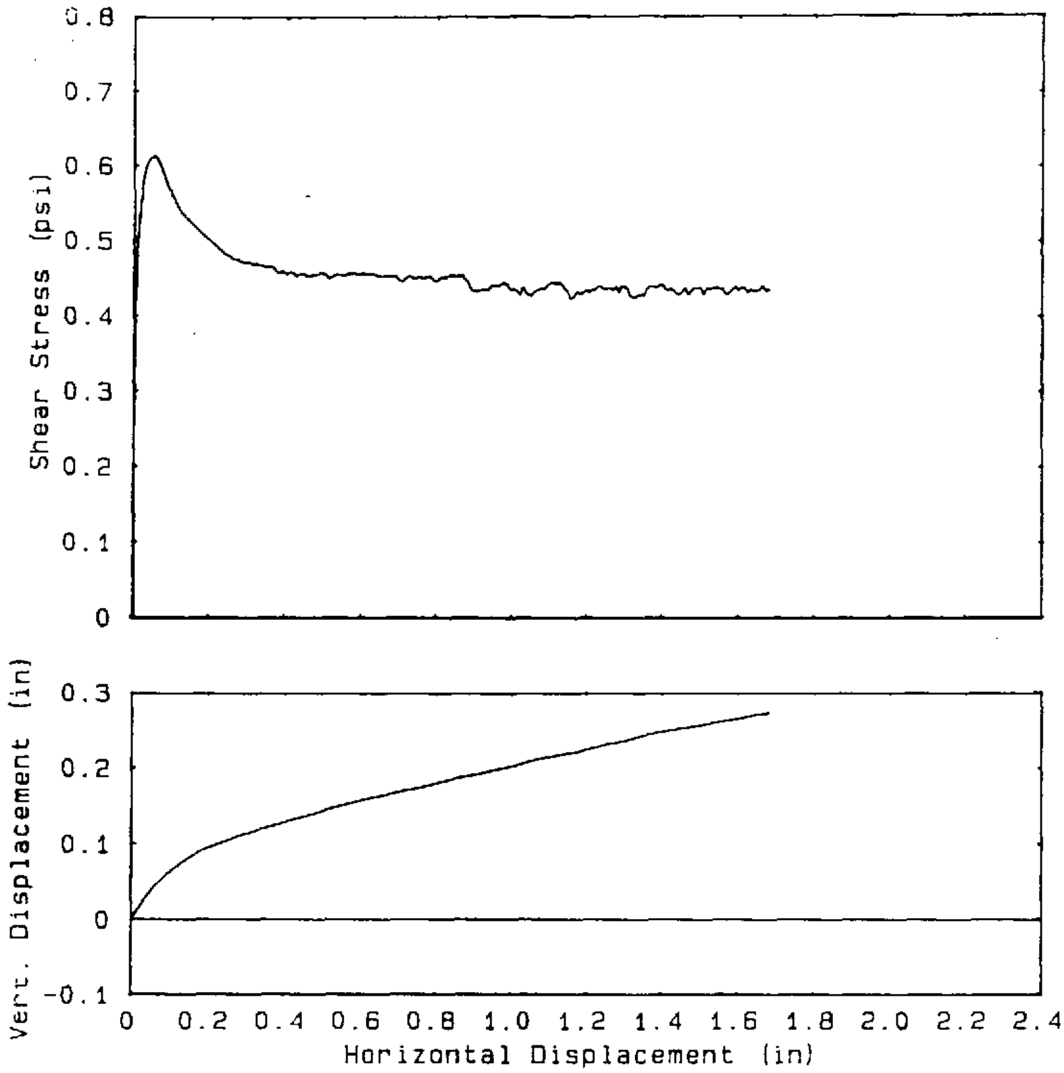
TEST T6



TESTS AT 0.73, 1.45 AND 2.90 psi
NORMAL STRESS

JOB No.: 903-6400	SCALE: N/A	**INTERFACE FRICTION ANGLES DIRECT SHEAR TEST T6**
DRAWN: RDT	DATE: 08/07/91	
CHECKED: KR	DWG. No.: MA01-828	
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST — FIGURE 11-E28

15695

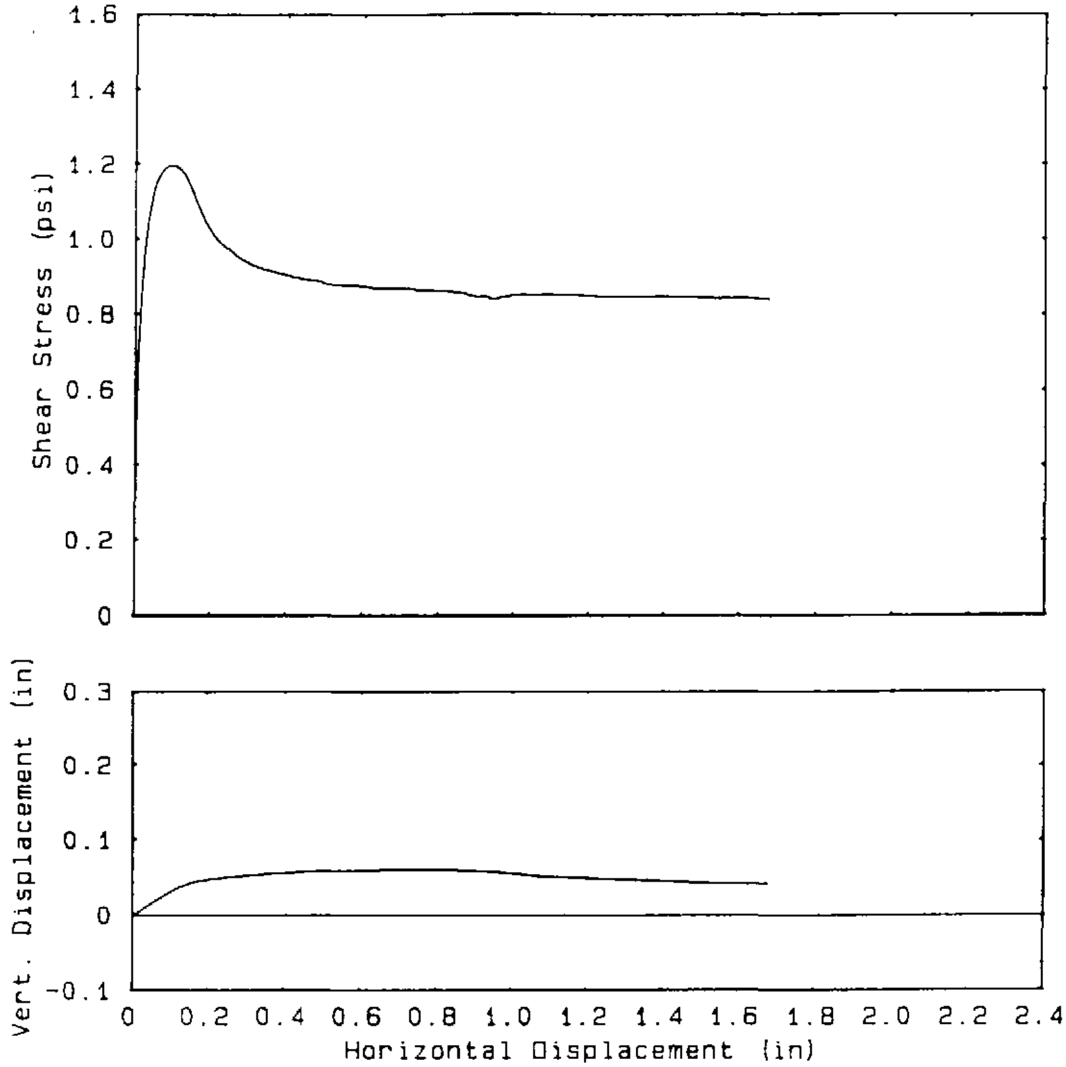


Large Direct Shear Test

Test Number	:	T7-05s
Test Date	:	02 Aug 1991
Shear Disp. Rate	:	2.42 in/hr
Sample Area	:	193.4 sq in
Sample Height	:	13.5 in
Normal Stress	:	.73 psi
Dry Density	:	103 pcf
Moisture Content	:	10 %
Sample Description	:	Loose Hubbardson Sand
	:	Shear test of soil only.
Test Description	:	
	:	Submerged.

JOB No.: 903-6400	SCALE: N/A	**INTERFACE FRICTION TESTING DIRECT SHEAR TEST T7-05s**
DRAWN: RDT	DATE: 08/07/91	
CHECKED:	DWG. No.: MA01-829	
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST
		FIGURE 11-E29

158696

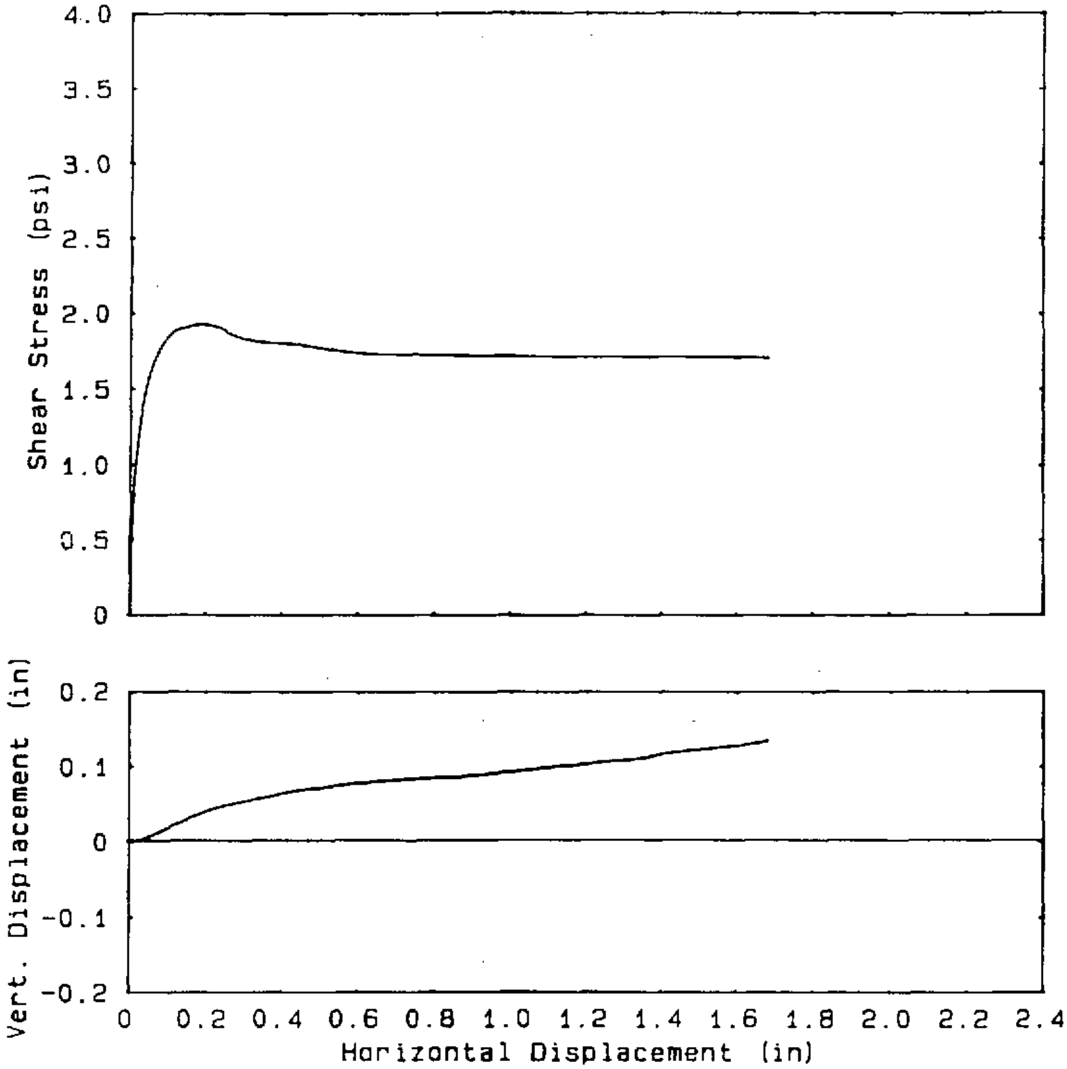


Large Direct Shear Test

Test Number	:	T7-10s
Test Date	:	02 Aug 1991
Shear Disp. Rate	:	2.42 in/hr
Sample Area	:	193.4 sq in
Sample Height	:	13.5 in
Normal Stress	:	1.45 psi
Dry Density	:	103 pcf
Moisture Content	:	10 %
Sample Description	:	Loose Hubbardson Sand
	:	Shear test of soil only.
Test Description	:	
	:	Submerged.

JOB No.: 903-6400	SCALE: N/A	INTERFACE FRICTION TESTING DIRECT SHEAR TEST T7-10s
DRAWN: RDT	DATE: 08/07/91	
CHECKED:	DWG. No.: MA01-830	
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST
		FIGURE 11-E30

158696



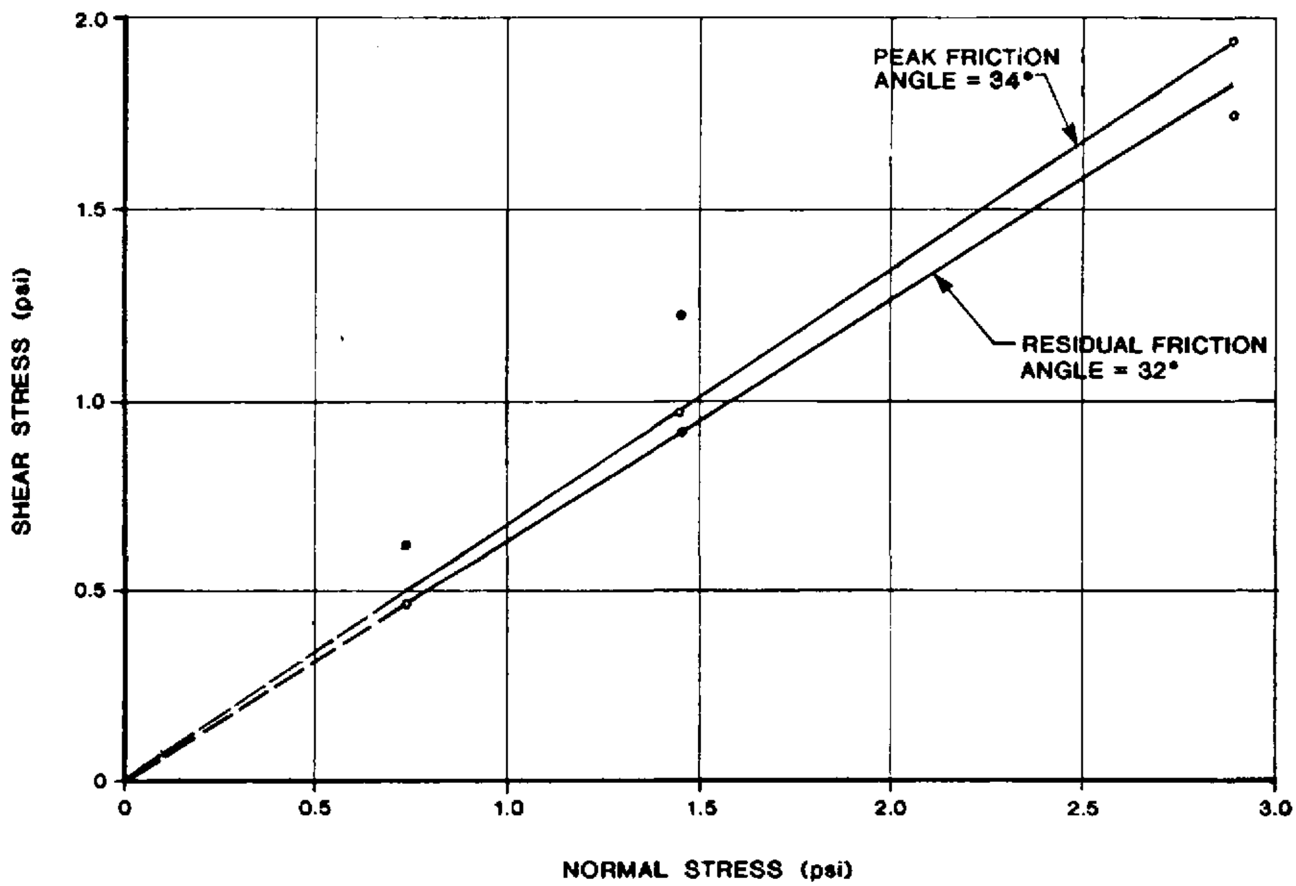
Large Direct Shear Test

Test Number	:	T7-20s
Test Date	:	02 Aug 1991
Shear Disp. Rate	:	2.35 in/hr
Sample Area	:	193.4 sq in
Sample Height	:	13.5 in
Normal Stress	:	2.9 psi
Dry Density	:	103 pcf
Moisture Content	:	10 %
Sample Description	:	Loose Hubbardson Sand
	:	Shear test of soil only.
Test Description	:	
	:	Submerged.

JOB No.: 903-6400	SCALE: N/A	INTERFACE FRICTION TESTING DIRECT SHEAR TEST T7-20s	
DRAWN: RDT	DATE: 08/07/91		
CHECKED:	DWG. No.: MA01-831		
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST	FIGURE 11-E31

15698

TEST T7



TESTS AT 0.73, 1.45 AND 2.90 psi
NORMAL STRESS

JOB No.: 903-6400	SCALE: N/A	INTERFACE FRICTION ANGLES DIRECT SHEAR TEST T7	
DRAWN: RDT	DATE: 08/07/91		
CHECKED:	DWG. No.: MA01-832		
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST	FIGURE 11-E32

APPENDIX 11-F

Geogrid Reinforced Slope Calculations

OBJECTIVE: EVALUATE THE FACTOR OF SAFETY FOR COVER STABILITY ON GEOGRID REINFORCED SLOPES.

METHOD: 1) CALCULATE REQUIRED TENSILE STRENGTH FOR FACTOR OF SAFETY EQUALS 1.5.

2) SELECT GEOGRID

3) CALCULATE FACTOR OF SAFETY FOR TENSILE CAPACITY OF GEOGRID

4) CALCULATE "RUNOUT" LENGTH ON SLOPE CREST TO ANCHOR GEOGRID.

ASSUMPTIONS AND CALCULATIONS FOLLOW

Golder Associates	SUBJECT Cover Stability - Geogrid Reinforcing		
	Job No. 903-6400.110	Made by RAC	Date 8/7/91
	Ref. ISRT	Checked BJD Reviewed	Sheet 2 of 7

1) CALCULATE THE REQUIRED TENSILE STRENGTH OF THE GEOGRID USED WHERE THE SLOPE OF THE PERMEABLE COVER IS STEEPER THAN 3H:1V.

PROCEDURE: USE METHOD OUTLINED IN THE PAPER "STRUCTURAL GEOGRIDS USED TO STABILIZE SOIL VENEER COVER," CHOUERY-CURTIS, V.E. AND BUTCHKO, S.T., CONFERENCE PROCEEDINGS FROM GEOSYNTHETICS '91.

ASSUMPTIONS:

1. ZERO COHESION AT CRITICAL SLIDING PLANE
2. RIGID BODY ANALYSIS (i.e. ASSUME SOIL COVER BEHAVES AS A SLIDING BLOCK)
3. TOTAL NORMAL FORCES IS RESISTED ON SIDE SLOPES (NEGLECT EFFECT OF RESISTING WEDGE AT TOE OF SLOPE, IF ANY)
4. THERE ARE NO SEEPAGE PRESSURES IN THE PROTECTIVE SOIL COVER LAYER.

EQUATIONS:

T = REQUIRED TENSILE STRENGTH
L = TOTAL SLOPE LENGTH
F_D = DRIVING FORCE
W = WEIGHT OF SOIL COVER
F_F = RESISTING FORCE
β = SLOPE ANGLE
Z = THICKNESS OF SOIL COVER
$F_D = W \sin \beta$
$F_F = W \cos \beta \tan \phi_c'$
$F_R = F_F + T$
ϕ_c' = CRITICAL INTERFACE FRICTION ANGLE

CALCULATIONS:

L = per UNIT LENGTH OF SLOPE
MAX SLOPE = 2.5H:1V (40%)
β = 21.8°
γ = 103 pcf

ϕ'_e = INTERFACE FRICTION ∡ BETWEEN NONWOVEN GEOTEXTILE AND COVER SOIL
= 30°

Z = 16 in = 1.33 ft

W = L · Z · γ = 1.33' x 103 pcf = 137 lb/ft/unit length

F_D = (137 lb/ft/unit length) sin 21.8 = 51 lb/ft/unit length

F_F = (137 lb/ft/unit length) cos 21.8 tan 30°
= 73.4 lb/ft/unit length

F_R = FS * F_D
= 1.5 x F_D
= 1.5 x 51 lb/ft/unit length
= 76.5 lb/ft/unit length

F_R = F_F + T
76.5 = 73.4 + T

T = 3.1 lb/ft/unit length

FOR A MAXIMUM SLOPE = 80 ft

T = 3.1 lb/ft/unit length x 80 ft
= 248 lb/ft

2) TENSAR GEOGRID UX1400 HAS A LONG TERM TENSILE = 1200 lb/ft

1200 lb/ft > 248 lb/ft

∴ UX1400 HAS SUFFICIENT TENSILE CAPACITY FOR THIS DESIGN.

3) CALCULATE FACTOR OF SAFETY FOR TENSILE CAPACITY OF THE GEOGRID

PROCEDURE: USING THE FREE BODY ANALYSIS, INPUT TENSILE CAPACITY (T) CALCULATE FS USING SIMILAR RIGID BODY ANALYSIS. SOLVING FOR FACTOR OF SAFETY USING THE FOLLOWING EQUATIONS

ASSUMPTIONS:

1. ZERO COHESION AT CRITICAL SLIDING PLANE
2. RIGID BODY ANALYSIS (IE ASSUME SOIL COVER BEHAVES AS A SLIDING BLOCK)
3. TOTAL NORMAL FORCES IS RESISTED ON SIDE SLOPES (NEGLECT EFFECT OF RESISTING WEDGE AT TOE OF SLOPE, IF ANY)
4. THERE ARE NO SEEPAGE PRESSURES IN THE PROTECTIVE SOIL COVER LAYER
5. SLOPE EQUALS 2H:1V
6. ϕ_c' = CRITICAL INTERFACE FRICTION ANGLE BETWEEN COVER SOIL AND GEOGRID = 30°
7. T_{REQD} = 3.1 lb/ft/unit length

CALCULATIONS:

F_R = RESISTING FORCE = F_D FS

F_D = DRIVING FORCE = 51 lb/ft, UNIT LENGTH

F_F = FRICTIONAL RESISTING FORCE = 13.4 lb/ft/UNIT LENGTH

T = TENSILE CAPACITY = 1200 lb/ft, 80 ft SLOPE
= 15 lb/ft/UNIT LENGTH

Golder Associates	SUBJECT Cover Stability - Geogrid Reinforcing		
	Job No. 913-6400.110 Ref.	Made by RAC Checked RJD Reviewed PER	Date 8/7/91 Sheet 5 of 7

CALCULATIONS (cont'd)

$$F_R = FS \times F_D = F_F + T$$

$$F_S = (F_F + T)/F_D$$

$$= (73.4 + 15)/51$$

$$\underline{\underline{FS = 1.73}}$$

4) CALCULATE RUNOUT LENGTH ON SLOPE CREST TO ANCHOR GEOGRID

PROCEDURE: USE RUNOUT CALCULATION METHOD AS DESCRIBED IN DESIGNING WITH GEOSYNTHETICS, ROBERT M. KOERNER, 2nd EDITION, PRENTICE HALL, P. 426, 1990.

ASSUMPTIONS: SAME AS PREVIOUSLY MENTIONED IN (3) ABOVE.

EQUATIONS:

$$T_{ALLOW} \times L_S(\cos\beta) = q_L \tan\phi_c' (L_{RO}) + T_{ALLOW} \times L_S(\sin\beta \tan\phi_c')$$

SOLVING FOR L_{RO}

$$L_{RO} = T_{ALLOW} \times L_S(\cos\beta - \sin\beta\tan\phi_c') / q_L \tan\phi_c'$$

WHERE

L_{RO} = LENGTH OF RUNOUT

$T_{ALLOW} = T_{REQ'D}$ = REQUIRED TENSILE FORCE TO PREVENT SOIL COVER FROM SLIDING USING A FS = 1.5 = 3.1 lb/ft/unit len

β = SLOPE ANGLE = 21.8° (2.5H:1V) (40% THIS IS MAX SLOPE)

ϕ_c' = CRITICAL INTERFACE FRICTION ANGLE BETWEEN COVER SOIL AND GEOGRID = 30°. BASED ON GEOTEXTILE ϕ_r = 30° AND (SEE ...) RELATIVELY HIGHER VALUES FOR GEOGRID. SEE PAGE 7

q_L = THE SURCHARGE PRESSURE = $d_{cs} \times \gamma_{cs} \times \cos\theta$

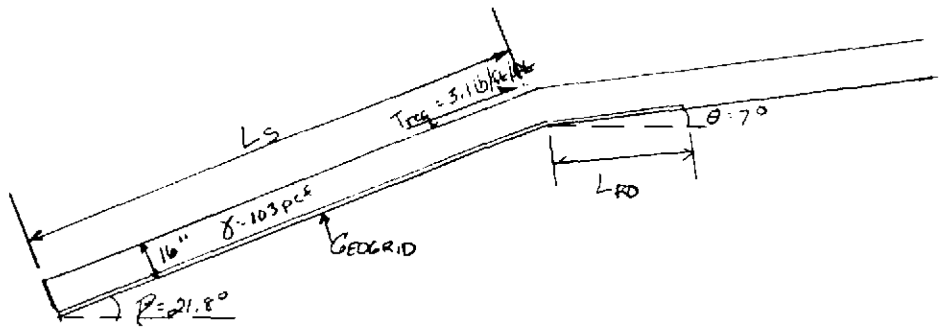
d_{cs} = DEPTH OF COVER SOIL = 16 in

EQUATIONS (cont'd)

γ_{cs} = UNIT WEIGHT OF COVER SOIL = 103 pcf
θ = SLOPE ANGLE BEYOND CREST = $7°$

q_L = 16"/12"/ft × 103 pcf × cos 7 = 136 psf

L_S = SLOPE LENGTH = VARIES



$L_{RO} = T_{allow} \times L_S\,(\cos\beta - \sin\beta \tan\phi_c')\,/\,q_L \tan\phi_c'$

$= 3.1\ \text{lb/ft/ft} \times L_S\,(\cos 21.8 - \sin 21.8 \tan 30)\,/\,136\ \text{psf} \times \tan 30$

$= 0.028\ \text{ft/ft}\ L_S$

WEST HIDE PILE - MAXIMUM SLOPE DISTANCE = 80'

L_{RO} = 0.028 ft/ft × 80 ft = 2.24 ft → 3 ft

SOUTH HIDE PILE - MAXIMUM SLOPE DISTANCE = 80'

L_{RO} = 0.028 ft/ft × 80 ft = 2.24 ft → 3 ft

L_{RO} = 3 ft BEYOND APPROXIMATE CREST OF SLOPE

TO ALLOW FOR VARIATIONS IN SLOPE CREST INCREASE
L_{RO} TO 10 ft

TABLE I
RANGE OF VALUES OF INTERFACE FRICTION ANGLES*

INTERFACES	FRICTION ANGLE
Geosynthetic/Soil	
Stiff Geogrid/Sand	23° to 34°
HDPE FML (smooth)/Sand	18° to 26°
PVC FML/Sand	20° to 28°
Nonwoven Fabric/Sand	21° to 29°
HDPE FML (smooth)/Clay	12° to 19°
PVC FML/Clay	13° to 20°
Nonwoven Fabric/Clay	14° to 22°
Geosynthetic/Geosynthetic	
Nonwoven Fabric/HDPE FML (smooth)	9° to 16°
Nonwoven Fabric/PVC FML	12° to 18°
Nonwoven Fabric/Drainage Net	10° to 16°
HDPE FML (smooth)/Drainage Net	8° to 15°

*NOTE: The value of interface friction angles are product dependent. Testing is recommended based on project specifics and final intended use of the various geosynthetic products.

<u>Geosynthetic Components</u>: As noted above, a variety of planar, polymer based synthetic materials are commonly utilized in construction of municipal and hazardous waste facilities. These materials are FMLs, fabrics, drainage nets, and geogrids. Although all are polymer based materials, the manufacturing processes and selected resins can vary widely between and within each category. The desired, manufactured properties of the geosynthetics are dictated by their respective functions.

The four categories of geosynthetics commonly used in waste facilities and their primary function are summarized below. All geosynthetics must be resistant to chemical and biological degradation for utilization in waste containment. A FML is used for containment and must have a very low permeability, so as to provide adequate leachate containment for the design life of the structure. A fabric is used for separation and must be capable of passing fluid through it while retaining soil above it. A drainage net is used for drainage and must be able to transmit large flow under high compressive loading. A geogrid is used to provide tensile reinforcement. This function classifies geogrids as structural elements, which is a unique classification, in comparison to the other planar geosynthetics.

Within this paper, the category of geogrids is reviewed in detail. The physical properties of geogrids required for performance in their applications are defined. Comparisons are made on how these functions vary and complement the functions of other geosynthetic material used in waste containment applications.

APPENDIX 11-G

Soil Erosion Calculations

Golder Associates	SUBJECT Soil Erosion		
	Job No. 903-6400.110	Made by ACK	Date 12/13/91
	Ref. ISRT/Woburn/Mass	Checked VEE7	Sheet 1 of 8
		Reviewed RCR	

Objective: estimate the amount of soil loss on the steepest slope section of the permeable cover, which is on the West Hide Pile

Method: calculate the average annual erosion using the Universal Soil Loss Equation (USDA)

$A = R \times K \times LS \times C \times P$

where A = avg. annual soil loss, tons/acre

R = rainfall and runoff erosivity index

K = soil erodibility factor, tons/acre

LS = slope length-steepness factor

C = cover management factor

P = erosion control practice factor

References: 1. "Evaluating Cover Systems for Solid and Hazardous Waste, (EPA-SW-867), R.J. Lutton

2. Standards for Soil Erosion and Sediment Control in New Jersey, 1987

3. Sheet 11-7 from this report

Assumptions: $R = 125$ (from fig. 20 of ref 1)

$K = 0.24$ (from Table 5 of ref 1 for a sandy loam with 2% organic content)

$C = 0.004$ (from Table 7 of ref 1 for grass and legume meadow, high productivity)

$P = 1.0$ (from Table 8 of ref 1 - conservative value)

steepest slope section [see sheet 11-7A at section I-I' — 168' up slope starting at 74' contour (toe of slope)]

Golder Associates	SUBJECT Soil Erosion		
	Job No. 903-6400.110	Made by ALK	Date 12/13/91
	Ref. ISRT/Woburn/Mass	Checked VEE7 Reviewed	Sheet 2 of 8

Calculations;

slope segments

1	~2% slope	56 ft. long
2	~7% slope	56 ft. long
3	33% slope	56 ft. long

model as 3 equal segments – total length = 168'

from Table AI-3 of ref 2 (which is used because it has a greater # of values for LS)

$LS_{2\%} = 0.234$ for length = 168 ft

$LS_{7\%} = 1.08$ for length = 168 ft

$LS_{33\%} = 13.0$ for length = 168 ft

multiply these values by factors from p. 38 of ref. 1 for 3 segments to determine an effective LS value

$LS_{eff} = (0.58 \times 0.234 + 1.06 \times 1.08 + 1.37 \times 13.0)/3 = 6.36$

$A = R \times K \times LS \times C \times P$

$= 125 \times 0.24 \times 6.36 \times 0.004 \times 1.0$ (tons/acre/yr)

$= 0.76$ (tons/acre/yr) < 2 (tons/acre/yr)

∴ calculated soil loss is less than permitted value stated in the RDAP

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Not only is erosion objectionable in itself but erosion can degrade the cover and seriously reduce its effectiveness.

Evaluate Erosion Potential — Step 19

The USDA universal soil loss equation (USLE) is a convenient tool for use in evaluating erosion potential. The USLE predicts average annual soil loss as the product of six quantifiable factors. The equation is:

$$A = R\ K\ L\ S\ C\ P$$

where A = average annual soil loss, in tons/acre
R = rainfall and runoff erosivity index
K = soil erodibility factor, tons/acre
L = slope-length factor
S = slope-steepness factor
C = cover-management factor
P = practice factor

The data necessary as input to this equation are available to the evaluator in a figure and tables included below. Note that the evaluations in Step 8 on soil composition and Steps 25-32 on vegetation all impact on the evaluation of erosion also.

Factor R in the USLE can be calculated empirically from climatological data. For average annual soil loss determinations, however, R can be obtained directly from Figure 20. Factor K, the average soil loss for a given

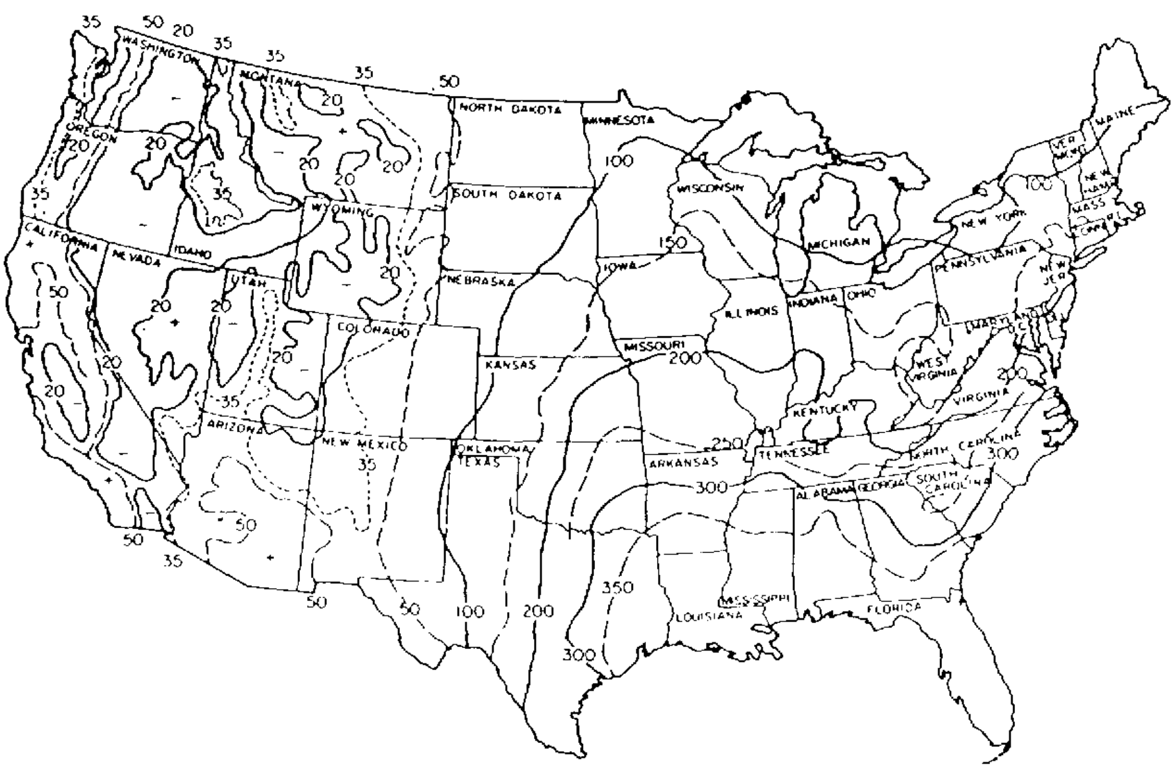

Figure 20. Average annual values of rainfall-erosivity factor R.[11]

from reference I

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soil in a unit plot, pinpoints differences in erosion according to differences in soil type. Long-term plot studies under natural rainfall have produced K values generalized in Table 5 for the USDA soil types.

TABLE 5. APPROXIMATE VALUES OF FACTOR K FOR USDA TEXTURAL CLASSES[11]

Texture class	Organic matter content		
	<0.5%	2%	4%
	K	K	K
Sand	0.05	0.03	0.02
Fine sand	.16	.14	.10
Very fine sand	.42	.36	.28
Loamy sand	.12	.10	.08
Loamy fine sand	.24	.20	.16
Loamy very fine sand	.44	.38	.30
Sandy loam	.27	.24	.19
Fine sandy loam	.35	.30	.24
Very fine sandy loam	.47	.41	.33
Loam	.38	.34	.29
Silt loam	.48	.42	.33
Silt	.60	.52	.42
Sandy clay loam	.27	.25	.21
Clay loam	.28	.25	.21
Silty clay loam	.37	.32	.26
Sandy clay	.14	.13	.12
Silty clay	.25	.23	.19
Clay		0.13-0.29	

The values shown are estimated averages of broad ranges of specific-soil values. When a texture is near the borderline of two texture classes, use the average of the two K values.

The evaluator must next consider the shape of the slope in terms of length and inclination. The appropriate LS factor is obtained from Table 6. A nonlinear slope may have to be evaluated as a series of segments, each with uniform gradient. Two or three segments should be sufficient for most engineered landfills, provided the segments are selected so that they are also of equal length (Table 6 can be used, with certain adjustments). Enter Table 6 with the total slope length and read LS values corresponding to the percent slope of each segment. For three segments, multiply the chart LS values for the upper, middle, and lower segments by 0.58, 1.06, and 1.37, respectively. The average of the three products is a good estimate of the

from reference I

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TABLE 6. VALUES OF THE FACTOR LS FOR SPECIFIC COMBINATIONS OF SLOPE LENGTH AND STEEPNESS[11]

% Slope	Slope length (feet)											
	25	50	75	100	150	200	300	400	500	600	800	1000
0.5	0.07	0.08	0.09	0.10	0.11	0.12	0.14	0.15	0.16	0.17	0.19	0.20
1	0.09	0.10	0.12	0.13	0.15	0.16	0.18	0.20	0.21	0.22	0.24	0.26
2	0.13	0.16	0.19	0.20	0.23	0.25	0.28	0.31	0.33	0.34	0.38	0.40
3	0.19	0.23	0.26	0.29	0.33	0.35	0.40	0.44	0.47	0.49	0.54	0.57
4	0.23	0.30	0.36	0.40	0.47	0.53	0.62	0.70	0.76	0.82	0.92	1.0
5	0.27	0.38	0.46	0.54	0.66	0.76	0.93	1.1	1.2	1.3	1.5	1.7
6	0.34	0.48	0.58	0.67	0.82	0.95	1.2	1.4	1.5	1.7	1.9	2.1
8	0.50	0.70	0.86	0.99	1.2	1.4	1.7	2.0	2.2	2.4	2.8	3.1
10	0.69	0.97	1.2	1.4	1.7	1.9	2.4	2.7	3.1	3.4	3.9	4.3
12	0.90	1.3	1.6	1.8	2.2	2.6	3.1	3.6	4.0	4.4	5.1	5.7
14	1.2	1.6	2.0	2.3	2.8	3.3	4.0	4.6	5.1	5.6	6.5	7.3
16	1.4	2.0	2.5	2.8	3.5	4.0	4.9	5.7	6.4	7.0	8.0	9.0
18	1.7	2.4	3.0	3.4	4.2	4.9	6.0	6.9	7.7	8.4	9.7	11.0
20	2.0	2.9	3.5	4.1	5.0	5.8	7.1	8.2	9.1	10.0	12.0	13.0
25	3.0	4.2	5.1	5.9	7.2	8.3	10.0	12.0	13.0	14.0	17.0	19.0
30	4.0	5.6	6.9	8.0	9.7	11.0	14.0	16.0	18.0	20.0	23.0	25.0
40	6.3	9.0	11.0	13.0	16.0	18.0	22.0	25.0	28.0	31.0	--	--
50	8.9	13.0	15.0	18.0	22.0	25.0	31.0	--	--	--	--	--
60	12.0	16.0	20.0	23.0	28.0	--	--	--	--	--	--	--

Values given for slopes longer than 300 feet or steeper than 18% are extrapolations beyond the range of the research data and, therefore, less certain than the others.

overall effective LS value. If two segments are sufficient, multiply by 0.71 and 1.29.

Factor C in the USLE is the ratio of soil loss from land cropped under specified conditions to that from clean-tilled, continuous fallow. Therefore, C combines effects of vegetation, crop sequence, management, and agricultural (as opposed to engineering) erosion-control practices. On landfills, freshly covered and without vegetation or special erosion-reducing procedures of cover placement, C will usually be about unity. Where there is vegetative cover or significant amounts of gravel, roots, or plant residues or where cultural practices increase infiltration and reduce runoff velocity, C is much less than unity. Estimate C by reference to Table 7 for anticipated cover management, but also consider changes that may take place in time. Meadow values are usually most appropriate. See Reference 1 for additional guidance.

Factor P in the USLE is similar to C except that it accounts for additional erosion-reducing effects of land management practices that are superimposed on the cultural practices, e.g., contouring, terracing, and contour strip-cropping. Approximate values of P, related only to slope steepness,

from reference 1

From Reference 2

TABLE A1-3

VALUES OF THE TOPOGRAPHIC FACTOR "LS"

Length of Slope (L) Ft.	Percent Slope (S) 0.2	0.3	0.4	0.5	1.0	2.0	3.0	4.0	5.0	6.0	8.0	10.0	12.0	14.0	16.0	18.0	20.0	25.0	30.0	40.0	50.0	60.0
20	.05	.05	.06	.06	.08	.12	.18	.21	.24	.30	.44	.61	.81	1.0	1.3	1.6	1.8	2.6	4	6	8	10
40	.06	.07	.07	.08	.10	.15	.22	.28	.34	.43	.63	.87	1.2	1.4	1.8	2.2	2.6	3.5	5	8	11	15
60	.07	.08	.08	.09	.11	.17	.25	.33	.41	.52	.77	1.0	1.4	1.8	2.2	2.6	3.0	4.5	6	10	14	18
80	.08	.08	.09	.09	.12	.19	.27	.37	.48	.60	.89	1.2	1.6	2.1	2.6	3.0	3.6	5.5	7	11	16	21
100	.08	.09	.09	.10	.13	.20	.29	.40	.54	.67	.99	1.4	1.8	2.4	2.9	3.5	4.2	6.0	8	13	18	23
110	.08	.09	.10	.10	.13	.21	.30	.42	.56	.71	1.0	1.5	2.0	2.5	3.0	3.7	4.5	6	9	14	19	25
120	.09	.09	.10	.10	.14	.21	.30	.43	.59	.74	1.0	1.6	2.1	2.6	3.3	4.0	4.6	7	9	14	20	26
130	.09	.09	.10	.11	.14	.22	.31	.44	.61	.77	1.2	1.6	2.2	2.8	3.4	4.1	4.9	7	9	15	20	27
140	.09	.10	.10	.11	.14	.22	.32	.46	.63	.80	1.2	1.7	2.3	2.9	3.6	4.3	5.1	7	10	15	21	29
150	.09	.10	.11	.11	.15	.23	.32	.47	.66	.82	1.2	1.8	2.4	3.0	3.7	4.5	5.3	8	10	16	23	30
160	.09	.10	.11	.11	.15	.23	.33	.48	.68	.85	1.2	1.9	2.5	3.1	3.9	4.7	5.5	8	10	17	24	31
180	.10	.10	.11	.12	.15	.24	.34	.51	.72	.90	1.4	1.9	2.6	3.3	4.1	5.0	6.0	9	12	18	26	33
200	.10	.11	.11	.12	.16	.25	.35	.53	.76	.95	1.4	2.1	2.8	3.6	4.4	5.3	6.3	9	12	18	27	35
300	.11	.12	.13	.14	.18	.28	.40	.62	.93	1.2	1.8	2.7	3.6	4.5	5.6	6.8	8	12	16	25	35	45
400	.12	.13	.14	.15	.20	.31	.44	.70	1.0	1.4	2.0	3.2	4.2	5.4	6.7	8.0	10	14	19	30	42	54
500	.13	.14	.15	.16	.21	.33	.47	.76	1.2	1.6	2.2	3.7	4.9	6.2	7.6	9.2	11	16	21	34	47	61
600	.14	.15	.16	.17	.22	.34	.49	.82	1.4	1.6	2.4	4.1	5.4	6.9	8.5	10.3	12	16	24	38	53	68
700	.15	.16	.17	.18	.23	.36	.52	.87	1.4	1.8	2.6	4.5	6.0	7.5	9.3	11.3	13	18	26	41	58	75
800	.15	.16	.17	.18	.24	.38	.54	.92	1.6	2.0	2.8	4.9	6.4	8.2	10.1	12.2	14	20	28	45	58	81
900	.16	.17	.18	.19	.25	.39	.56	.96	1.6	2.0	3.0	5.2	6.9	9.8	10.8	13.1	16	22	30	48	67	87
1000	.16	.18	.19	.20	.26	.40	.57	1.0	1.6	2.2	3.0	5.6	7.4	9.3	11.6	14.0	17	24	32	51	72	93

When the length of slope exceeds 400 feet and (or) percent of slope exceeds 24 percent, soil loss estimates are speculative as these values are beyond the range of research data.

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TABLE 7. GENERALIZED VALUES OF FACTOR C FOR STATES EAST OF THE ROCKY MOUNTAINS[11]

Crop, rotation, and management	Productivity level: High	Productivity level: Mod.
	C value	
Base value: continuous fallow, tilled up and down slope	1.00	1.00
CORN		
C, RdR, fall TP, conv	0.54	0.62
C, RdR, spring TP, conv	.50	.59
C, RdL, fall TP, conv	.42	.52
C, RdR, wc seeding, spring TP, conv	.40	.49
C, RdL, standing, spring TP, conv	.38	.48
C-W-M-M, RdL, TP for C, disk for W	.039	.074
C-W-M-M-M, RdL, TP for C, disk for W	.032	.061
C, no-till pl in c-k sod, 95-80% rc	.017	.053
COTTON		
Cot, conv (Western Plains)	0.42	0.49
Cot, conv (South)	.34	.40
MEADOW		
Grass & Legume mix	0.004	0.01
Alfalfa, lespedeza or Sericia	.020	
Sweet clover	.025	
SORGHUM, GRAIN (Western Plains)		
RdL, spring TP, conv	0.43	0.53
No-till pl in shredded 70-50% rc	.11	.18
SOYBEANS		
B, RdL, spring TP, conv	0.48	0.54
C-B, TP annually, conv	.43	.51
B, no-till pl	.22	.28
C-B, no-till pl, fall shred C stalks	.18	.22
WHEAT		
W-F, fall TP after W	0.38	
W-F, stubble mulch, 500 lbs rc	.32	
W-F, stubble mulch, 1000 lbs rc	.21	

Abbreviations defined:

B - soybeans
C - corn
c-k - chemically killed
conv - conventional
cot - cotton
F - fallow
M - grass & legume hay
pl - plant
W - wheat
wc - winter cover

lbs rc - pounds of crop residue per acre remaining on surface after new crop seeding
% rc - percentage of soil surface covered by residue mulch after new crop seeding
70-50% rc - 70% cover for C values in first column; 50% for second column
RdR - residues (corn stover, straw, etc.) removed or burned
RdL - all residues left on field (on surface or incorporated)
TP - turn plowed (upper 5 or more inches of soil inverted, covering residues)

from reference 1

are listed in Table 8. These values are based on rather limited field data, but P has a narrower range of possible values than the other five factors.

TABLE 8. VALUES OF FACTOR P[11]

Practice	Land slope (percent)				
	1.1-2	2.1-7	7.1-12	12.1-18	18.1-24
	(Factor P)				
Contouring (P_c)	0.60	0.50	0.60	0.80	0.90
Contour strip cropping (P_{sc})					
R-R-M-M[1]	0.30	0.25	0.30	0.40	0.45
R-W-M-M	0.30	0.25	0.30	0.40	0.45
R-R-W-M	0.45	0.38	0.45	0.60	0.68
R-W	0.52	0.44	0.52	0.70	0.90
R-O	0.60	0.50	0.60	0.80	0.90
Contour listing or ridge planting (P_{cl})	0.30	0.25	0.30	0.40	0.45
Contour terracing (P_t)[2]	[3] $0.6/\sqrt{n}$	$0.5/\sqrt{n}$	$0.6/\sqrt{n}$	$0.8/\sqrt{n}$	$0.9/\sqrt{n}$
No support practice	1.0	1.0	1.0	1.0	1.0

[1] R = rowcrop, W = fall-seeded grain, O = spring-seeded grain, M = meadow. The crops are grown in rotation and so arranged on the field that rowcrop strips are always separated by a meadow or winter-grain strip.
[2] These P_t values estimate the amount of soil eroded to the terrace channels and are used for conservation planning. For prediction of off-field sediment, the P_t values are multiplied by 0.2.
[3] n = number of approximately equal-length intervals into which the field slope is divided by the terraces. Tillage operations must be parallel to the terraces.

Example: An owner/operator proposes to close one section of his small landfill with a sandy clay subsoil cover having the surface configuration shown in Figure 21. The factor R has been established as 200 for this locality. The evaluator questions anticipated erosion along the steep side and assigns the following values to the other factors in the USLE after inspecting Tables 5 through 8:

$$K = 0.14 \quad LS = 8.3 \quad C = 1.00 \quad P = 0.90$$

The rate of erosion for the steep slope of the landfill is calculated as follows:

$$A = 200\ (0.14\ \text{tons/acre})\ (8.3)\ (1.00)\ (0.90) = 209\ \text{tons/acre}$$

This erosion not only exceeds a limit recommended by the permitting authority but also indicates a potential

from reference 1

APPENDIX 11-H

Settlement Calculations

One-Dimensional Calculations

Golder Associates	SUBJECT PERMEABLE COVER – SETTLEMENT CALCULATIONS	
Job No. 893-6260	Made by RAC	Date 9/27/90
Ref.	Checked JBF	Sheet 1 of 3
	Reviewed PCR	

SETTLEMENT OF HIDE PILES

OBJECTIVE: ESTIMATE SETTLEMENTS AND MAXIMUM DIFFERENTIAL SETTLEMENTS OF HIDE PILES TO ENSURE ADEQUATE DRAINAGE GRADIENTS.

ASSUMPTIONS:

1. WORST CASE IS RELATIVELY FLAT CREST OF HIDE PILE.
2. UNIFORM STRESS INCREMENT THROUGHOUT SURFICIAL MATERIALS AND FILL AND HIDE RESIDUE. (VALID FOR WIDE LOAD)
3. OUTWASH SAND AND GLACIAL TILL INCOMPRESSIBLE.

CALCULATIONS:

MAXIMUM THICKNESS OF FILL & HIDE RESIDUE = 28 ft

MAXIMUM RATE OF CHANGE OF THICKNESS = 4 ft in 60 ft

SURFICIAL MATERIAL THICKNESS = 4 ft (max)

Section D–D' Fig 6-3

LOADING: PERMEABLE CAP = $(16\,in \times \frac{1\,ft}{12\,in}) \times 103\,pcf$ = 137 psf

BACKFILL (max) = $4\,ft \times 125\,pcf$ = 500 psf

637 psf

SETTLEMENT: $\delta = C_c H / 1+e_o \left[\log (P_o' + \Delta q) / P_o' \right]$

C_c = COMPRESSION INDEX

e_o = INITIAL VOID RATIO

H = LAYER THICKNESS

P_o' = OVERBURDEN [illegible]

Δq = STRESS INCREMENT

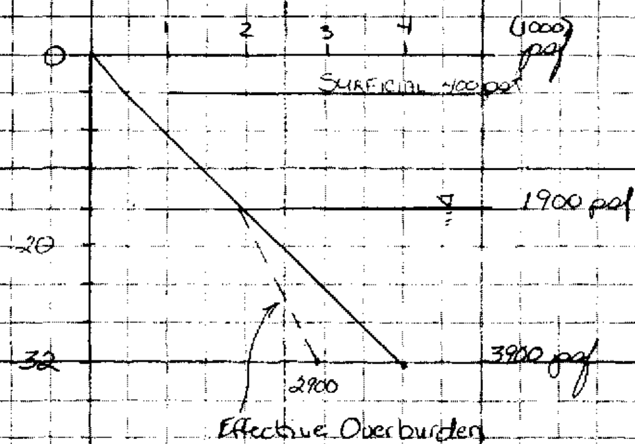
From Task S-2 Report (Golder Associates, 1990)

Surficial Materials : $C_c = 0.5$
$e_o = 1.5$

Fill & Hide Residue : $C_c = 0.3$
$e_o = 0.76 \rightarrow 1.55$ Average 1.10

Overburden Pressure

γ_T (surficial) = 100 pcf
γ_T (Hide & Fill) = 125 pcf



Case 1 : Thickness of Fill & Hide Residue = 28 ft

	C_c	H	e_o	P_o'	Δq	Δs (ft)
Surficial	0.5	4	1.5	200	637	0.50
F&H (above WT)	0.3	12	1.1	1150	637	0.33
F&H (below WT)	0.3	16	1.1	2400	637	0.23
						1.06 ft

Case 2 : Thickness of Fill & Hide Residue = 24 ft

	C_c	H	e_o	P_o'	Δq	Δs (ft)
Surficial	0.5	4	1.5	200	637	0.50
F&H (above wt)	0.3	12	1.1	1150	637	0.33
F&H (below wt)	0.3	12	1.1	2275.6	637	0.18
						1.01 ft

Golder Associates

SUBJECT Permeable Cover - Settlement Calculations		
Job No.	Made by RAC	Date 9/27/90
Ref.	Checked JBF	Sheet 3 of 3
	Reviewed PCR	

DIFFERENTIAL SETTLEMENT

1.06 - 1.01 ft = 0.05 ft in 60 ft

= 0.083 %

∴ 3% GRADES ARE NOT MATERIALLY AFFECTED BY MAX POSSIBLE DIFFERENTIAL SETTLEMENT.

Schmertmann Method

Golder Associates	SUBJECT SETTLEMENT CALCULATIONS - EPA COMMENT RESPONSE 30% DESIGN REPORT		
	Job No. 903-6400	Made by RAC	Date 1/18/91
	Ref. West Hide Pile	Checked UEE7	Sheet 1 of 16
		Reviewed [signature]	

OBJECTIVE: TO DETERMINE THE AMOUNT OF DIFFERENTIAL SETTLEMENT OF THE WEST HIDE PILE USING THE SCHMERTMANN METHOD BASED ON STANDARD PENETRATION RESISTANCE.

ASSUMPTIONS:

1) USE PDI BOREHOLES 1 AND 2 LOCATED AT THE CREST OF THE WEST HIDE PILE;

2) USE SCHMERTMANN METHOD ASSUMING AN AXISYMMETRIC LOADING.

3) USE SIEVE ANALYSIS RESULTS FROM TASK S-2 PDI INTERIM FINAL REPORT TO DETERMINE D_{50}.

4) MAXIMUM STRESS INCREMENT Δp DUE TO THE COVER AND MAXIMUM FILL THICKNESS.

REFERENCES

1) "IMPROVED STRAIN INFLUENCE FACTOR DIAGRAMS", JOHN H. SCHMERTMANN, JOHN PAUL HARTMAN AND PHILIP R. BROWN, AUGUST 1978

2) "SPT-CPT CORRELATIONS", ROBERTSON, CAMPANELLA AND WIGHTMAN, 1984.

3) "STATIC CONE TO COMPUTE STATIC SETTLEMENT OVER SAND", SCHMERTMANN, 1970

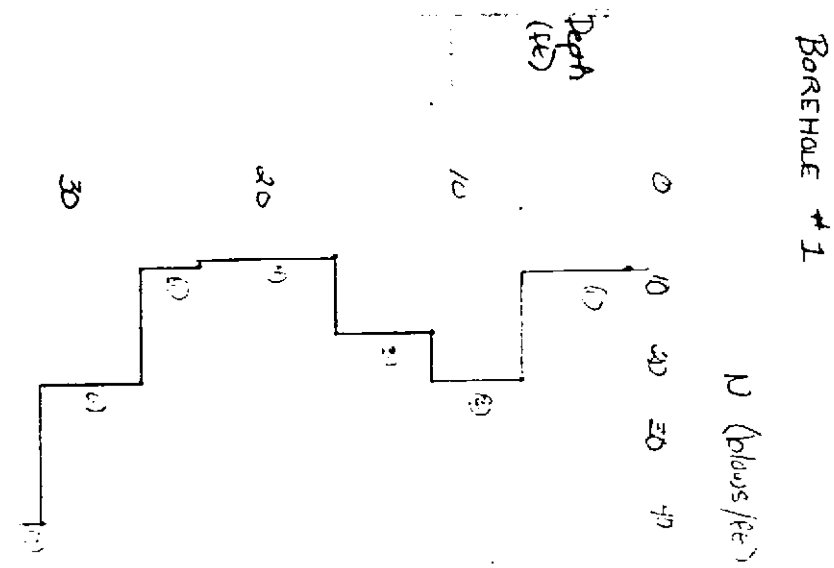
4) PRE-DESIGN INVESTIGATION, TASK S-2, STABILITY OF HIDE PILES, INTERIM FINAL REPORT, GOLDER ASSOCIATES, INC. 1990

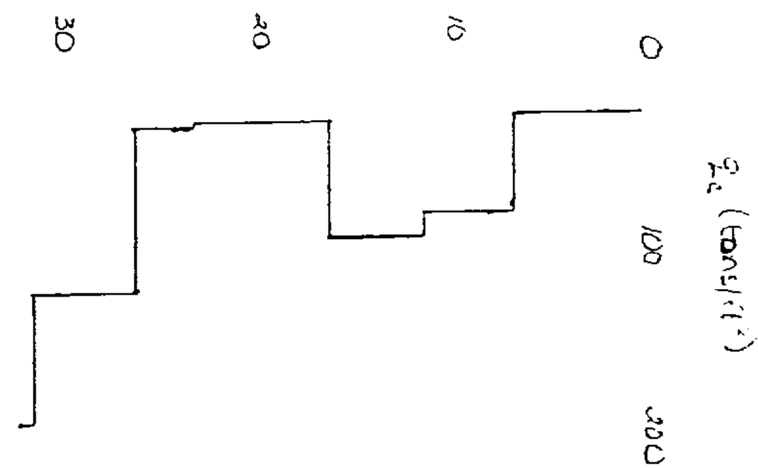
CALCULATIONS:

SEE ATTACHED PAGES.

Golder Associates

SUBJECT Settlement Calculations		
Job No. 903-6400	Made by RAC	Date 1/17/91
Ref. EPA-Comment Response	Checked VEEF	Sheet 2 of 16
	Reviewed RCR	





Golder Associates	SUBJECT SETTLEMENT CALCULATIONS		
	Job No. 903-6400	Made by RAL	Date 1/17/91
	Ref. EPA - COMMENT RESPON.	Checked RJH 0427 Reviewed [illegible]	Sheet 3 of 16

BOREHOLE # 1

LAYER	SAMPLE ID	D_{50} (mm)	N	q_c/N *	q_c (tsf)	$E = 2.5(q_c)$ ^ (tsf)
1	S-1	0.1	8	3.9	31.2	78
2	USE BULK (0-5')	0.1	22	3.9	85.8	215
3	ST-1	0.33	16	5.4	86.4	216
4	S-4	0.33	6	5.4	32.4	81
5	ST-2	0.26	7	5.2	36.4	91
6	ST-2	0.26	22	5.2	114.4	286
7	S-7	0.17	40	4.5	180	450

* Robertson and Campanella (1984)

CALCULATING I_z

$$I_{zp} = 0.5 + 0.1 \left(\frac{\Delta p}{\sigma'_{zp}}\right)^{\frac{1}{2}}$$

^ SCHMERTMANN (1978)

$$\Delta p = \gamma_{cover} \times t_{cover} + \gamma_{fill} \times t_{fill(max)}$$

$$= 103\,pcf \times \left(\frac{16''}{12''/ft}\right) + 125\,pcf \times 4\,ft$$

$$= 637\,psf$$

σ'_{zp} @ B/2

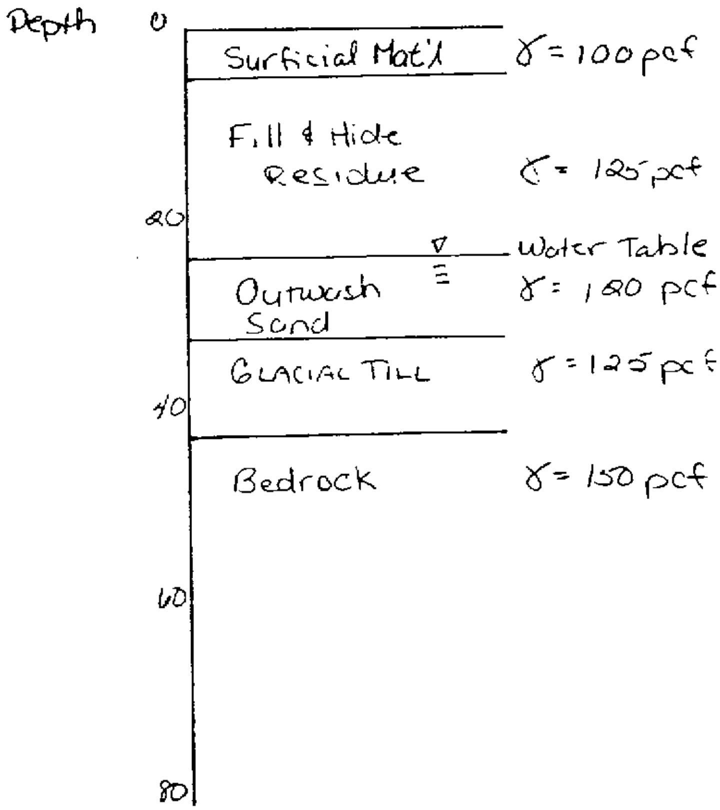
B = 315 ft (avg of length and width of crest of WHP)

B/2 = 157.5

Golder Associates

SUBJECT Settlement Calculations		
Job No. 903-6400	Made by RAC	Date 1/17/90
Ref.	Checked [illegible]	Sheet 4 of 16
	Reviewed	

BOREHOLE #1



σ'_{zp} @ B/2

$$= 100\,pcf\,(5\,ft) + 125\,pcf\,(18.7\,ft) + (120-62.4)(pcf)(8.3\,ft) + (125-62.4)(10.3\,ft) + (150-62.4\,pcf)(115.2\,ft)$$

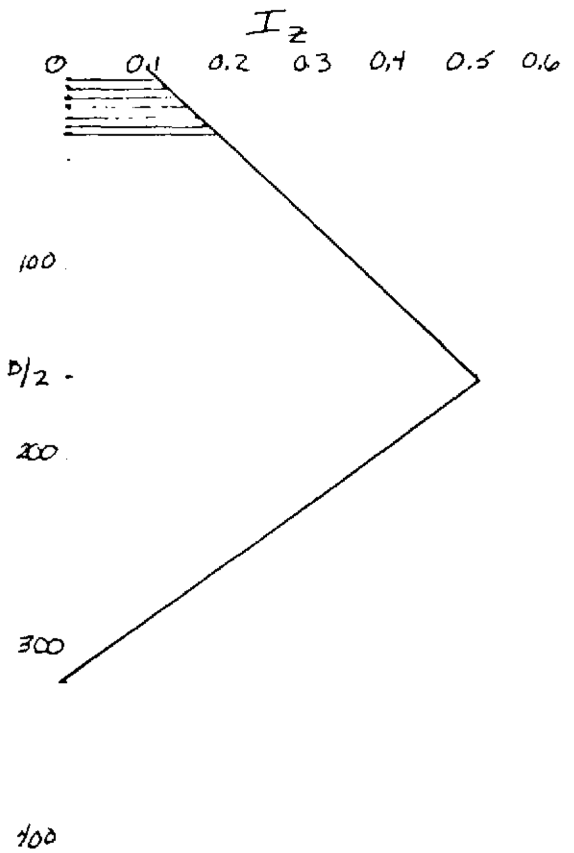
$$= 14051.8\ psf \checkmark$$

$$I_{zp} = 0.5 + 0.1\left(\frac{637}{14051.8}\right)^{1/2}$$

$$I_{zp} = 0.52 \checkmark$$

Golder Associates	SUBJECT SETTLEMENT CALCULATIONS		
	Job No. 903-6400 Ref.	Made by RAC Checked [illegible] Reviewed [illegible]	Date 1/17/91 Sheet 5 of 16

BOREHOLE #1



Layer	ΔZ (ft)	q_c (tsf)	E (tsf)	Depth of Center Below Ground Surface	I_z	$(\frac{I_z}{E})\Delta Z$
1	6.5	31	78	3.25	0.123	0.103
2	5	86	215	9	0.13	0.03
3	5	86	216	14	0.151	0.0035
4	7	32	81	20	0.166	0.014
5	3	36	91	25	0.179	0.0059
6	5	114	286	29	0.189	0.0033
7	1	180	450	32	0.197	0.00044

$\sum(\frac{I_z}{E})\Delta z = 0.0404$

$$S_d = C_1 C_2 \Delta p \Sigma \left(\frac{I_z}{E}\right)_i \Delta z_i$$ (Schmertmann, 1970)

Δp = net cover pressure

$= (103 \text{pcf})\left(\frac{16 \text{in}}{12 \text{in/ft}}\right) + (125 \text{pcf})(4 \text{ft})$
$= 637 \text{psf}$ ✓

$C_1 = 1 - 0.5\left(\frac{\sigma'_o}{\Delta p}\right) \geq 0.5$

C_1 IS AN EMBEDDMENT CORRECTION FACTOR. IT IS CONSERVATIVE TO TAKE $C_1 = 1$

$C_2 = 1 + 0.2 \log_{10}\left(\frac{t}{0.1}\right)$ let t=30 years

$C_2 = 1 + 0.2 \log_{10}\left(\frac{30}{0.1}\right)$
$= 1.5$ ✓

$S_d = (1)(1.5)(637 \text{pcf} \times \frac{100}{2000,0})[0.0404]$

$= 0.0193 \text{ft} = 0.23 \text{in}$ in 30 years

PROJECT: INDUSTRI-PLEX

PROJECT LOCATION: WOBURN

PROJECT NUMBER: 893-6255

RECORD OF BOREHOLE 1

BORING DATE: 04/23/90

BORING LOCATION: N: 564,712 E: 696,866

SHEET: 1 OF 1

DATUM: MSL

7/16

DEPTH FEET	BORING METHOD	SOIL PROFILE DESCRIPTION	USCS	GRAPHIC LOG	ELEV / DEPTH	SAMPLE NUMBER	TYPE	BLOWS / 6 in	N	REC/ATT	PIEZOMETER OR STANDPIPE INSTALLATION
0					92.30 / 0.00						
		0.00-5.0 ft. Loose, moderate brown to yellowish-orange, SILT and m-f SAND, few roots, some pockets of white clayey silt, (SM-ML). FILL.	SM-ML			1	DO	2,4,4,8	8	45	
5					87.30 / 5.00						
		5.0-23.7 ft. Loose to compact, black, m-f SAND, little to some silt, few hairs, occasional micas, some roots (SP-SM). FILL AND HIDE RESIDUE.	SP-SM			2	DO	10,13,9,9	22	30	
10						3	DO	5,8,8,9	16	80	
						ST-1	TO	SHELBY TUBE		100	
15						4	DO	2,3,3,4	6	65	
20						ST-2	TO	SHELBY TUBE		100	
						5	DO	4,3,4,7	7	95	
					68.60 / 23.70						
25		23.7-32.0 ft. Compact becoming dense, black becoming olive-gray, f-SAND, little silt, few micas (SP). OUTWASH SAND. Water encountered at approximately 23.7 ft.	SP			6	DO	6,9,13,18	22	90	
30						7	DO	13,20,20,30	40	100	
					60.30 / 32.00						
		BOREHOLE TERMINATED AT 32.0 FT. BELOW GROUND SURFACE.									
35											
40											

PENETRATION RESISTANCE BLOWS/FT ■ 10 20 30 40 50 60 70 80 90

WATER CONTENT, PERCENT Wp W Wl 10 20 30 40 50 60 70 80 90

DRILL RIG: Mobile B-57 ATV

DRILLING CONTRACTOR: Geologic

DRILLER: T. Paquette

Golder Associates

LOGGED: RJI

CHECKED: JEW

DATE: 05/29/90

F. M. Kasim, et. al. (1986)

…ic data is shown in Table 1. Impurities such as clay lumps or …ntonite "driller's mud" were removed before performing the tests. It is to be noted that from both this study and studies made for other nearby portions of the greater project site, the percentage fines for the hydraulic fill sand is typically about 10, whereas the natural sand typically has about 20% fines content.

Occasionally, blow counts were not included in the correlation be-

TABLE 2.—Penetration Data

Depth (ft)[a] (1)	Measured N[b] (2)	Corrected N (3)	q_c (kg/cm^2) (4)
(a) Test Location 1			
2	25	25	106
5	27	27	123
8	28	28	119
16.6	26	26	113
17.1	6	6	41
34.2*	10	10	40
37.2*	53	53	184
(b) Test Location 2			
2	29	29	156
5	62	62	271
8	31	31	186
11.1	18	18	108
14.1	21	21	79
17.1	8	8	38
28.6*	9	9	30
31.2*	24	24	54
34.2*	31	31	118
(c) Test Location 3			
3	39	39	152
6(1)	47	47	185
9	36	36	133
12	19	19	76
15.1(2)	7	7	38
18.1(3)	2	2	4
33.2*(4)	4	4	9
37.2*	27	27	67
(d) Test Location 4			
3	12	12	80
6	28	28	155
9	25	25	157
12(5)	19	19	99
15.1	30	30	145
29.2*(6)	26	26	66
34.2*	40	40	133
39.2*	32	32	196

Depth (ft)[a] (5)	Measured N[b] (6)	Corrected N (7)	q_c (kg/cm^2) (8)
(e) Test Location 5			
3	19	19	115
6(7)	9	9	38
35.2*	48	48	88
(f) Test Location 6			
3	31(1)	41.3	236
6	26(1)	34.7	142
9(8)	22(1)	29.3	96
33.7*	41(1)	54.7	180
(g) Test Location 7			
3	34(1)	45.3	254
6	39(1)	52	285
9	40(1)	53.3	295
12.1	47(1)	62.7	242
18.1(9)	15(1)	20	76
21.1*	5(1)	6.7	30
26.1*	18(1)	24	171
(h) Test Location 8			
2.5(10)	17(1)	22.7	153
5.5	33(1)	44	230
8.5	43(1)	57.3	231
13.6	54	54	300
17.6	12(1)	16	42
20.6*	5(1)	6.7	17
23.6*(11)	13(1)	17.3	76
36.6*	9(1)	12	241
(i) Test Location 9			
6.1	21(2)	35	119
7.6	32(2)	53.3	85
9.1	19(2)	31.7	79
10.7	6(2)	10	63
15.2	23(2)	38.3	180
17.2(12)	13(2)	21.7	116
19.2*	11(2)	18.3	47
20.7*(13)	16(2)	26.7	75
23.2*	28(2)	46.7	172
31.3*	25(2)	41.7	107
40.3(14)	10(2)	16.7	68

[a]Depth is to midpoint of SPT. An asterisk (*) next to depth denotes natural sand, otherwise material is hydraulic fill sand. A number in parentheses next to depth is the sample number for grain size analyses, which are summarized in Table 1.

[b]SPT procedures: SPT conducted with safety hammer and three wraps, N corresponds to the standard energy (56%). (1) SPT conducted with safety hammer and two wraps, N corrected to the standard donut hammer and two wraps according to Kovacs; or (2) SPT conducted using no liners for sampler to be used with liners, safety hammer and two wraps used; N value corrected according to Kovacs, et al.

Borehole #1

cause the encountered conditions were considered to in[illegible]lidate the test results, such as changes in material types discovered during the penetration test. A total of 65 test data points were selected for the correlations. A summary of the field measurements is given in Table 2.

Analysis

The correlation between q_c and N was made by averaging q_c over the same 12-in. (30-cm) length as the N values were recorded, i.e., from the 6-in. (15-cm) to the 18-in. (30-cm) penetration levels of the sampler.

The data resulting from this work was expressed in terms of q_c/N versus the mean grain diameter and was superimposed on the average curve and data points (solid circles) by Robertson and Campanella (3), as shown in Fig. 1. The data points from this work are denoted by open circles and triangles for the hydraulic and natural sands, respectively, and are indexed by q_c in kg/cm^2 (see Tables 1 and 2).

Examination of the relationship in Fig. 1 shows that although the Robertson and Campanella (3) curve represents a good average for the study site, significant scatter around the mean curve is evident. Such scatter could indicate either that correlation relationships are difficult to obtain because of inherent variabilities in both types of tests or that the effect of some other soil property that is not completely defined by the mean grain diameter. The entire data bank in Table 2 was analyzed to investigate the possible dependence of the q_c/N on the numerical value of q_c. The results were inconclusive.

Another plot was developed to evaluate the possible dependence of q_c/N on the fines content. For that purpose, we have utilized only that

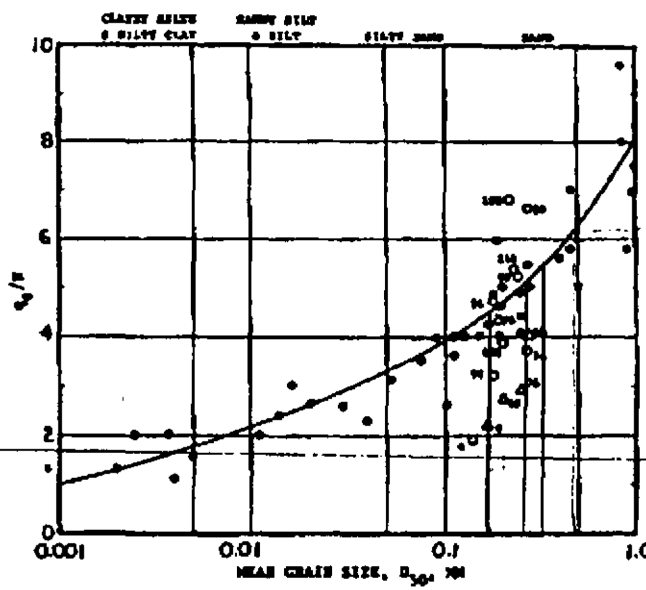
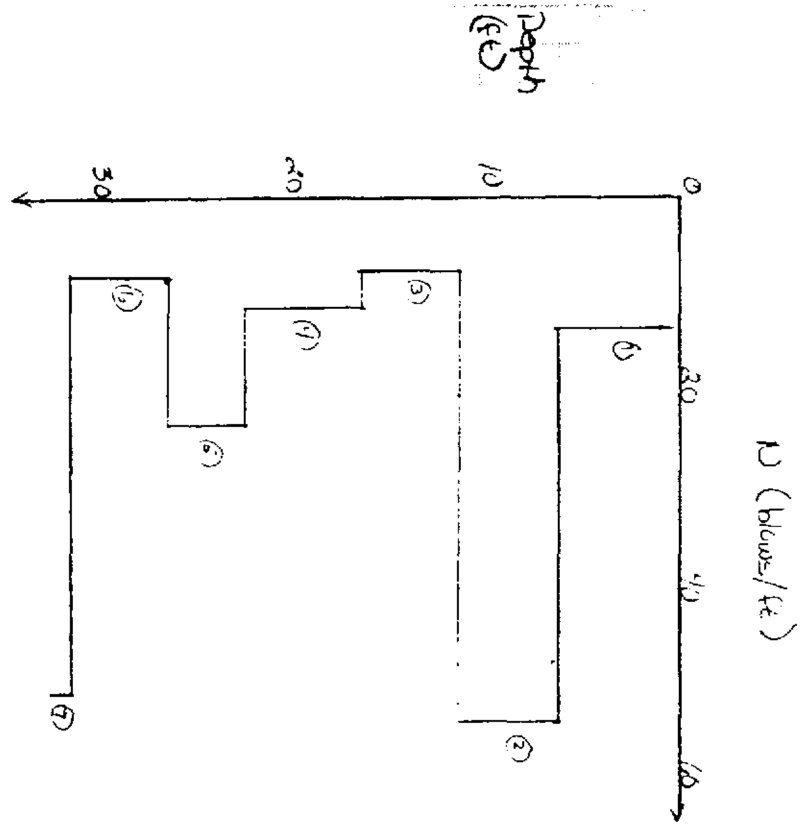


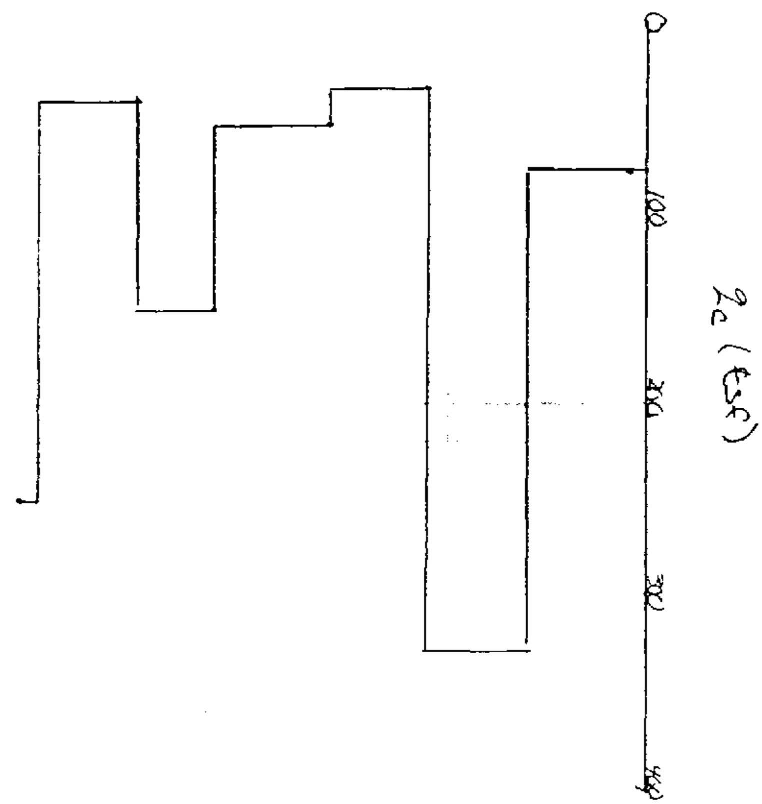
FIG. 1.—Variation of q_c/N with Mean Gra[illegible]

Golder Associates

SUBJECT Settlement Calculations		
Job No. 903-6400	Made by RAC	Date 11/7/90
Ref.	Checked VE	Sheet 9 of 16
	Reviewed	

BOREHOLE #2





Golder Associates	SUBJECT Settlement Calculations (WHP)		
	Job No. 903-6460	Made by RAC	Date 1/17/91
	Ref.	Checked VEE?	Sheet 10 of 16
		Reviewed PCR	

Borehole #2

Layer	Sample ID	D_{50} mm	N	q_c/N *	q_c (tsf)	$E = 2.5 q_c$ °
1	S-1	0.045	13	3.2	41.6	104
2	S-1	0.045	54	3.2	173	432
3	S-3	0.28	7	5.2	36.4	91
4	ST-2	0.28	11	5.2	57.2	143
5	S-5	0.6	23	6.8	156	391
6	ST-3	0.28	8	5.2	41.6	104
7	S-7	0.25	51	5	255	638

* Robertson & Companella (1984)
° Schmertmann (1978)

Calculate I_{zp} according to Schmertmann 1978

$$I_{zp} = 0.5 + 0.1\left(\frac{\Delta p}{\sigma_{zp}}\right)^{1/2}$$

$$\Delta p = \gamma_{cover} \times t_{cover} + \gamma_{FILL} \times t_{fill(max)}$$

$$= (103\,pcf)\left(\frac{16''}{12''/ft}\right) + (125\,pcf)(4\,ft)$$

$$= 637\text{ psf}$$

σ'_{zp} @ $B/2$

Assume load to be axisymmetric

$B = 315$ ft (average value of length and width of crest of WHP)

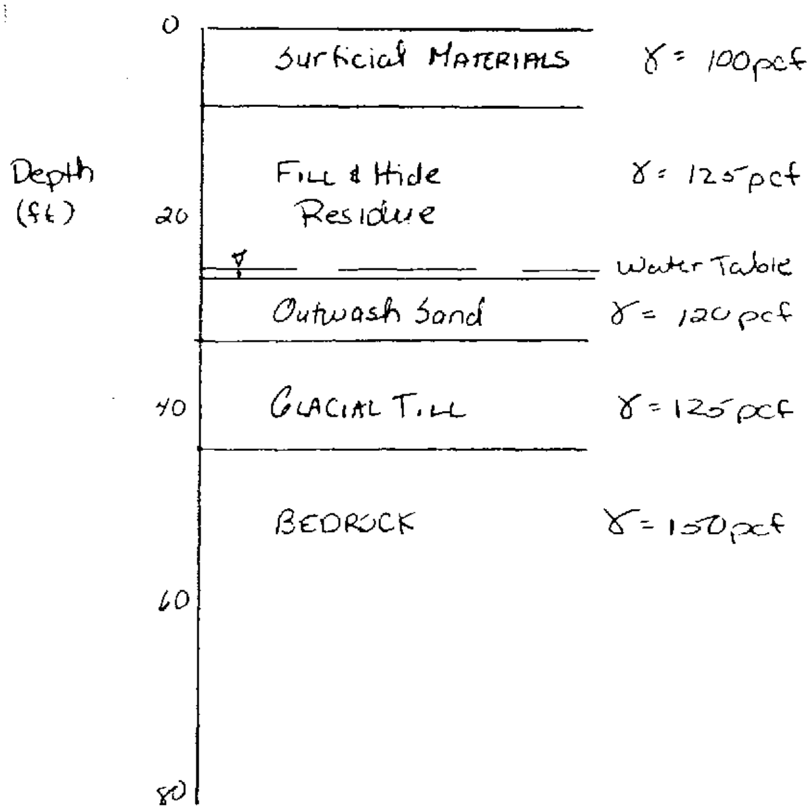
$B/2 = 157.5$ ft

Golder Associates

SUBJECT Settlement Calculations (WHP)		
Job No. 903-6400	Made by RAC	Date 1.7.91
Ref.	Checked V447	Sheet 11 of 14
	Reviewed [signature]	

Borehole #2

Assume bedrock to be @ EL 50ft



$$\sigma'_{zp} = 100pcf(7.5ft) + 125pcf(17') + (125-62.4)(1.3') + (120-62.4)(6.2') + (125-62.4pcf)(11.2ft) + (150-62.4)(114.3ft)$$

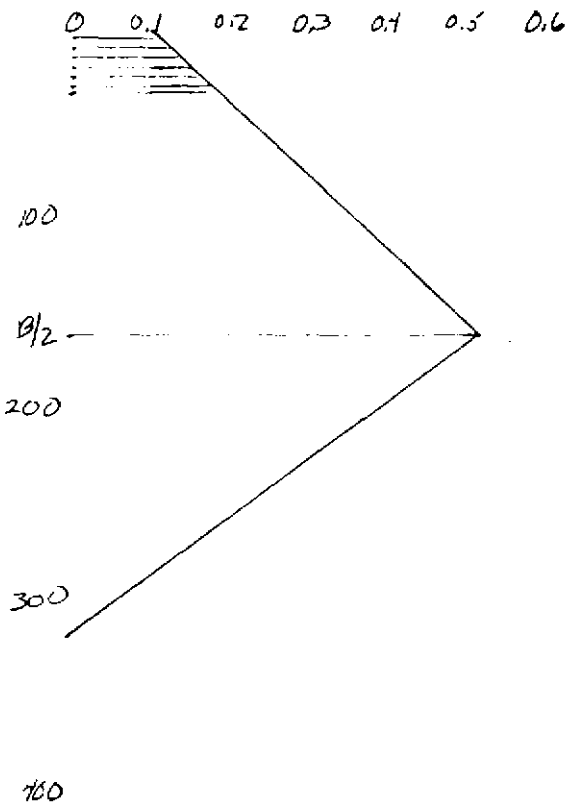
$$= 14027.3 psf$$

$$I_z = 0.5 + 0.1\left(\frac{637}{14027.3}\right)^{1/2}$$

$$= 0.52$$

Golder Associates	SUBJECT Settlement Calculations		
	Job No. 903-6400	Made by RAC	Date 1/17/91
	Ref.	Checked VEE7	Sheet 12 of 14
		Reviewed PLR	

Borehole #2



Layer	Δz	q_c (tsf)	E (tsf)	Depth of Center Below Ground Surface	I_z	$\left(\frac{I_z}{E}\right)\Delta z$
1	6	42	104	3	0.11	0.0063
2	5	173	432	8.5	0.13	0.0015
3	5	36	91	13.5	0.14	0.0077
4	6	57	143	19	0.15	0.0063
5	4	156	391	24	0.164	0.0017
6	5	42	104	28.5	0.176	0.00[illegible]
7	1	255	638	31.5	0.184	0.0003

$\Sigma\left(\frac{I_z}{E}\right)\Delta z = 0.0323$

Golder Associates	SUBJECT Settlement Calculations		
	Job No. 903-6400 Ref.	Made by RAC Checked DEE7 Reviewed PCR	Date 1/17/91 Sheet 13 of 14

BOREHOLE #2

$$S_d = C_1 C_2 \Delta p \Sigma \left(\frac{I_z}{E}\right)_i z_i \qquad \text{Schmertmann (1970)}$$

$$C_1 = 1 - 0.5\left(\frac{\sigma_0'}{\Delta p}\right) \geq 0.5$$

C_1 IS AN EMBEDDMENT CORRECTION FACTOR. IT IS CONSERVATIVE TO TAKE $C_1 = 1$

$$C_2 = 1 + 0.2 \log_{10}\left(\frac{t}{0.1}\right) \qquad t = 30 \text{ years}$$

$$= 1 + 0.2 \log\left(\frac{30}{0.1}\right)$$

$$= 1.5$$

$$S_d = (1)(1.5)(637 \text{psf} \times \frac{100}{400000}) (0.0323)$$

$$= 0.015 \text{ ft} = 0.18 \text{ in} \quad \text{in 30 years}$$

PROJECT: INDUSTRI-PLEX

RECORD OF BOREHOLE 2

SHEET: 1 OF 1

PROJECT LOCATION: WOBURN

BORING DATE: 04/22/90

DATUM: MSL

PROJECT NUMBER: 893-6255

BORING LOCATION: N: 554,550
E: 696,759

14/16

SOIL PROFILE

DEPTH SCALE FEET	DESCRIPTION	USCS	ELEV / DEPTH
0			93.20 / 0.00
0–7.5	0.00-7.5 ft. Firm to very dense, moderate brown to dark gray, SILT, some f-sand, slight organic odor. Frequent pockets of white clayey silt, roots present, (ML). FILL.	ML	85.70 / 7.50
7.5–25.8	7.5-25.8 ft. Loose to compact, black, f-m SAND, some silt, hair and hair clumps, occasional roots, frequent pockets of white clayey silt, few micas, (SM). FILL AND HIDE RESIDUE. Water at approximately 24.8 ft. at completion of drilling.	SM	67.40 / 25.80
25.8–32.0	25.8-32.0 ft. Loose becoming very dense, black to light olive gray, m-f SAND, trace silt, few roots (SP). OUTWASH SAND.	SP	61.20 / 32.00
32	BORING TERMINATED AT 32 FT. BELOW GROUND SURFACE.		

SAMPLES

NUMBER	TYPE	BLOWS / 6 in	N	REC/ATT
1	DO	1,6,7,10	13	100
2	DO	10,26,28,26	54	100
ST-1	TO	SHELBY TUBE		85
3	DO	5,2,5,8	7	75
4	DO	4,3,8,5	11	40
ST-2	TO	SHELBY TUBE		85
5	DO	4,15,8,9	23	65
6	DO	3,5,3,3	8	100
ST-3	TO	SHELBY TUBE		100
7	DO	10,19,32,46	51	100

PENETRATION RESISTANCE BLOWS/FT ■ 10 20 30 40 50 60 70 80 90

WATER CONTENT, PERCENT Wp W Wl 10 20 30 40 50 60 70 80 90

PIEZOMETER OR STANDPIPE INSTALLATION

DRILL RIG: Mobile B-57 ATV
DRILLING CONTRACTOR: Geologic
DRILLER: T. Paquette

LOGGED: RJI
CHECKED: JEW
DATE: 05/29/90

Golder Associates

FROM NASIM, et al.

…asic data is shown in Table 1. Impurities such as clay lumps or …ntonite "driller's mud" were removed before performing the tests. It is to be noted that from both this study and studies made for other nearby portions of the greater project site, the percentage fines for the hydraulic fill sand is typically about 10, whereas the natural sand typically has about 20% fines content.

Occasionally, blow counts were not included in the correlation because the encountered conditions were considered to invalidate the test results, such as changes in material types discovered during the penetration test. A total of 65 test data points were selected for the correlations. A summary of the field measurements is given in Table 2.

TABLE 2.—Penetration Data

Depth (ft)[a] (1)	Measured N[b] (2)	Corrected N (3)	q_c (kg/cm^2) (4)
(a) Test Location 1			
3	25	25	106
5	27	27	123
8	28	28	119
14.6	26	26	113
17.1	6	6	41
34.7*	10	10	46
37.7*	53	53	184
(b) Test Location 2			
2	29	29	156
5	62	42	271
8	31	31	186
11.1	18	18	108
14.1	21	21	79
17.1	8	8	50
20.6*	9	9	30
31.2*	24	24	56
34.2*	31	31	118
(c) Test Location 3			
3	39	39	152
6(1)	47	47	185
9	36	36	133
12	19	19	76
15.1(2)	7	7	34
18.1(3)	2	2	4
33.2*(4)	4	4	9
37.2*	27	27	67
(d) Test Location 4			
3	12	12	48
6	28	28	155
9	25	25	157
12(5)	19	19	99
15.1	30	30	165
29.2*(6)	26	26	66
34.7*	40	40	133
38.2*	32	32	196

Depth (ft)[a] (5)	Measured N[b] (6)	Corrected N (7)	q_c (kg/cm^2) (8)
(e) Test Location 5			
3	19	19	115
6(7)	9	9	38
35.2*	40	40	88
(f) Test Location 6			
3	31(1)	41.3	236
6	26(1)	34.7	142
9(8)	22(1)	29.3	96
33.7*	43(1)	56.7	188
(g) Test Location 7			
3	34(1)	45.3	254
6	39(1)	52	265
9	40(1)	53.3	295
12.1	47(1)	62.7	242
18.1(9)	15(1)	20	74
21.1*	5(1)	6.7	33
26.1*	18(1)	24	171
(h) Test Location 8			
2.5(10)	17(1)	22.7	153
5.5	33(1)	44	230
8.5	43(1)	57.3	231
13.6	56	56	300
17.6	12(1)	16	42
20.6*	5(1)	6.7	17
23.6*(11)	13(1)	17.3	76
26.6*	9(1)	12	241
(i) Test Location 9			
6.1	21(2)	35	119
7.6	32(2)	53.3	85
9.1	19(2)	31.7	79
10.7	6(2)	10	63
15.2	23(2)	38.3	180
17.2(12)	13(2)	21.7	116
19.2*	11(2)	18.3	47
20.7*(13)	16(2)	26.7	75
23.2*	28(2)	46.7	172
31.3*	25(2)	41.7	107
40.3(14)	10(2)	16.7	68

[a]Depth is to midpoint of SPT. An asterisk (*) next to depth denotes natural sand, otherwise material is hydraulic fill sand. A number in parentheses next to depth is the sample number for grain size analyses, which are summarized in Table 1.

[b]SPT procedure: SPT conducted with safety hammer and three wraps, N corresponds to the standard energy (56%). (1) SPT conducted with safety hammer and two wraps, N corrected to the standard donut hammer and two wraps according to Kovacs; or (2) SPT conducted using no liners for sampler to be used with liner, safety hammer and two wraps used; N value corrected according to Kovacs, et al.

Analysis

The correlation between q_c and N was made by averaging q_c over the same 12-in. (30-cm) length as the N values were recorded, i.e., from the 6-in. (15-cm) to the 18-in. (30-cm) penetration levels of the sampler.

The data resulting from this work was expressed in terms of q_c/N versus the mean grain diameter and was superimposed on the average curve and data points (solid circles) by Robertson and Campanella (3), as shown in Fig. 1. The data points from this work are denoted by open circles and triangles for the hydraulic and natural sands, respectively, and are indexed by q_c in kg/cm^2 (see Tables 1 and 2).

Examination of the relationship in Fig. 1 shows that although the Robertson and Campanella (3) curve represents a good average for the study site, significant scatter around the mean curve is evident. Such scatter could indicate either that correlation relationships are difficult to obtain because of inherent variabilities in both types of tests or that the effect of some other soil property that is not completely defined by the mean grain diameter. The entire data bank in Table 2 was analyzed to investigate the possible dependence of the q_c/N on the numerical value of q_c. The results were inconclusive.

Another plot was developed to evaluate the possible dependence of q_c/N on the fines content. For that purpose, we have utilized only that

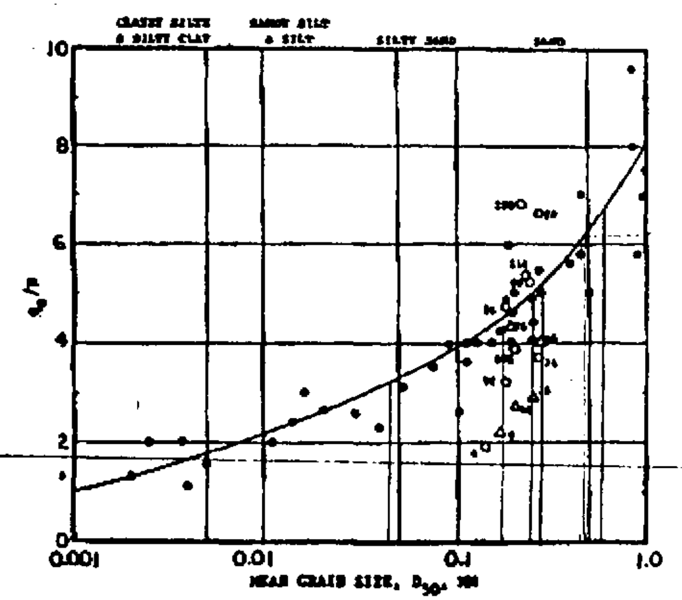


FIG. 1.—Variation of q_c/N with Mean Grain Size

Golder Associates	SUBJECT Settlement Calculations (WHP)		
	Job No. 903-6700	Made by RAC	Date 1/17/91
	Ref.	Checked UYE7	Sheet 16 of 16
		Reviewed RCR	

RESULTS:

TOTAL DIFFERENTIAL SETTLEMENT BETWEEN BH#1 AND BH#2 OVER 30 YEARS

$$0.0193\ \text{ft} - 0.015\ \text{ft} = 0.0043\ \text{ft}$$

DISTANCE BETWEEN BH#1 AND BH#2 = 194 ft

Percent Differential Settlement

$$= 0.0043\ \text{ft} / 194\ \text{ft} = 0.0022\ \%$$

APPENDIX 11-I

Equivalent Cover Pavement Rating

Golder Associates	SUBJECT ISRT- COVER EQUIVALENT PAVEMENT RATING		
	Job No. 903-6400	Made by PCN	Date 7/15/91
	Ref.	Checked RJD	Sheet 1 of 18
		Reviewed pcn	

OBJECTIVE: CONDUCT FIELD EVALUATION OF PAVEMENT CONDITION, AND POSSIBLE NEED FOR MAINTENANCE OR REPAIR.

METHOD:
- RATING BASED ON METHODS OUTLINED IN THE FOLLOWING ASPHALT INSTITUTE PUBLICATIONS

  IS-169 "A PAVEMENT RATING SYSTEM FOR LOW VOLUME ASPHALT ROADS"

  [illegible]-15 "MAINTENANCE AND REPAIR OF ASPHALT PARKING LOTS"

- RATINGS COMPLETED DURING MARCH AND JULY, 991 SITE VISITS.

ASSUMPTIONS:

1) CONSIDER AREAS OF PAVEMENT ONLY WITHIN THE PERMEABLE COVER.

2) MODIFY IS-169 SYSTEM TO ACCOUNT FOR PARKING LOT USAGE RATHER THAN ROADWAY. MODIFICATIONS AS FOLLOW:

   a) COMBINE DEFECTS FOR LOW VOLUME ASPHALT ROADS INTO ONE CATEGORY FOR PARKING LOTS.

   (LINEAR CRACKING = TRANSVERSE & LONGITUDINAL CRACKING)

   b) ELIMINATE DEFECTS USED FOR LOW VOLUME ASPHALT ROADS FROM PARKING LOT RATING SYSTEM

   (OVERALL RIDING QUALITY)

   c) DECREASE D[illegible] [illegible] THE [illegible] [illegible] [illegible] [illegible] PARKING LOT.

ASSUMPTIONS (CONTINUED)

d) SPECIFIC COMBINATIONS OF DEFECTS USED IN REVISED RATING SYSTEM

LOW VOLUME PAVEMENT COMBINED DEFECT RATING.		PARKING LOT REVISED DEFECT RATING
TRANSVERSE & LONGITUDINAL CRACKING	=	LINEAR CRACKING
ALLIGATOR & SHRINKAGE	=	ALLIGATOR.
CORRUGATIONS & SHOVING	=	UPHEAVAL
POTHOLES & RIDING QUALITY	=	POT HOLES.
RAVELING & EXCESS ASPHALT & POLISHED AGGRAGATE	=	RAVELING.
RUTTING & DEFICIENT DRAINAGE	=	GRADE DEPRESSION

e) DUE TO ADJUSTMENTS IN SCALE UNITS FOR ALLIGATOR CRACKING AND POTHOLES, THE COMBINED VALUES FOR THE DEFECT RATING WERE ADJUSTED. ALLIGATOR CRACKING DEFECT RATING VALUES WERE INCREASED BY 1.25 AND THE POTHOLE RATING VALUES WERE DOUBLED.

ASPHALT INSTITUTE RATING FORM IS SHOWN ON PAGE 3 FOR INFORMATION. REVISED RATING PRESENTED ON PAGES 4 - 18.

2/18.

ASPHALT PAVEMENT RATING FORM

STREET OR ROUTE ____________________ CITY OR COUNTY ____________

LENGTH OF PROJECT ____________________ WIDTH ____________________

PAVEMENT TYPE ____________________ DATE ____________________

(Note: A rating of "0" indicates defect does not occur)

DEFECTS		RATING
Transverse Cracks	0-5	______
Longitudinal Cracks	0-5	______
Alligator Cracks	0-10	______
Shrinkage Cracks	0-5	______
Rutting	0-10	______
Corrugations	0-5	______
Raveling	0-5	______
Shoving or Pushing	0-10	______
Pot Holes	0-10	______
Excess Asphalt	0-10	______
Polished Aggregate	0-5	______
Deficient Drainage	0-10	______
Overall Riding Quality (0 is excellent; 10 is very poor)	0-10	______
	Sum of Defects	______

Condition Rating = 100 – Sum of Defects
= 100 – ____________

Condition Rating = $\boxed{}$

Figure 1. Asphalt pavement rating form.

4/10

ASPHALT PARKING LOT RATING SYSTEM

[Revised from Asphalt Institute (IS–169) and (CL–15)]

Location	10 Atlantic Ave.	City	Woburn, MA
Area of Coverage	14, 764 ft^2	Owner	Atlantic Ave. Trust
Pavement Type	AC	Date	3/3/91

Referred to Photos 6, limited areas of exposed cover

(Note A rating of "0" indicates defect does not occur)

DEFECTS		RATING
Lineal Cracking	0–10	5
Alligator Cracks	0–20	5
Upheaval	0–20	2
Pot Holes	0–10	0
Raveling	0–20	7
Grade Depressions	0–20	3
	Sum of Defects	22

Condition Rating = 100 – Sum of Defects

= 100 – 22

Condition Rating $\boxed{78}$

5/18

ASPHALT PARKING LOT RATING SYSTEM

[Revised from Asphalt Institute (IS-169) and (CL-15)]

Location	15 Atlantic Ave.	City	Woburn, MA
Area of Coverage	150 ft^2	Owner	Atlantic Ave. Assoc.
Pavement Type	AC	Date	3/3/91

(Note A rating of "0" indicates defect does not occur)

DEFECTS		RATING
Lineal Cracking	0-10	0
Alligator Cracks	0-20	0
Upheaval	0-20	0
Pot Holes	0-10	0
Raveling	0-20	3
Grade Depressions	0-20	1
	Sum of Defects	4

Condition Rating = 100 - Sum of Defects

= 100 - 4

Condition Rating $\boxed{96}$

6/18

ASPHALT PARKING LOT RATING SYSTEM

[Revised from Asphalt Institute (IS-169) and (CL-15)]

Location	20 Atlantic Ave.	City	Woburn, MA
Area of Coverage	78,430 ft^2	Owner	Winter Hill Store House
Pavement Type	AC	Date	3/3/91

(Note A rating of "0" indicates defect does not occur)

DEFECTS		RATING
Lineal Cracking	0–10	4
Alligator Cracks	0–20	8
Upheaval	0–20	2
Pot Holes	0–10	2
Raveling	0–20	6
Grade Depressions	0–20	4
	Sum of Defects	26

Condition Rating = 100 – Sum of Defects

= 100 – 26

Condition Rating $\boxed{74}$

7/10

ASPHALT PARKING LOT RATING SYSTEM

[Revised from Asphalt Institute (IS-169) and (CL-15)]

Location	130 Commerce Wy	City	Woburn, MA
Area of Coverage	27,203	Owner	Sunder & Hiro Gauglani
Pavement Type	AC	Date	3/3/91

(Note A rating of "0" indicates defect does not occur)

DEFECTS		RATING
Lineal Cracking	0–10	3
Alligator Cracks	0–20	6
Upheaval	0–20	1
Pot Holes	0–10	3
Raveling	0–20	4
Grade Depressions	0–20	7
	Sum of Defects	24

Condition Rating = 100 – Sum of Defects

= 100 – 24

Condition Rating $\boxed{76}$

8/18

ASPHALT PARKING LOT RATING SYSTEM

[Revised from Asphalt Institute (IS-169) and (CL-15)]

Location	210 New Barton	City	Woburn, MA
Area of Coverage	30,565 ft²	Owner	Pebco Co.
Pavement Type	AC	Date	2/18/91

(Note A rating of "0" indicates defect does not occur)

DEFECTS		RATING
Lineal Cracking	0–10	4
Alligator Cracks	0–20	4
Upheaval	0–20	2
Pot Holes	0–10	2
Raveling	0–20	2
Grade Depressions	0–20	3
	Sum of Defects	17

Condition Rating = 100 – Sum of Defects

= 100 – 17

Condition Rating $\boxed{83}$

9/18

ASPHALT PARKING LOT RATING SYSTEM

[Revised from Asphalt Institute (IS-169) and (CL-15)]

Location	211 New Boston	City	Woburn, MA
Area of Coverage	1104 ft^2	Owner	Dagata
Pavement Type	AC	Date	3/5/91

(Note A rating of "0" indicates defect does not occur)

DEFECTS		RATING
Lineal Cracking	0-10	5
Alligator Cracks	0-20	5
Upheaval	0-20	1
Pot Holes	0-10	5
Raveling	0-20	5
Grade Depressions	0-20	2
	Sum of Defects	23

Condition Rating = 100 - Sum of Defects

= 100 - 23

Condition Rating $\boxed{77}$

10/10

ASPHALT PARKING LOT RATING SYSTEM

[Revised from Asphalt Institute (IS–169) and (CL–15)]

Location	216 New Boston	City	Woburn
Area of Coverage	88,757 ft²	Owner	PX Realty
Pavement Type	AC	Date	2/18/91

(Note A rating of "0" indicates defect does not occur)

DEFECTS		RATING
Lineal Cracking	0–10	3
Alligator Cracks	0–20	3
Upheaval	0–20	1
Pot Holes	0–10	2
Raveling	0–20	2
Grade Depressions	0–20	2
	Sum of Defects	13

Condition Rating = 100 – Sum of Defects

= 100 – 13

Condition Rating 87

11/18

ASPHALT PARKING LOT RATING SYSTEM

[Revised from Asphalt Institute (IS–169) and (CL–15)]

Location	217 New Boston	City	Woburn, MA
Area of Coverage	18,406 ft²	Owner	J. Koster
Pavement Type	AC	Date	2/18/91

(Note A rating of "0" indicates defect does not occur)

DEFECTS			RATING
Lineal Cracking	(1+1)/2	0–10	1
Alligator Cracks	(0+2)/2 * 1.25	0–20	1
Upheaval	(3+4)/2	0–20	4
Pot Holes	[(1+1)/2] /2	0–10	1
Raveling	(2+2)/2	0–20	2
Grade Depressions	(1+2)/2	0–20	2
		Sum of Defects	11

Condition Rating = 100 – Sum of Defects

= 100 – 11

Condition Rating $\boxed{89}$

12/18

ASPHALT PARKING LOT RATING SYSTEM

[Revised from Asphalt Institute (IS-169) and (CL-15)]

Location	219 New Boston	City	Woburn, MA
Area of Coverage	5,134	Owner	J. Koster
Pavement Type	AC	Date	2/18/91

(Note A rating of "0" indicates defect does not occur)

DEFECTS			RATING
Lineal Cracking	(1+2)/2	0-10	2
Alligator Cracks	(1+1)/2 * 1.25	0-20	1
Upheaval	(2+1)/2	0-20	2
Pot Holes	[(2+1)/2]/2	0-10	1
Raveling	(1+1)/2	0-20	1
Grade Depressions	(3+2)/2	0-20	3
		Sum of Defects	11

Condition Rating = 100 - Sum of Defects

= 100 - 11

Condition Rating $\boxed{89}$

3/19

ASPHALT PARKING LOT RATING SYSTEM

[Revised from Asphalt Institute (IS-169) and (CL-15)]

Location	223 New Boston	City	Woburn, MA
Area of Coverage	10,230 ft^2	Owner	Aero Realty
Pavement Type	AC	Date	2/18/91

(Note A rating of "0" indicates defect does not occur)

DEFECTS		RATING
Lineal Cracking	0–10	4
Alligator Cracks	0–20	6
Upheaval	0–20	2
Pot Holes	0–10	3
Raveling	0–20	3
Grade Depressions	0–20	1
	Sum of Defects	19

Condition Rating = 100 – Sum of Defects

= 100 – 19

Condition Rating $\boxed{81}$

14/10

ASPHALT PARKING LOT RATING SYSTEM

[Revised from Asphalt Institute (IS-169) and (CL-15)]

Location	225/227 New Boston	City	Woburn, MA
Area of Coverage	18 194	Owner	J. Koster
Pavement Type	AC	Date	3/5/91

(Note A rating of "0" indicates defect does not occur)

DEFECTS			RATING
Lineal Cracking	(2+1)/2	0–10	2
Alligator Cracks	[(2+1)/2] * 1.25	0–20	3
Upheaval	(1+1)/2	0–20	1
Pot Holes	[(1+0)/2] /2	0–10	1
Raveling	(2+) / 2	0–20	3
Grade Depressions	(2+2)/2	0–20	2
		Sum of Defects	12

Condition Rating = 100 - Sum of Defects

= 100 - 12

Condition Rating $\boxed{88}$

ASPHALT PARKING LOT RATING SYSTEM

[Revised from Asphalt Institute (IS-169) and (CL-15)]

Location	229/231 New Boston	City	Woburn, MA
Area of Coverage	14,183 ft^2	Owner	J. Koster
Pavement Type	AC	Date	2/5/91

(Note A rating of "0" indicates defect does not occur)

DEFECTS		RATING
Lineal Cracking	0–10	2
Alligator Cracks	0–20	1
Upheaval	0–20	2
Pot Holes	0–10	1
Raveling	0–20	3
Grade Depressions	0–20	3
	Sum of Defects	12

Condition Rating = 100 – Sum of Defects

= 100 – 12

Condition Rating $\boxed{88}$

16/160

ASPHALT PARKING LOT RATING SYSTEM

[Revised from Asphalt Institute (IS–169) and (CL–15)]

Location	204 Merrimac	City	Woburn, MA
Area of Coverage	42,400	Owner	BFI
Pavement Type	AC	Date	2/27/91

Referred to Photos for limited areas of [illegible]

(Note A rating of "0" indicates defect does not occur)

DEFECTS		RATING
Lineal Cracking	0–10	2
Alligator Cracks	0–20	1
Upheaval	0–20	1
Pot Holes	0–10	0
Raveling	0–20	1
Grade Depressions	0–20	1
	Sum of Defects	6

Condition Rating = 100 – Sum of Defects

= 100 – 6

Condition Rating $\boxed{94}$

17/18

ASPHALT PARKING LOT RATING SYSTEM

[Revised from Asphalt Institute (IS-169) and (CL-15)]

Location	225 Merrimac	City	Woburn, MA
Area of Coverage	45101	Owner	PX Realty
Pavement Type	AC	Date	3/9/91

(Note A rating of "0" indicates defect does not occur)

DEFECTS		RATING
Lineal Cracking	0–10	4
Alligator Cracks	0–20	5
Upheaval	0–20	5
Pot Holes	0–10	1
Raveling	0–20	0
Grade Depressions	0–20	4
	Sum of Defects	25

Condition Rating = 100 – Sum of Defects

= 100 – [illegible]

Condition Rating [illegible]

18/16.

ASPHALT PARKING LOT RATING SYSTEM

[Revised from Asphalt Institute (IS-169) and (CL-15)]

Location	Atlantic Ave.	City	Woburn, MA
Area of Coverage	2,312	Owner	Kennedia(?) Trust
Pavement Type	AC	Date	7/15/91

(Note A rating of "0" indicates defect does not occur)

DEFECTS		RATING
Lineal Cracking	0-10	4
Alligator Cracks	0-20	4
Upheaval	0-20	0
Pot Holes	0-10	1
Raveling	0-20	1
Grade Depressions	0-20	0
	Sum of Defects	10

Condition Rating = 100 - Sum of Defects

= 100 - 10

Condition Rating: 90

APPENDIX 11-J

Evaluation of Geotextile Clogging Potential

Golder Associates	SUBJECT ISRF - CLOGGING POTENTIAL OF GEOTEX		
	Job No. 903-6400 Ref.	Made by RJD Checked [illegible] Reviewed [illegible]	Date 8-2-91 Sheet 1 of 5

OBJECTIVE: EVALUATE GEOTEXTILE CLOGGING POTENTIAL OF SOILS USED IN INTERFACE FRICTION TESTING.

METHOD: APPLY 3 METHODS AS OUTLINED IN KOERNER, 1990, DESIGNING WITH GEOSYNTHETICS 2 ED pgs 121-122.

ALL METHODS BASED ON COMPARISON OF GEOTEXTILE APPARENT OPENING SIZE (A.O.S.) 5% RETAINED SIZE OF GEOTEXTILE. (O_{95})

ASSUMPTIONS: A.O.S = #70 OR #100 BASED UPON REVIEW OF GEOTECHNICAL FABRICS REPORT, 1991 SPECIFIER'S GUIDE, DEC. 1990.

CALCULATIONS:

GRAIN SIZE CURVES ON SHEETS 4 & 5

METHOD 1: ASTM TASK FORCE 25

BASED ON % PASSING 200 SIEVE,

HUDSARDSTON 2%

QUINN PRAIRIE 1%

SOIL ≤ 50% PASSING #200

AOS ≥ #60 SIEVE.

∴ #70 OR #100 > #60 BOTH SOILS MEET CRITERIA.

Golder Associates	SUBJECT CLOGGING POTENTIAL OF GEOTEXTILES		
	Job No. 903-6400 Ref.	Made by RJD Checked JAC Reviewed KR	Date 8-2-91 Sheet 2 of 5

METHOD 2: CARROLL METHOD

$$O_{95} < (2 \text{ or } 3) d_{85}$$

O_{95} = #70 = .212 mm

O_{95} = #100 = .150 mm.

HUBBARDSTON d_{85} = 1.7 mm.

1.7 (2 or 3) > .212 or .150 MEETS CRITERIA

QUINN PERKINS d_{85} = 2.3 mm.

2.3 (2 or 3) > .212 or .150 MEETS CRITERIA

METHOD 3: GIROUD METHOD.

BASED ON CRITERIA OF DENSITY IN PLACE.

AND UNIFORMITY COEFF $CU = d_{60}/d_{10}$

ASSUMING INTERMEDIATE DENSITY 50% < D_R < 80%

FOR $1 < CU < 3$: $O_{95} < 1.5 (CU)(d_{50})$

FOR $CU > 3$: $O_{95} < (13.5 d_{50})/CU$

ASSUMING INTERMEDIATE DENSITY D_r ~ 60%

HUBBARSTON. $d_{10} = 0.17$ mm $d_{50} = 0.5$ mm $d_{60} = 0.65$ mm.

$C_U = 0.65/0.17 = 3.8$

.150 or .212 < 13.5 (0.5) / 3.8.

.150 or .212 < [illegible] MEETS CRITERIA,

Golder Associates			
SUBJECT CLOGGING POTENTIAL OF GEOTEXTILES			
Job No. 903-6400	Made by RJD	Date 8-2-91	
Ref.	Checked JJC	Sheet 3	of 5
	Reviewed JCR		

QUINN PERKINS

$d_{10} = 0.2mm \quad d_{50} = 0.7mm \quad d_{60} = 0.95mm.$

$CU = 0.95/0.2 = 4.75$

$.150 \text{ or } .212 < 13.5(0.7)/4.75$

$.150 \text{ or } .212 < 1.98$ MEET CRITERIA

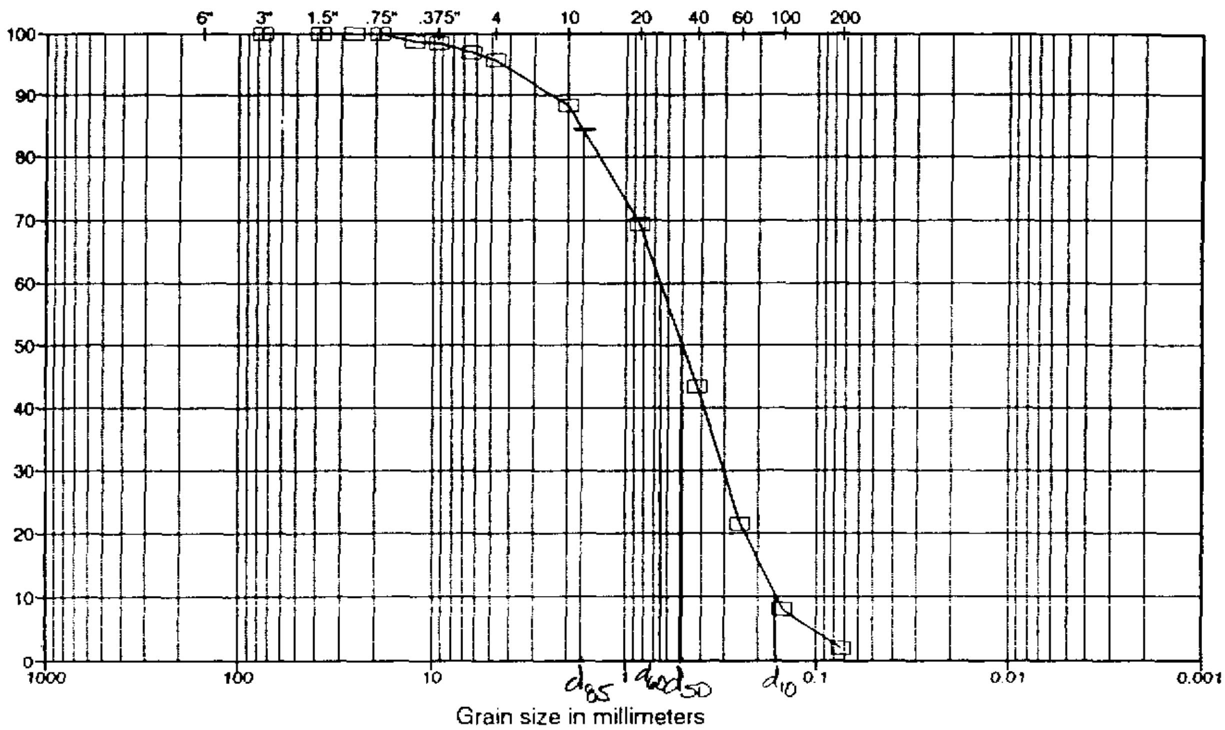
CONCLUSION: BOTH HUBBARDSTON SAND AND QUINN PERKINS CONCRETE SAND EASILY MEET ALL CLOGGING CRITERIA.

CLOGGING IS NOT A PROBLEM FOR TYPICAL GEOTEXTILES.

RJD
8/2/91
4/5

PARTICLE SIZE DISTRIBUTION ASTM D-421 AND 422

US STANDARD SIEVE OPENING SIZES



USCS

COBBLES	GRAVEL		SAND			FINES (Silt or Clay)
	Coarse	Fine	C	Med	Fine	

TECH: DL
DATE: 7/18/91
CHECKED: RMW
REVIEWED: RJC

SAMPLE ID	W%	LL	PL	PI	Other	DESCRIPTION
HUBBARDSTON SAND	1.69					Dark yellowish orange m-f SAND, trace f gravel trace fines
Sample Type: BULK		Date Tested: 7/17/91			USCS:	

INDUSTRI-PLEX/WOBURN/MA
903-6400.110

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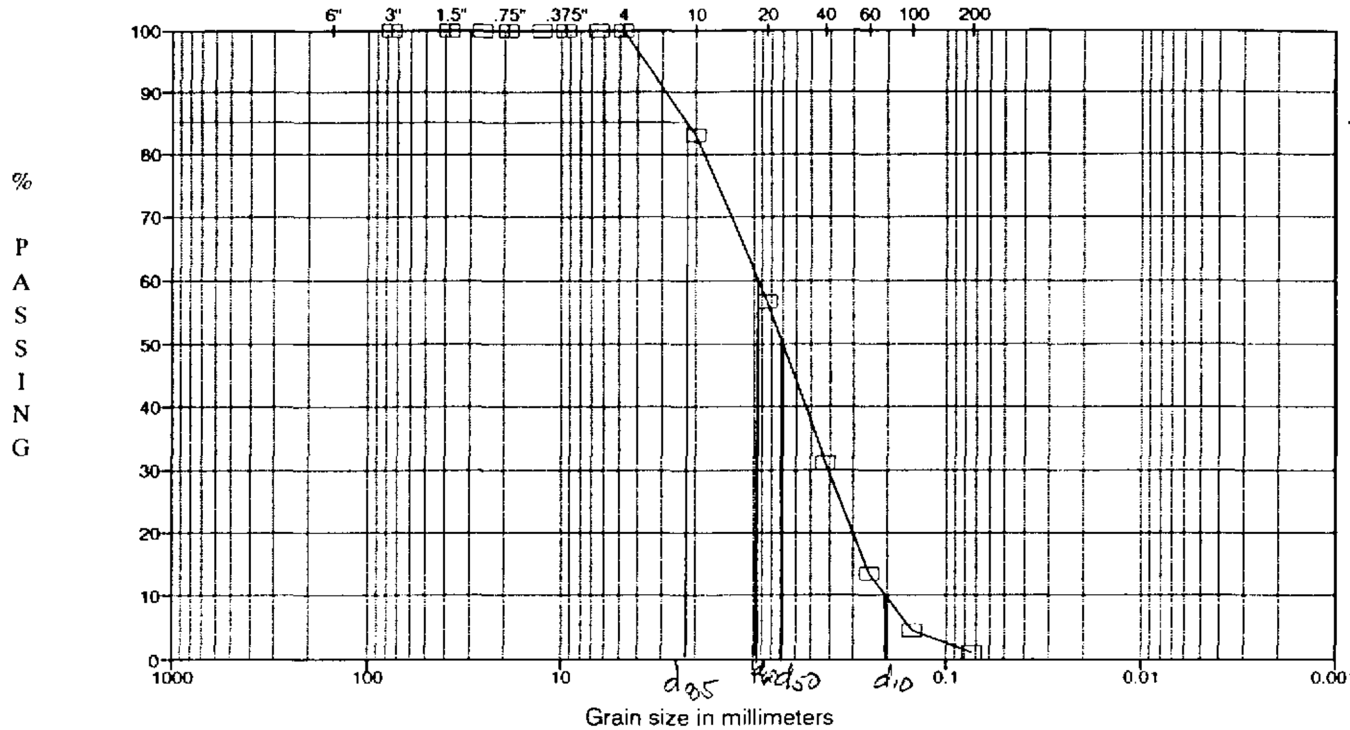
Form LABMI 1 2

RJD 5/5
8/2/91
PR

PARTICLE SIZE DISTRIBUTION ASTM D-421 AND 422

US STANDARD SIEVE OPENING SIZES



USCS

	Coarse	Fine	C	Med	Fine	
COBBLES	GRAVEL		SAND			FINES (Silt or Clay)

TECH: DL
DATE: 7/18/91
CHECKED: RMW
REVIEWED: DJF.

SAMPLE ID	W%	LL	PL	PI	Other	DESCRIPTION
QUINN PERKINS	2.09					Greyish orange c-f SAND, trace fines trace f gravel
Sample Type:	BULK	Date Tested:	7/17/91	USCS:		

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Form LAB41 1 2

APPENDIX 11-K
Hydrological Design Calculations

Objective: determine the size and shape of the drainage swale between the East-Central Hide Pile and the extension of Commerce Way and determine the size of the culvert to drain the channel into Aberjona River

LOCATIONS OF DRAINAGE SWALE AND CULVERT ARE SHOWN ON FIGURE 1.

Method: Use SCS TR55 to determine the maximum flow the swale will be required to accommodate and then use a spreadsheet which implements Manning's equation to determine the shape and size of the channel; reference 2 is used to determine the culvert size

References:

1. USDA SCS TR55 "Urban Hydrology for Small Watersheds" computer program

2. Concrete Pipe Design Manual, © American Concrete Pipe Association, 1985

Assumptions:

1. the drainage swale shall be grass or rip rap lined
2. the swale shall have side slopes of 2:1 and a design depth of 1.5 feet (2.0 feet shall be total depth including freeboard)
3. the culvert shall end above the water level of the river and therefore shall be designed for inlet control

Drainage Area for Swale 1:
SEE FIGURE 1 FOR LOCATIONS

I : 181,200 ft^2 = 4.16 ac = 0.0065 mi^2

II : 60,600 ft^2 = 1.39 ac = 0.0022 mi^2

0.0087 mi^2

Curve Number:

a. For cover soil: grassland with fair hydrologic conditions.
(or open space fair condition) CN = 79 for Type C soil

b. to be conservative, use a CN = 86 :

Since a geocomposite drainage layer will be installed where the slope of the remediated East-Central Hide Pile is 25% or steeper, an CN = 86 will be used. This drainage layer will behave almost like an impervious transport medium (CN = 98); hence a weighted average between 79 and 98 was selected. Also factored into the chosen CN value is the fact that once the roadway is paved, the average hydraulic condition will change to a slightly less pervious condition.

The CN = 86 is consistent with the value used in the total Site hydrological analysis used in the HEC-1 model.

Worksheet 3: Time of concentration (T_c) or travel time (T_t)

3/12

Project ISRT ECHP Swale By RD Date 8-2-91

Location Woburn, Mass Checked ACK Date 9/6/91

Circle one: Present Developed permeable cover

Circle one: T_c T_t through subarea Area I

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

Sheet flow (Applicable to T_c only)

Area I

		IIA	IIB	IIC	IID	
Segment ID		IIA	IIB	IIC	IID	
1. Surface description (table 3-1)		short grass	short grass	short grass	short grass	
2. Manning's roughness coeff., n (table 3-1) ..		0.15	0.15	0.15	0.15	
3. Flow length, L (total L ≤ 300 ft)	ft	40	90	95	75	
4. Two-yr 24-hr rainfall, P_2	in	3.3	3.3	3.3	3.3	
5. Land slope, s	ft/ft	0.013	0.044	0.084	0.120	
6. $T_t = \frac{0.007\,(nL)^{0.8}}{P_2^{0.5}\, s^{0.4}}$ Compute T_t	hr	0.092 +	0.108 +	0.087 +	0.062	= 0.35

Shallow concentrated flow

Segment ID		IIE	IIF	IIG	
7. Surface description (paved or unpaved)		unpaved	unpaved	unpaved	
8. Flow length, L	ft	23	107	45	
9. Watercourse slope, s	ft/ft	0.130	0.075	0.102	$\frac{72-67.4}{45} = 0.10$
10. Average velocity, V (figure 3-1)	ft/s	5.8	4.4	5.6	
11. $T_t = \frac{L}{3600\ V}$ Compute T_t	hr	0.001 +	0.007 +	0.002	= 0.010

Channel flow

Segment ID				
12. Cross sectional flow area, a	ft^2			
13. Wetted perimeter, p_w	ft			
14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r	ft			
15. Channel slope, s	ft/ft			
16. Manning's roughness coeff., n				
17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V	ft/s			
18. Flow length, L	ft			
19. $T_t = \frac{L}{3600\ V}$ Compute T_t	hr	+	=	

20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19) .... hr 0.36

Worksheet 3: Time of concentration (T_c) or travel time (T_t)

Project ISRT ECHP Swale　By RV　Date 8-2-91

Location Woburn, Mass.　Checked ALK　Date 8/6/91

Circle one: Present (Developed)　permeable cover

Circle one: (T_c) T_t through subarea　Area II

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

Area II

Sheet flow (Applicable to T_c only)					
Segment ID		IIA	IIB	IIC	IID
1. Surface description (table 3-1)		short grass	short grass	short grass	short grass
2. Manning's roughness coeff., n (table 3-1) ..		0.15	0.15	0.15	0.15
3. Flow length, L (total L ≤ 300 ft)	ft	40	65	125	70
4. Two-yr 24-hr rainfall, P_2	in	3.3	3.3	3.3	3.3
5. Land slope, s IIA (104.5 - 104)/40 = 0.013	ft/ft	0.013	0.062	0.096	0.043
6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5}\ s^{0.4}}$ Compute T_t	hr	0.092 +	0.072 +	0.103 +	0.089 = 0.350

Shallow concentrated flow					
Segment ID		IIE	IIF	IIG	IIH
7. Surface description (paved or unpaved)		unpaved	unpaved	unpaved	unpaved
8. Flow length, L	ft	120	30	6	305
9. Watercourse slope, s	ft/ft	0.042	0.067	0.33	.035 $\left(\frac{78-67.4}{305}\right)$
10. Average velocity, V (figure 3-1)	ft/s	3.3	4.2	9.3	3.0
11. $T_t = \frac{L}{3600\ V}$ Compute T_t	hr	0.010 +	0.002 +	0.0003 +	0.028 = 0.040

Channel flow			
Segment ID			
12. Cross sectional flow area, a	ft^2		
13. Wetted perimeter, P_w	ft		
14. Hydraulic radius, $r = \frac{a}{P_w}$ Compute r	ft		
15. Channel slope, s	ft/ft		
16. Manning's roughness coeff., n			
17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V	ft/s		
18. Flow length, L	ft		
19. $T_t = \frac{L}{3600\ V}$ Compute T_t	hr	+	=

20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19) .... hr 0.40

5/12

Table 2-2a.—Runoff curve numbers for urban areas[1]

Cover description		Curve numbers for hydrologic soil group—			
Cover type and hydrologic condition	Average percent impervious area[2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only)[4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)[5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

[1]Average runoff condition, and $I_a = 0.2S$.

[2]The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.

[3]CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.

[4]Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.

[5]Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4, based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

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Table 2-2c.—Runoff curve numbers for other agricultural lands[1]

Cover description		Curve numbers for hydrologic soil group—			
Cover type	Hydrologic condition	A	B	C	D
Pasture, grassland, or range—continuous forage for grazing.[2]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow—continuous grass, protected from grazing and generally mowed for hay.	—	30	58	71	78
Brush—brush-weed-grass mixture with brush the major element.[3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	[4]30	48	65	73
Woods—grass combination (orchard or tree farm).[5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods.[6]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	[4]30	55	70	77
Farmsteads—buildings, lanes, driveways, and surrounding lots.	—	59	74	82	86

[1] Average runoff condition, and $I_a = 0.2S$.

[2] *Poor:* <50% ground cover or heavily grazed with no mulch.
Fair: 50 to 75% ground cover and not heavily grazed.
Good: >75% ground cover and lightly or only occasionally grazed.

[3] *Poor:* <50% ground cover.
Fair: 50 to 75% ground cover.
Good: >75% ground cover.

[4] Actual curve number is less than 30; use CN = 30 for runoff computations.

[5] CN's shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pasture.

[6] *Poor:* Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair: Woods are grazed but not burned, and some forest litter covers the soil.
Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

Sheet flow

Sheet flow is flow over plane surfaces. It usually occurs in the headwater of streams. With sheet flow, the friction value (Manning's n) is an effective roughness coefficient that includes the effect of raindrop impact; drag over the plane surface; obstacles such as litter, crop ridges, and rocks; and erosion and transportation of sediment. These n values are for very shallow flow depths of about 0.1 foot or so. Table 3-1 gives Manning's n values for sheet flow for various surface conditions.

For sheet flow of less than 300 feet, use Manning's kinematic solution (Overton and Meadows 1976) to compute T_t:

$$T_t = \frac{0.007\,(nL)^{0.8}}{(P_2)^{0.5}\,s^{0.4}} \qquad \text{[Eq. 3-3]}$$

Table 3-1.—Roughness coefficients (Manning's n) for sheet flow

Surface description	n[1]
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≤20%	0.06
Residue cover >20%	0.17
Grass:	
→ Short grass prairie	0.15
Dense grasses[2]	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods:[3]	
Light underbrush	0.40
Dense underbrush	0.80

[1]The n values are a composite of information compiled by Engman (1986).

[2]Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.

[3]When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

where

T_t = travel time (hr),
n = Manning's roughness coefficient (table 3-1),
L = flow length (ft),
P_2 = 2-year, 24-hour rainfall (in), and
s = slope of hydraulic grade line (land slope, ft/ft).

This simplified form of the Manning's kinematic solution is based on the following: (1) shallow steady uniform flow, (2) constant intensity of rainfall excess (that part of a rain available for runoff), (3) rainfall duration of 24 hours, and (4) minor effect of infiltration on travel time. Rainfall depth can be obtained from appendix B.

Shallow concentrated flow

After a maximum of 300 feet, sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from figure 3-1, in which average velocity is a function of watercourse slope and type of channel. For slopes less than 0.005 ft/ft, use equations given in appendix F for figure 3-1. Tillage can affect the direction of shallow concentrated flow. Flow may not always be directly down the watershed slope if tillage runs across the slope.

After determining average velocity in figure 3-1, use equation 3-1 to estimate travel time for the shallow concentrated flow segment.

Open channels

Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where blue lines (indicating streams) appear on United States Geological Survey (USGS) quadrangle sheets. Manning's equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity is usually determined for bank-full elevation.

8/12

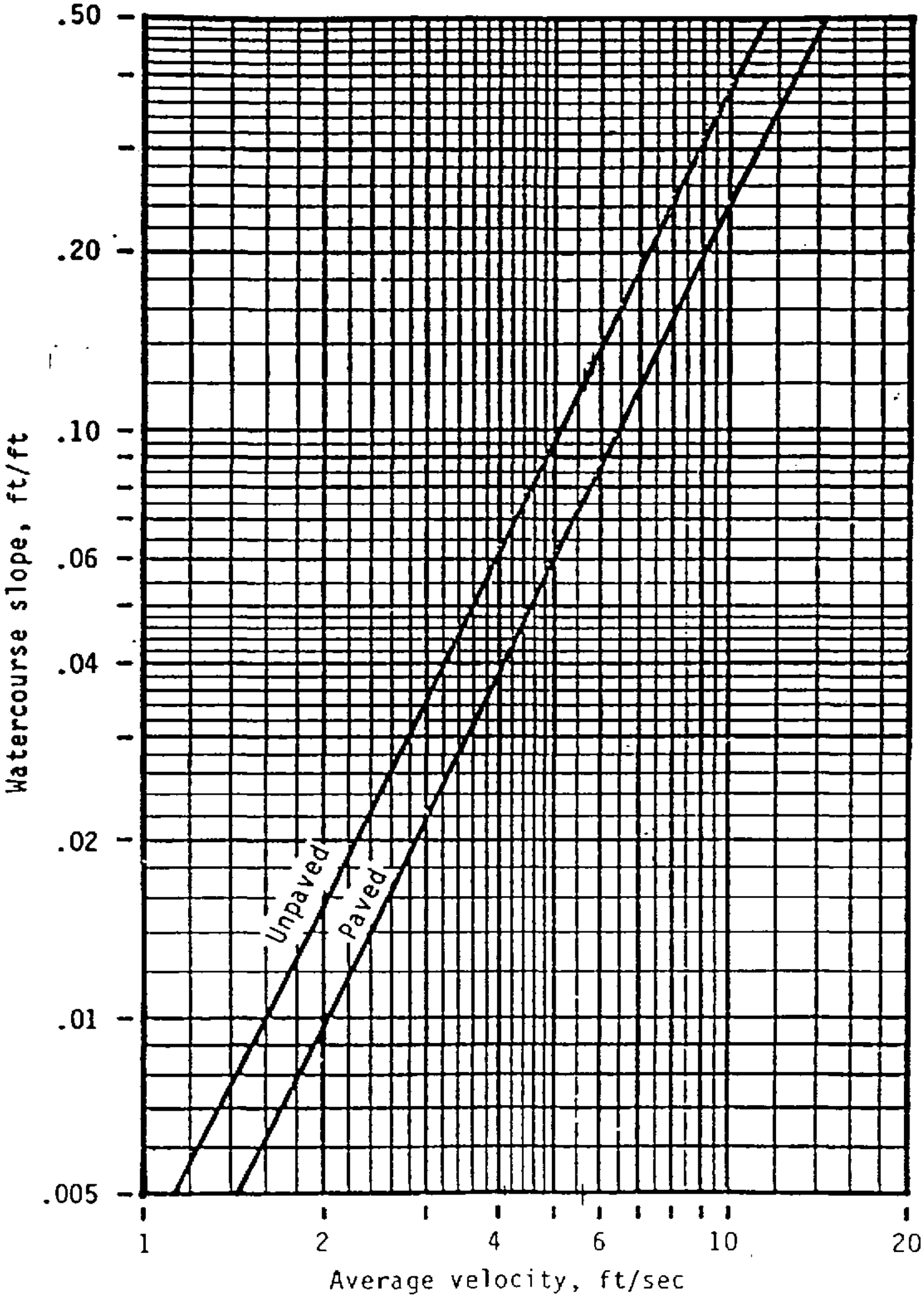


Figure 3-1.—Average velocities for estimating travel time for shallow concentrated flow.

TR-55 TABULAR DISCHARGE METHOD VERSION 1.11

1/

P--oject : isrt User: rd Date: 08-02-91
-nty : middlesex State: ma Checked: ALK Date: 08/06/91
title: east-central hp drainage swale 1

Total watershed area: 0.009 sq mi Rainfall type: III Frequency: 100 years

	Subareas	
	I	II
Area(sq mi)	0.01	0.00
Rainfall(in)	6.6	6.6
Curve number	86	86
Runoff(in)	4.98	4.98
Tc (hrs)	0.36	0.40
(Used)	0.40	0.40
TimeToOutlet	0.00	0.00
Ia/P	0.05	0.05
(Used)	0.10	0.10

Time (hr)	Total Flow	Subarea Contribution to Total Flow (cfs) I	II
11.0	1	1	0
11.3	1	1	0
11.6	1	1	0
11.9	3	2	1
12.0	4	3	1
12.1	5	4	1
2	8	6	2
3	13	10	3
12.4	19	14	5P
12.5	20P	15P	5
12.6	19	14	5
12.7	15	11	4
12.8	12	9	3
13.0	7	5	2
13.2	4	3	1
13.4	4	3	1
13.6	3	2	1
13.8	3	2	1
14.0	3	2	1
14.3	3	2	1
14.6	1	1	0
15.0	1	1	0
15.5	1	1	0
16.0	1	1	0
16.5	1	1	0
17.0	1	1	0
17.5	1	1	0
18.0	1	1	0
.0	0	0	0
.0	0	0	0
0	0	0	0
26.0	0	0	0

P - Peak Flow

file: ECSW100
proj: 903-6400
subj: DRAINAGE SWALE DESIGN ADJACENT TO EAST-CENTRAL HIDE PILE
date: DEC. 11, 1991
rev: DEC. 11, 1991

Mannings equation

REACH	side slope 1	side slope 2	base width (ft)	channel slope (ft/ft)	riprap D50 (ft)	Manning's n	design depth (ft)	required discharge (cfs)	Qmax (cfs)	velocity (ft/s)	tau	Eta	Phi (deg)
IA	2	2	1	0.010	0.5	0.035	1.5	20	21.4	3.6	0.49	0.20	42
IB	2	2	1	0.015	0.5	0.035	1.5	20	26.3	4.4	0.73	0.30	42
IC	2	2	1	0.020	0.5	0.035	1.5	20	30.3	5.1	0.97	0.40	42

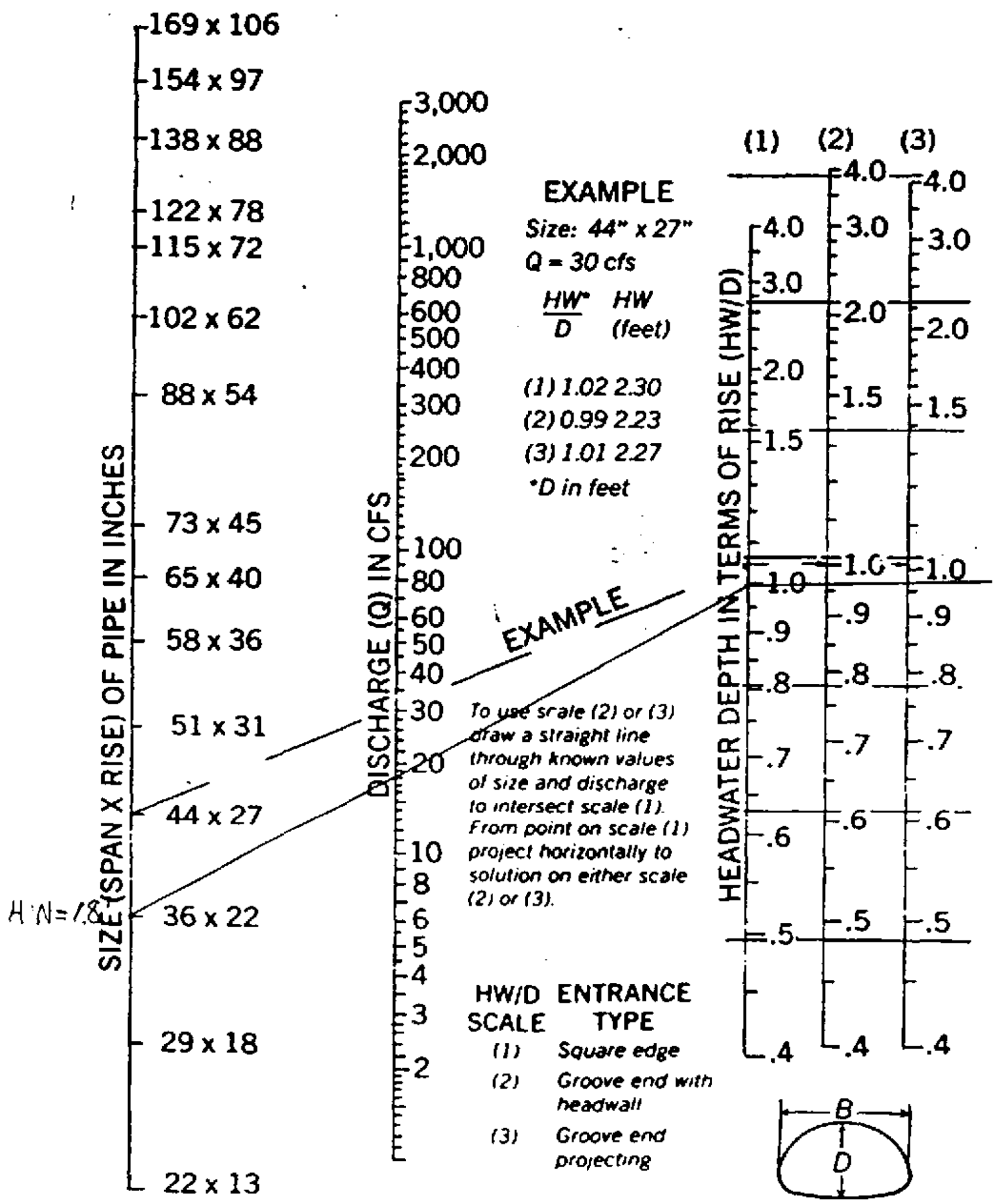
GRASS-LINED

REACH	side slope 1	side slope 2	base width (ft)	channel slope (ft/ft)	riprap D50 (ft)	Manning's n	design depth (ft)	required discharge (cfs)	Qmax (cfs)	velocity (ft/s)	tau	Eta	Phi (deg)
IA	2	2	0	0.010		0.024	1.5	20	21.4	4.7	0.42		
IB	2	2	0	0.015		0.024	1.5	20	26.1	5.8	0.63		

11/12

FIGURE 36

HEADWATER DEPTH FOR CONCRETE ARCH CULVERTS WITH INLET CONTROL

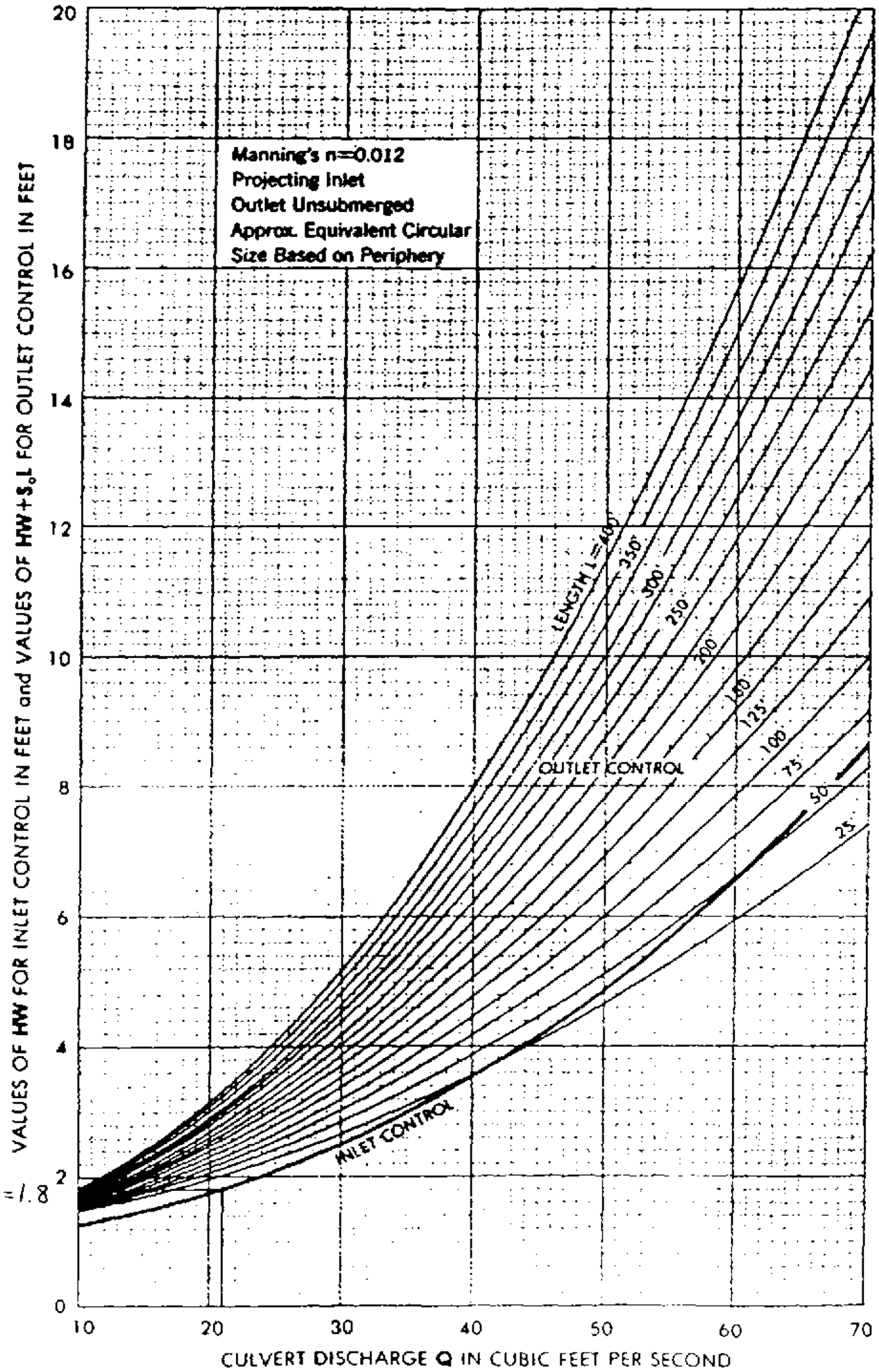


From Concrete Pipe Design Manual, 1985 (CRM)

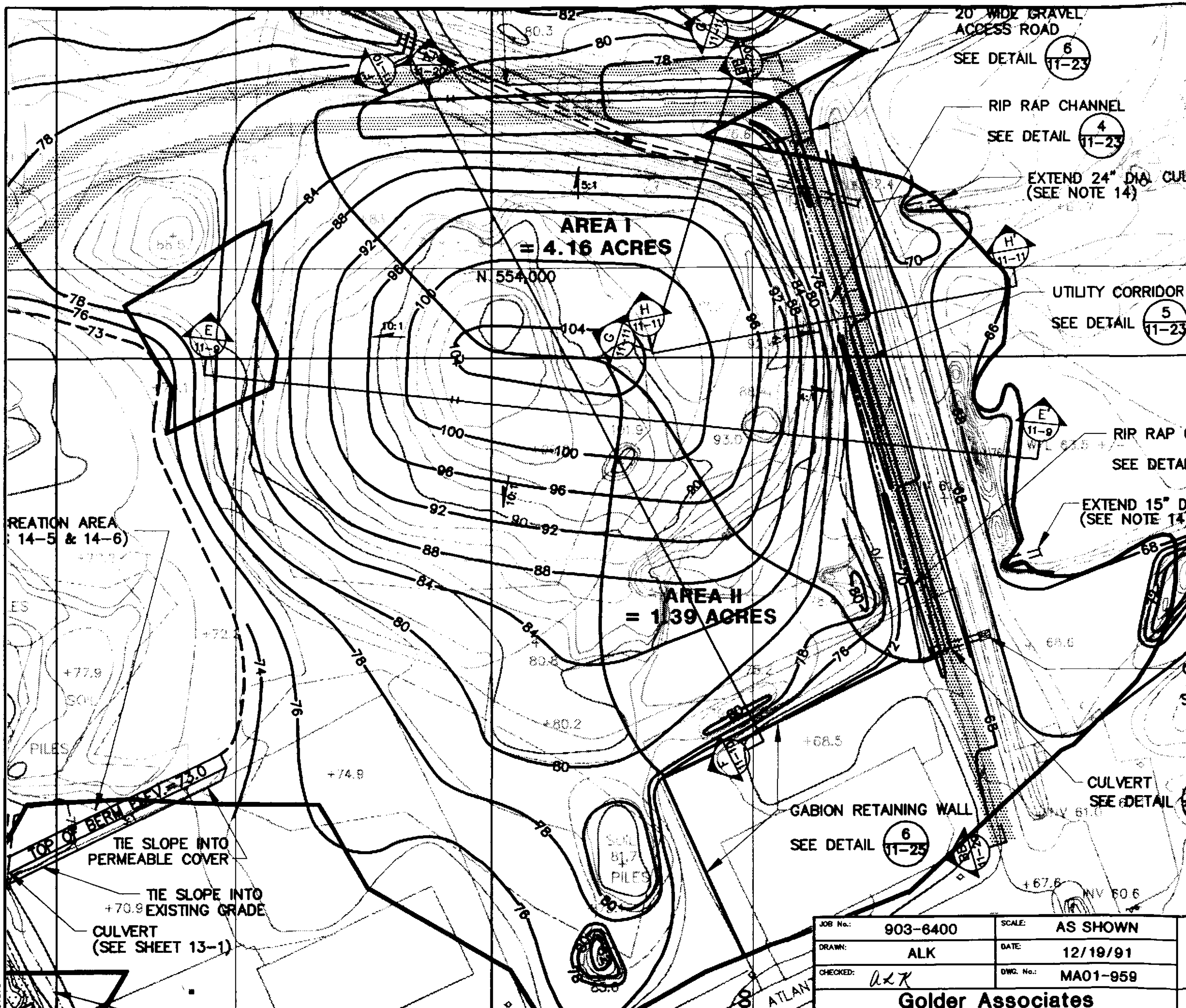
12 | 16

FIGURE 91

CULVERT CAPACITY
22 x 36-INCH (RISE x SPAN) ARCH
EQUIVALENT 30-INCH CIRCULAR



From Concrete Pipe Design Manual, 1985 (CPM)



APR 2 5 1992

100 0 100
scale feet

JOB No.: 903-6400	SCALE: AS SHOWN	DRAINAGE AREAS	
DRAWN: ALK	DATE: 12/19/91		
CHECKED: ALK	DWG. No.: MA01-959		
Golder Associates		INDUSTRI-PLEX SITE REMEDIAL TRUST	FIGURE 1

161576