DRAFT

Geotechnical Engineering Analyses and Recommendations
Third Runway Embankment
Seattle-Tacoma International Airport
SeaTac, Washington

HARTCROWSER
Delivering smarter solutions

Prepared for
HNTB

December 4, 2000
J-4978-28

AR 049659

CONTENTS

Page

INTRODUCTION 1

SUMMARY 1

Area Covered by this Report 1
Embankment Fill Materials 2
Slope Stability Analyses 2
Analysis of Areas with Potential Foundation Instability 3
Recommended Subgrade Improvements 4
Other Construction Recommendations 5
Work in Progress 5
Organization of This Report 5

GEOTECHNICAL OVERVIEW OF SUBSURFACE CONDITIONS 6

Areas Addressed in this Report 6
Existing Information 6
Subgrade Improvement Needed for Specific Soil Conditions to Provide Stability 7
Occurrence of Problematic Subgrade Conditions Impacting Slope Stability 7

GEOTECHNICAL RECOMMENDATIONS FOR CONSTRUCTION 12

Embankment Site Preparation 12
Subgrade Improvement 15
Embankment Underdrain 16
Embankment Fill 17
Slope Face Treatment 17

USE OF THIS REPORT 18

REFERENCES 20

TABLES

1 Soil Gradations for Embankment Fill Material Groups 22
2 Water Level Data 23

AR 049660

CONTENTS (Continued) <u>Page</u>

FIGURES

1 Vicinity Map
2 Site and Exploration Plan, Embankment Slope Between NSA and West MSE Walls
3 Site and Exploration Plan, Embankment Slope Between West and South MSE Walls
4 Site and Exploration Plan, Phase 4 Fill
5 Recommended Phase 4 Subgrade Improvement Area

APPENDIX A
SLOPE STABILITY AND DEFORMATION ANALYSES A-1

Introduction A-1
Limit Equilibrium Slope Stability Analyses A-1
Additional Special Explorations A-6
Statistical Analysis of Liquefaction A-6
Deformation Analysis for Discontinuous Liquefaction Conditions A-9

FIGURE

A-1 Illustration of Probabilistic Based Liquefaction 9% Probability of Liquefaction

TABLES

A-1 Limit Equilibrium Analyses A-1
A-2 Summary of Input Soil Parameters A-4
A-3 Generalized Summary of Limit Equilibrium Results A-5
A-4 Results of Liquefaction Analysis for Different Earthquake Events A-7
A-5 Soil Properties Used in the Displacement-Based (FLAC) Model at Station 193+19 A-10
A-6 Displacements Calculated for the 175-Year Seismic Event (9% liquefaction) A-11
A-7 Displacements Calculated for the 475-Year Seismic Event (33% liquefaction) A-11
A-8 Displacements Calculated Using the Composite Strength Method A-12
A-9 Displacements Calculated Using the Probabilistic Strength Method A-13

CONTENTS (Continued) **Page**

APPENDIX B
STABILITY ANALYSES CROSS SECTIONS

FIGURES

B-1 2H:1V Embankment Section Used for Stability Analyses, Section 170+23
B-2 2H:1V Embankment Section Used for Stability Analyses, Section 187+60
B-3 2H:1V Embankment Section Used for Stability Analyses, Section 193+19
B-4 2H:1V Embankment Section Used for Stability Analyses, Section 206+44
B-5 2H:1V Embankment Section Used for Stability Analyses, Section 210+00
B-6 2H:1V Embankment Section Used for Stability Analyses, Section 212+50

APPENDIX C
EMBANKMENT INFILTRATION AND SEEPAGE STUDIES

Introduction C-1
Approach C-1
Soil Properties C-3
Weather Data C-6
Simulations C-6
Model Results C-8
Conclusions C-10
References for Appendix C C-13

TABLES

C-1 Soil Properties used for Developing Input to Rosetta Model C-14
C-2 HELP Output Summary for Existing Conditions (Baseline) C-15
C-3 HELP Output Summary for Group 1B Embankment Fill C-16
C-4 HELP Output Summary for Group 3 Embankment Fill C-17
C-5 HELP Output Summary for Group 4 Embankment Fill C-18
C-6 HELP Output Summary for Various Embankment Heights C-19

CONTENTS (Continued) Page

FIGURES

C-1 Embankment Slope Showing Water Balance Components
C-2 Soil Moisture/Conductivity Characteristic Curves for Outwash Silty Sand
C-3 Soil Moisture/Conductivity Characteristic Curves for Glacial Till
C-4 Grain Size Envelope for Group 1A Fill Material
C-5 Grain Size Envelope for Group 1B Fill Material
C-6 Grain Size Envelope for Group 2 Fill Material
C-7 Grain Size Envelope for Group 3 Fill Material
C-8 Grain Size Envelope for Group 4 Fill Material
C-9 Soil Moisture/Conductivity Characteristic Curves for Group 1B Fill Material
C-10 Soil Moisture/Conductivity Characteristic Curves for Group 3 Fill Material
C-11 Soil Moisture/Conductivity Characteristic Curves for Group 4 Fill Material
C-12 Simulated Groundwater Discharge Rates for Existing Conditions and Group 1B Fill
C-13 Simulated Groundwater Discharge Rates for Existing Conditions and Group 3 Fill
C-14 Simulated Groundwater Discharge Rates for Existing Conditions and Group 4 Fill
C-15 Cumulative Plot of Simulated Groundwater Discharge Rates

APPENDIX D
ALTERNATIVE ACCEPTANCE CRITERIA FOR EMBANKMENT FILL MATERIALS

GEOTECHNICAL ENGINEERING ANALYSES AND RECOMMENDATIONS THIRD RUNWAY EMBANKMENT SEATTLE-TACOMA INTERNATIONAL AIRPORT SEATAC, WASHINGTON

INTRODUCTION

This report presents the results of Hart Crowser's geotechnical analyses and recommendations for construction of the proposed embankment to support the new Third Runway at Seattle-Tacoma International Airport, in SeaTac, Washington.

This report provides geotechnical design and construction information for the Phase 4 and subsequent phases of embankment construction. Geotechnical recommendations are based on results of subsurface explorations and tests that are presented in other reports (see Hart Crowser, 2000j and other references listed at the end of this text). Geotechnical recommendations for previous phases of embankment construction are presented in other reports (AGI, 1998; Hart Crowser, 1999a and 2000a).

SUMMARY

Phase 4 embankment construction will include permanent slopes as well as construction of interior fill portions of the embankment. Design requirements for these permanent slopes include seismic design considerations that differ from the previously constructed temporary slopes that have been constructed in the interior of the embankment. Accordingly, the main part of this report is focused on the proposed permanent slopes, including foundation support for these slopes, on the west side of the proposed embankment.

Significant elements of this report are summarized below. The main text, tables and figures, appendices, and referenced documents should be referred to for significant additional information that supports and/or qualifies the following summary.

Area Covered by this Report

This report addresses the interior of the embankment (Phase 4) and permanent fill slopes along the west side of the project. Inclination of the embankment slopes discussed in this report is nominally 2 horizontal to 1 vertical (2H:1V). Analyses and recommendations for design and construction of the proposed

MSE walls that will retain portions of the embankment are discussed elsewhere (Hart Crowser, 2000f and 2000h).

Figures 1, 2, and 3 show the embankment slope areas that are addressed in this report. Figure 4 shows the Proposed Phase 4 embankment footprint. The proposed permanent slope areas addressed in this report are located as follows (projected west from the approximate runway centerline stations noted):

- Station 187+00 to 213+00 (between the NSA and West MSE walls);
- Station 149+00 to 171+00 (between the South and West MSE walls); and
- Station 133+00 to 139+00 (south of the South MSE wall).

Embankment Fill Materials

Embankment fill materials and methods of construction will generally be the same as have been used for previous phases of embankment construction. Table 1 summarizes the specified gradation and recommended use for different types of fill material. The embankment is to be constructed in zones, with different compaction criteria applied to different fill materials and zones as discussed elsewhere.

The specified fill soils are relatively "well-graded." Well-graded means that these soils are a blend of fine- and coarser-grained particle sizes. Hart Crowser has proposed test specifications that could be used to broaden the range of acceptable fill materials. These draft specifications and supporting research are presented in Appendix D. Except for addition of the proposed specification in Appendix D, Hart Crowser does not recommend any changes to the fill material specifications or compaction criteria that are currently being used in Phase 3 construction (Port of Seattle, 2000).

Slope Stability Analyses

Embankment design was based on slope stability and deformation analyses. Limit equilibrium analyses of slope stability were accomplished for representative cross section to verify the proposed embankment slopes met target values for factor of safety. Various drainage and seismic conditions were assessed as discussed in Appendix A, in accordance with conventional geotechnical engineering practice.

The purpose of the slope stability analyses discussed in this report was to:

- Verify that permanent slopes constructed with the specified materials and methods would be stable for all anticipated design loads; and

- Define the need for and extent of any needed subgrade improvement to foundation soils below the embankment slopes.

The design seismic event for the Third Runway project is a large earthquake with a probability of exceedence of 10 percent in 50 years, or an equivalent return period of about once in 475 years. This design earthquake would have a Richter magnitude on the order of about 7.5 and is estimated to produce a peak horizontal acceleration of 0.36 g (see Hart Crowser, 1999d).

Overall stability of the proposed embankment slopes met target factors of safety except in one specific area (extending from the area adjacent to approximately runway stations 204+00 to 214+00) where existing native soils are liquefaction susceptible and/or include areas of low-strength silty clay. Recommended subgrade improvements in this area will provide stable slopes. Smaller scale seismic instability (slumps on slopes less than about 60 feet in height) may result in other areas, as a result of isolated, discontinuous zones of soil liquefaction.

Analysis of Areas with Potential Foundation Instability

Initial analyses showed embankment slope stability would meet target factors of safety except where problems might exist with native soils that provide foundation support to the slope. Hart Crowser accomplished an extensive assessment of available soils information to identify location of problematic conditions, and to verify adequacy of the available information to identify such areas. This assessment focused on areas underlying the embankment slope because our design approach assumed that a stable toe section was necessary to buttress the interior of the embankment fill.

Analyses of soils along the embankment showed isolated areas of loose to medium dense, saturated sandy soils that are susceptible to liquefaction, but which are not contiguous or extensive enough to produce significant embankment instability. Hart Crowser estimated post-liquefaction shear strength that would result from different size earthquakes, and used a probabilistic deformation-based analysis to demonstrate that the composite strength of areas with mixed liquefaction susceptible and non-susceptible soils would limit embankment deformations to acceptable levels, even after a very large earthquake.

Analysis of potential liquefaction also showed areas of loose to medium dense, saturated sandy soils that are not susceptible to liquefaction under existing groundwater conditions, but which might be susceptible in the event groundwater levels rise. Significant changes in groundwater levels are not anticipated, based on groundwater monitoring in the area. In addition, infiltration modeling (see Appendix C) indicates that construction of the embankment would not produce long-term increases in groundwater levels below the embankment.

Other potentially problematic subgrade soil conditions that were evaluated include limited areas of (1) clay and peat soils with low shear strength, and (2) silt and clay soils where excess pore pressures might reduce shear strength during construction.

Recommended Subgrade Improvements

Subgrade improvement are recommended in the area(s) where existing soils could affect the embankment slopes due to potential liquefaction, excess pore pressures in silts and clays, and/or low shear strength organic soils (peat). The available methods to accomplish subgrade improvement where needed for the Third Runway MSE walls (see Hart Crowser, 2000f) are also generally applicable for soils supporting the embankment slopes.

For the Third Runway project overall, Hart Crowser recommended subgrade improvements consist of either 1) overexcavation and replacement of unsuitable soils with compacted structural fill, or 2) use of stone columns to improve subgrade soils in place. Either of these approaches is suitable in the limited areas where subgrade improvement is needed for stability of the permanent embankment slopes.

Subsequent to the Hart Crowser's initial review of alternatives, independent reviewers suggested that dynamic compaction be re-evaluated. Our analysis indicates this approach would be feasible to mitigate potential liquefaction only, but would not be effective in areas where silt, clay, and/or peat soils could contribute to instability. A variation on this approach, referred to as "dynamic replacement," is currently being evaluated.

Recommended subgrade improvements are focused on conditions affecting stability of the permanent embankment slopes. Conditions within the interior of the embankment are generally of less concern because:

1) Our design approach assumes that stability of subgrade soils along the perimeter of the embankment is necessary to assure stability of the embankment as a whole; and
2) Subsurface explorations within the interior of the embankment generally encountered more competent soils (higher denser and shear strength) compared to soils along the west edge of the embankment.

Other Construction Recommendations

In addition to subgrade improvements, this report summarizes Hart Crowser's previous recommendations for embankment construction. These recommendations include:

- Site preparation should include removal of vegetation prior to placement of fill, proof rolling, and limited stripping of topsoil along the toe of the embankment;
- Provide a continuous drainage layer below the base of the embankment;
- Remove or improve in-place any areas of peat or other soft unsuitable soils that may be encountered prior to fill placement;
- Abandon any existing utility pipes below the embankment by filling them with grout; and
- Continue to use the fill material criteria and compaction approach as specified for Phase 3 of the project. See Appendix D for recommended additions to the fill material acceptance criteria.

Work in Progress

An independent Geotechnical Review Board reviewed geotechnical work to date and work in progress, in November 2000. At the time this report is being issued as a draft for review and discussion, some related work is ongoing to address questions raised by the GRB and/or to improve cost-effectiveness of the design and construction. Recommendations in this report may be modified depending on results of this additional work.

Organization of This Report

This report is organized as follows:

- Following this summary, the text of this report discusses subsurface conditions in the areas analyzed, focusing particularly on conditions that could require subgrade improvement; and
- The final section of the main text of this report provides geotechnical recommendations for construction.

Appendix A provides a detailed geotechnical discussion of the slope stability and deformation analyses accomplished for the permanent embankment slopes. This includes design criteria, analysis methods, assumptions, and input soil parameters.

Appendix B presents generalized cross sections used for the stability analyses discussed in this report.

Appendix C discusses the groundwater modeling accomplished by Hart Crowser to assess the potential for long-term changes in groundwater level below the embankment. The modeling was accomplished to assess potential changes in risk of seismic liquefaction, and potential for changes in base flow to Miller Creek and adjacent wetlands west of the embankment.

Appendix D presents a recommended draft specification and supporting information that may be used to broaden the fill material acceptance criteria.

GEOTECHNICAL OVERVIEW OF SUBSURFACE CONDITIONS

Areas Addressed in this Report

This report addresses the Third Runway embankment areas shown on Figures 1, 2, and 3. The specific areas represented in our analyses are as follows:

- **Phase 4 Area between the NSA Wall and the West Wall.** This area includes the permanent embankment slopes from about Station 194+50 to 213+00. Subgrade improvements in the area north of the existing 156th Street and below the West MSE Wall, and most of the embankment fill in the area between the NSA and West MSE Walls will be constructed as part of the Phase 4 contract.

- **South of the West MSE Wall.** The remainder of the embankment will be part of a subsequent construction phase(s). This includes permanent fill slopes from about Station 149+00 to 171+00 (between the West and South Walls) and a smaller area of permanent fill slopes from about Station 133+00 to 138+50, south of the South MSE Wall.

Existing Information

Information on subsurface conditions in the area covered by this report is presented in several reports, including: AGI, 1998; Hart Crowser, 1999a, 2000b, 2000d, 2000e, 2000g, and 2000j.

Subgrade conditions below the embankment in general, and in the Phase 4 area specifically, typically consist of up to about 20 feet of loose to medium dense surficial soils overlying very dense or hard glacially overridden soils. In some areas, the surficial soils include some existing fill that typically consists of local soils that have been disturbed.

The surficial soils generally consist of sands interbedded with a mixture of silts, clays, and peat. These "sands" actually range in gradation from non-silty to silty or clayey sands, and contain varying amounts of gravel.

- Typically the surficial soils are more likely to contain lenses or interbeds of fine-grained silt and clay, and organic peat, along the north and west sides of the project where the embankment is adjacent to wetlands which border Miller Creek.
- The surficial soils are more typically loose sand (with varying fines content but fewer distinct lenses of silt and clay) in the southwest part of the embankment.

Generally very dense or hard, glacially overridden sands and silts are encountered within 10 to 20 feet of the existing surface. Both the glacially overridden soils and the shallow surficial soils include zones of perched water and a shallow regional aquifer that contribute base flow to Miller Creek and Walker Creek, west of the embankment.

Subgrade Improvement Needed for Specific Soil Conditions to Provide Stability

Soil conditions in some areas require subgrade improvement to provide stable support for the Third Runway embankment. Such conditions identified in some explorations for the project include:

- Relatively loose, predominantly sand soils below the water table, which are potentially susceptible to strength loss due to seismic shaking ("liquefaction");
- Compressible silt and clay soils which are potentially susceptible to short term ("end of construction") strength loss due to pore pressures developed from embankment loads; and
- Low strength, compressible organic soils, referred to as peat.

Occurrence of Problematic Subgrade Conditions Impacting Slope Stability

Most of the surficial soils and all of the underlying glacially overridden soils will provide good embankment support. However, locally there are some near-surface soils that will not provide good embankment support. A summary of these potentially problematic subgrade conditions is provided below.

AR 049670

Liquefaction

As used in this report "liquefaction" refers to partial or complete loss of strength in granular soils when excess pore pressures are created as a result of seismic shaking. Figures 2 and 3 show the extent of seismic hazard conditions (including liquefaction-susceptible soils) identified during the EIS for the Third Runway, (FAA, 1995). Explorations for the embankment that encountered liquefaction-susceptible soils are also shown on Figures 2 and 3.

General Conditions of Interest

Liquefaction-susceptible soils, consisting of loose to medium dense sand and slightly silty to silty sand below the groundwater surface, were encountered (1) in isolated, widely spaced explorations along the proposed embankment slope between the South and West MSE walls, (2) more often but still discontinuously in explorations between the West MSE wall and proposed runway Station 205+00 (about where the existing South 156th Way crosses the proposed embankment), and (3) consistently in an area extending north of about runway Station 205+00 and south of the proposed NSA wall.

Analysis of Liquefaction

Hart Crowser used the Seed and Harder method of analysis to determine susceptibility of soils to liquefaction (Hart Crowser, 2000i). Our analyses used results of Standard Penetration Test (SPT N-values) and Cone Penetrometer Tests (CPT) and considered the effect of different magnitude earthquakes having return periods ranging from 72 years up to the 475-year design event.

Hart Crowser's analysis concluded that the extent of liquefaction-susceptible surficial soils along the permanent embankment slope is largely dependent on the extent of saturation in the loose to medium dense, predominantly sandy surficial soils. Pre-1998 water level observations in the project area are reported in AGI (1998). Hart Crowser installed and monitored additional observation wells to measure groundwater levels in the area of interest in 1999 and 2000 (see Table 2).

Hart Crowser's interpretation of existing water levels to indicate potential areas susceptible to liquefaction is conservative for two reasons:

- Review of local precipitation records indicates the winter of 1999 and 2000 was wetter than average (42 inches of precipitation compared with the long-term (69-year) average of 38.4 inches). Groundwater monitoring in the southern part of the embankment was initiated in the winter and spring of

2000, during the time when water levels are seasonally highest. Water level monitoring over the rest of the embankment area has been regularly accomplished for more than a year.

- Hart Crowser also completed a detailed hydrologic modeling analysis to assess whether infiltration through the embankment would affect long-term groundwater levels after construction. Our analyses indicate the embankment is unlikely to materially change groundwater levels compared to existing conditions (see Appendix C).

As a result, our interpretation of risk of liquefaction based on existing conditions is conservative.

Finally, Hart Crowser evaluated stability of areas where soil liquefaction is anticipated to occur. We evaluated post-liquefaction residual shear strength in areas of potentially complete liquefaction, and composite shear strength of zones where liquefaction may occur discontinuously, for use in slope stability and deformation analyses. These analyses and results are presented in Appendix A. These analyses generally indicate the embankment slope will be stable for the design seismic event, except in one area between runway Stations 204+00 to 214+00. As a result of these analyses, Hart Crowser recommends subgrade improvements to prevent liquefaction and achieve target factors of safety along this portion of the west slope of the proposed embankment (see Figure 5).

Hart Crowser anticipates there will be some liquefaction in isolated areas below the embankment south of runway Station 205+00; see Figures 2 and 3. Our analysis of composite strength indicates that discontinuous areas of liquefaction would not produce large-scale slope instability such as would impact the operability of the runway. However, less significant problems may result in areas where subgrade improvement is not provided.

- Parametric analysis indicates that smaller scale instability involving slopes up to about 60 feet in height could result from local discontinuous zones of liquefaction. Depending on where liquefaction occurs and the magnitude of strength reduction, such events could limit use of the airfield perimeter road and require local slope maintenance.
- Where slope instability does not occur, there is some potential that ground surface movements may occur ("sand boils") within the interior or slopes of the embankment. Surface movements are not observed where the depth of well-compacted fill is on the order of a few tens of feet or more. This is not anticipated to be a significant problem for the proposed runway since liquefaction-susceptible soils are primarily anticipated in areas of thick fill adjacent to the western side of the embankment.

Recommended Mitigation

Figure 5 depicts the proposed location for subgrade improvement to mitigate liquefaction in the Phase 4 construction area. Recommended mitigation in this area could consist of either (1) removal and replacement of soils to a depth of about 15 feet, or (2) use of stone columns.

<u>End of Construction Excess Pore Pressures</u>

Compressible silt and clay soils are potentially susceptible to short-term ("end of construction") strength loss due to pore pressures developed from embankment loads.

General Conditions of Interest

Soft and medium stiff to stiff silt and clay subgrade soils in the north part of Phase 4, south of the NSA wall, were evaluated to assess potential for construction-induced pore pressures could reduce soil shear strength below acceptable values. Potential for occurrence of this problem below the embankment slopes (i.e., outside the MSE wall subgrade support areas which are discussed in Hart Crowser, 2000f) is limited to the area north of about runway Station 205+00 (about where the existing South 156th Way crosses the proposed embankment. This is roughly the same area discussed above, where liquefaction-susceptible sandy soils are interbedded with relatively low strength silty clay.

Silt and clay soils were also observed in the embankment slope area between Stations 202+50 and approximately 197+00. However, the silt/clay in this area is stiff to hard and the results of Hart Crowser's analyses indicated adequate factors of safety for undrained conditions due to the associated higher soil shear strength (refer to Table A-1 in Appendix A).

Analysis of Construction-Induced Pore Pressures

Potential embankment stability problems due to excess pore pressures during construction could occur if embankment fill placement occurs at a rate faster than pore pressures can dissipate. Hart Crowser created a spreadsheet model to calculate the rate of pore pressure build-up based on consolidation theory. Consolidation parameters were obtained from laboratory tests and CPT tests. Maximum pore pressure values over time were calculated by comparing the incremental increase in pore pressure due to daily fill placement with continuous pore pressure dissipation due to consolidation. Rate of fill placement was adjusted to determine limiting values.

Hart Crowser found excess construction-induced excess pore pressures will not cause instability where the embankment fill is placed at a rate of less than 4 feet per day.

Recommended Mitigation

Potential for instability due to construction-induced excess pore pressure is not anticipated within the Phase 4 area except within the area where subgrade improvement is recommended for mitigation of seismic liquefaction. The mitigation alternatives (remove and replace, or stone columns) would also eliminate potential pore pressure concerns in the clay soils interbedded with the sands in this area.

Provided mitigation for liquefaction is provided as recommended above, it would not be necessary to limit rate of fill placement to avoid excess pore pressures. No other mitigation is needed.

Low Shear Strength, Compressible Peat

Peat (organic-rich soils) was encountered discontinuously in isolated areas within the embankment footprint, including some areas along the proposed permanent slope. Peat is typically associated with wetland soils, but is not present in all wetlands.

General Conditions of Interest

The occurrence of peat is uncommon within the proposed embankment subgrade, and will generally not be a stability problem for permanent slopes. Limited thicknesses of peat (typically less than a foot in thickness) have been observed discontinuously in a few isolated borings below the proposed embankment.

(Peat is more pervasive in some of the wall subgrade areas. See Hart Crowser (2000b, 2000e, and 2000f) for discussion of peat below portions of the NSA and West MSE walls, and below part of the right of way for the relocated South 156th Street).

Recommended Mitigation

Where peat is encountered below the main embankment it could be either (1) removed and replaced with compacted fill, or (2) left in-place and filled over with coarse gravel or quarry spalls, prior to placement of embankment fill. The second approach is generally appropriate where the peat is less than about 2 feet in thickness or about 25 feet in lateral extent. Hart Crowser recommends that we be consulted prior to filling over thicker or more extensive peat deposits left in-place.

GEOTECHNICAL RECOMMENDATIONS FOR CONSTRUCTION

This section of the report discusses recommendations for geotechnical aspects of embankment construction.

For the most part, previous Hart Crowser recommendations (Hart Crowser, 1999a and 2000a) for embankment construction Phases 2 and 3 are applicable to the proposed Phase 4 and remaining embankment construction. As needed, these recommendations are reiterated and/or modified herein.

Hart Crowser provides recommendations for construction in the following areas:

- Site preparation;
- Subgrade improvement;
- Drainage layer construction;
- Fill materials, placement and compaction; and
- Slope face treatment.

Embankment Site Preparation

Recommended site preparation for embankment construction includes (1) removal of vegetation and debris, (2) limited topsoil removal, (3) proof rolling, (4) filling over, or local overexcavation and replacement, of unsuitable soils; and (5) abandonment of wells and buried pipelines by grouting.

Removal of Vegetation and Debris

Prior to placement of any fill, Hart Crowser recommends that all vegetation be removed within the fill footprint.

- Close-cut trees and vegetation, and process into wood chips;

- Rake and remove loose organic debris resulting from clearing and mowing activities. Grubbing or removal of topsoil is not required except as noted below.

The chipped organic debris can be reused as mulch, for dust or mud control on haul roads, or incorporated into non-structural fill.

Remove and dispose of all rubbish encountered during removal of vegetation. Minor amounts of concrete or masonry demolition debris may be incorporated into the embankment fill only if it is less than 6 inches in size and can be placed and compacted in such a manner to prevent formation of voids. (Note that a significant amount of concrete debris was incorporated into a test fill in the NSA in 1998. Contact Hart Crowser if you wish to develop specifications for placement of concrete debris in Phase 4 or subsequent embankment construction).

Limited Removal of Topsoil

Within a 50-foot-wide zone underlying the toe of the fill and within the footprint of future airfield pavement areas, Hart Crowser recommends the following:

- Clear and grub all stumps, roots, buried logs, brush, matted roots, and other unsatisfactory materials; and
- Remove soft/loose, organic-rich topsoil at the ground surface to expose medium dense to dense granular soils. We estimate that typically a half-foot to about 1-foot nominal thickness would need to be removed, based on the subsurface information available, including the test pits in the area. However, we recommend there be some provision to do additional stripping as needed based on observations at the time of construction.

This is a good construction procedure because it enables close observation of subgrade conditions in area critical to overall slope stability. Also, decay over time of organic material in the topsoil may reduce strength of the topsoil (if left in-place) and possibly lead to instability. Stability analyses previously accomplished by Hart Crowser (for Phase 3) indicate embankment stability is relatively insensitive to the width of topsoil stripping, thus the 50-foot width is considered reasonable for the embankment slopes.

Note that future airfield pavement will be located above part of the Phase 4 fill. In our opinion, the risk of future settlements due to organic decomposition causing potential long-term pavement damage is probably low, but it is not possible to specifically quantify the magnitude of such risk. We recommend the base preparation noted above extend over the area defined by projecting down and outward from the edge of runway and taxiway pavement at 0.5H to 1V, to avoid risk of possible future pavement damage related to settlements resulting from organic decomposition.

Proof Rolling

Prior to placement of any new fill, we recommend that the subgrade be proof rolled with a heavy roller (nominal 15,000-pound static dead weight) after removal of vegetation, topsoil, and any other overexcavation. The purpose of the proof rolling is to identify any local areas of unacceptably soft, loose, or wet soils that may need to be treated prior to fill placement.

We recommend an experienced person representing the Port observe proof rolling and initial fill placement and compaction, so that local overexcavation and replacement may be accomplished if unsuitable conditions are encountered.

Treatment of Unsuitable Soils

Hart Crowser anticipates that some areas within the embankment footprint may be soft or wet or otherwise unsuitable for fill placement.

Where peat or other soft soils are encountered below the embankment, these areas could be either (1) excavated and replaced with compacted fill, or (2) left in-place and filled over with coarse gravel or quarry spalls, prior to placement of embankment fill. The second approach is generally appropriate where the peat or other soft soils are less than about 2 feet in thickness or about 25 feet in lateral extent. Hart Crowser recommends that we be consulted prior to filling over thicker or more extensive peat deposits, in the event that any such areas are encountered during construction.

Seepage is anticipated in some areas within the embankment footprint. Where subgrade soils are generally competent, the embankment underdrain material can be placed directly on the subgrade to allow continued seepage below the main fill. Where seepage occurs in soft soil or peat deposits, we recommend that a filter geotextile be placed between the subgrade and the underdrain to prevent piping of fine-grained soil into the underdrain.

Depending on condition of the subgrade, it may be necessary to increase the thickness of the first lift of fill material, and/or to accept a somewhat lower degree of compaction than otherwise specified. Regardless of whether topsoil is left in-place, it may be difficult to achieve the specified minimum level of compaction for the initial 1 to 2 feet of fill. Sometimes it is necessary to build up some thickness of good fill to bridge over soft subgrade, to achieve specified compaction. Where the specified compaction cannot be achieved within the first 1 to 2 feet, we recommend the Contractor be allowed to either:

1. Overexcavate and replace the soft subgrade; or
2. Stabilize the soft area with quarry spalls; and/or

3. Use a geotextile between the subgrade and new fill, to obtain a relatively non-yielding foundation to support the fill.

Abandon Buried Utility Pipes and Wells

Prior to fill construction, existing monitoring wells and any abandoned water supply wells that may be discovered, should be abandoned in accordance with Washington State Department of Ecology regulations (Chapter 173-160 WAC).

Hart Crowser also recommends that any abandoned underground pipes within the fill footprint be filled with cement grout to prevent them from becoming a possible conduit for underground erosion that could produce settlements. Typically this is done by completely filling such pipes with grout, from the lowest point to the highest point. Less complete filling, such as installation of one or more intermittent grout plugs at the lowest end of the pipe is sometimes acceptable, but provides a lower degree of protection.

Subgrade Improvement

Hart Crowser recommends that subgrade improvement of unsuitable soils be accomplished by either (1) removal of the unsuitable soil and replacement with compacted structural fill, or (2) improvement in-place by installation of stone columns.

Overexcavation and Replacement

For this alternative, soil would be excavated and replaced with Type 1 fill placed and compacted as specified for the embankment underdrain. Dewatering needed to accomplish overexcavation and replacement should consist of either well points or wells installed by an experienced dewatering contractor. Please refer to the draft dewatering specifications that Hart Crowser has previously submitted to HNTB for further information.

Stone Columns

As an alternative to overexcavation and backfill, the installation of stone columns could be used to mitigate soil liquefaction and avoid concerns about potential excess pore pressures in clayey soils.

Hart Crowser recommends that stone columns used for subgrade improvement below the toe of the embankment slope be designed and installed using the same criteria as previously recommended for subgrade improvements below the MSE walls (Hart Crowser, 2000f). We recommend the stone columns be nominal 42-inch-diameter, installed on a triangular pattern, 8 feet on center, to produce an area replacement ratio of 17 percent. Please refer to the draft stone column specifications that Hart Crowser has previously submitted to HNTB for further information.

Embankment Underdrain

Hart Crowser recommends that an embankment underdrain, consisting of a minimum 3-foot-thick layer of free-draining fill (Type 1) be placed under the entire footprint of the embankment. The purpose of the underdrain layer is to collect and discharge seepage without inducing any excess pore pressures in the embankment.

The underdrain should daylight in a drainage swale along the toe of the embankment, and each section of the swale should be sloped to enable gravity drainage. Locally the exposed face of the day-lighted underdrain should be protected from erosion because stability analyses for the Phase 3 fill indicated some potential for initiation of shallow instability associated with excess flow in the underdrain.

Contract provisions for thickening the drainage layer are recommended to accommodate variations in topographic relief of the existing ground surface and seeps that will be encountered. Seeps encountered within the embankment fill area should be hydraulically connected to the underdrain, and locally the drainage layer thickness may need to be increased to achieve this.

Prior to placing the underdrain, Hart Crowser recommends the surface of the area to be filled be graded as needed to prevent drainage within the underdrain from being impeded by topographic high points. An example of this would be where the fill will cross the existing ROW for 12th Avenue South that is higher in elevation that the native ground surface to the east and west.

Where existing seepage occurs in peat areas that will be stabilized with quarry spalls, Hart Crowser recommends excavation of finger drains which can be backfilled with Type 1 drain material to avoid any build-up of pore pressures as the peat consolidates. The underdrain layer in these areas should be protected with a filter geotextile as previously recommended.

In the event that Group 1 soils are not readily available, it may be possible to modify the specification for underdrain fill. In this case the drainage layer soil gradation should meet established filter criteria to ensure that a) drainage layer has adequate permeability relative to the overlying protected soil, and b) drainage layer has gradation that is resistant to piping erosion of overlying protected soil. Rather than attempt to cover all possible contingencies in the construction specification, Hart Crowser recommends specifying Group 1 soils for the underdrain, and addressing alternate materials through submittal review in the event this is required.

Embankment Fill

Table 1 summarizes the fill material gradation criteria used in the Phase 3 construction specifications. Hart Crowser recommends the Port and HNTB consider modifying the existing specifications (Port of Seattle, 2000) to broaden the allowable gradation range of acceptable fill material – see Appendix D.

Hart Crowser has reviewed field test data collected by Terra Associates for the 1998, 1999, and 2000 fill construction (Phases 1, 2, and 3). We find that the materials specified in the existing Phase 3 fill specification are readily available and can be placed and compacted within the specified limits.

Review of the test results and field notes indicates that problems with compaction below the specified minimum density are relatively infrequent overall. However, when such problems have occurred, this is consistently associated with soils wet of the specified limits. Accordingly, we do not recommend increasing the recommended compaction moisture contents or other fill placement and compaction parameters.

Slope Face Treatment

Hart Crowser recommends that the specifications include a requirement for overbuilding and trimming back the face of the embankment. This is a widely used construction practice that has been voluntarily implemented by one Contractor on some of the Third Runway embankment work to date.

Overbuilding and trimming the embankment provides some confinement that improves density of the fill at the final slope surface. This, along with slope track walking and revegetation reduces potential for erosion and instability of the slope face.

Many embankment slope faces are constructed without benching, and appear to perform well in most cases. Hart Crowser recommends the permanent embankment slope be benched to reduce potential erosion-related instability and to facilitate maintenance in the event this is required (Hart Crowser, 2000c). We understand HNTB is now reviewing bench arrangements that would allow an overall average slope of 2H:1V, with somewhat steeper sections between benches, and we anticipate Hart Crowser will assess stability of this configuration when it is completed.

USE OF THIS REPORT

This report is for the exclusive use of the Port of Seattle, HNTB, and their consultants for specific application to the subject project and site. We completed this study in accordance with generally accepted geotechnical engineering practices for the nature and conditions of the work completed in the same or similar localities, at the time the work was performed. We make no other warranty, express or implied.

In our opinion the explorations provide a reasonable basis for design and showing the proposed extent of subgrade improvements on the construction plans. Note, however, that the explorations performed for this study reveal subsurface conditions only at discrete locations across the project site and that actual conditions in other areas could vary. Furthermore, the nature and extent of any such variations would not become evident until additional explorations are performed or until construction activities have begun. If significant variations are observed at that time, we may need to modify our conclusions and recommendations accordingly to reflect actual site conditions.

If you have questions or if we can be of further assistance, please call.

Sincerely,

HART CROWSER, INC.

JAMES R. BEAVER, E.I.T.
Project Engineer

DOUG D. LINDQUIST, E.I.T.
Senior Staff Engineer

MICHAEL J. BAILEY, P.E.
Project Manager

BARRY S. CHEN, PH.D., P.E.
Senior Associate Engineer

F:\docs\jobs\497828\Analyses_Recommendations.doc

REFERENCES

AGI, 1998. Geotechnical Design Recommendations Phase 1 Embankment construction, Third Runway Project, Sea-Tac International Airport, Seatac, WA. January 22, 1998.

Hart Crowser, 1999a. Use of Taxiway Soils in 1999 Fill, Third Runway Embankment, Memorandum to Jim Thomson, HNTB, March 4, 1999.

Hart Crowser, 1999b, Subsurface Conditions Data Report, 404 Permit Support, Third Runway Embankment, Sea-Tac International Airport, SeaTac, Washington, July 1999.

Hart Crowser, 1999c. Geotechnical Engineering Report, 404 Permit Support, Third Runway Embankment, Sea-Tac International Airport, SeaTac, Washington, July 9, 1999.

Hart Crowser, 1999d. Sea-Tac Airport Third Runway, Probabilistic Seismic Hazard Analysis Results, Memorandum to Jim Thomson, HNTB, October 9, 1999.

Hart Crowser, 2000a. DRAFT Geotechnical Engineering Report, Phase 3 (Y2000) Fill, Third Runway Embankment, Sea-Tac International Airport, January 27, 2000.

Hart Crowser, 2000b. DRAFT Subsurface Conditions Data Report, North Safety Area, Third Runway Embankment, Sea-Tac International Airport, SeaTac, Washington, March 20, 2000.

Hart Crowser, 2000c. Memorandum: Bench Study for Slope Embankment; March 31, 2000

Hart Crowser, 2000d. DRAFT Subsurface Conditions Data Report, South MSE Wall and Adjacent Embankment, Third Runway Project, Sea-Tac International Airport, SeaTac, Washington, April 7, 2000.

Hart Crowser, 2000e. DRAFT Subsurface Conditions Data Report, West MSE Wall, Third Runway Embankment, Sea-Tac International Airport, SeaTac, Washington, June 2000.

Hart Crowser, 2000f. Preliminary Stability and Settlement Analyses, Subgrade Improvements, MSE Wall Support, Third Runway Project, June 2000.

Hart Crowser, 2000g. DRAFT Subsurface Conditions Data Report, Additional Field Explorations and Advanced Testing, Third Runway Embankment, Sea-Tac International Airport, Hart Crowser, August 2000.

Hart Crowser, 2000h. DRAFT Geotechnical Input into MSE Wall and Reinforced Slope Design, August 21, 2000.

Hart Crowser, 2000i. Memorandum: Methods and Results of Liquefaction Analyses, Third Runway Embankment, Sea-Tac International Airport, Hart Crowser, September 7, 2000.

Hart Crowser, 2000j. DRAFT Subsurface Conditions Data Report, Phase 4 Fill, Third Runway Embankment, Sea-Tac International Airport, Hart Crowser, November 29, 2000.

Kramer, S.L. Geotechnical Earthquake Engineering. Prentice Hall Inc., 2000a.

Port of Seattle, 2000. Project Manual Including Specifications for Third Runway: Embankment construction - Phase 3. March 6, 2000.

Sharma, S. 1994. XSTABL 5.0, Interactive Software Designs, Inc.

FAA, 1995. DRAFT Environmental Impact Statement for Proposed Master Plan Update Development Actions at Seattle-Tacoma International Airport. US Department of Transportation, Federal Aviation Administration, April, 1995.

Table 1 - Soil Gradations for Embankment Fill Material Groups

Sieve Size	Percent Passing
	Embankment Fill (Not Reinforced)
Group 1A	
6-inch	100
4-inch	-
3-inch	70 to 100
1¼-inch	-
¾-inch	50 to 77
U.S. No. 4	30 to 50
U.S. No. 40	3 to 15
U.S. No. 200(1)	0 to 5
Group 1B	
6-inch	100
4-inch	-
3-inch	70 to 100
1¼-inch	-
¾-inch	35 to 80
U.S. No. 4	20 to 55
U.S. No. 40	3 to 30
U.S. No. 200(1)	0 to 8
Group 2	
6-inch	100
4-inch	-
3-inch	70 to 100
1¼-inch	-
¾-inch	50 to 85
U.S. No. 4	30 to 65
U.S. No. 40	5 to 30
U.S. No. 200(1)	0 to 12
Group 3	
6-inch	100
U.S. No. 4	50 to 100
U.S. No. 40	20 to 60
U.S. No. 200(1)	0 to 35
Group 4	
6-inch	100
¾-inch	75 to 100
U.S. No. 4	50 to 100
U.S. No. 40	20 to 70
U.S. No. 200(1, 2)	0 to 50

1. The fine-grained soil percentage passing the U.S. No. 200 is based on the fraction of the soil passing the ¾-inch sieve
2. P.I. < 4 for fine-grained fraction.

F:\docs\jobs\497828\Analyses_Recommendations.doc

Table 2 - Water Level Data

	AT96-B1		AT96A-B8		AT96A-B10		AT97-B41		AT97-B42		AT97-B57		HC99-B31	
	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet
Measuring Point	0.00	*407.7*	0.00	*412.7*	0.00	*319.7*	0.00	*312.2*	0.00	*325.2*	0.00	*235.7*	0.00	266.24
Ground Level*	*-0.3*	*408*	*-0.3*	*413*	*-0.3*	*320*	3.2	*309*	3.2	*322*	*-0.3*	*236*	2.5	263.7
Top of Screen*	*77.7*	*330*	*49.7*	*363*	*23.7*	*296*	*81.2*	*231*	*22.7*	*303*	*13*	*395*	17.5	248.7
Bottom of Screen*	*87.7*	*320*	*60.7*	*352*	*33.7*	*286*	*83.2*	*229*	*27.7*	*298*	*23*	*385*	27.5	238.7
Date: 3/8/1999	-	-	-	-	-	-	Flowing	>312	21.21	303.9	-	-	2.38	263.86
3/10/1999	dry	dry	-	-	8.15	311.6	-	-	-	-				
4/5/1999	dry	dry	-	-	8.11	311.6	Flowing	>312	21.59	303.6	-	-	2.41	263.83
5/4/1999	dry	dry	-	-	8.35	311.4	0.91	311.2	22.17	303.0	-	-	2.58	263.66
5/15/1999	-	-	-	-	-	-	-	-	22.22	302.9				
6/14/1999	dry	dry	47.86	364.8	8.74	311.0	1.27	310.9	22.58	302.6	2.11	233.6	2.93	263.31
7/13/1999	dry	dry	48.87	363.8	8.81	310.9	1.41	310.7	Decommissioned		-	-	2.98	263.26
8/13/1999	dry	dry	48.10	364.6	9.06	310.6	1.57	310.6	Decommissioned		3.10	232.6	3.11	263.13
9/14/1999	dry	dry	48.26	364.4	9.44	310.3	1.73	310.4	Decommissioned		3.61	232.1	3.30	262.94
10/13/1999	dry	dry	48.49	364.2	9.74	310.0	1.77	310.4	Decommissioned		3.72	232.0	2.97	263.27
11/11/1999	87.88	319.8	48.75	364.0	10.00	309.7	1.52	310.6	Decommissioned		-	-	2.23	264.01
12/9/1999	dry	dry	48.88	363.8	9.50	310.2	0.72	311.4	Decommissioned		-	-	2.19	264.05
1/13/2000	dry	dry	49.10	363.6	9.18	310.5	0.69	311.5	Decommissioned		-	-	2.25	263.99
2/14/2000	dry	dry	50.25	362.5	9.02	310.7	0.49	311.7	Decommissioned		-	-	2.38	263.86
3/9/2000	dry	dry	49.05	363.7	8.63	311.1	0.48	311.7	Decommissioned		-	-	2.37	263.87
4/11/2000	dry	dry	48.63	364.1	8.65	311.1	0.88	311.3	Decommissioned		0.04	235.7	2.72	263.52
5/9/2000	dry	dry	48.40	364.3	8.86	310.8	1.01	311.1	Decommissioned		0.46	235.2	2.47	263.77
6/19/2000	dry	dry	48.23	364.5	8.91	310.8	1.37	310.8	Decommissioned		1.29	234.4	2.75	263.49
7/10/2000	62.93	344.8	48.18	364.5	9.21	310.5	1.52	310.6	Decommissioned		4.0	231.7	2.96	263.28
10/10/2000	dry	dry	48.64	364.1	10.71	309.0	2.00	310.2	Decommissioned		4.08	231.6	3.00	263.24

Italics = Estimated

Depth* All depths are below measuring point (NOT below the ground surface)

\- Indicates data not available.

Table 2 - Water Level Data

	HC99-B32		HC99-B33		HC99-B34		HC99-B35		HC99-B36		HC99-B41		HC99-B43A	
	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet
Measuring Point	0.00	266.29	0.00	265.65	0.00	267.63	0.00	294.58	0.00	275.03	0.00	330.8	0.00	295.58
Ground Level*	3.1	263.2	2.9	262.8	2.4	265.2	2.0	292.6	2.4	272.6	3	328	3	293
Top of Screen*	13.1	253.2	11.9	253.8	7.4	260.2	15.0	279.6	6.4	268.6	28	302.8	27.0	268.6
Bottom of Screen*	23.1	243.2	21.9	243.8	17.4	250.2	25.0	269.6	10.4	264.6	38	292.8	37.0	258.6
Date: 3/8/1999	3.55	262.74	2.71	262.94	4.72	262.91	4.69	289.89	4.73	270.30	31.87	308.86	-	-
3/10/1999											-	-	-	-
4/5/1999	3.51	262.78	2.64	263.01	4.68	262.95	5.13	289.45	5.01	270.02	32.57	308.16	Under Pressure	
5/4/1999	4.14	262.15	3.19	262.46	5.44	262.19	5.58	289.00	5.83	269.20	33.17	307.56	-10.1	306
5/15/1999											33.24	307.49	-	-
6/14/1999	4.75	261.54	2.61	263.04	5.88	261.75	6.48	288.10	6.23	268.80	33.56	307.17	-9.2	304.81
7/13/1999	4.83	261.46	3.72	261.93	5.86	261.77	6.79	287.79	6.42	268.61	33.77	306.96	-9.5	305.04
8/13/1999	5.05	261.24	3.90	261.75	6.12	261.51	7.29	287.29	6.68	268.35	33.97	306.76	-10.4	305.96
9/14/1999	5.21	261.08	4.09	261.56	6.16	261.47	7.76	286.82	7.85	267.18	24.18	306.52	-9.2	304.81
10/13/1999	4.77	261.52	3.70	261.95	5.89	261.74	7.79	286.79	6.91	268.12	24.29	306.53	-9.5	305.04
11/11/1999	3.02	263.27	2.90	262.75	4.32	263.31	6.40	288.18	5.60	269.43	24.00	306.82	-9.7	305.27
12/9/1999	3.08	263.21	1.44	264.21	4.35	263.28	Destroyed		4.29	270.74	23.06	307.76	-10.1	305.73
1/13/2000	3.32	262.97	1.65	264.00	4.54	263.09	Destroyed		4.31	270.72	23.00	307.82	-10.1	305.73
2/14/2000	3.56	262.73	1.81	263.84	4.81	262.82	Destroyed		4.50	270.53	22.68	308.14	-10.4	305.96
3/9/2000	3.63	262.66	1.82	263.83	4.92	262.71	Destroyed		4.49	270.54	22.62	308.20	-10.6	306.19
4/11/2000	4.32	261.97	2.48	263.17	5.55	262.08	Destroyed		5.71	269.32	23.16	307.66	-10.3	305.85
5/9/2000	3.48	262.81	1.91	263.74	4.67	262.96	Destroyed		5.87	269.16	23.38	307.44	-10.1	305.73
6/19/2000	4.51	261.78	2.69	262.96	5.73	261.90	Destroyed		6.14	268.89	23.76	307.06	-10.1	305.73
7/10/2000	4.73	261.56	3.00	262.65	5.84	261.79	Destroyed		6.48	268.55	23.94	306.88	-10.1	305.73
10/10/2000	4.65	261.64	2.80	262.85	5.77	261.86	Destroyed		6.97	268.06	24.53	306.29	-10.1	305.73

Table 2 - Water Level Data

	HC99-B44		HC99-B45		HC99-B46		HC99-B47		HC99-B48		HC99-B50		HC99-B51	
	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet
Measuring Point	0.00	275.82	0.00	285.29	0.00	332.93	0.00	281.22	0.00	270.02	0.00	287.9	0.00	278.56
Ground Level*	2.6	273.2	3.1	282.2	2.1	330.8	2.4	278.8	2.4	267.6	2.7	285.2	2.7	275.9
Top of Screen*	42.6	233.2	8.1	277.2	30.1	302.8	7.4	273.8	9.9	260.1	15.7	272.2	15.7	262.9
Bottom of Screen*	52.6	223.2	13.1	272.2	40.1	292.8	12.4	268.8	14.9	255.1	20.7	267.2	20.7	257.9
Date: 3/8/1999	-	-	6.70	278.59	22.01	310.81	-	-	-	-	-	-	-	-
3/10/1999			-	-	-	-	-	-						
4/5/1999	-	-	7.50	277.79	22.48	310.34	-	-	-	-	-	-	-	-
5/4/1999	-	-	7.93	277.36	23.09	309.73	6.26	274.96	-	-	-	-	-	-
5/15/1999			-	-	-	-	-	-						
6/14/1999	-	-	8.99	276.30	23.75	309.07	7.44	273.78	-	-	-	-	-	-
7/13/1999	-	-	9.00	276.29	24.17	308.65	6.99	274.23	-	-	-	-	-	-
8/13/1999	-	-	9.43	275.86	24.53	308.29	7.56	273.66	-	-	-	-	-	-
9/14/1999	-	-	11.09	274.20	24.84	307.98	9.03	272.19	-	-	-	-	-	-
10/13/1999	-	-	9.81	275.48	25.21	307.72	8.59	272.63	-	-	-	-	-	-
11/11/1999	-	-	8.38	276.91	25.33	307.60	5.12	276.10	-	-	-	-	-	-
12/9/1999	4.85	270.97	6.52	278.77	24.42	308.51	3.48	277.74	4.22	265.80	6.42	281.48	1.48	277.08
1/13/2000	4.92	270.90	7.34	277.95	23.85	309.08	3.13	278.09	4.63	265.39	6.68	281.22	1.81	276.75
2/14/2000	5.10	270.72	7.54	277.75	23.31	309.62	4.62	276.60	5.06	264.96	7.17	280.73	1.52	277.04
3/9/2000	5.05	270.77	7.34	277.95	23.14	309.79	3.51	277.71	5.11	264.91	7.27	280.63	1.75	276.81
4/11/2000	5.49	270.33	8.95	276.34	23.41	309.52	6.37	274.85	5.96	264.06	8.09	279.81	2.73	275.83
5/9/2000	5.38	270.44	9.18	276.11	23.77	309.16	6.72	274.50	5.08	264.94	8.41	279.49	3.02	275.54
6/19/2000	5.78	270.04	9.85	275.44	Decommissioned		7.06	274.16	6.39	263.63	8.79	279.11	3.88	274.68
7/10/2000	5.98	269.84	11.09	274.20	Decommissioned		7.81	273.41	6.76	263.26	9.61	278.29	4.49	274.07
10/10/2000	6.32	269.50	dry	dry	Decommissioned		12.29	268.93	6.76	263.26	12.75	275.15	7.62	270.94

497828\SeaTacWaterLevels.xls - NSA-404 Area

Table 2 - Water Level Data

	HC99-B52		HC99-B54		HC99-B55		HC99-B56		HC99-B57		HC99-B58		HC99-B61	
	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet
Measuring Point	0.00	287.34	0.00	277.94	0.00	289.81	0.00	295.25	0.00	296.08	0.00	293.60	0.00	303.94
Ground Level*	2.8	284.6	2.2	275.8	2.3	287.5	2.5	292.7	1.8	294.3	2.3	291.3	2.1	301.8
Top of Screen*	7.8	279.6	40.2	237.8	12.3	277.5	6.0	289.2	6.8	289.3	6.3	287.3	9.1	294.8
Bottom of Screen*	12.8	274.6	50.2	227.8	17.3	272.5	11.0	284.2	11.8	284.3	8.3	285.3	14.1	289.8
Date: 3/8/1999	-	-	-	-	-	-	-	-	-	-	-	-	-	-
3/10/1999														
4/5/1999	-	-	-	-	-	-	-	-	-	-	-	-	-	-
5/4/1999	-	-	-	-	-	-	-	-	-	-	-	-	-	-
5/15/1999														
6/14/1999	-	-	-	-	-	-	-	-	-	-	-	-	-	-
7/13/1999	-	-	-	-	-	-	-	-	-	-	-	-	-	-
8/13/1999	-	-	-	-	-	-	-	-	-	-	-	-	-	-
9/14/1999	-	-	-	-	-	-	-	-	-	-	-	-	-	-
10/13/1999	-	-	-	-	-	-	-	-	-	-	-	-	13.30	290.64
11/11/1999	-	-	-	-	-	-	-	-	-	-	-	-	13.58	290.36
12/9/1999	4.92	282.42	5.03	272.91	10.25	279.56	3.66	291.59	2.56	293.52	dry	dry	9.40	294.54
1/13/2000	5.29	282.05	5.04	272.90	10.32	279.49	4.09	291.16	2.36	293.72	dry	dry	7.89	296.05
2/14/2000	5.83	281.51	5.05	272.89	10.03	279.78	4.25	291.00	2.38	293.70	dry	dry	7.62	296.32
3/9/2000	5.81	281.53	5.06	272.88	10.20	279.61	4.16	291.09	2.50	293.58	dry	dry	7.55	296.39
4/11/2000	6.91	280.43	5.46	272.48	10.97	278.84	5.18	290.07	2.85	293.23	dry	dry	8.85	295.09
5/9/2000	7.36	279.98	5.45	272.49	11.68	278.13	5.51	289.74	2.53	293.55	dry	dry	9.72	294.22
6/19/2000	8.15	279.19	5.81	272.13	-	-	6.15	289.10	3.28	292.80	-	-	-	-
7/10/2000	9.07	278.27	5.98	271.96	Decommissioned		6.51	288.74	4.00	292.08	Decommissioned		Decommissioned	
10/10/2000	9.5	277.84	6.43	271.51	Decommissioned		6.81	288.44	3.91	292.17	Decommissioned		Decommissioned	

Table 2 - Water Level Data

	HC99-B64		HC99-B65		HC99-B71		HC99-B72		HC99-B73		HC00-B165		HC00-B173	
	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet
Measuring Point	0.00	294.2	0.00	348.12	0.00	304.46	0.00	283.81	0.00	293.80	0.00	278.48	0.00	276.13
Ground Level*	2.2	292.0	2.5	345.6	2.5	302.0	2.3	281.6	2.1	291.7	2.1	276.38	2.5	273.63
Top of Screen*	12.2	282.0	34.5	313.6	9.5	295.0	12.3	271.6	14.1	279.7	7.1	271.4	12.5	263.6
Bottom of Screen*	17.2	277.0	44.5	303.6	19.5	285.0	17.3	266.6	24.1	269.7	12.1	266.4	17.5	258.6
Date: 3/8/1999	-	-	-	-	-	-	-	-	-	-				
3/10/1999			-	-										
4/5/1999	-	-	-	-	-	-	-	-	-	-				
5/4/1999	-	-	-	-	-	-	-	-	-	-				
5/15/1999			-	-										
6/14/1999	-	-	-	-	-	-	-	-	-	-				
7/13/1999	-	-	-	-	-	-	-	-	-	-				
8/13/1999	-	-	-	-	-	-	-	-	-	-				
9/14/1999	-	-	-	-	-	-	-	-	-	-				
10/13/1999	-	-	40.23	307.89	14.53	289.93	-	-	10.38	283.42				
11/11/1999	-	-	40.44	307.68	14.60	289.86	-	-	10.10	283.70				
12/9/1999	6.87	287.3	39.64	308.48	7.20	297.26	5.35	278.46	5.25	288.55				
1/13/2000	6.05	288.2	38.95	309.17	6.76	297.70	4.32	279.49	4.50	289.30				
2/14/2000	6.22	288.0	38.51	309.61	6.66	297.80	4.28	279.53	4.50	289.30	4.67	273.81	9.82	266.31
3/9/2000	6.31	287.9	38.28	309.84	6.72	297.74	4.34	279.47	4.47	289.33	4.68	273.80	9.61	266.52
4/11/2000	7.36	286.8	38.44	309.68	7.87	296.59	6.06	277.75	5.64	288.16	5.01	273.47	11.12	265.01
5/9/2000	7.8	286.4	38.86	309.26	8.64	295.82	6.51	277.30	6.36	287.44	5.00	273.48	11.9	264.23
6/19/2000	8.15	286.1	39.38	308.74	10.60	293.86	6.89	276.92	7.18	286.62	5.23	273.25	12.35	263.78
7/10/2000	8.55	285.7	39.59	308.53	11.20	293.26	7.08	276.73	7.85	285.95	5.81	272.67	12.66	263.47
10/10/2000	9.34	284.9	40.44	307.68	Decommissioned		7.63	276.18	10.04	283.76	8.37	270.11	13.17	262.96

AR 049690

Table 2 - Water Level Data

	AT94A-B3		AT96-B4		AT97-B69		HC99-B37		HC99-B38		HC99-B39		HC99-B40	
	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet
Measuring Point	0.00	*273.4*	0.00	*279.7*	0.00	*337.2*	0.00	237.65	0.00	230.88	0.00	230.80	0.00	250.63
Ground Level*	1.4	*272*	*-0.3*	*280*	3.2	*334*	3.1	234.6	3.3	227.6	-0.3	231.1	2.0	248.6
Top of Screen*	23.4	*250.0*	36.7	*243.0*	*27.7*	*310*	9.1	228.6	12.3	218.6	4.7	226.1	14.0	236.6
Bottom of Screen*	33.4	*240.0*	46.7	*233.0*	*29.7*	*308*	19.1	218.6	22.3	208.6	14.7	216.1	24.0	226.6
Date: 3/8/1999					-	-	3.52	234.13	4.40	226.48	0.69	230.11	4.88	245.75
3/10/1999					6.18	331.0								
4/5/1999					6.59	330.6	3.58	234.07	4.41	226.47	0.74	230.06	5.26	245.37
5/4/1999					7.43	329.8	3.82	233.83	4.60	226.28	0.86	229.94	5.75	244.88
5/15/1999					-	-								
6/14/1999					8.08	329.1	5.12	232.53	5.90	224.98	1.68	229.12	6.89	243.74
7/13/1999					8.41	328.8	4.72	232.93	5.93	224.95	2.05	228.75	7.18	243.45
8/13/1999					8.83	328.4	5.70	231.95	6.08	224.80	2.18	228.62	7.13	243.50
9/14/1999					9.16	328.0	6.47	231.18	6.48	224.40	2.51	228.29	7.67	242.96
10/13/1999					9.12	328.1	4.50	233.15	5.98	224.90	2.09	228.71	7.32	243.31
11/11/1999					8.13	329.1	3.22	234.43	4.25	226.63	2.90	227.90	5.80	244.83
12/9/1999					6.80	330.4	3.27	234.38	4.38	226.50	0.27	230.53	5.00	245.63
1/13/2000					6.48	330.7	3.20	234.45	4.35	226.53	0.54	230.26	4.86	245.77
2/14/2000					6.54	330.7	3.12	234.53	4.33	226.55	0.59	230.21	4.49	246.14
3/9/2000	Flowing	>272	Flowing	>280	6.82	330.4	3.17	234.48	4.43	226.45	0.61	230.19	5.57	245.06
4/11/2000	Flowing	>272	Flowing	>280	7.45	329.8	3.35	234.30	4.60	226.28	0.88	229.92	5.08	245.55
5/10/2000	Flowing	>272	Flowing	>280	7.78	329.4	3.19	234.46	4.32	226.56	0.88	229.92	5.14	245.49
6/19/2000	-7.15	280.6	Flowing	>280	8.40	328.8	3.76	233.89	4.91	225.97	1.15	229.65	6.01	244.62
7/10/2000	-6.00	279.4	-1.73	281.4	8.84	328.4	3.96	233.69	5.72	225.16	1.61	229.19	6.50	244.13
10/10/2000	-1.38	274.8	-0.81	280.5	9.90	327.3	3.84	233.81	5.99	224.89	2.17	228.63	8.39	242.24

Italics = Estimated
Depth* All depths are below measuring point (NOT below the ground surface)
- Indicates data not available.

Table 2 - Water Level Data

	HC00-B106 Depth* in Feet	HC00-B106 Elevation in Feet	HC00-B111 Depth* in Feet	HC00-B111 Elevation in Feet	HC00-B118 Depth* in Feet	HC00-B118 Elevation in Feet	HC00-B120 Depth* in Feet	HC00-B120 Elevation in Feet	HC00-B121 Depth* in Feet	HC00-B121 Elevation in Feet	HC00-B123 Depth* in Feet	HC00-B123 Elevation in Feet	HC00-B125 Depth* in Feet	HC00-B125 Elevation in Feet
Measuring Point	0.00	315.81	0.00	286.06	0.00	298.61	0.00	236.93	0.00	231.78	0.00	237.64	0.00	257.8
Ground Level*	1.7	314.1	0.8	285.3	1.0	297.7	2.9	234.0	2.1	229.7	2.9	234.7	-0.4	258.2
Top of Screen*	11.7	304.1	9.8	276.3	7.0	291.7	17.6	219.3	6.8	225.0	14.0	223.7	3.6	254.2
Bottom of Screen*	21.7	294.1	19.8	266.3	12.0	286.7	22.6	214.3	16.8	215.0	24.0	213.7	8.6	249.2
Date: 3/8/1999														
3/10/1999														
4/5/1999														
5/4/1999														
5/15/1999														
6/14/1999														
7/13/1999														
8/13/1999														
9/14/1999														
10/13/1999														
11/11/1999														
12/9/1999														
1/13/2000														
2/14/2000	5.49	310.32	6.80	279.26	6.63	291.98								
3/9/2000	5.50	310.31	6.94	279.12	6.71	291.90								
4/11/2000	6.21	309.60	8.34	277.72	7.84	290.77	5.1	231.83	0.80	230.98	2.06	235.58	3.74	254.06
5/10/2000	6.38	309.43	8.59	277.47	7.64	290.97	4.92	232.01	0.62	231.16	1.90	235.74	3.89	253.91
6/19/2000	7.04	308.77	9.17	276.89	8.52	290.09	5.52	231.41	1.09	230.69	2.25	235.39	4.79	253.01
7/10/2000	7.47	308.34	9.95	276.11	9.12	289.49	6.1	230.83	1.53	230.25	2.61	235.03	6.39	251.41
10/10/2000	8.52	307.29	10.83	275.23	9.85	288.76	6.23	230.70	1.53	230.25	2.59	235.05	6.83	250.97

Italics = Estimated
Depth* All depths are below measuring point (NOT below the ground surface)
- Indicates data not available.

Table 2 - Water Level Data

	HC00-B126		HC00-B129		HC00-B130		HC00-B132		HC00-B133		HC00-B137		HC00-B141	
	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet
Measuring Point	0.00	251.56	0.00	245.83	0.00	225.46	0.00	229.96	0.00	243.47	0.00	267.21	0.00	258.64
Ground Level*	1.4	250.2	2.6	243.2	2.3	223.1	2.6	227.4	2.4	241.1	2.8	264.5	0.9	257.8
Top of Screen*	8.4	243.2	9.6	236.2	7.3	218.1	13.6	216.4	7.4	236.1	9.8	257.5	12.9	245.8
Bottom of Screen*	13.4	238.2	14.6	231.2	11.4	214.1	18.6	211.4	12.4	231.1	17.8	249.5	22.9	235.8
Date: 3/8/1999														
3/10/1999														
4/5/1999														
5/4/1999														
5/15/1999														
6/14/1999														
7/13/1999														
8/13/1999														
9/14/1999														
10/13/1999														
11/11/1999														
12/9/1999														
1/13/2000														
2/14/2000	2.02	249.54	3.07	242.76					2.51	240.96			7.94	250.70
3/9/2000	1.82	249.74	3.15	242.68					2.44	241.03			8.91	249.73
4/11/2000	1.97	249.59	3.59	242.24	1.73	223.73	2.65	227.31	2.53	240.94			10.21	248.43
5/10/2000	2.07	249.49	3.6	242.23	1.67	223.79	2.55	227.41	2.61	240.86			10.75	247.89
6/19/2000	2.54	249.02	4.56	241.27	1.92	223.54	2.91	227.05	3.97	239.50	2.54	264.67	11.50	247.14
7/10/2000	2.68	248.88	5.01	240.82	2.14	223.32	3.24	226.72	5.01	238.46	4.12	263.09	11.88	246.76
10/10/2000	5.15	246.41	6.6	239.23	1.88	223.58	3.29	226.67	5.94	237.53			12.99	245.65

Italics = Estimated

Depth* All depths are below measuring point (NOT below the ground surface)

\- Indicates data not available.

497828\SeaTacWaterLevels.xls - West Wall

AR 049693

Table 2 - Water Level Data

	HC00-B142		HC00-B143		HC00-B144		HC00-B145		HC00-B146		HC00-B301		HC00-B308	
	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet
Measuring Point	0.00	272.72	0.00	242.27	0.00	248.99	0.00	265.11	0.00	263.55				
Ground Level*	2.7	270.1	3.2	239.1	2.4	246.6	2.3	262.8	2.9	260.7				
Top of Screen*	17.2	255.6	7.2	235.1	8.9	240.1	12.3	252.8	11.9	251.7				
Bottom of Screen*	22.2	250.6	12.2	230.1	13.9	235.1	17.3	247.8	16.9	246.7				
Date: 3/8/1999														
3/10/1999														
4/5/1999														
5/4/1999														
5/15/1999														
6/14/1999														
7/13/1999														
8/13/1999														
9/14/1999														
10/13/1999														
11/11/1999														
12/9/1999														
1/13/2000														
2/14/2000							2.94	262.17						
3/9/2000	3.27	269.45			2.98	246.01	3.14	261.97	3.81	259.74				
4/11/2000	3.53	269.19			3.17	245.82	3.51	261.60	4.02	259.53				
5/10/2000	3.58	269.14			3.00	245.99	3.18	261.93	3.79	259.76				
6/19/2000	4.57	268.15	4.23	238.04	4.11	244.88	3.95	261.16	4.56	258.99				
7/10/2000	5.17	267.55	5.96	236.31	5.76	243.23	4.59	260.52	5.92	257.63				
10/10/2000	6.19	266.53			7.07	241.92	5.84	259.27	6.26	257.29	3.64		9.83	

Italics = Estimated

Depth* All depths are below measuring point (NOT below the ground surface)

\- Indicates data not available.

Table 2 - Water Level Data

	HC00-B309 Depth* in Feet	HC00-B309 Elevation in Feet	HC00-B310 Depth* in Feet	HC00-B310 Elevation in Feet	HC00-B311 Depth* in Feet	HC00-B311 Elevation in Feet
Measuring Point						
Ground Level*						
Top of Screen*						
Bottom of Screen*						
Date: 3/8/1999						
3/10/1999						
4/5/1999						
5/4/1999						
5/15/1999						
6/14/1999						
7/13/1999						
8/13/1999						
9/14/1999						
10/13/1999						
11/11/1999						
12/9/1999						
1/13/2000						
2/14/2000						
3/9/2000						
4/11/2000						
5/10/2000						
6/19/2000						
7/10/2000						
10/10/2000	35.66		9.49		10.01	

Table 2 - Water Level Data

	AT94A-B1		AT97-B8		AT97-B59		AT97-B61		AT97-B63		HC00-B203		HC00-B205	
	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet
Measuring Point	0.00	*356.2*	0.00	*379.2*	0.00	*312.9*	0.00	*328.0*	0.00	*330.5*	0.00	310.95	0.00	306.19
Ground Level*	1.2	*355*	2.2	*377*	2.9	*310*	3.0	*325*	2.5	*328*	2.0	309.0	2.8	303.4
Top of Screen*	74.2	282.0	15.2	364.0	22.4	290.5	32.0	296.0	42.0	288.5	33.0	278.0	12.8	293.4
Bottom of Screen*	84.2	272.0	20.2	359.0	24.4	288.5	34.0	294.0	44.0	286.5	38.0	273.0	17.8	288.4
Date: 3/10/2000	62.50	293.7	4.65	374.6	14.35	298.6	33.01	295.0	37.94	292.6	25.32	285.63	13.55	292.64
4/11/2000	62.51	293.7	5.50	373.7	14.87	298.0	33.40	294.6	37.98	292.5	25.63	285.32	16.45	289.74
5/10/2000	62.60	293.6	5.83	373.4	15.03	297.9	dry		38.03	292.5	25.82	285.13	dry	
6/20/2000	62.85	293.4	7.02	372.2	15.41	297.5	dry		38.19	292.3	26.19	284.76	16.6	289.59
7/10/2000	62.93	293.3	7.88	371.3	15.59	297.3	dry		38.26	292.2	26.51	284.44	dry	
10/10/2000	63.49	292.7	10.39	368.8	16.18	296.7	dry		38.59	291.9	27.51	283.44	dry	

Italics = Estimated

Depth* All depths are below measuring point (NOT below the ground surface)

\- Indicates data not available.

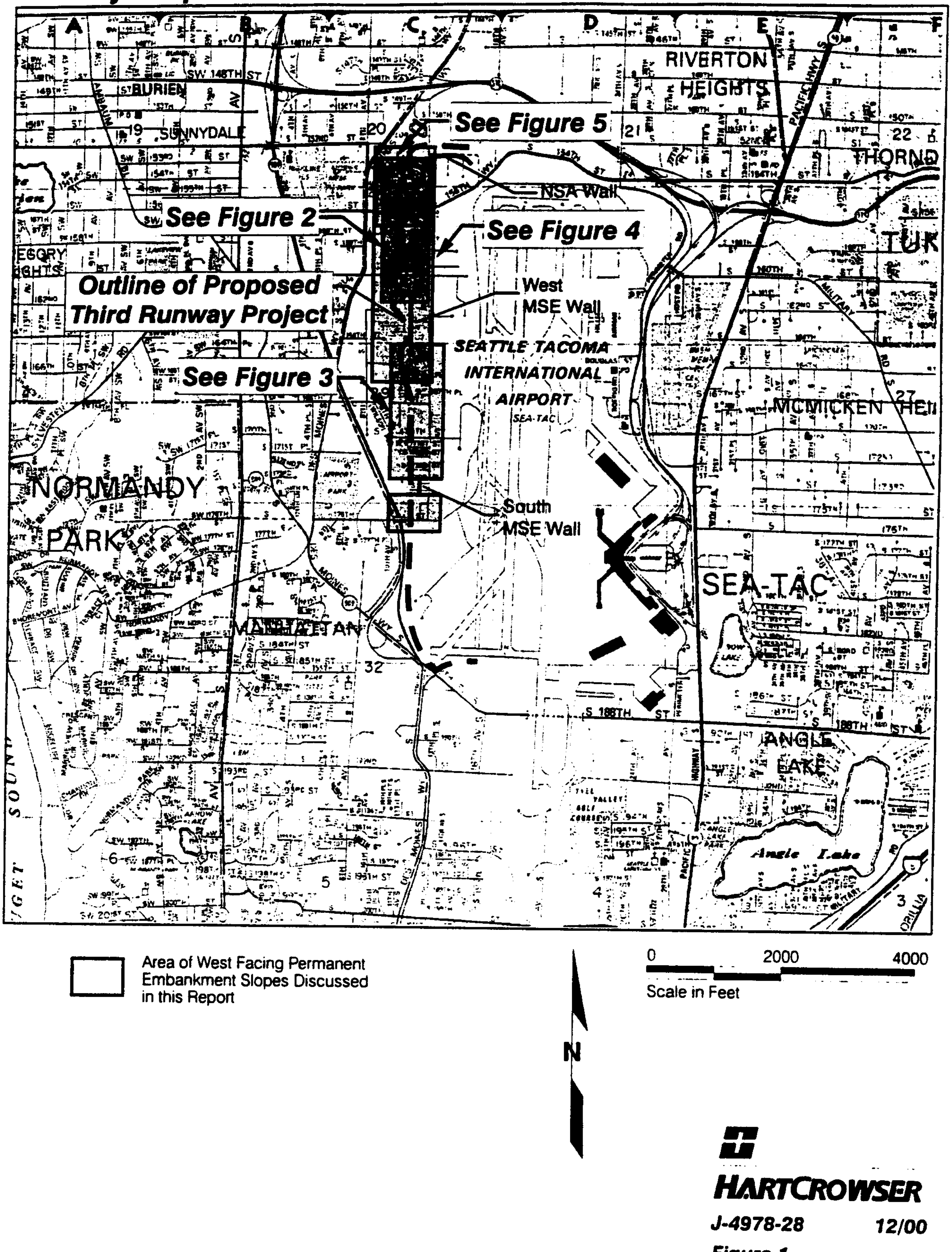
497820\SeaTacWaterLevels.xls\South Wall

AR 049696

Table 2 - Water Level Data

		HC00-B208		HC00-B211		HC00-B213	
		Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet	Depth* in Feet	Elevation in Feet
Measuring Point		0.00	278.67	0.00	301.70	0.00	313.35
Ground Level*		2.4	276.3	2.3	299.4	2.4	311.0
Top of Screen*		29.9	248.8	16.3	285.4	12.4	301.0
Bottom of Screen*		34.9	243.8	21.3	280.4	22.4	291.0
Date:	3/10/2000	0.43	278.24	1.51	300.19	15.47	297.88
	4/11/2000	0.6	278.07	2.2	299.50	16.08	297.27
	5/10/2000	0.71	277.96	2.29	299.41	16.22	297.13
	6/20/2000	0.98	277.69	2.96	298.74	16.37	296.98
	7/10/2000	1.14	277.53	3.42	298.28	16.48	296.87
	10/10/2000	1.62	277.05	4.31	297.39	16.81	296.54

Vicinity Map



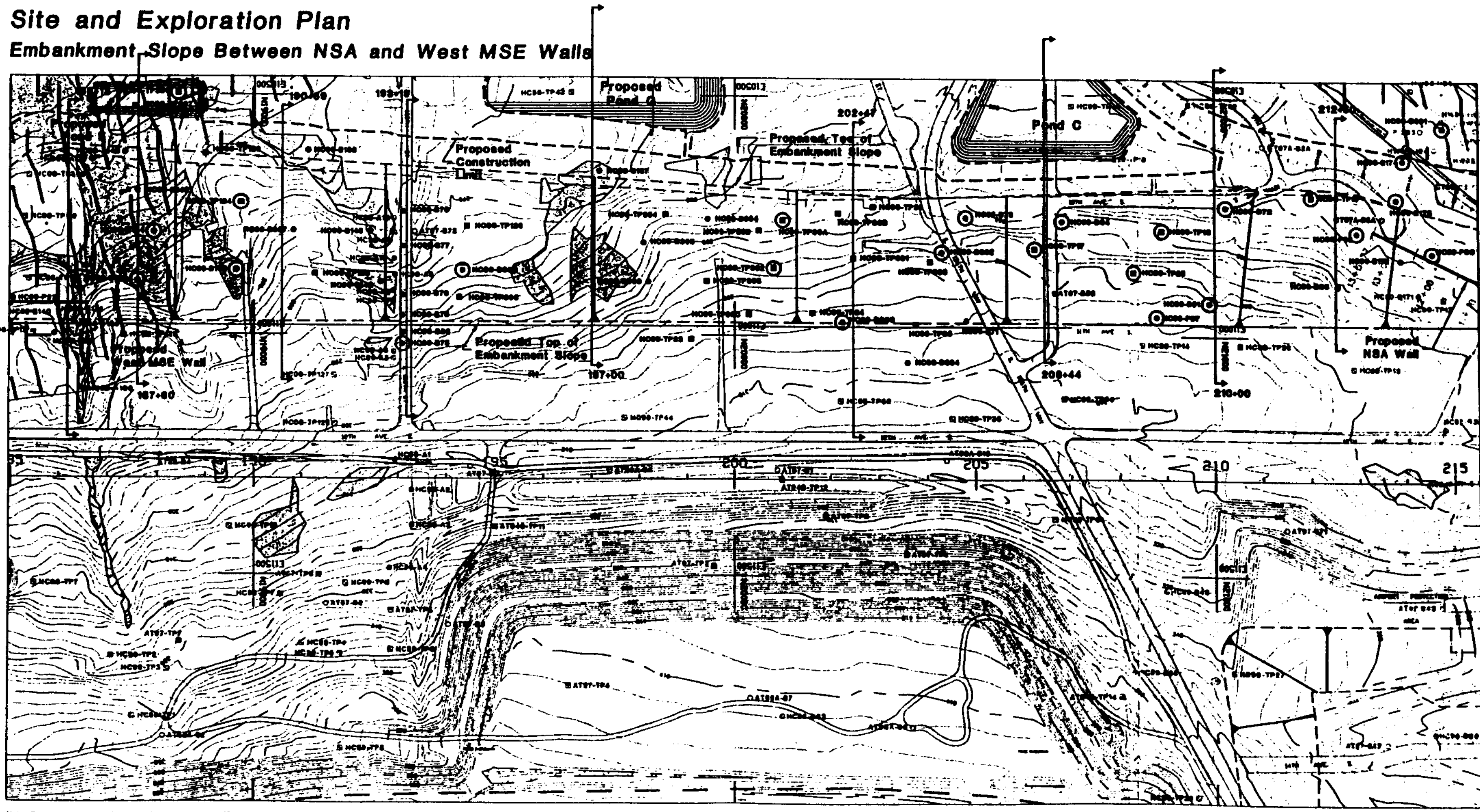
497828BB.cdr RC 12/1/00

HARTCROWSER

J-4978-28 12/00

Figure 1

Site and Exploration Plan
Embankment Slope Between NSA and West MSE Walls



Note: Base map prepared from drawing provided by HNTB entitled "Topo_Full.dwg", dated October 4, 1999. Wetland delineation provided Parametric drawing entitled, "W_110800.dwg", dated November 8, 2000.

Estimated Liquefaction Susceptibility (10% Probability of Exceedence in 50 Years)

HC00-TP86A Exploration with Existing Liquefaction Susceptible Soils

EIS Seismic Hazard Area

HC00-P00 Cone Probe Location and Number

HC00-B000 Boring Location and Number

HC00-TP100 Test Pit Location and Number

Wetland Location

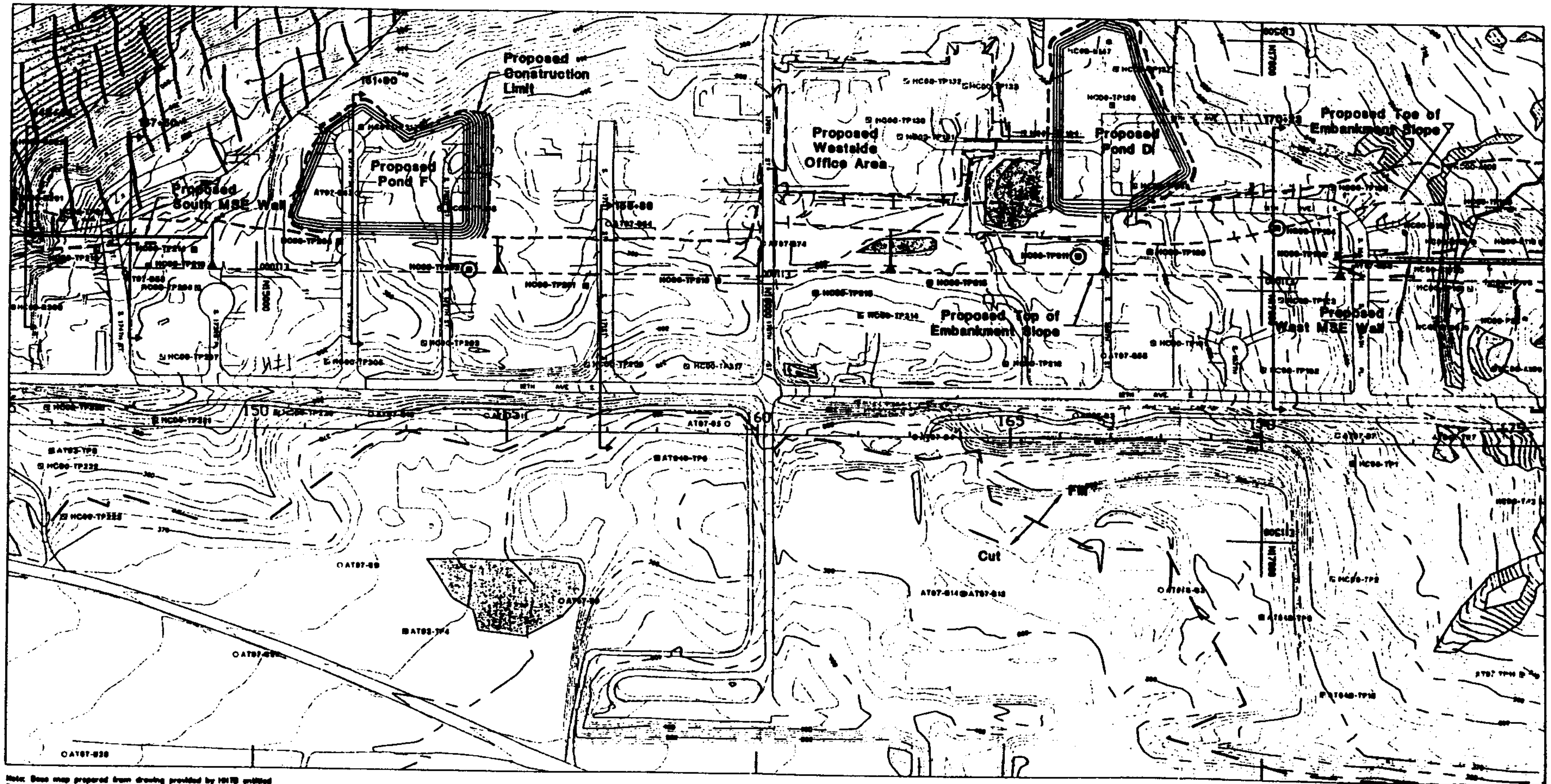
210+00 Cross Section Location and Designation

0 200 400
Scale in Feet

HARTCROWSER
J-4978-28 12/00
Figure 2

Site and Exploration Plan

Embankment Slope Between West and South MSE Walls



Note: Base map prepared from drawing provided by HNTB entitled "Topo_full.dwg", dated October 4, 1999. Wetland delineation provided Parametrix drawing entitled, "W_110800.dwg", dated November 8, 2000.

Estimated Liquefaction Susceptibility (10% Probability of Exceedence in 50 Years)

HC00-TP88A Exploration with Existing Liquefaction Susceptible Soils

EIS Seismic Hazard Area

Cone Probe Location and Number

Boring Location and Number

Test Pit Location and Number

Wetland Location

Cross Section Location and Designation

N

0 200 400

Scale in Feet

HARTCROWSER

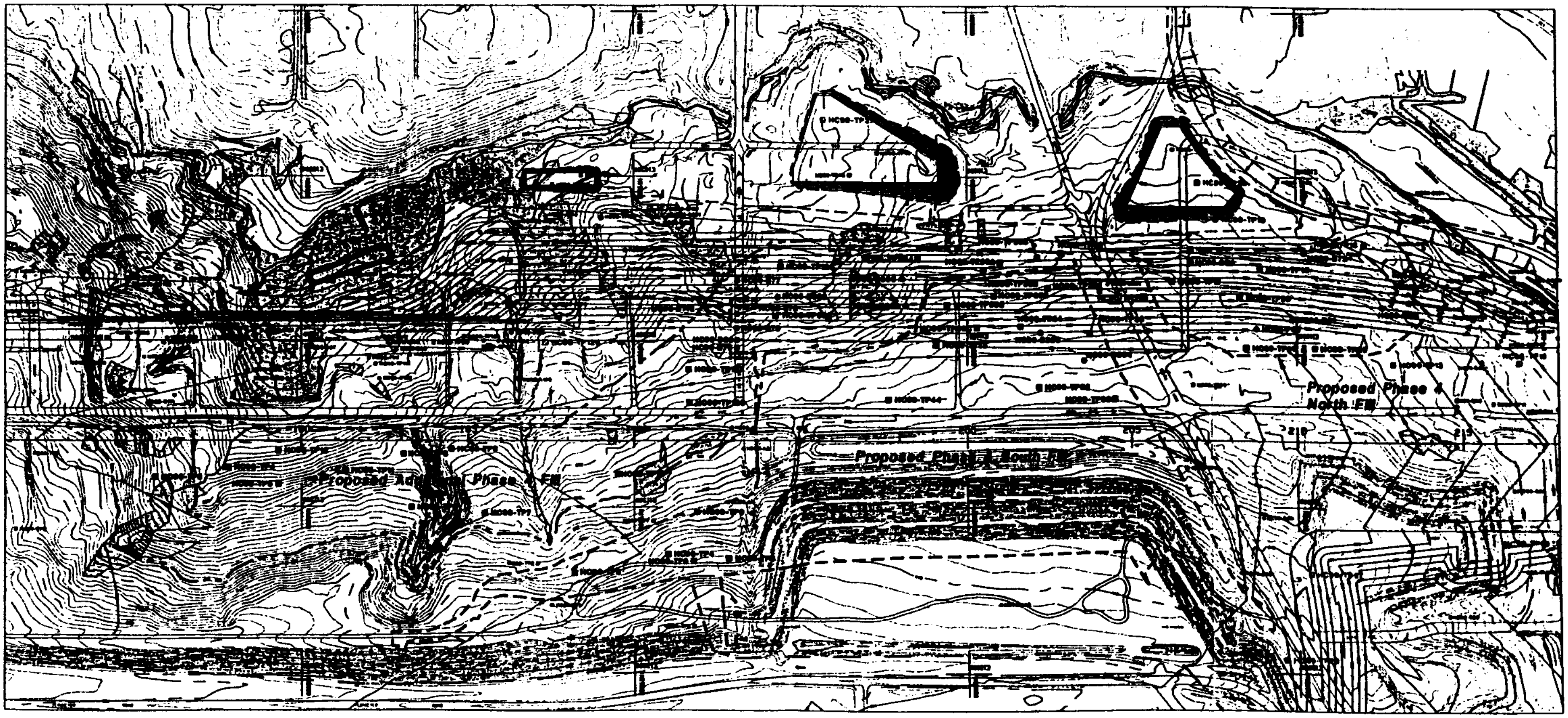
J-4978-28 12/00

Figure 3

AR 049700

Site and Exploration Plan

Phase 4 Fill



Notes:

1) Base map prepared from drawing provided by HNTB entitled "Topo_Full.dwg", dated October 4, 1999. Wetlands delineations prepared from drawing provided by Parametrix entitled, "w_110800.dwg", dated November 8, 2000.

2) Phase 4 Fill limits based on drawing provided by HNTB, November 2000.

HC00-TP8 Exploration Location and Designation

N

0 300 600

Scale in Feet

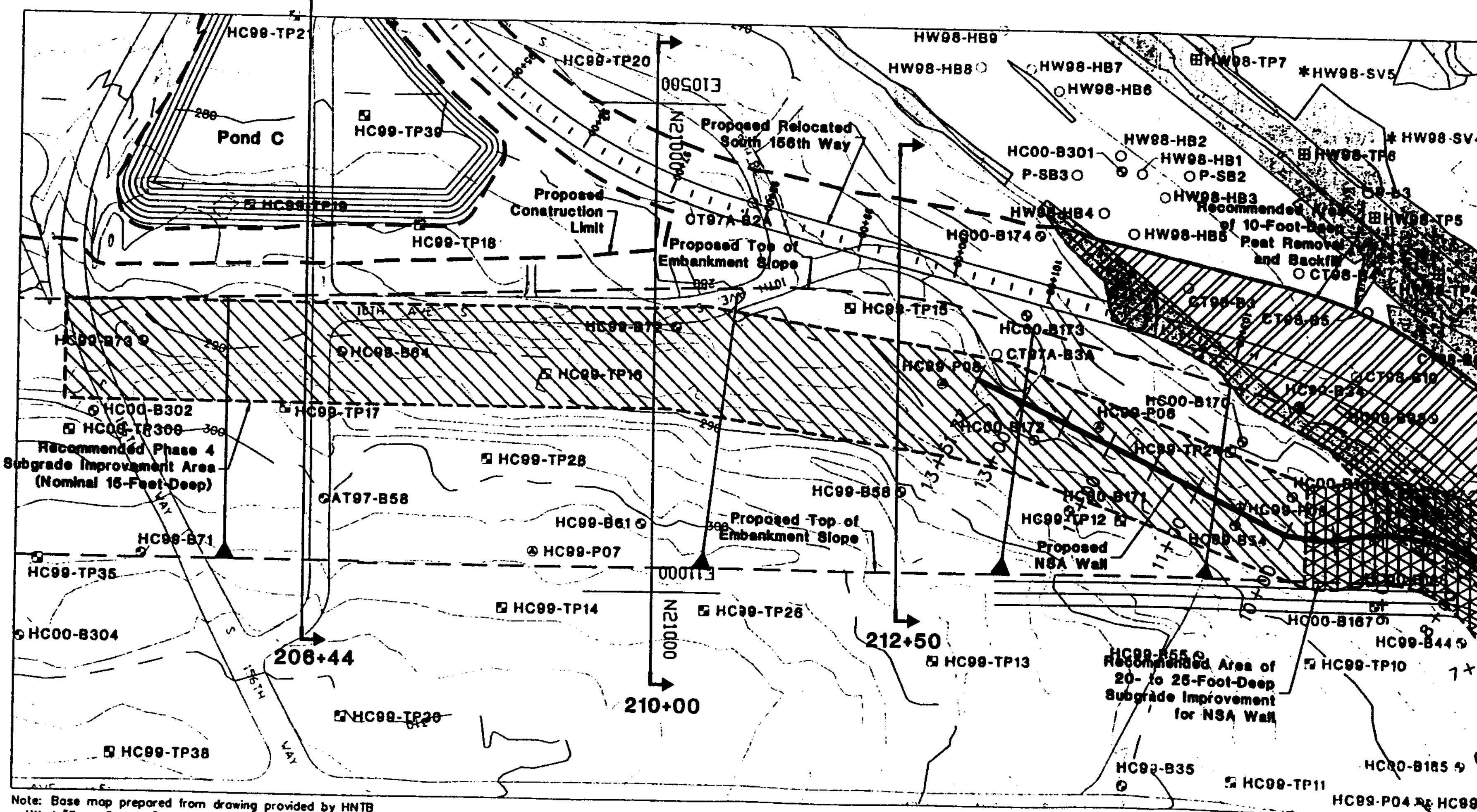
HARTCROWSER

J-4978-28 12/00

Figure 4

AR 049701

Recommended Phase 4 Subgrade Improvement Area



Note: Base map prepared from drawing provided by HNTB entitled "Topo_Full.dwg", dated October 4, 1999. Wetland delineation prepared from drawing provided by Parametrix entitled, "W_110800.dwg", dated November 8, 2000.

HC99-P08	Cone Probe Location and Number
HC00-B306	Boring Location and Number
HC00-TP126	Test Pit Location and Number
	Wetland Location
210+00	Cross Section Location and Designation

N

0 100 200
Scale in Feet

HARTCROWSER
J-4978-28 12/00
Figure 6

AR 049702

APPENDIX A
SLOPE STABILITY AND DEFORMATION ANALYSES

APPENDIX A
SLOPE STABILITY AND DEFORMATION ANALYSES

Introduction

This appendix provides a geotechnical description of the slope stability and deformation analyses used for design of the permanent embankment slopes for the Third Runway. This discussion includes the design assumptions, methods of analyses, design criteria and input soil parameters. Two major types of analyses are included:

- Limit equilibrium analyses accomplished with various computer codes that include the methods of analysis developed by Spencer, Bishop, and Janbu; and
- A finite difference method for analysis of stress and deformation, using the FLAC computer code.

The objective of our stability analysis was to verify stability of the proposed embankment and, as needed, to define areas of subgrade improvement to assure foundation stability.

Limit Equilibrium Slope Stability Analyses

Limit equilibrium analyses were accomplished with the computer programs Slopes/W and XSTABL.

Types of Analyses

Slope stability analyses and target factors of safety are shown in Table A-1. A complete discussion of these analysis conditions is provided in Hart Crowser (2000f).

Table A-1 - Limit Equilibrium Analyses

Analysis Condition	**Target FS**
Undrained end of construction (EOC)	> 1.3
Partially drained EOC	> 1.3
Pseudo-static	> 1.0 (1)
Liquefaction	> 1.1
Steady state conditions	> 1.5

(1) Note target factor of safety: greater than 1.1 for global stability for pseudo-static conditions; greater than 1.0 for infinite slope type failure through toe.

For the pseudo-static case, Hart Crowser found that factors of safety were consistently around 1.05 for shallow infinite slope-type failures near the toe of the slope. We accepted a slightly lower minimum factor of safety of 1.0 for this case for the 2H:1V slope, since the overall integrity of the embankment was not compromised, and a minimum FS of 1.0 for a large seismic event is consistent with accepted standards of practice for slope stability analysis.

Hart Crowser developed slope cross sections to depict changes in slope geometry and subsurface conditions at intervals of about 250 to 400 feet where the slope height exceeded about 50 feet, and additional representative sections where slope heights were less than 50 feet in height. These sections were closely reviewed to identify potential problematic subsurface conditions and seven sections were selected for detailed analyses. (The selected sections are shown in Appendix B. Note these illustrations are general in nature and actual details were varied for analyses.)

Cross sections at Stations 193+19 and 206+44 were analyzed for the complete suite of analysis conditions indicated in Table A-1. Based on the results of these representative cross sections, we proceeded to analyze the other sections for the pseudo-static, liquefaction, and steady state conditions. Since subsurface conditions in the area of Stations 170+23 and 187+60 did not include clay/silt soils, the EOC and partially drained EOC were not analyzed for these sections.

Input Soil Parameters

Input soil parameters were selected based on the results of field and laboratory tests, specifically correlations of SPT blow counts and CPT measurements for granular soils, and laboratory triaxial tests for fine-grained soils (see Hart Crowser, 2000b, 2000d, 2000e, and 2000g).

Susceptibility to Liquefaction

Susceptibility to liquefaction was determined as shown in Hart Crowser (2000i). In determining depth to groundwater, data from monitoring wells that have been measured over an extended period of time were considered to provide the best quality data. Groundwater levels measured in recently installed wells, groundwater observations at the time of drilling or test pit excavation, and groundwater elevation contour maps (i.e. Hart Crowser, 2000b) were used as a secondary data source to interpolate design groundwater levels.

Analysis of Excess Pore Pressure in Silt and Clay

For the partially drained and more conservative EOC cases, we used undrained shear strength for the silt/clay soil assuming the embankment is constructed so rapidly that the load is transferred to pore water. The soft to medium stiff silt and clay was assigned 1,000 psf for this case, while the stiff to hard silt/clay was assigned 3,500 psf, based on CU and UU triaxial testing.

For the three sections north of Station 205+00 where EOC stability was below the target value of factor of safety, our approach was to estimate the build-up of pore pressure in the silt/clay considering actual construction rates that would be expected for the embankment. To simulate the drainage characteristics in the silt/clay, we used one dimensional consolidation theory to estimate the buildup of excess pressure in the silt/clay for an assumed rate of loading. *In situ* piezocone pressure dissipation test and laboratory consolidation tests were used to determine a value of coefficient of consolidation to represent the permeability of the silt/clay.

The layer thickness we assumed was either 5 or 8 feet for the affected cross sections based on borings, giving some consideration to the presence of thin sandier layers in the silt/clay that would tend to enhance drainage assuming adequate lateral extent. We assumed double drainage conditions since the silt/clay is generally surrounded by sand soil. Using a rate of fill placement up to 4 feet per day, the excess pore pressure under full embankment loads approaches 5,700 psf (>90 feet of water) for an 8-foot-thick clay layer and 1,800 psf (>29 feet of water) for a 5-foot-thick clay layer. For this modeled condition of partial drainage, factors of safety are above target values.

Other Limit Equilibrium Assumptions

Other assumptions used in our stability analyses include:

- Seismic basis of design was based on a 10 percent probability of exceedence in 50 years (Hart Crowser, 1999d) with a pseudo-static acceleration equal to 50 percent of the peak horizontal ground acceleration.
- Laterally extensive areas of peat would be removed or, if left in-place they would be filled over with gravel or quarry spalls compacted into the peat.
- A minimum 3-foot-thick drainage layer will prevent development of hydrostatic positive pore pressures within the embankment.
- For cases except the liquefaction case, the drainage layer is water-filled with hydrostatic groundwater conditions below. For the liquefaction case, maximum observed or projected existing groundwater level was used.

- Embankment soils will be compacted to a density and moisture level consistent with that previously specified, sufficient to provide a friction angle of phi = 35 degrees.

A summary of unit weight and shear strength parameters for the embankment analyses is listed in Table A-2.

Table A-2 - Summary of Input Soil Parameters

Soil Type	Unit Weight	Drained Strength		Undrained Strength	
		c'	ϕ'	c	ϕ
	in pcf	in psf	in deg.	in psf	in deg.
Existing Subgrade Soils					
Loose to Medium Dense Sand	125	0	32	675 (1)	0
Medium Dense to Dense Sand	130	0	35	-	-
Dense to Very Dense Sand	135	0	37	-	-
Glacial Till	130	250	40	-	-
Soft Peat or Organic Silt (Topsoil)	110	0	15	-	-
Medium Stiff Silt/Clay(2)	115	0	32	1000	0
Stiff to Hard Silt/Clay(2)	115	0	32	3500	0
Post Construction Soils					
Embankment Fill	135	0	35	-	-
Drainage Blanket	140	0	37	-	-
Improved Subgrade	135	0	35	-	-

(1) Undrained strength parameters were used for post-liquefaction residual strength for loose to medium dense sand for the liquefaction case, as applicable. This is discussed further in the text below.

(2) Undrained strength parameters were used for the end-of-construction cases, otherwise, drained strength properties were used.

Results of Limit Equilibrium Analyses

Table A-3 presents a general summary of the results of the limit equilibrium stability analyses for the 2H:1V embankment slopes.

Table A-3 - Generalized Summary of Limit Equilibrium Results.

Station	EOC, Undrained, (F.S. > 1.3)	EOC, Partially Drained, (F.S. > 1.3)	Pseudo-Static, (F.S. > 1.0)	Liquefaction, (F.S. > 1.1)	Steady State, (F.S. > 1.5)
170+23	N/A	N/A	OK	Global OK	OK
187+60	N/A	N/A	OK	Global OK	OK
193+19	OK	OK	OK	Global OK	OK
202+47	OK	OK	OK	Global OK	OK
206+44	NO	OK	OK	NO	OK
210+00	NO	OK	OK	NO	OK
212+50	NO	OK	OK	NO	OK

The results shown in Table A-3, indicate the need for subgrade improvements to prevent liquefaction and subgrade improvement or other mitigation to prevent excess pore pressure-related instability in the area extending from about Stations 205+00 to 213+50, as shown on Figure 5. Subgrade improvements are recommended north of the Station 205+00 because:

(1) Potentially liquefiable soils (and soils susceptible to development of excess pore pressures due to construction) are relatively continuous in this area; and

(2) Proximity of the embankment to an urban street (relocated South 154th Way) justifies a high degree of precaution to prevent potential injury or loss of life.

South of Station 205+00, the potential benefit of subgrade improvements is less clear: potentially liquefaction-susceptible soils are not contiguous over large areas; and the extent of liquefaction from the design seismic event (475 year return period) would not produce global (large-scale) instability of the embankment slope.

Based on discussions with HNTB and other members of the design team, Hart Crowser undertook a series of special studies to further define the potential extent of liquefaction, and its effects, south of Station 205+00. These studies included:

- Additional special explorations;
- Statistical analysis of liquefaction-susceptibility and estimated post-liquefaction residual strength; and
- Probabilistic based deformation analysis of a representative slope section with non-continuous liquefaction conditions.

Additional Special Explorations

Test pits used to explore subsurface conditions allow subjective assessment but are not directly amenable to quantitative assessment of liquefaction susceptibility. To check interpretation of conditions observed in test pits, Hart Crowser accomplished additional special explorations (borings and CPT probes) to verify our interpretation of conditions in along the embankment slope south of Station 205+00. Borings were accomplished with special attention to quality of the SPT data. The CPT work is in progress at the time this report is being prepared.

Statistical Analysis of Liquefaction

Analysis of the effect of liquefaction on stability was based on two different approaches:

(1) Analysis of global liquefaction with single residual strength value; and
(2) A "composite strength" approach that more accurately represents the variability in existing subsurface conditions.

In general, as the recurrence interval of the earthquake decreases, the magnitude of the earthquake decreases, the extent of liquefaction-susceptible soil decreases, and the average post-liquefaction (undrained) residual strength decreases. This is an important observation since it explains why the Port should anticipate that some small slope failures are likely to occur along the toe of the 2H:1V slope in areas which analyses show have acceptable factors of safety against large scale (global) instability. To illustrate:

- In a small earthquake, very loose sands might liquefy and they would have very low residual strengths.
- In a larger earthquake, more dense sands may also liquefy such that average residual strength would increase because it includes both very loose sands and more dense sands.

The cross sections analyzed south of Station 205+00 have acceptable post-liquefaction global stability based on exceeding the minimum target factor of safety. However, local failures at the toe of the embankment could occur where liquefaction occurs over limited areas that are not extensive enough to produce large-scale instability. The effect for small earthquakes is limited: while the average undrained residual strength decreases, the spatial extent of liquefaction also decreases, and small soil slumps may occur in areas where complete liquefaction does not occur.

Conservative Assumption of Saturated Conditions Used for Comparison

Hart Crowser found that the depth and continuity of potentially liquefaction-susceptible soils varied widely south of Station 205+00. In addition, both susceptibility to liquefaction and magnitude of post-liquefaction residual shear strength varied by location and, importantly, with the size of the earthquake. To illustrate this, Hart Crowser assessed results of liquefaction analysis for all the samples from borings below the embankment slope, south of Station 205+00. ***To provide a large enough database to support statistical comparisons, all samples above the glacially overridden soils were included in this analysis, including those that are above the groundwater table. Inclusion of unsaturated soils is to enable comparison only and does not represent the actual prevalence of liquefaction-susceptible soils below the embankment slope.*** Using results of borings only below the embankment slope eliminates the subjectivity of using soils information from the test pits and possible differences in soil conditions elsewhere on the project site. Note that at the time this report is being prepared, additional CPT explorations are in progress, which will produce a large enough data set that the non-saturated samples can be eliminated from the analysis presented below. Table A-4 presents results of the analysis (assuming complete saturation of all samples).

Table A-4 Results of Liquefaction Analysis for Different Earthquake Events

Seismic Return Interval	Proportion of Liquefaction-Susceptible Samples	Post-liquefaction Undrained Residual Shear Strength in psf
475-year	33%	675
300-year	21%	500
175-year	9%	290
72-year	0%	N/A

Note: see Hart Crowser (2000i) for details of the method of analysis used.

The percentage of liquefaction-susceptible samples, and the post-liquefaction residual strength for those samples were used in the analyses discussed below.

- A parametric analysis was accomplished to evaluate the maximum slope height that would meet the target factor of safety - assuming liquefaction occurred for all the soil below the slope.
- A second parametric analysis assessed the effect of partial liquefaction through use of a composite shear strength. This composite strength was obtained using both stress dependent (i.e., soil strength based on a normal load and friction angle, phi, for the non-liquefied soil) and stress independent (i.e., undrained residual strength, for the liquefied soil) components.

Note that results of these analyses are very conservative because actual degree of saturation and hence the number of liquefaction susceptible soils is much lower than assumed to generate the values used in the analyses.

Assumption of Complete Liquefaction

For the assumption of complete liquefaction of foundation soils, the limit equilibrium analyses showed that the factor of safety for a global failure through subgrade soils is a function of embankment height. For the design value of 675 psf representing the post-liquefaction residual shear strength for the 475-year seismic event, Hart Crowser found that factor of safety exceeded the target values for slopes less than about 60 feet in height. This result indicates that the area south of the west wall would not suffer large instability due to liquefaction for the design event. However, some local instability may result for less than complete liquefaction, where lower residual shear strength occurs over limited areas.

We used the cross section at Station 170+23, which has a fill height of about 40 feet, as a check to validate the parametric analysis.

Composite Strength Analysis for Partial Liquefaction

For the case of partial or discontinuous liquefaction, stability analyses were accomplished with a composite soil shear strength calculated as a function of embankment height.

The composite shear strength value represents an average value in which the proportion of liquefiable soil on the critical failure surface is assigned an undrained residual shear strength, while the remaining proportion of soil is assigned a drained shear strength for non-liquefiable soils (i.e., phi = 32 degrees for loose to medium dense sand or medium stiff silt and clay).

Based on the statistical analysis noted above, the 475-year event would cause liquefaction of 33 percent of the subgrade soil samples analyzed, and 67 percent non-liquefiable soil. For this condition:

$$\tau = 1/3 * c + 2/3 * \sigma_v * \tan(\phi)$$

Since the contribution to total shear strength for the frictional component of is dependent upon embankment height, we performed a parametric analysis for a representative cross section (i.e., Station 193+19) to determine factor of safety for given embankment heights. We averaged the variation in height of the embankment slope by using one-half of the embankment height to calculate the vertical overburden stress. The results are shown below.

- Height of 40 feet, S_{avg} = 2,472 psf
- Height of 65 feet, S_{avg} = 3,885 psf
- Height of 85 feet, S_{avg} = 5,015 psf
- Height of 115 feet (max for Station 193+19), S_{avg} = 6,709 psf

Using estimated composite shear strength for partial liquefaction based on the design event, Hart Crowser obtained acceptable factors of safety for all slopes south of Station 205+00.

Deformation Analysis for Discontinuous Liquefaction Conditions

South of Station 205+00, Hart Crowser did not find any basis for predicting location of liquefaction-susceptible soils, other than at the specific location of the explorations. Hart Crowser used a stress-deformation model (FLAC) to further assess the potential effects of discontinuous liquefaction of foundation soils below the embankment slope.

FLAC is a two-dimensional explicit finite difference program for engineering mechanics computation. This program simulates the behavior of soil, rock, or other materials. Materials are represented by elements, which form a grid that is adjusted to the geometry of the specific problem. FLAC has a number of built-in constitutive models for modeling various materials.

In contrast to limit equilibrium slope stability analyses, which result in a factor of safety, FLAC calculates forces and displacements directly according to a material's constitutive model and the site's geometry.

Model Parameters

The FLAC model was evaluated at Section 193+19 of the 2H:1V embankment slope. The material properties were similar to those used in the slope stability calculations with minor simplification of the subsurface profile.

Figure A-1 illustrates the generalized embankment section and subsurface profile used in the FLAC analyses. In the FLAC analyses the embankment "Drainage Layer" was incorporated into the "Embankment Fill" and the "Loose to medium SAND," "Medium stiff to stiff SILT," and "Medium dense to dense SAND" were combined into a single potentially liquefiable layer for which random distribution of liquefaction was applied.

The values used for the elastic modulus of the subgrade were based on the results of pressuremeter tests (Hart Crowser, 2000g). The elastic modulus for the liquefied soils was calculated from the stress strain model of Byrne (1991) as presented in Kramer (1996). The dilation angles were selected based on typical values for sands and silty sands.

The probability of liquefaction for different seismic events and their corresponding residual strengths are shown in Table A-4. The soil properties used in the FLAC analyses are shown in Table A-5.

Table A-5 - Soil Properties Used in the Displacement-Based (FLAC) Model at Station 193+19

Soil Description	Unit Weight in psf	Friction Angle in Degrees	Cohesion in psf	Dilation Angle in Degrees	Elastic Modulus in ksi	Poisson's Ratio
Embankment Fill and Drainage Layer	135	35	0	15	6	0.3
Potentially Liquefiable Soils (static)*	125	32	0	10	4	0.3
Potentially Liquefiable Soils (residual strength)*	125	0	**	10	***	0.3
Dense to very dense SAND	135	37	0	15	30	0.3

* A probabilistic combination of static and residual strengths were used.

** Cohesions of 293, 643, and 963 psf were used for the incremental samples that liquefy in the 175, 300, and 475-yr event.

*** Elastic Moduli of 0.01, 0.04, and 0.08 ksi were used for the incremental samples that liquefy in the 175, 300, and 475-yr event.

FLAC Analyses

The FLAC modeling sequence progressed as follows:

(1) Set up model geometry and boundary conditions;
(2) Initialize displacements to a post-construction condition;
(3) Randomly distribute post-liquefaction or non-liquefaction shear strength in the potentially liquefiable soil layer, according to the expected probability of liquefaction;
(4) Calculate the resulting deformations; and
(5) Repeat Steps 3 and 4 to observe the effects of the random distribution.

A random number generator was used in Step 3 to determine whether each grid cell in the FLAC model would liquefy. When the random number was below a cutoff value, the cell was identified to liquefy and the shear strength was changed to the post-liquefaction residual strength value. A small grid size was used in the analyses such that there were 1500 grid cells in the partially liquefiable layer. An example of the profile of liquefaction for the 9% probability of liquefaction (175-year event) in the potentially liquefiable layers is illustrated on Figure A-1.

Deformation Due to Random Distribution of Discontinuous Liquefaction

The FLAC analyses were performed using a random distribution of liquefaction for both the 175-year and 475-year events. Each analysis was iterated 10 times to assess the effect of variability in the results. The displacements were measured at the toe, mid-height, and top of the slope. The results for the two seismic events are presented in Tables A-6 and A-7.

Table A-6 - Displacements Calculated for the 175-year Seismic Event (9% liquefaction)

Number	Horizontal Displacement in Inches		
Number	toe of slope	midheight of slope	top of slope
1	0.3	0.8	0.5
2	0.9	0.7	0.5
3	10.7	0.8	0.5
4	0.7	0.7	0.5
5	2.3	1.0	0.5
6	0.5	0.7	0.5
7	1.7	0.6	0.4
8	1.0	0.7	0.5
9	0.8	0.8	0.6
10	0.5	0.7	0.5
mean	1.9	0.7	0.5
median	0.9	0.7	0.5
st. dev.	3.1	0.1	0.0

Table A-7 - Displacements Calculated for the 475-year Seismic Event (33% liquefaction)

Run	Horizontal Displacement in Inches		
Number	toe of slope	midheight of slope	top of slope
1	4.2	5.4	3.4
2	7.3	5.1	3.4
3	17.7	6.7	4.1
4	6.6	5.9	4.3
5	10.9	7.2	4.7
6	14.7	14.7	11.0
7	6.5	3.9	2.4
8	7.7	5.2	3.3
9	6.5	4.2	3.0
10	4.6	4.0	2.2
mean	8.7	6.2	4.2
median	6.9	5.3	3.4
st. dev.	4.4	3.2	2.5

These results indicate that deformations from the 175-year event will likely be in the 1- to 2-inch range; however, they could be a foot or greater. Deformations from the 475-year event will likely be in the 4- to 8-inch range; however, they could be 2 feet or greater. It is important to note that if the zones of liquefaction are not randomly distributed (i.e., if there is a significant zone of liquefaction in critical area, such as occurs north of Station 205+00) actual deformations would be greater than these calculated deformations for discontinuous liquefaction.

Predicted Deformations as a Function of Composite Shear Strength

The FLAC model was also run for a number of different composite strengths ranging from 1250 to 4000 psf. The displacements were calculated at the toe, mid-height, and top of the slope. The results are presented in Table A-8.

Table A-8 - Displacements Calculated using the Composite Strength Method

Composite (Residual) Strength in psf	Horizontal Displacement in Feet		
	toe of slope	midheight of slope	top of slope
4000	0.0	0.0	0.0
2000	0.0	0.1	0.1
1750	0.1	0.3	0.2
1500	12.5	15.1	9.0
1250	unstable	unstable	unstable

These results indicate that the composite strength needs to be approximately 1,750 psf or larger to keep displacements to a reasonable level (on the order of inches) during the design level seismic event. Use the proportional approach to estimating composite strength discussed above for the 475-, 300- and 175-year return period events, Hart Crowser found that this minimum value of composite shear strength would be exceeded for all critical failure surfaces, for slopes greater than about 50 feet in height. This analysis puts a probable upper bound on the size of the local instability anticipated to result from discontinuous liquefaction.

Deformation for Different Probabilities of Liquefaction

Finally, the FLAC model was run with the probability of liquefaction varied in the liquefiable layer. In the base case discussed above, the probability of a sample being liquefaction-susceptible (including the very conservative assumption of complete saturation) was 33 percent. Change in deformation was checked for various probabilities of liquefaction ranging from 20 to 60 percent. The displacements were calculated at the toe, mid-height, and top of the slope. The results are presented in Table A-9.

Table A-9 - Displacements Calculated Using the Probabilistic Strength Method

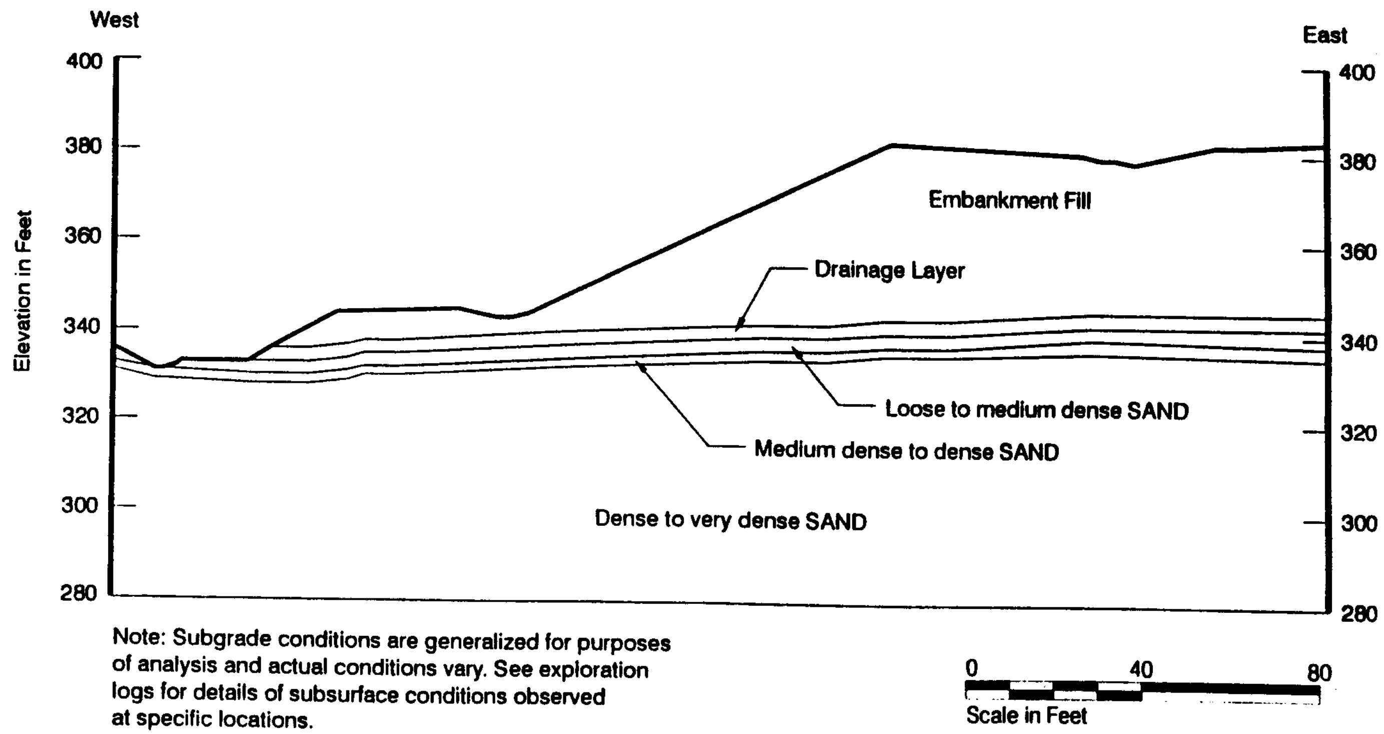
Percent of Liquefaction Zone that Liquefies with Sr=675 psf	Horizontal Displacement in Feet		
	toe of slope	midheight of slope	top of slope
20	0.0	0.1	0.0
40	0.3	0.2	0.1
50	4.9	3.8	1.1
60	unstable	unstable	unstable

These results indicate that displacements on the order of a foot or less are anticipated for the probability of liquefaction of 40 percent or lower during the design level seismic event (475-year return period). Above 50 percent liquefaction, deformations on the order of several feet would be expected, and instability of the slope is anticipated when the probability of liquefaction is on the order of 60 percent or greater. These results appear very consistent with the limit equilibrium analyses that show acceptable factors of safety against instability for composite shear strength based on 33 percent liquefaction for the design seismic event.

F:\docs\jobs\497828\Analyses_Recommendations.doc

APPENDIX B
STABILITY ANALYSES CROSS SECTIONS

2H:1V Embankment Section Used for Stability Analyses
Section 170+23



Note: Subgrade conditions are generalized for purposes of analysis and actual conditions vary. See exploration logs for details of subsurface conditions observed at specific locations.

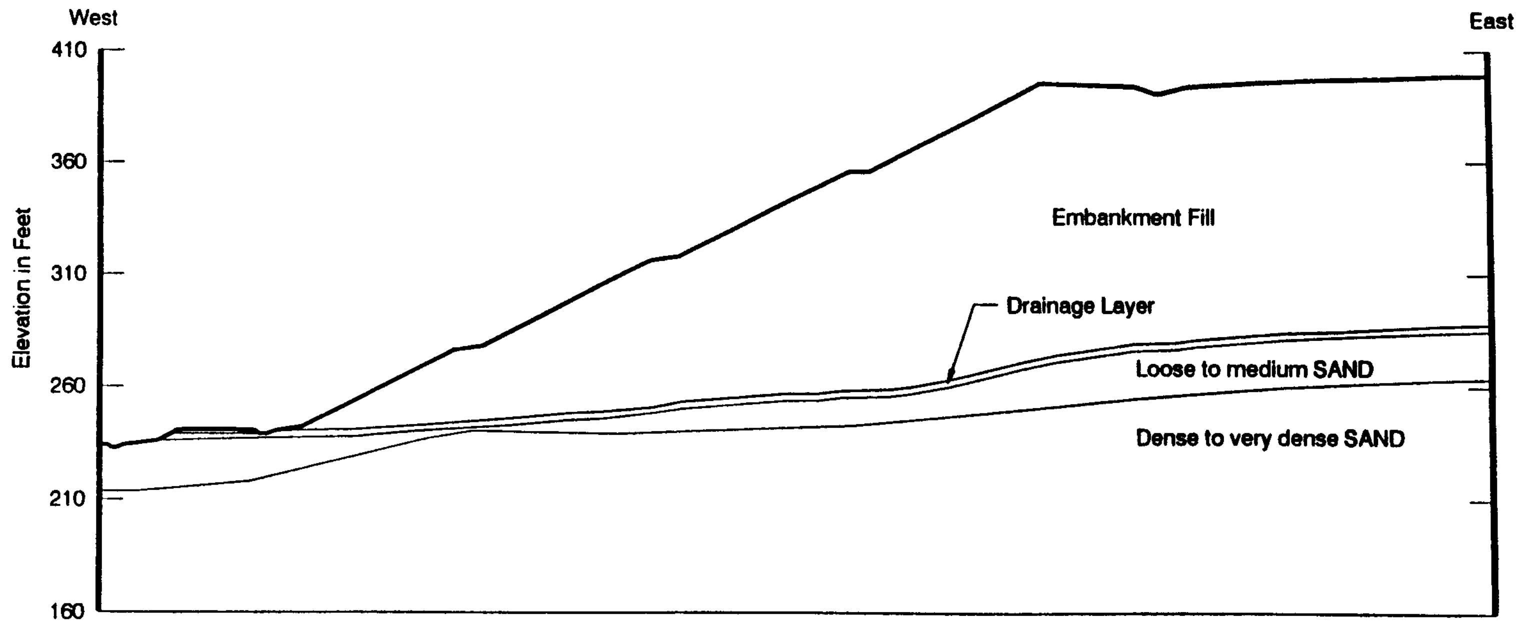
HARTCROWSER
J-4978-28 12/00
Figure B-1

AR 049718

RC 12/1/00 497828A CDR

2H:1V Embankment Section Used for Stability Analyses

Section 187+60



Note: Subgrade conditions are generalized for purposes of analysis and actual conditions vary. See exploration logs for details of subsurface conditions observed at specific locations.

HARTCROWSER

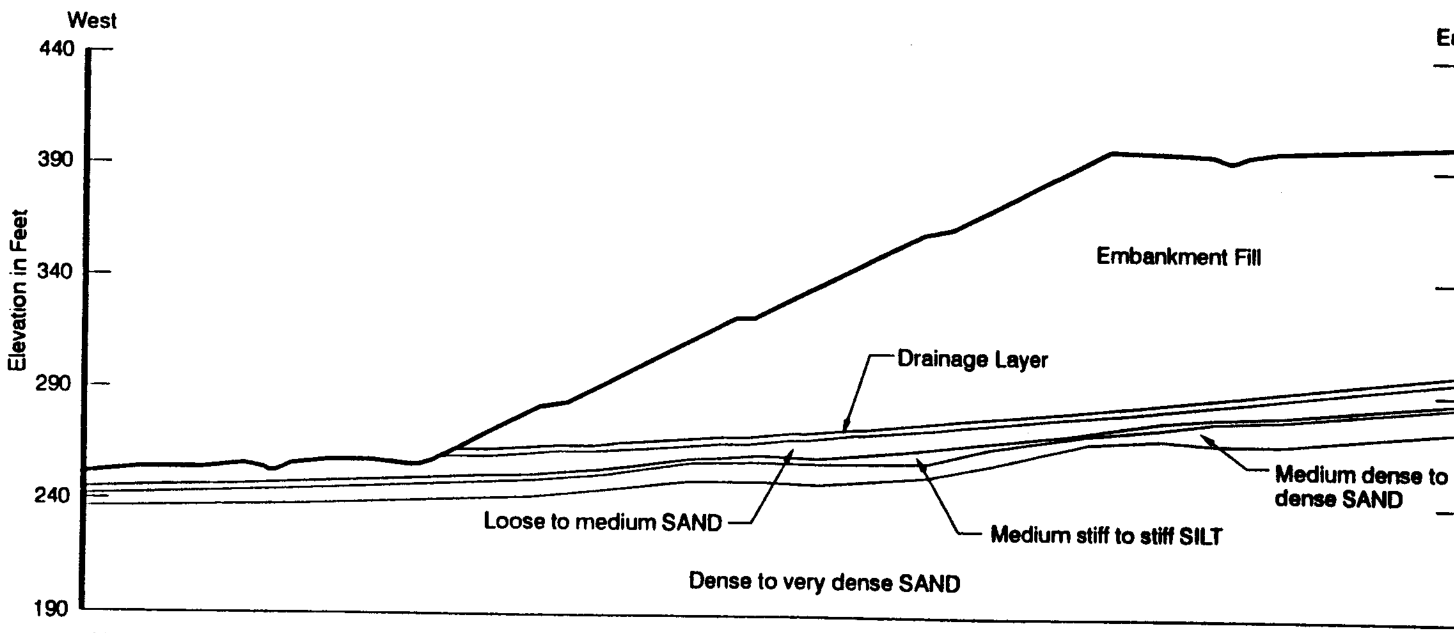
J-4978-28 12/00

Figure B-2

AR 049719

RC 12/1/00 497828B.CDR

2H:1V Embankment Section Used for Stability Analyses
Section 193+19



Note: Subgrade conditions are generalized for purposes of analysis and actual conditions vary. See exploration logs for details of subsurface conditions observed at specific locations.

0 80 160
Scale in Feet

HARTCROWSER

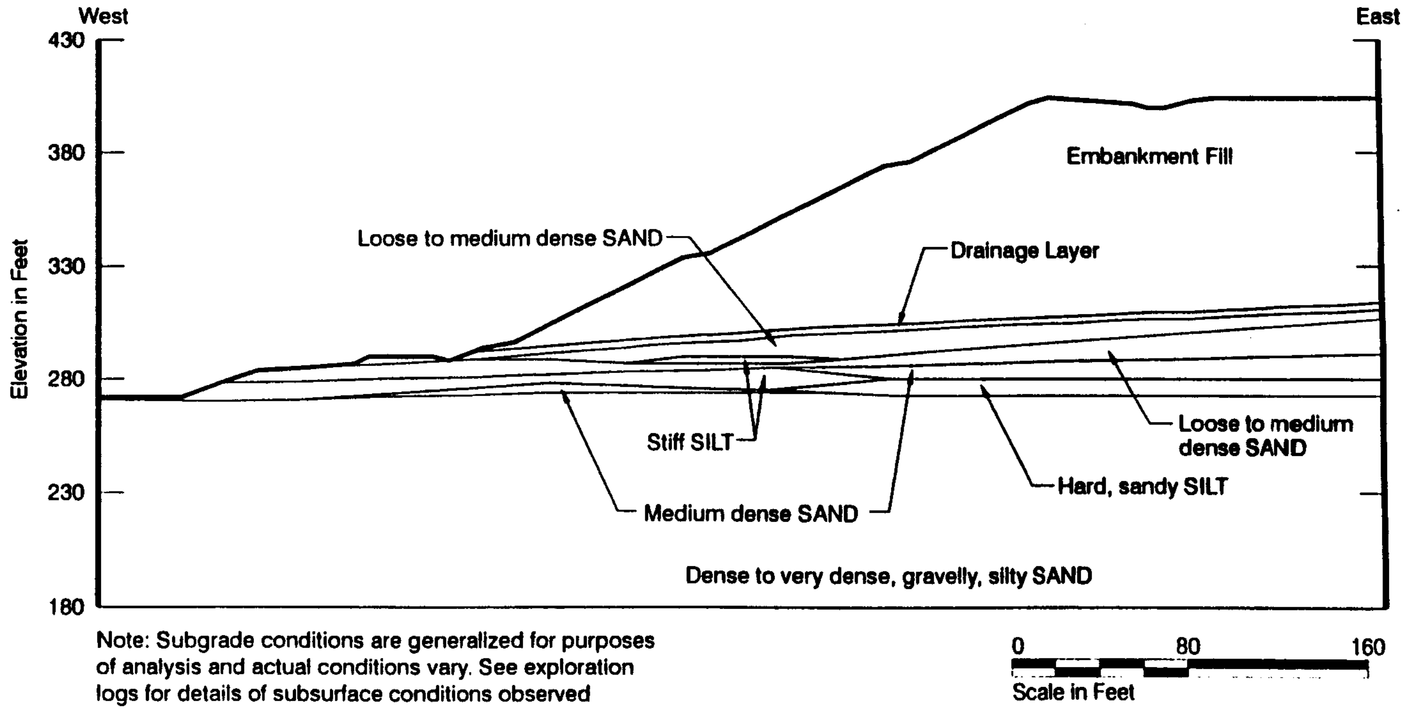
J-4978-28 12/00

Figure B-3

AR 049720

RC 12/1/00 497828C.CDR

2H:1V Embankment Section Used for Stability Analyses
Section 206+44



Note: Subgrade conditions are generalized for purposes of analysis and actual conditions vary. See exploration logs for details of subsurface conditions observed at specific locations.

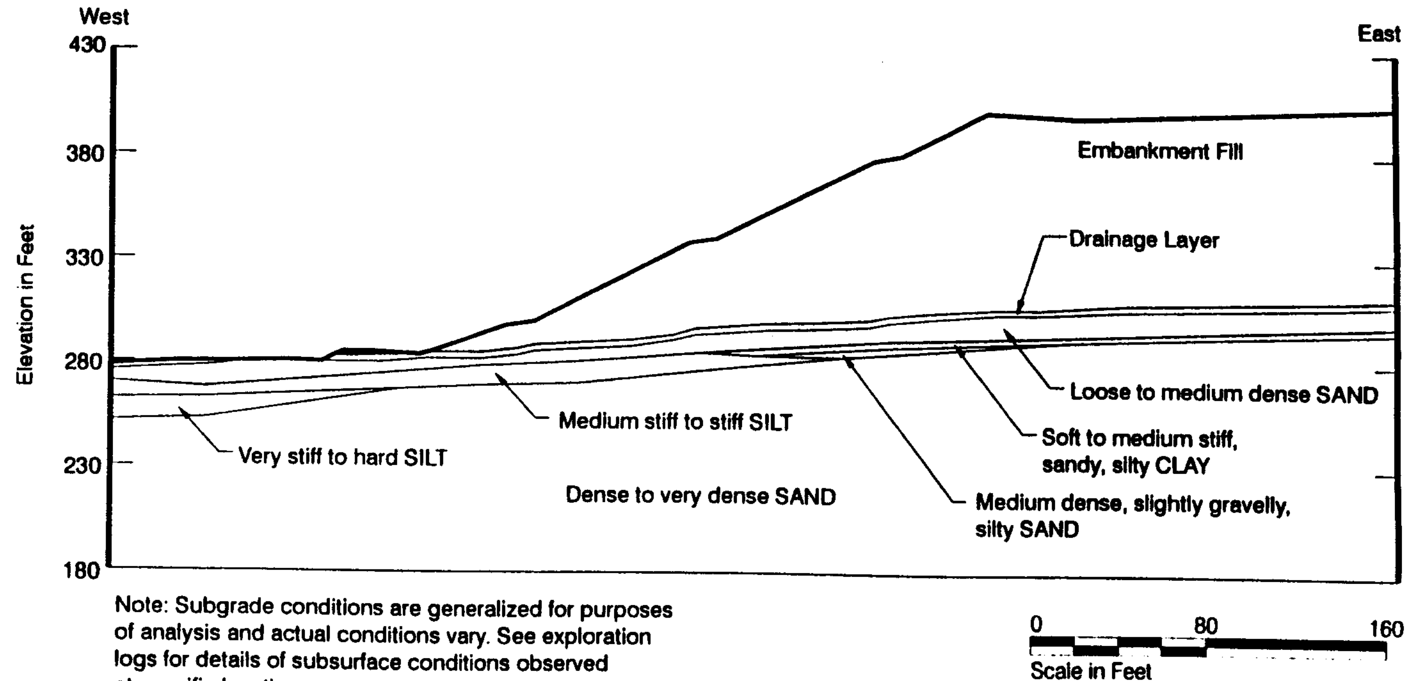
RC 12/1/00 497828D CDR

HARTCROWSER

J-4978-28 12/00

Figure B-4

2H:1V Embankment Section Used for Stability Analyses
Section 210+00



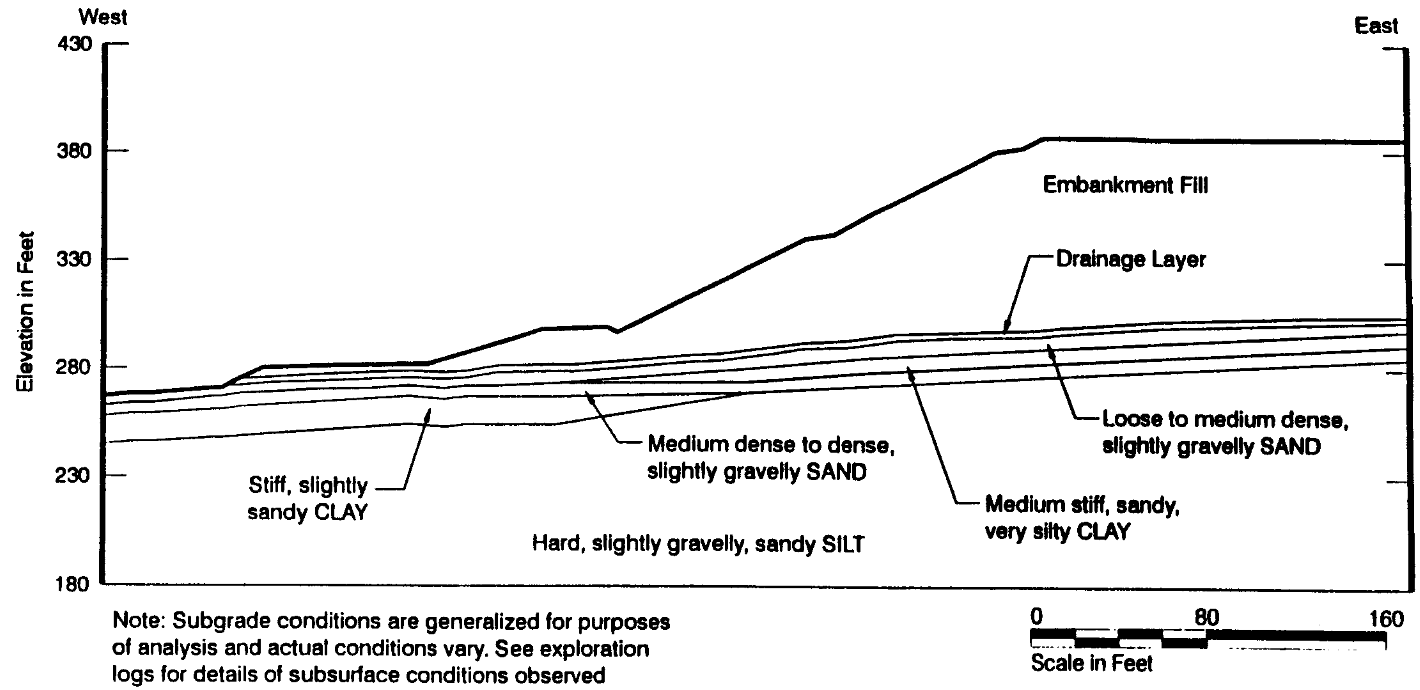
Note: Subgrade conditions are generalized for purposes of analysis and actual conditions vary. See exploration logs for details of subsurface conditions observed at specific locations.

RC 12/1/00 497828E.CDR

HARTCROWSER
J-4978-28 12/00
Figure B-5

AR 049722

2H:1V Embankment Section Used for Stability Analyses
Section 212+50



Note: Subgrade conditions are generalized for purposes of analysis and actual conditions vary. See exploration logs for details of subsurface conditions observed at specific locations.

HARTCROWSER
J-4978-28 12/00
Figure B-6

RC 12/1/00 497828F.CDR

APPENDIX C
EMBANKMENT INFILTRATION AND SEEPAGE STUDIES

APPENDIX C
EMBANKMENT INFILTRATION AND SEEPAGE STUDIES

Introduction

This appendix presents the results of seepage analyses designed to track changes in the infiltration and deep percolation of moisture occurring as a result of constructing the proposed Third Runway embankment. Understanding of these changes is important for a number of reasons:

- Different soil types proposed for the embankment fill will result in different amounts of infiltration and runoff. The surface soil type will also affect rates of evapotranspiration.

- The percolation of moisture through the embankment could potentially create zones of saturation were pore pressures could build up, with consequent risk to the stability of slope faces.

- The rate and timing of recharge to groundwater beneath the embankment could change, affecting the groundwater level beneath the fill. This could affect the extent of areas susceptible to liquefaction during earthquake events, and/or affect base flow to wetlands and Miller Creek.

The analyses presented in this appendix are designed to address:

- The relative quantities of moisture percolating downward through the embankment and into the underlying drainage layer;
- The proportion of moisture that flows along the drainage layer and discharges at the embankment toe;
- The proportion and timing of groundwater recharge occurring as downward seepage from the drainage layer into the native soils beneath the embankment; and
- The water table elevation maintained in the existing subgrade soils after embankment construction.

Approach

The movement of moisture into and through the Third Runway embankment represents a complex interplay of hydrologic processes occurring at and beneath the soil surface, which are listed and defined below. Figure C-1 shows a representative cross section through the embankment and illustrates the water balance components used in the model.

- **Precipitation (P).** The occurrence of rainfall is the main driver for the infiltration process.
- **Evaporation (E).** A portion of the precipitation evaporates without infiltrating or running off, this includes interception storage on leaves and in shallow surface ponds.
- **Runoff (R_o).** The occurrence of runoff from the surface of the embankment (excluding the effect of impervious surfaces) depends on a number of factors, including:
  - The intensity and duration of each precipitation event;
  - The prevailing moisture content of the surface soil, as influenced by antecedent conditions;
  - The type and density of vegetation;
  - Surface slope; and
  - The hydraulic conductivity of the surface soil, as influenced by grain size, soil fabric, macro-porosity, and degree of compaction.
- **Infiltration (I).** The amount of water infiltrating into the soil surface is complimentary to the runoff, and is largely dependent on the same factors.
- **Transpiration (T).** A portion of the moisture in the upper soil layer(s) is taken up by the vegetation and lost back into the atmosphere.
- **Percolation (P).** Excess moisture in the upper soil zone(s) is available to move downward under the influence of gravity and the pressure gradient created by soil moisture tension in the unsaturated vadose zone within the body of the embankment. The moisture content in the vadose zone continually adjusts to the rate of percolation to achieve a dynamic balance with the unsaturated hydraulic conductivity.
- **Seepage (S).** Locally saturated conditions can occur within or beneath the embankment where deep percolation encounters lower-permeability layers (e.g., silty or clayey soils or very dense soils such as glacial till), potentially creating zones of saturation in which water can perch and move laterally.
- **Drain Flow (DF).** Seepage within the underdrain is identified as drain flow. There is both a horizontal and vertical component of drain flow.
- **Groundwater Flow (GW).** Seepage into the native soils below the underdrain becomes groundwater flow (horizontal or base flow component).
- **Deep Percolation (DP).** Deep percolation is the vertical component of groundwater flow that goes down into the ground below the surficial water-bearing zone to recharge deeper regional aquifers.

The approach taken to analyzing embankment infiltration and seepage uses a sequence of three models to represent these processes, recognizing that unsaturated flow conditions likely predominate within the embankment.

Rosetta. The USDA has developed a "neural network" database model to generate soil moisture and hydraulic conductivity characteristic curves from grain size and soil density information (Schaap and Bouten, 1996). These curves define the fundamental moisture-conductivity-suction relationships that control infiltration and unsaturated percolation in the embankment, and are needed as input to simulation models, such as SoilCover and SEEP/W.

HELP. The EPA has developed a program for studying runoff, infiltration, and evapotranspiration as an aid to the design of landfill covers (Schroeder et al., 1994). The program, called HELP (Hydrologic Evaluation of Landfill Performance) has since been widely used to calculate groundwater recharge. It is applicable to the Third Runway embankment design in that it allows the direct simulation of lateral drainage layers within the embankment.

SoilCover. SoilCover is a soil-atmosphere flux model that links the subsurface saturated/unsaturated groundwater system and the atmosphere above the soil in a rigorous mathematical algorithm that represents the physical processes that occur between the soil and the atmosphere. These include: precipitation, infiltration, runoff, transpiration, and evaporation. The model calculates moisture fluxes within an unsaturated soil profile, as driven by day-to-day variations in atmospheric conditions, including precipitation, temperature, humidity, and solar radiation.

Soil Properties

Infiltration and seepage of moisture into the proposed embankment are controlled primarily by atmospheric conditions and soil properties. The soil properties of interest are those that govern the physical processes occurring at the soil surface, namely runoff, infiltration, and evapotranspiration. These processes are controlled primarily by the relative hydraulic conductivity of the soil layer, where the hydraulic conductivity of the unsaturated soil varies with the moisture content of the soil. The relative hydraulic conductivity is some fraction of the saturated hydraulic conductivity of the soil.

In recent years, numerous attempts have been made to define the unsaturated characteristics of soils using mathematical relationships among the three key parameters: moisture content, matric suction, and hydraulic conductivity. The computer program Rosetta was used to determine unsaturated hydraulic parameters from the grain-size distributions of the proposed fill materials (van Genuchten, 1980). Once the parameters were obtained, relationships (also known as soil characteristic curves) between matric potential (also known as soil suction or tension) and volumetric water content were constructed using the van Genuchten method, and between matric potential and unsaturated hydraulic conductivity using the Mualem (1976) method. Rosetta input requires percentages of sand, silt, and clay along with the bulk density for the soil(s) of interest. The program uses a limiting maximum bulk density value of 2.0 g/cm^3 (128 pcf).

Existing Soils

Hart Crowser reviewed the results of more than 50 test pits and borings in the proposed embankment foot print area, and identified two soil types that are representative of the overall embankment subgrade. The existing embankment subgrade soils of interest for the infiltration and seepage study are as follows:

- **Outwash Sand and Silty Sand.** Outwash sand and silty sand are the predominant surficial soil type within the embankment footprint. A representative sample of this soil type was chosen for use as input to the analyses based on a review of grain-size analyses. Sample S-2 from a depth of 8 feet in boring HC00-B115 was chosen. Gradation for this sample was comprised of 74 percent sand and 26 percent silt, with an estimated bulk density of 106 pcf (1.7 g/cm^3). These parameters were run through the Rosetta model to develop the characteristic unsaturated moisture content/matric potential/hydraulic conductivity curves shown on Figure C-2.

- **Dense Glacial Till.** Surficial soils at the embankment site are underlain at relatively shallow depth (5 to 20 feet) by glacially overridden advance outwash and glacial till soils, generally consisting of silty sand and sandy silt. For the HELP runs, the "glacial till" was represented using a default soil type available within the HELP program (Material 24 - a sand-silt-clay loam mixture with a saturated hydraulic conductivity of 2.7×10^{-6} cm/sec.). This material is considered representative of the conductivity expected for glacial tills and silty advance deposits of the type observed at the embankment site. The moisture/conductivity characteristic curves for this soil generated within SoilCover, using field capacity and wilting point data from HELP, are shown on Figure C-3.

Fill Materials

Four generalized soil groups are proposed for the Third Runway embankment construction, with Group 1 soils split into two subgroups (see Hart Crowser, 2000):

- **Group 1A.** This is a free-draining sand and gravel with less than 5 percent fines (i.e., passing the US No. 200 sieve) conforming to the grain size envelope presented on Figure C-4. Group 1A soils are required to be used for the embankment drainage layer.

- **Group 1B.** This is a sand and gravel with less than 8 percent fines conforming to the grain size envelope presented on Figure C-5.

  Soils from Groups 1A and 1B will be used as select fill in the reinforced zone for the West MSE wall, may be used in the reinforced zone for the South MSE and NSA walls, and as wet weather fill for the embankment.

- **Group 2.** This is a sand and gravel with up to 12 percent fines conforming to the grain size envelope presented on Figure C-6. Group 2 soils may be used in the reinforced zones for the NSA and South MSE walls, and will be used as common embankment fill except during wet weather.

- **Group 3.** This is a silt, sand, and gravel with up to 35 percent fines conforming to the grain size envelope presented on Figure C-7. Group 3 soils are intended for use as common embankment fill, except during wet weather.

- **Group 4.** This is a clay, silt, sand, and gravel with up to 50 percent fines conforming to the grain size envelope presented on Figure C-8. Group 4 soils may be used as common embankment fill, except during wet weather.

For each of these soil groups, a median grain size distribution was selected to be representative of the respective group (as shown on the figures listed above). This median grain size distribution was extrapolated into the fines region and used to define the proportions of gravel, sand, silt, and clay for each soil group. These proportions are listed in Table C-1. The Rosetta model was then used to generate the unsaturated moisture content/matric potential/hydraulic conductivity characteristic curves for each representative median soil type. Curves for soil Groups 1B, 3, and 4 are shown on Figures C-9 through C-11.

The soils proposed as fill material for the Third Runway embankment have significant percentages of gravel (up to 80 percent in Group 1A), which is ignored in the inputs to the Rosetta program. Rosetta deals only with the sand-silt-clay fractions, so the percentages listed in Table 1 were normalized to discount the presence of gravel before being input to Rosetta. As a result, the Rosetta model tends to slightly underpredict the unsaturated hydraulic parameters to a degree that is proportional to the gravel content.

A method was devised to account for the effect of gravel content on the hydraulic properties calculated by the Rosetta model. The parameter that can be manipulated in Rosetta without affecting the grain size distribution of the soil is the bulk density. A correction factor for the percentage of gravel contained in the soil was therefore applied to the saturated hydraulic conductivity value calculated initially by Rosetta, after the method of Brakensiek et al. (1974). This correction factor was determined by:

Correction Factor = 1 + (% gravel) / 100

As needed, the bulk density value for each soil group was then reduced to below limiting value of 2.0 g/cm^3 and Rosetta was rerun to produce a new parameter set with a saturated hydraulic conductivity equal to the corrected value. The reduction in bulk density represents in part the reduced degree of compaction achieved among the sand-silt-clay fraction in soils with increasing gravel content.

Note that hydraulic conductivity of the glacial till was not analyzed with the Rosetta model because we used a default conductivity value from the HELP model for the till. This is acceptable because the unsaturated hydraulic properties of the glacial till would not be affected by the presence of the embankment. The Rosetta model was used for the embankment fill materials and the native surficial soil (outwash) so that the HELP model output would accurately represent conditions following embankment construction.

Weather Data

Precipitation, temperature, humidity, and solar radiation are the main atmospheric drivers controlling the surficial soil moisture. Data collected at SeaTac for the most recent 11 years (1987 through 1997) and published by NCDC (1998 and 1999) were used to the extent possible. Data are incomplete for the years 1998 and 1999; however, the total precipitation in those years was similar to 1995 and 1991, respectively. We therefore reused data from 1995 and 1991 to extend the data record to the end of 1999.

Simulations

The HELP model was used to simulate infiltration and seepage under existing conditions at the site of the proposed embankment, and to study changes in infiltration and seepage that will occur following construction of the embankment. HELP works by routing the products of precipitation, apportioning them between runoff, evapotranspiration, and percolation. In the model, precipitation is applied as inches of rainfall and is thus independent of the surface area under consideration. To maintain consistency in the model, all other fluxes are measured in inches of water per unit time.

Existing Conditions (Baseline)

The infiltration and seepage analysis was applied to existing subgrade soils in the embankment area to establish a baseline for post-construction comparisons. Natural vegetation conditions at the embankment site were approximated in HELP with a leaf area index (of 4.5 for Western Washington forested lands) and an evaporative zone depth of 20 inches. Net infiltration from the surface water balance currently sustains the shallow groundwater table typically found in the outwash sands and silts, perched on the underlying till layer, as noted in observation wells.

Existing hydrogeologic conditions in the proposed embankment area are characterized as follows:

- Moderately sloping ground surface, dropping down from the airfield elevation (~400 feet) to the toe of the west slope of the proposed embankment (between 280 and 320 feet elevation).
- Vegetation cover is generally deciduous forest with a moderate understory
- Shallow soils are typically outwash sands and silts, 5 to 20 feet thick, overlying dense glacial till that is 5 to 15 feet thick

The following soil profile was simulated in HELP:

- **Layer 1.** 5 feet of outwash sand and silt - vertical percolation layer;
- **Layer 2.** 10 feet of outwash sand and silt - lateral drainage layer that transmits base flow in the existing condition; and
- **Layer 3.** 5 feet of glacial till - generally an aquitard or barrier soil layer with only limited ability to transmit deeper percolation vertically.

The model was configured to allow ponding and lateral flow of water in Layer 2, as representative of the perched groundwater conditions observed overlying the glacial till. In calibration runs, the hydraulic conductivity of the glacial till had to be reduced to 5×10^{-7} cm/sec to develop the typical range in saturated thickness (listed in Table C-2 as Head on top of Layer 3) that was comparable to field observations in monitoring wells (i.e., 1 to 10 feet).

Constructed Conditions

The infiltration and seepage analysis was also applied to anticipated soil conditions to assess changes that would occur as a result of embankment construction. The following generalized soil profile was simulated in HELP:

- **Layer 1.** 100 feet of embankment fill - vertical percolation layer;

- **Layer 2.** 3 feet of sand and gravel - lateral drainage layer;
- **Layer 3.** 5 feet of outwash sand and silt - native surficial soil layer that might act as a nominal barrier layer, depending on its conductivity relative to the overlying embankment soils;
- **Layer 4.** 10 feet of outwash sand and silt - existing soils that act as a lateral drainage layer (transmitting base flow to Miller Creek); and
- **Layer 5.** 5 feet of glacial till - existing barrier soil layer.

Three different types of embankment fill material were simulated, representing median conditions and probable extremes in terms of grain size distribution for the bulk of the fill material:

- Group 1B represents the coarsest material likely to be used within the main body of the embankment;
- Group 3 represents the median soil type that may be expected to predominate in embankment construction (based on 1998 (Phase I) and 1999 (Phase II) construction records); and
- Group 4 represents the finest gradation material likely to be used within the embankment.

A long-term vegetated surface condition was modeled for each soil group with a leaf area index of 2.0 (representing a fair stand of grass) with an evaporative zone depth of 20 inches.

Layer 2 immediately beneath the fill represents the drainage layer, comprised of Group 1A material.

The lower layers (3, 4, and 5) in the post-construction model represent the same soils as in the existing conditions (see previous section). A limitation of the HELP model requires that a barrier soil layer must underlie any lateral drainage layer. This does not affect the soil properties, except that HELP considers a barrier soil to be permanently at 100 percent saturation.

We elected not to model the Group 2 soil material because it is very similar in grain size distribution to the Group 1B material, and because quantities used in embankment construction to date have been relatively minor.

Model Results

The models were used to simulate hydrologic conditions as they affect the existing water table beneath the embankment. Predicted model flux rates calculated in HELP are markedly affected by the initially assumed moisture content distribution in the unsaturated soil profile at the start of the simulations;

this effect lasted for between 1 and 3 years into the simulation period, depending on soil type. Our comparison of results, therefore, focuses on the last 10 years of the simulation period (1990 through 1999).

Existing Conditions

The lateral drainage rate from Layer 2 of the HELP model for the existing conditions is equated to groundwater base flow or discharge in the shallow water table aquifer. The predicted rate ranges between 3.8 and 20.0 inches per year as shown highlighted in Table C-2. This forms the baseline we used for comparison with possible changes that are predicted due to the placement of various embankment fill configurations in the constructed condition.

Embankment Conditions

The lateral drainage rate from Layer 4 of the HELP model for the constructed conditions is equated to groundwater base flow or discharge in the shallow water table aquifer beneath the embankment. The abbreviated annual output from HELP for each year of the simulation period is listed in Tables C-3, C-4, and C-5 for the respective embankment soil groups. The predicted discharge rates are highlighted in each table, ranging between 5.1 to 21.3 inches per year, and groundwater of 5.4 to 18.3 inches per year, for the different fill soils modeled.

Group 1B

The embankment profile composed of Group 1B material exhibits minimal runoff and slightly lower evapotranspiration than the other fill materials. The lower evapotranspiration is attributed to higher porosity and steeper soil moisture characteristic curves (see Figure C-9), which limit soil moisture utilization in the active near-surface soil zone. As a result, the amount of deep percolation remaining that can move downward through the embankment is higher than for the finer-grained Group 3 material.

Group 3

The embankment profile composed of Group 3 material exhibits a minor amount of runoff and slightly more evapotranspiration than for the Group 1B soil. As a result, the amount of deep percolation remaining that can move downward through the embankment is lower than for the coarser-grained Group 1B material.

Group 4

The embankment profile composed of Group 4 material exhibits substantial runoff and moderate to low evapotranspiration. Plant growth in Group 4 material is least able to extract moisture from the active surface layer because unit changes in matric suction yield the smallest volume of moisture, due to the relative flatness of the soil moisture characteristic curve (See Figure C-11). Taking into account the water lost as runoff, the amount of deep percolation remaining that can then move downward into the passive mass of the embankment is less than for the Group 3 material, but more than the Group 1B material.

For all fill soils, the seasonality of the groundwater recharge/flow component from the embankment (also called the hydroperiod) is strongly impacted, with reduced peaks and troughs that are shifted by 3 to 6 months relative to the existing conditions (see Figures C-12, C-13, and C-14). These changes reflect the delay and buffering effect created by time for percolation through and storage within the full thickness of the embankment.

Conclusions

The results of the model show groundwater base flow rates for existing and post-construction conditions, indicating substantial differences on a month-by-month basis, but the overall long-term average amounts are generally very similar. The differences are the seasonal lag which produces a net benefit of more base flow to Miller Creek in the summer and early fall. The overall long-term similarity is best illustrated by cumulative plots of groundwater discharge for each fill type for a 10-year simulation period, as plotted on Figure C-15.

Implications for Underlying Water Table Conditions

Close examination of the cumulative plots (Figure C-15) indicates the groundwater flowrates beneath the proposed embankment will generally be similar to existing conditions but that slight differences are predicted depending on whether annual precipitation is more or less on average, as discussed below.

Years with More than Average Precipitation (Wet Years)

Groundwater flowrates beneath the proposed embankment will generally be similar to or slightly lower than for existing conditions during wet years (1990; 1995-99). This implies that groundwater water levels beneath the toe of the embankment would be similar to or slightly lower than those observed in monitored wells over the past 12+ months (a relatively wet period in the precipitation record).

Years with Less than Average Precipitation (Dry Years)

The cumulative plots indicate that groundwater flowrates beneath the embankment would show a relative increase over existing conditions during dry years (1991-94). While this would result in higher water levels compared to existing conditions (i.e., a wet year), it should be noted that the absolute water levels during dry years would be lower than the levels recently observed in monitored wells over the past 12+ months.

It is, therefore, concluded that groundwater levels beneath the constructed embankment should become no higher than the peak levels observed over the last 12 moths or so, which means no increase in the area(s) susceptible to liquefaction is anticipated. Similarly, the effect of the embankment on hydraulic lag in precipitation becoming base flow will be most pronounced in dry years, when the increased water is most beneficial to the environment.

Effect of Different Fill Materials

Although the grain size and consequently the saturated hydraulic conductivity of the fill materials vary widely, there is a much narrower envelope of variation bounding their respective hydrologic behaviors under constructed conditions in the embankment.

Group 1B materials allow more recharge than would occur under existing conditions, but it is unlikely that a large portion of the embankment would be constructed of Group 1B materials.

Group 3 materials allow approximately the same recharge than would occur under existing conditions, and this is likely the most representative of the bulk materials that will be used in the embankment.

Group 4 materials result in less recharge than would occur under existing conditions, but use of Group 4 fill will not be allowed in wet weather conditions (i.e., when the less silty Group 1A or 1B materials must be used), which will limit the overall quantities of Group 4 soils that will be placed.

The reasons for the broad similarity in recharge response (compare Figures C-12 through C-14) relates to the mechanisms of unsaturated flow by which infiltrated water percolates through the embankment.

Deep percolation in the embankment is driven by the net flux leaving the surficial soil layer once the processes of runoff, evaporation, infiltration, and transpiration have been satisfied. This net flux is relatively insensitive to soil type, as long as the infiltration capacity is not too low. The net surface flux that moves downward into the body of the embankment causes the moisture content of the fill material to adjust under the physical constraints of unsaturated flow. This requires that the unsaturated hydraulic conductivity of the soil mass be approximately equal to the net surface flux. The moisture content and matric potential of the soil mass thus adjust in concert with the hydraulic conductivity, as governed by the soil characteristic functions (Figures C-9 through C-11). The result is differing soil moisture and matric suction distributions for the three soil types studied, but very similar unsaturated hydraulic conductivities, because the net flux rates are essentially similar.

This balance should not be significantly affected by layering of different fill materials as the embankment is constructed, as long as each layer is capable of passing the net flux entering from above. The limiting value for the saturated hydraulic conductivity of any discrete layer within the embankment should be no less than the net flux rate for deep percolation in the embankment. This rate is estimated using Soil Cover to be around 4.6 x 10^{-6} cm/sec, which is well below the value expected for any of the proposed embankment soils. In the event less permeable soils do become part of the fill (for instance, due to variability within an approved fill material source), the result would be creation of a local perched zone of limited extent within the embankment, with no loss in overall infiltration capacity. The frequent gradation checks accomplished as part of the embankment construction process prevent such an effect from extending over any significant area.

Effect of Different Fill Thicknesses

The simulation results presented above were for a nominal 100-foot-thick embankment fill. In reality, the embankment thickness will vary from zero to 160 feet. We made some additional runs of the HELP model using rainfall records for the year 1997, with Group 3 material in fill thickness of 150, 100, 60, 30, and 15 feet to see if there was a trend in seepage behavior, or a point at which the seepage behavior changed significantly.

Flux rates in the simulations of different fill thickness showed little variation (on the order of 2 to 5 percent) from the nominal 100-foot base case (see Table C-6). The results show a trend of increasing groundwater recharge rates with decreasing fill thickness, down to thicknesses of about 30 feet. Reduced thicknesses of fill in general, have less moisture storage capacity and so yield less water during a period of declining precipitation.

References for Appendix C

Brakensiek, D.L., W.J. Rawls, and G.R. Stephenson, 1974. A Note on Determining Soil Properties for Soils Containing Rock Fragments. J. Range Manage., Vol. 39, No. 5, pp. 408 - 409.

Hart Crowser, 2000. Geotechnical Input to MSE Wall and Reinforced Slope Design, Third Runway Embankment. Memorandum from J. Beaver, M Bailey, and B. Chen to J. Thompson of HNTB, May 12, 2000.

Mualem, Y., 1976. A new model predicting the hydraulic conductivity of unsaturated porous media. Water Resour. Res. 12:513-522.

NCDC, 1997. National Climatic Data Center, Weather Data Set 1987-1997 (CD ROM).

Schaap, M.G. and W. Bouten, 1996. Modeling water retention curves of sandy soils using neural networks. Water Resour. Res. 32:3033-3040.

Schroeder, P.R., C.M Lloyd, P.A. Zappi, and N.M. Aziz, 1994. Hydrologic Evaluation of Landfill Performance (HELP) Model. User's Guide for Version 3. Army Engineer Waterways Experiment Station, Vicksburg, MS. and Clemson Univ., SC. Dept. of Civil Engineering. 103p.

van Genuchten, M.Th., 1980. A closed-form equation for predicting the hydraulic conductivity of unsaturated soils. Soil Sci. Am. J. 44:892-898.

F:\docs\jobs\497806\infiltrationappendix.doc

Table C-1 - Soil Properties Used for Developing Input to Rosetta Model

	Size Fractions in %					
Material	Gravel	Sand	Silt	Clay	Bulk Density in gm/cm^3	Gravel Correction Factor
Group 1A	74	22	3	1	1.77	1.74
Group 1B	69	26	4	1	1.81	1.69
Group 2	62	31	5	2	1.85	1.62
Group 3	35	57	6	2	1.9	1.35
Group 4	37	38	20	5	1.91	1.37
Outwash	8	68	24	0	1.7	1.08

Notes: Bulk density value is based on the relative compaction of the sand - silt - clay fraction, adjusted by using the Gravel Correction Factor after Brakensiek et al. (1974), (see text).

497806/infiltrationtables - Table 1

Table C-2 - HELP Output Summary for Existing Conditions (Baseline)

	1987 INCHES	1988 INCHES	1989 INCHES	1990 INCHES	1991 INCHES	1992 INCHES	1993 INCHES	1994 INCHES	1995 INCHES	1996 INCHES	1997 INCHES	1998 INCHES	1999 INCHES
PRECIPITATION	29.9	33.0	34.7	44.8	35.4	32.8	28.8	34.8	42.6	50.3	43.3	44.8	42.6
RUNOFF	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
EVAPOTRANSPIRATION	14.3	18.4	16.5	17.8	16.2	17.1	19.5	15.8	18.1	16.8	21.1	17.9	17.8
DRAINAGE COLLECTED FROM LAYER 2	34.0	7.6	9.7	13.3	16.0	8.0	5.6	3.8	13.1	19.8	20.0	13.7	14.1
PERC./LEAKAGE THROUGH LAYER 3	13.6	7.9	8.3	9.1	9.7	8.0	6.8	6.6	9.1	10.5	10.5	9.2	9.3
AVG. HEAD ON TOP OF LAYER 3	71.6	16.0	20.4	28.0	33.5	16.9	11.8	8.0	27.6	41.5	42.0	28.9	29.6
CHANGE IN WATER STORAGE	-31.9	-1.0	0.2	4.6	-6.4	-0.3	-3.1	8.6	2.3	3.2	-8.4	4.0	1.4
SOIL WATER AT START OF YEAR	86.7	54.8	53.8	54.0	58.6	52.2	51.9	48.9	57.5	59.8	63.0	54.7	58.6
SOIL WATER AT END OF YEAR	54.8	53.8	54.0	58.6	52.2	51.9	48.9	57.5	59.8	63.0	54.7	58.6	60.0
SNOW WATER AT START OF YEAR	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
SNOW WATER AT END OF YEAR	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
ANNUAL WATER BUDGET BALANCE	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Highlight = Contribution of precipitation that becomes groundwater base flow. Note first 3 years of model results reflect "initial saturation" and are not representative of long-term conditions.

497806/infiltrationtables.xls - Table 2

AR 049739

Table C-3 - HELP Output Summary for Group 1B Embankment Fill

	1987 INCHES	1988 INCHES	1989 INCHES	1990 INCHES	1991 INCHES	1992 INCHES	1993 INCHES	1994 INCHES	1995 INCHES	1996 INCHES	1997 INCHES	1998 INCHES	1999 INCHES
PRECIPITATION	29.9	33.0	34.7	44.8	35.4	32.8	28.8	34.8	42.6	50.3	43.3	44.8	42.6
RUNOFF	0.0	0.0	0.0	0.3	0.1	0.0	0.0	0.0	0.0	0.1	0.0	0.3	0.0
EVAPOTRANSPIRATION	12.7	16.4	14.5	15.7	14.5	14.5	18.0	13.0	16.5	14.4	18.5	16.4	16.1
DRAINAGE COLLECTED FROM LAYER 2	0.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
PERC./LEAKAGE THROUGH LAYER 3	124.3	16.6	18.3	23.7	28.2	18.3	16.7	11.6	22.1	32.3	32.2	25.4	24.6
AVG. HEAD ON TOP OF LAYER 3	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DRAINAGE COLLECTED FROM LAYER 4	57.3	54.6	20.4	14.5	17.4	12.4	8.7	5.1	11.2	19.0	21.3	17.1	15.6
PERC./LEAKAGE THROUGH LAYER 5	18.6	18.0	10.6	9.4	10.0	8.9	8.1	7.3	8.6	10.3	10.8	9.9	9.6
AVG. HEAD ON TOP OF LAYER 5	119.9	114.0	42.9	30.4	36.4	25.9	18.3	10.7	23.4	39.6	44.5	35.8	32.7
CHANGE IN WATER STORAGE	-58.8	-56.1	-10.8	4.9	-6.7	-3.1	-6.0	9.4	6.3	6.4	-7.3	1.1	1.3
SOIL WATER AT START OF YEAR	416.7	357.9	301.8	290.9	295.9	289.2	286.1	280.1	289.5	295.8	302.2	294.9	296.0
SOIL WATER AT END OF YEAR	357.9	301.8	290.9	295.9	289.2	286.1	280.1	289.5	295.8	302.2	294.9	296.0	297.3
SNOW WATER AT START OF YEAR	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
SNOW WATER AT END OF YEAR	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
ANNUAL WATER BUDGET BALANCE	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Notes: Fill Height = 98 ft
Runoff Curve Number = 49

Highlight = Contribution of precipitation that becomes groundwater base flow. Note first 3 years of model results reflect "initial saturation" and are not representative of long-term conditions.

497806/infiltrationtables.xls - Table 3

Table C-4 - HELP Output Summary for Group 3 Embankment Fill

	1987 INCHES	1988 INCHES	1989 INCHES	1990 INCHES	1991 INCHES	1992 INCHES	1993 INCHES	1994 INCHES	1995 INCHES	1996 INCHES	1997 INCHES	1998 INCHES	1999 INCHES
PRECIPITATION	29.9	33.0	34.7	44.8	35.4	32.8	28.8	34.8	42.6	50.3	43.3	44.8	42.6
RUNOFF	0.1	0.0	0.0	1.0	0.5	0.0	0.0	0.0	0.1	0.6	0.1	1.0	0.1
EVAPOTRANSPIRATION	13.3	17.2	15.8	16.8	15.5	16.1	18.8	14.8	17.4	15.5	20.0	17.1	17.2
DRAINAGE COLLECTED FROM LAYER 2	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
PERC./LEAKAGE THROUGH LAYER 3	137.1	19.6	17.7	21.0	26.7	19.1	15.8	11.5	17.9	28.7	30.2	24.6	23.8
AVG. HEAD ON TOP OF LAYER 3	0.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DRAINAGE COLLECTED FROM LAYER 4	57.3	57.5	29.2	14.2	15.0	12.7	8.4	5.4	8.5	15.7	19.4	16.6	14.8
PERC./LEAKAGE THROUGH LAYER 5	18.6	18.7	12.5	9.3	9.4	9.0	8.0	7.4	8.1	9.6	10.4	9.8	9.4
AVG. HEAD ON TOP OF LAYER 5	119.9	120.0	61.2	29.7	31.2	26.6	17.5	11.3	17.8	32.7	40.6	34.7	30.9
CHANGE IN WATER STORAGE	-59.4	-60.4	-22.9	3.5	-5.0	-5.1	-6.4	7.2	8.5	8.9	-6.7	0.3	1.1
SOIL WATER AT START OF YEAR	393.3	333.9	273.4	250.6	254.1	249.1	244.1	237.7	244.9	253.3	262.2	255.6	255.8
SOIL WATER AT END OF YEAR	333.9	273.4	250.6	254.1	249.1	244.1	237.7	244.9	253.3	262.2	255.6	255.8	257.0
SNOW WATER AT START OF YEAR	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
SNOW WATER AT END OF YEAR	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
ANNUAL WATER BUDGET BALANCE	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Notes: Fill Height = 98 ft
Runoff Curve Number = 69

Highlight = Contribution of precipitation that becomes groundwater base flow. Note first 3 years of model results reflect "initial saturation" and are not representative of long-term conditions.

497806/infiltrationtables.xls - Table 4

AR 049741

Table C-5 - HELP Output Summary for Group 4 Embankment Fill

	1987 INCHES	1988 INCHES	1989 INCHES	1990 INCHES	1991 INCHES	1992 INCHES	1993 INCHES	1994 INCHES	1995 INCHES	1996 INCHES	1997 INCHES	1998 INCHES	1999 INCHES
PRECIPITATION	29.9	33.0	34.7	44.8	35.4	32.8	28.8	34.8	42.6	50.3	43.3	44.8	42.6
RUNOFF	0.6	0.2	0.3	2.6	1.2	0.2	0.1	0.5	1.0	2.2	0.8	2.6	1.0
EVAPOTRANSPIRATION	13.4	17.5	15.8	17.2	15.6	15.9	18.5	. 14.6	17.4	15.9	20.5	17.4	17.3
DRAINAGE COLLECTED FROM LAYER 2	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
PERC./LEAKAGE THROUGH LAYER 3	87.4	18.2	17.3	20.7	24.6	18.2	15.9	11.5	17.5	27.9	28.9	23.5	21.9
AVG. HEAD ON TOP OF LAYER 3	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DRAINAGE COLLECTED FROM LAYER 4	57.3	35.1	11.7	11.8	13.6	11.8	8.1	5.4	8.4	15.2	18.3	15.6	13.4
PERC./LEAKAGE THROUGH LAYER 5	18.6	13.8	8.8	8.8	9.2	8.8	8.0	7.4	8.0	9.5	10.2	9.6	9.1
AVG. HEAD ON TOP OF LAYER 5	119.9	73.4	24.6	24.7	28.4	24.7	17.0	11.3	17.5	31.6	38.3	32.6	28.1
CHANGE IN WATER STORAGE	-60.0	-33.6	-1.9	4.4	-4.1	-3.9	-5.9	7.0	7.8	7.6	-6.5	-0.4	1.7
SOIL WATER AT START OF YEAR	381.5	321.5	287.9	286.0	290.4	286.2	282.3	276.5	283.5	291.3	298.9	292.4	292.0
SOIL WATER AT END OF YEAR	321.5	287.9	286.0	290.4	286.2	282.3	276.5	283.5	291.3	298.9	292.4	292.0	293.8
SNOW WATER AT START OF YEAR	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
SNOW WATER AT END OF YEAR	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
ANNUAL WATER BUDGET BALANCE	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Notes: Fill Height = 98 ft
Runoff Curve Number = 79

Highlight = Contribution of precipitation that becomes groundwater base flow. Note first 3 years of model results reflect "initial saturation" and are not representative of long-term conditions.

497806/infiltrationtables.xls - Table 5

AR 049742

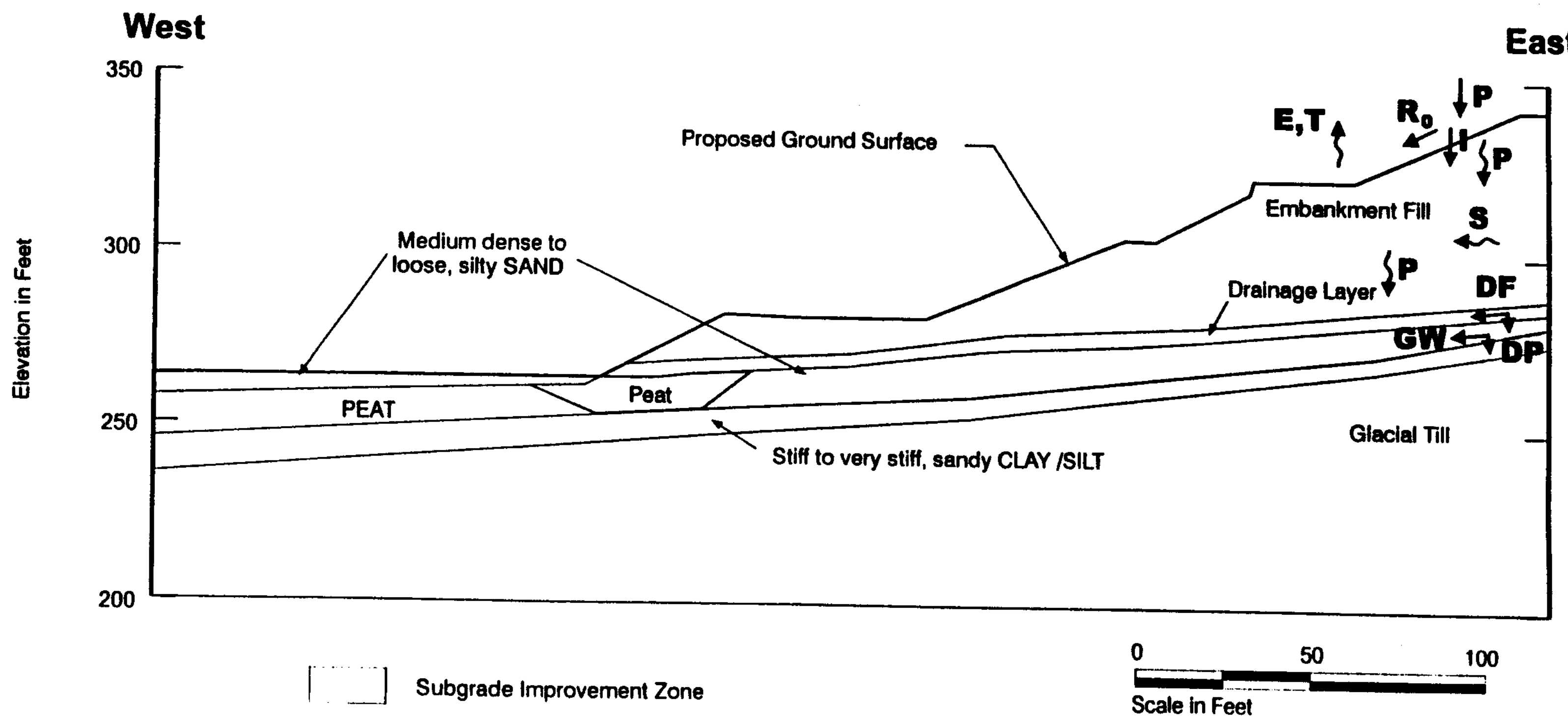
Table C-6 - HELP Output Summary for Various Embankment Heights

	Embankment Height in Feet				
	150	100	60	30	15
			Inches of H_2O		
PRECIPITATION	43.3	43.3	43.3	43.3	43.3
RUNOFF	0.0	0.0	0.0	0.0	0.0
EVAPOTRANSPIRATION	20.0	20.0	20.0	20.0	20.0
DRAINAGE COLLECTED FROM LAYER 2	0.0	0.0	0.0	0.0	0.0
PERC./LEAKAGE THROUGH LAYER 3	28.9	30.5	31.0	30.4	28.0
AVG. HEAD ON TOP OF LAYER 3	0.0	0.0	0.0	0.0	0.0
DRAINAGE COLLECTED FROM LAYER 4	17.9	19.6	20.6	21.0	20.9
PERC./LEAKAGE THROUGH LAYER 5	10.1	10.5	10.7	10.8	10.7
AVG. HEAD ON TOP OF LAYER 5	37.3	41.1	43.1	44.0	43.9
CHANGE IN WATER STORAGE	-4.7	-6.9	-8.1	-8.5	-8.4
SOIL WATER AT START OF YEAR	358.8	267.0	192.3	136.4	109.3
SOIL WATER AT END OF YEAR	354.1	260.1	184.2	127.9	100.9
SNOW WATER AT START OF YEAR	0.0	0.0	0.0	0.0	0.0
SNOW WATER AT END OF YEAR	0.0	0.0	0.0	0.0	0.0
ANNUAL WATER BUDGET BALANCE	0.0	0.0	0.0	0.0	0.0

Comparison is based on data for 1997.

497806/infiltrationtables.xls - Table 6

Embankment Slope Showing Water Balance Components
Shown for Section 101+20 (NSA Wall Stationing)

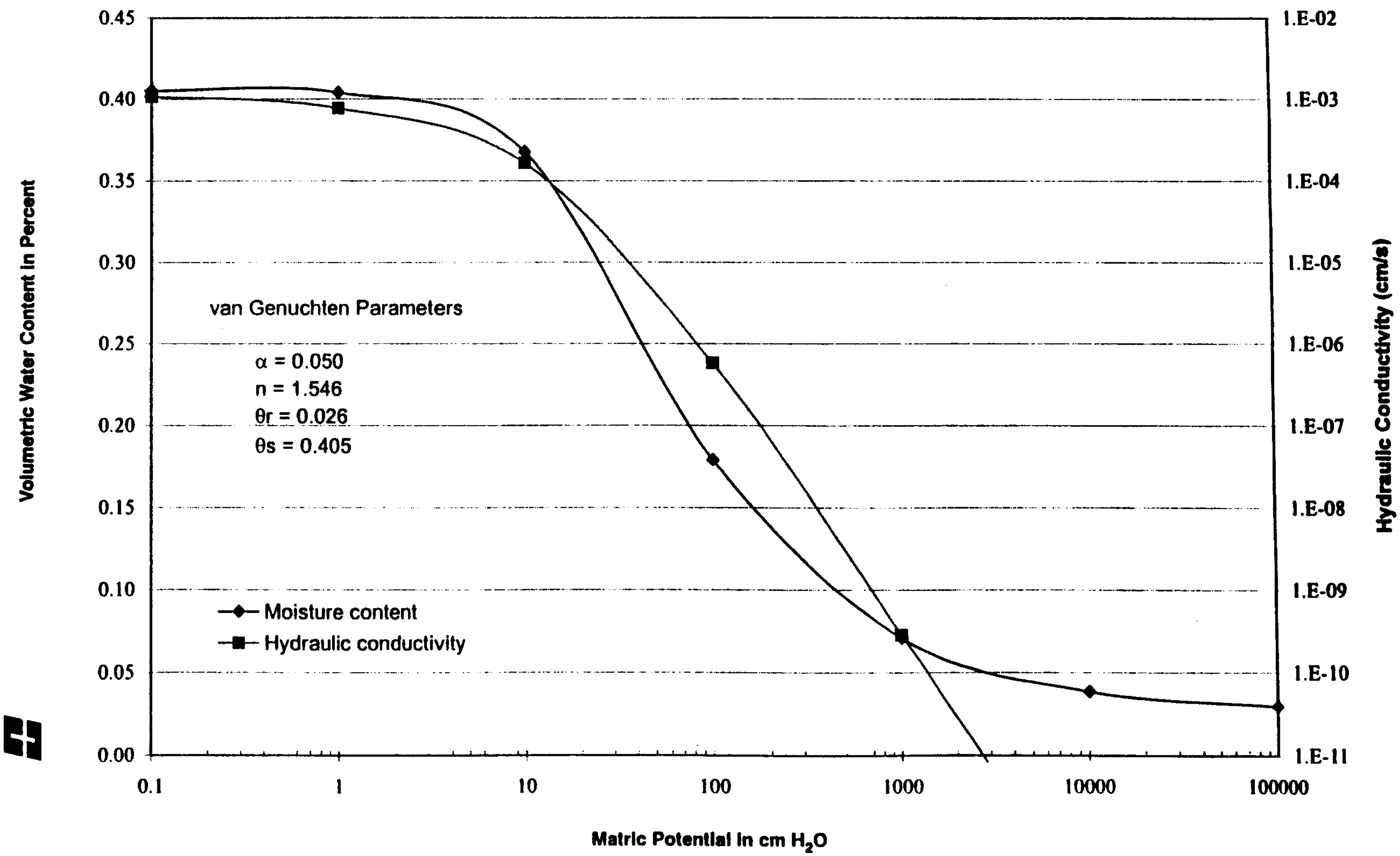


HARTCROWSER
J-4978-28 10/00
Figure C-1

DJH 10/12/00 4978280 cdr

AR 049744

Soil Moisture/Conductivity Characteristic Curves of Outwash Silty Sand



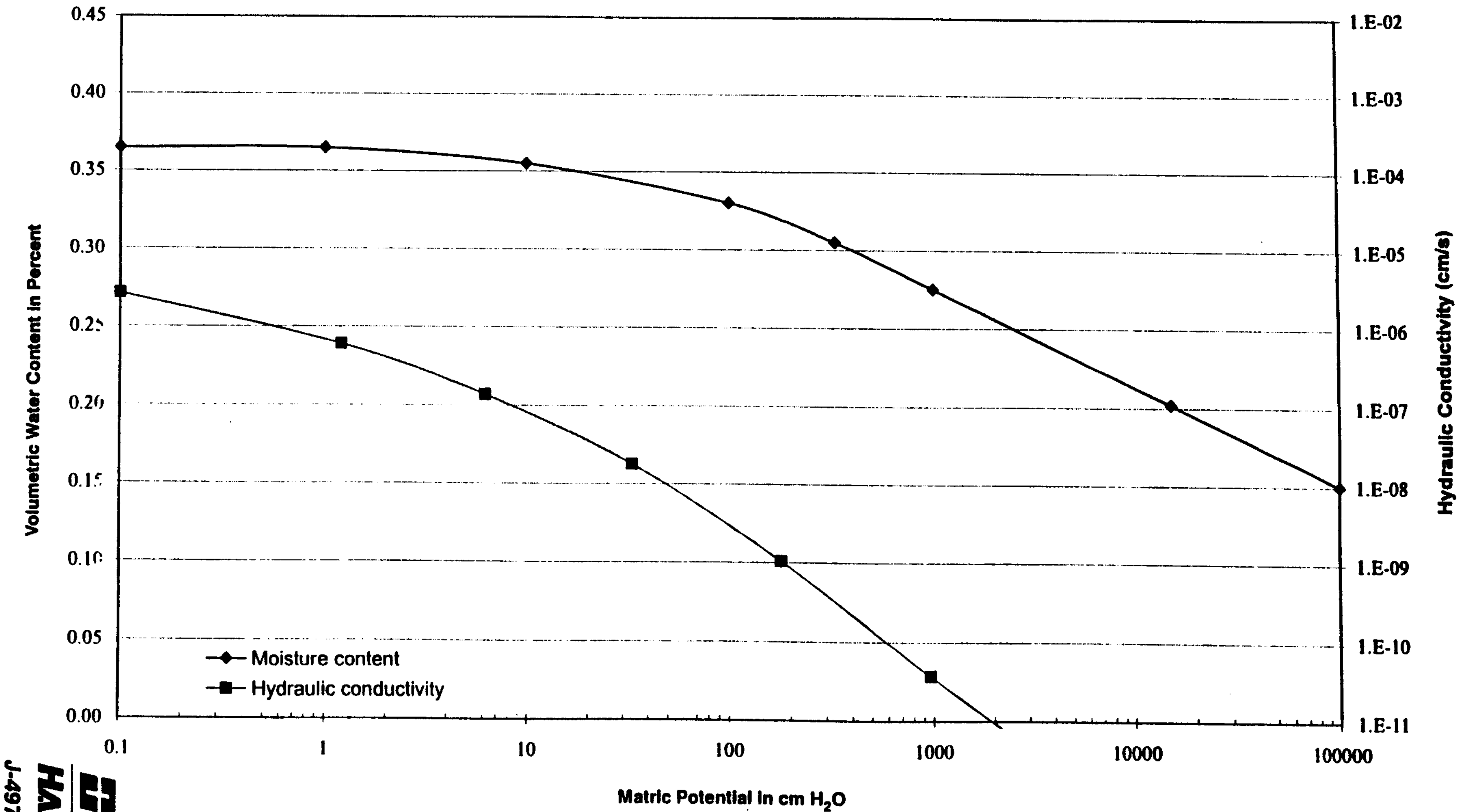
DJH 10/12/00 497828R.cdr

HARTCROWSER

J-4978-28 10/00

Figure C-2

Soil Moisture / Conductivity Characteristic Curves for Glacial Till



DJH 10/12/00 497828S.cdr

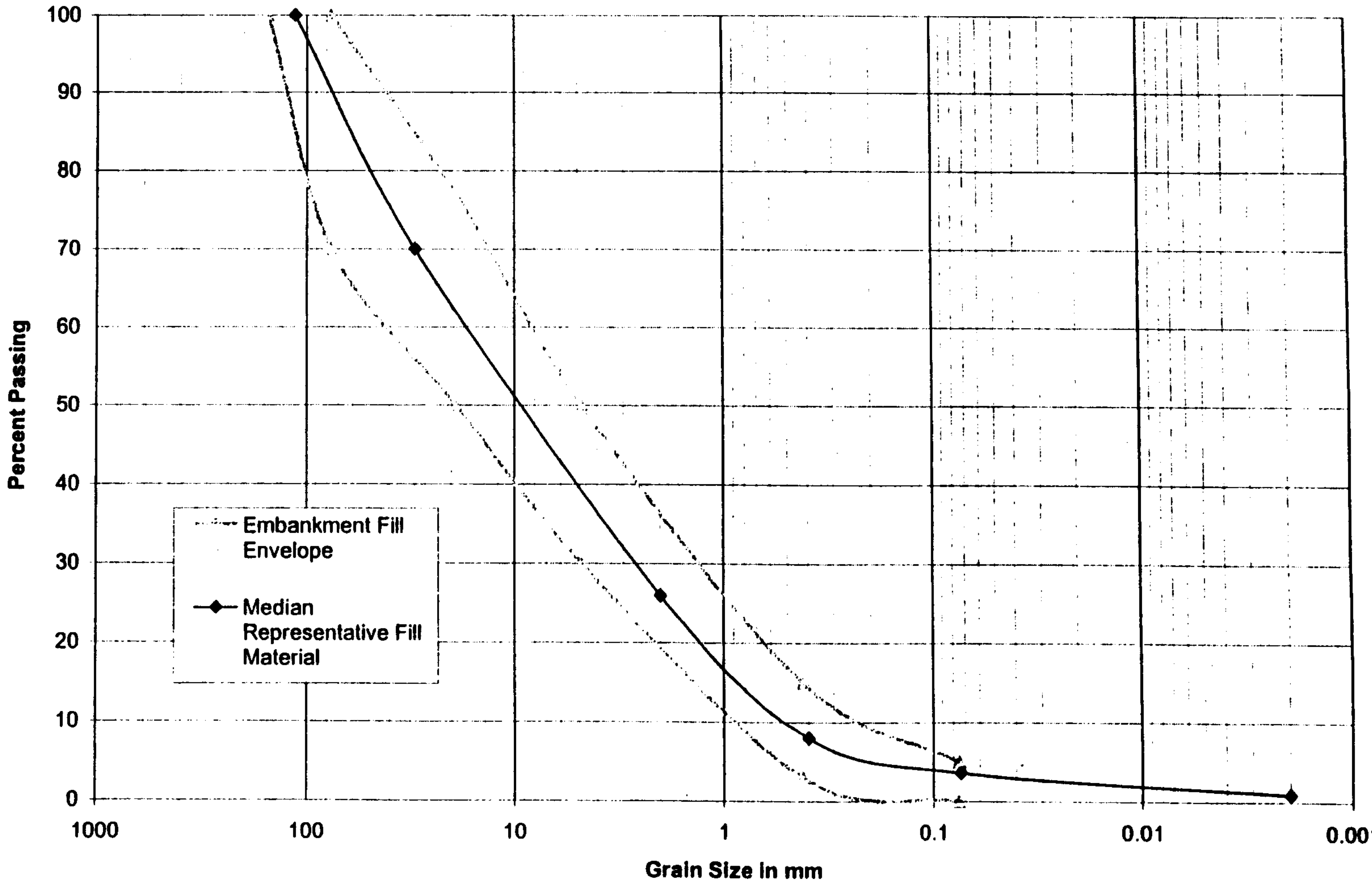
HARTCROWSER

J-4978-28 10/00

Figure C-3

AR 049746

Grain Size Envelope for Group 1A Fill Material

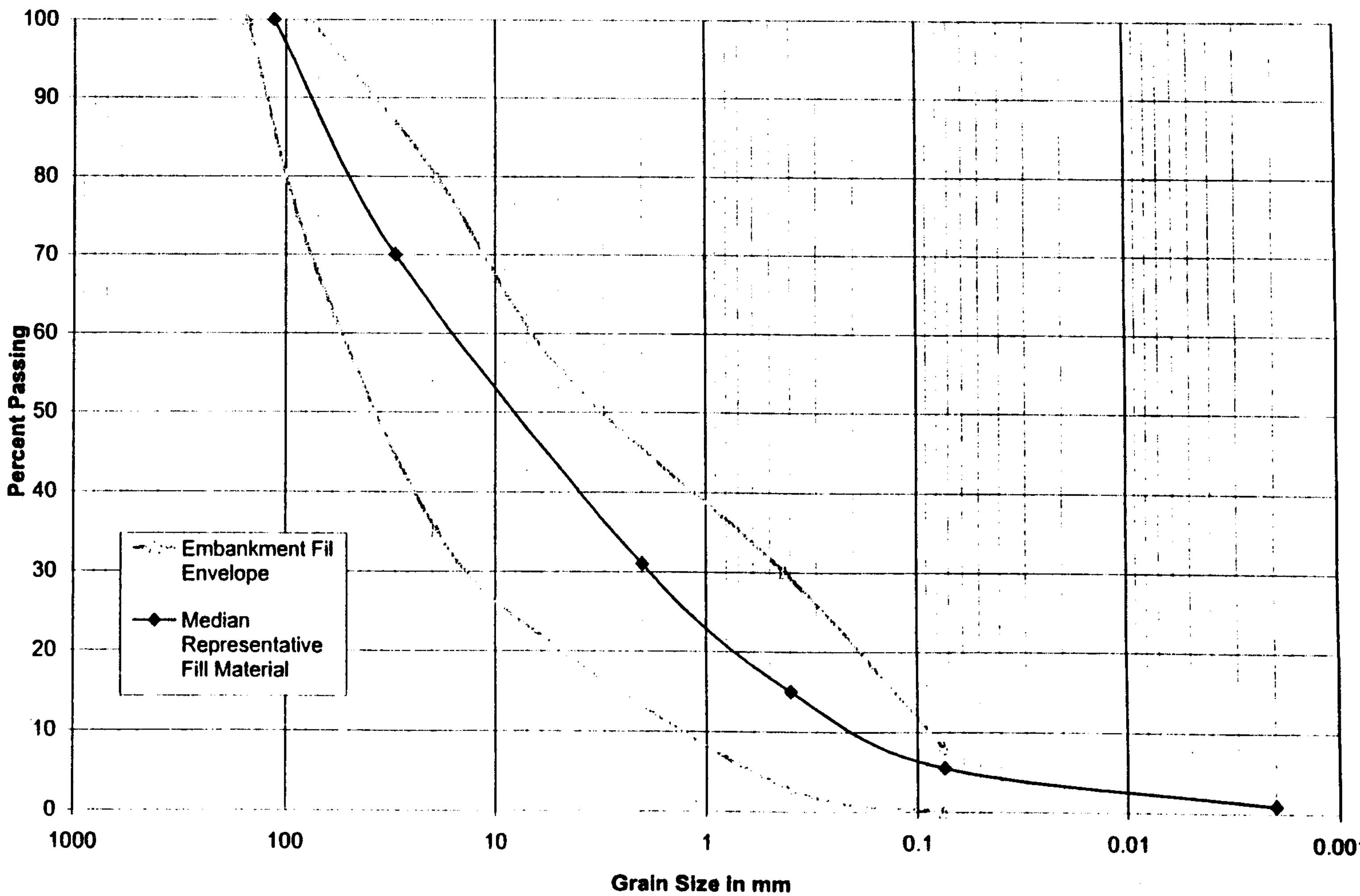


DJH 10/12/00 497828T.cdr

HARTCROWSER
J-4978-28 10/00
Figure C-4

AR 049747

Grain Size Envelope for Group 1B Fill Material



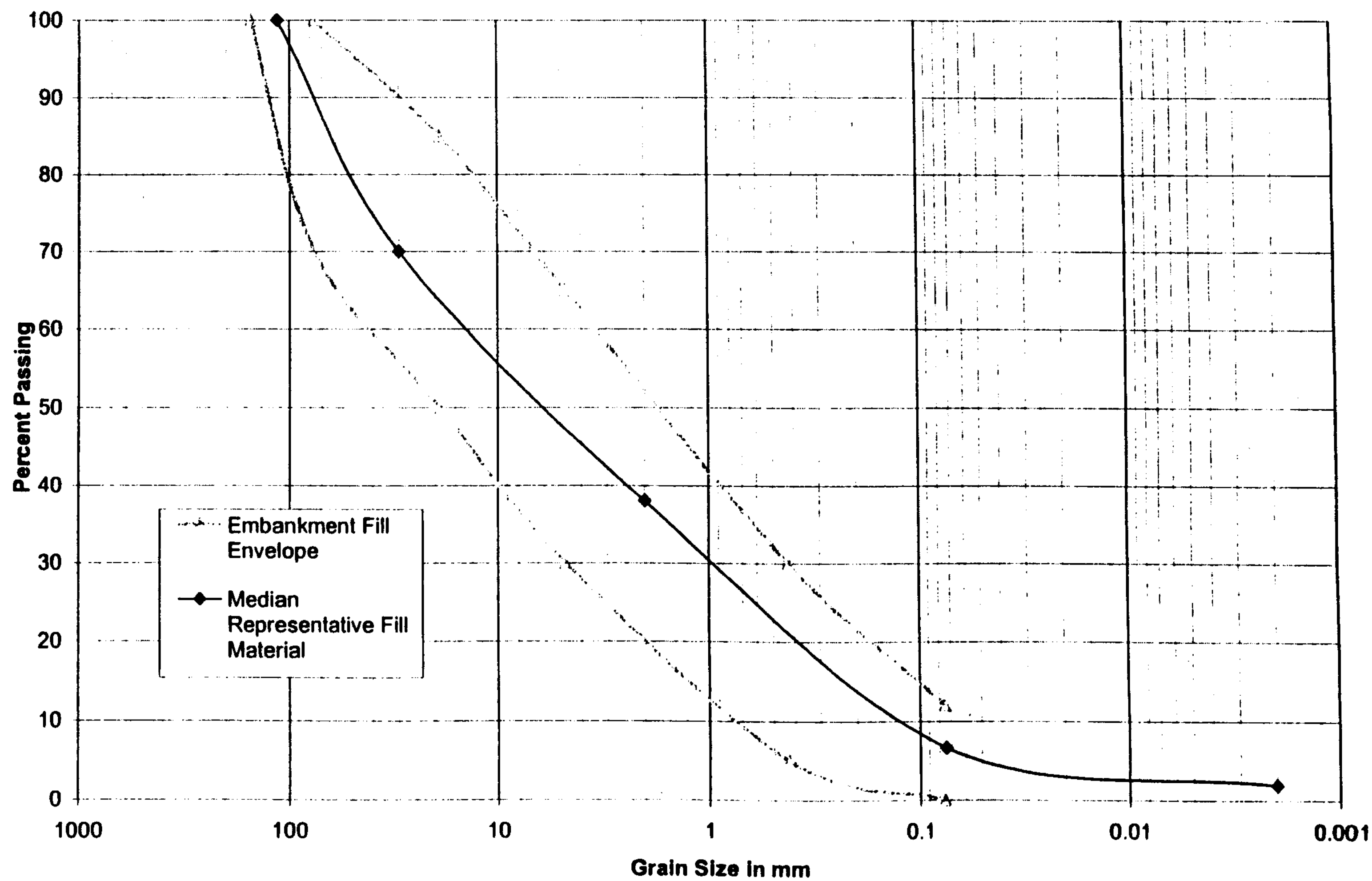
HARTCROWSER

J-4978-28 10/00

Figure C-5

DJH 10/12/00 497828U.cdr

Grain Size Envelope for Group 2 Fill Material



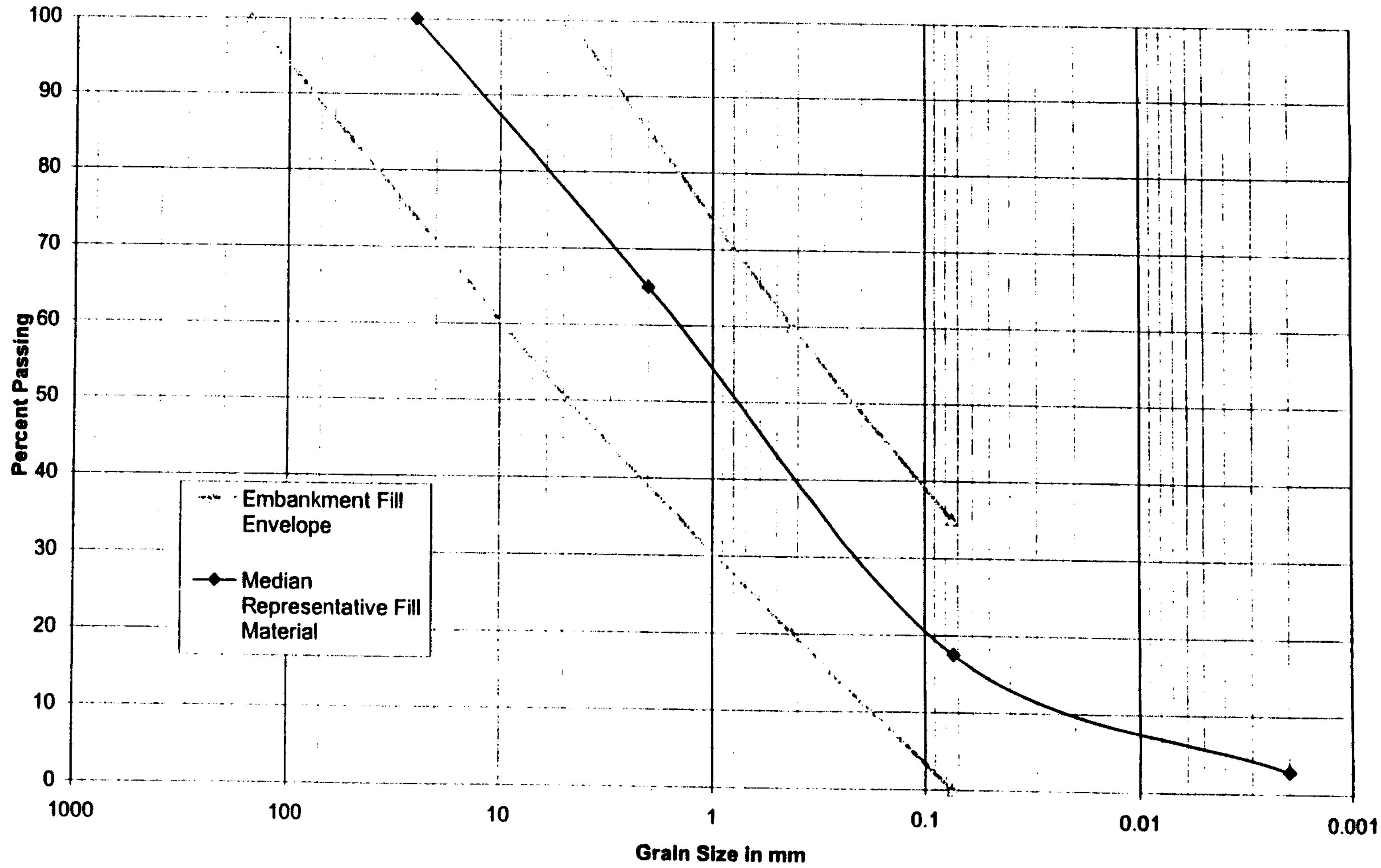
DJH 10/12/00 497828Vcdr

HARTCROWSER

J-4978-28 10/00

Figure C-6

Grain Size Envelope for Group 3 Fill Material



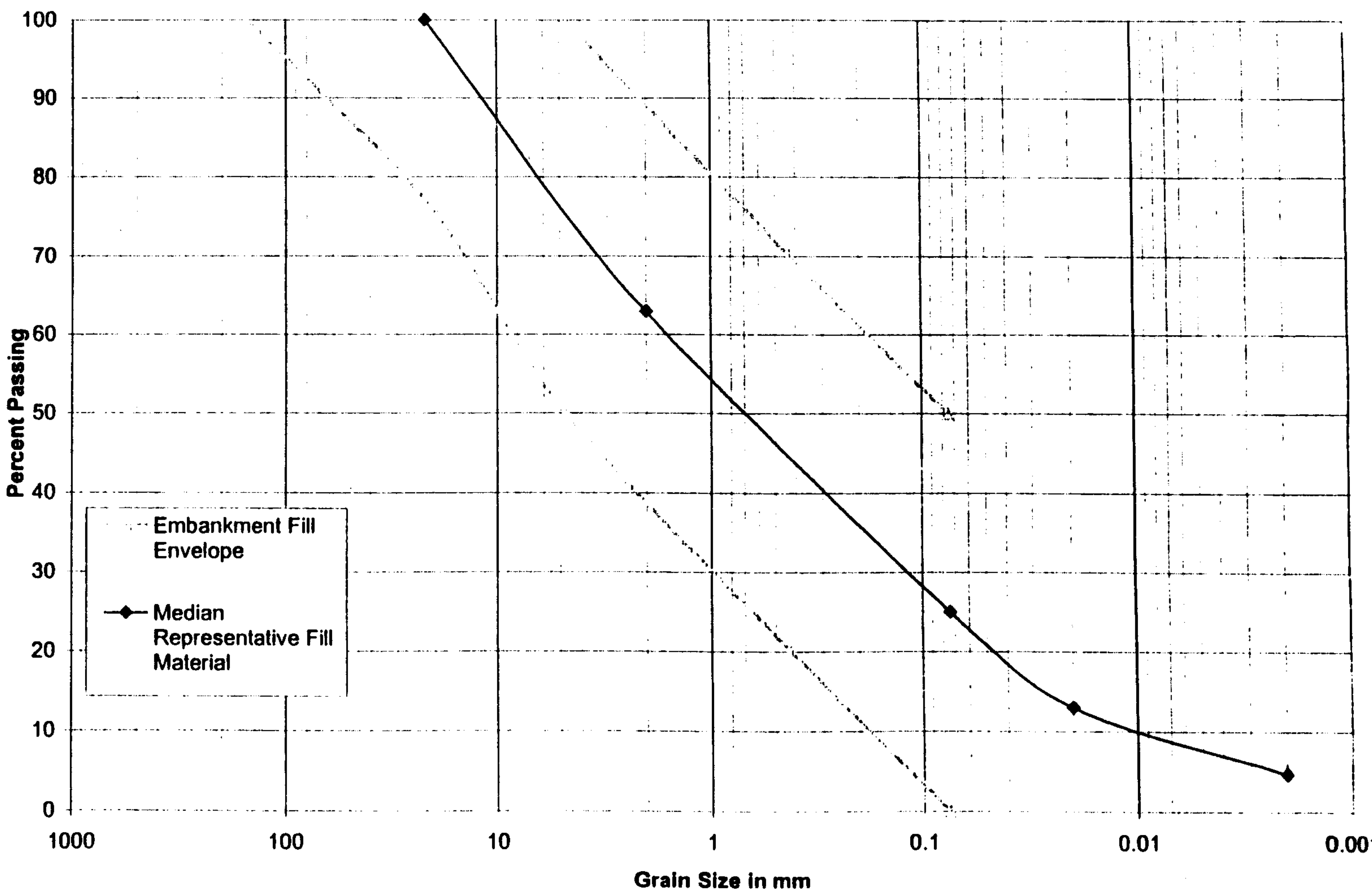
DJH 10/12/00 497828W.cdr

HARTCROWSER

J-4978-28 10/00

Figure C-7

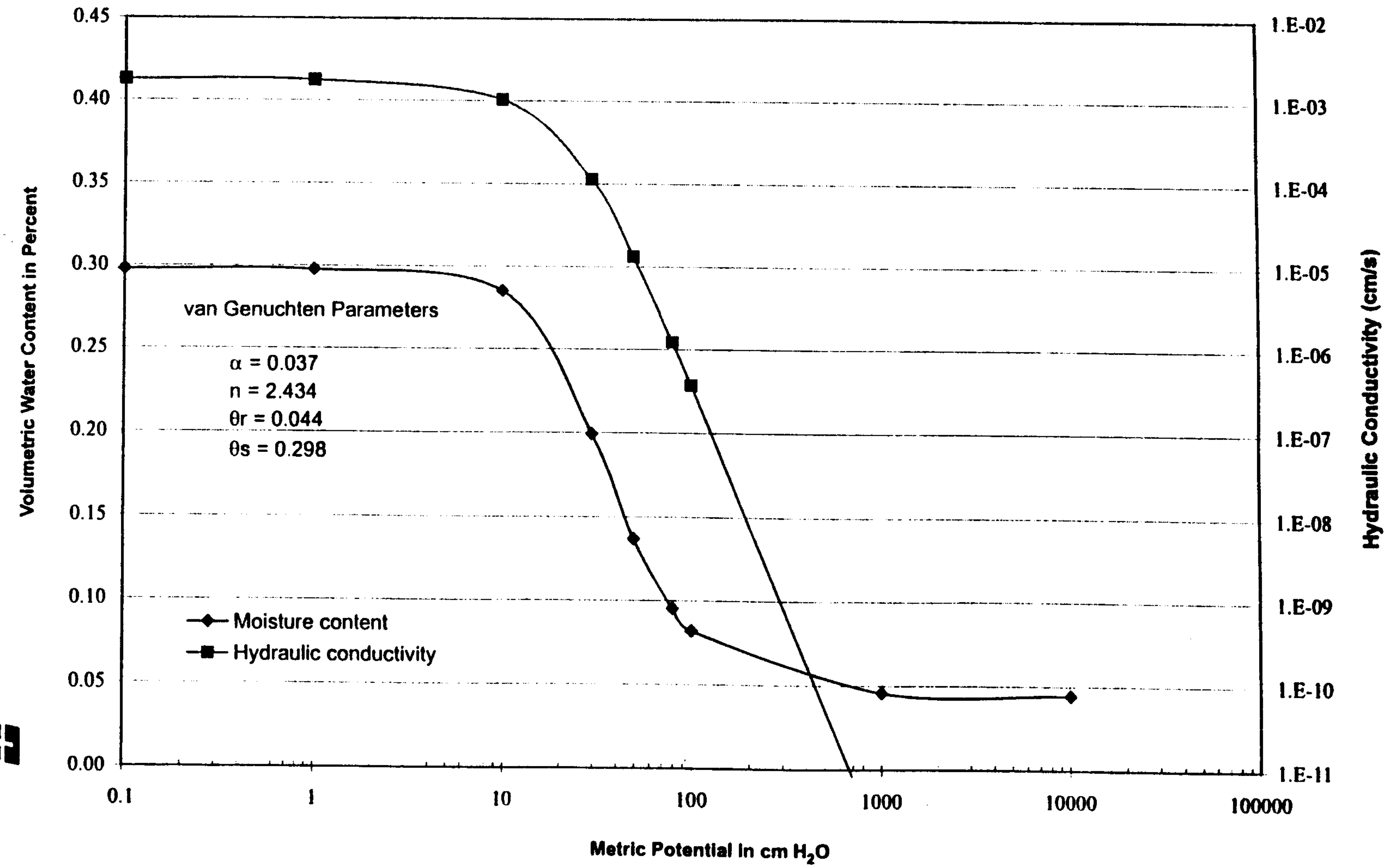
Grain Size Envelope for Group 4 Fill Material



HARTCROWSER
J-4978-28 10/00
Figure C-8

DTN 10/12/00 497828X.cdr

Soil Moisture/Conductivity Characteristic Curves for Group 1B Fill Material



DJH 10/12/00 497828Y.cdr

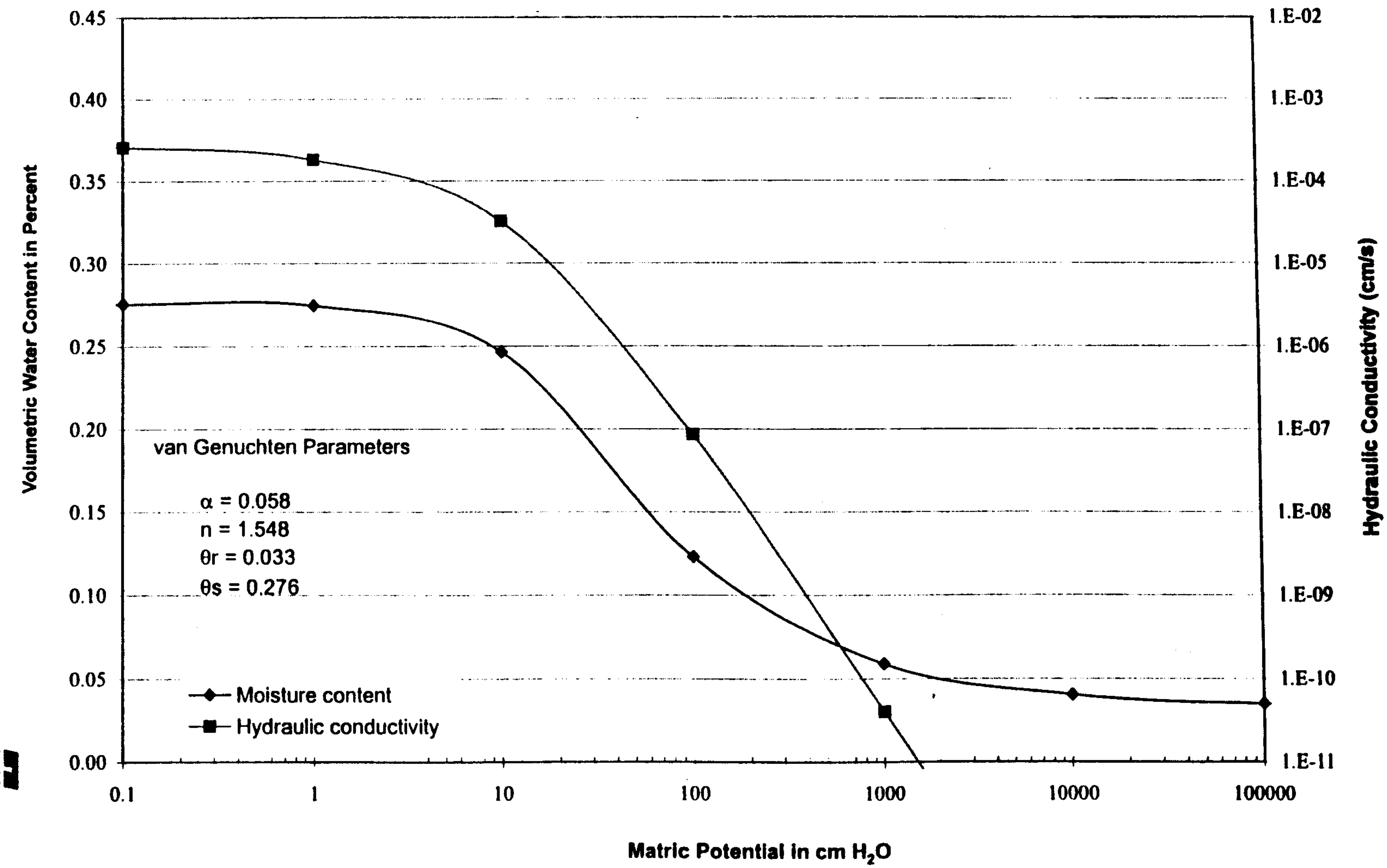
HARTCROWSER

J-4978-28 10/00

Figure C-9

AR 049752

Soil Moisture/Conductivity Characteristic Curves for Group 3 Fill Material

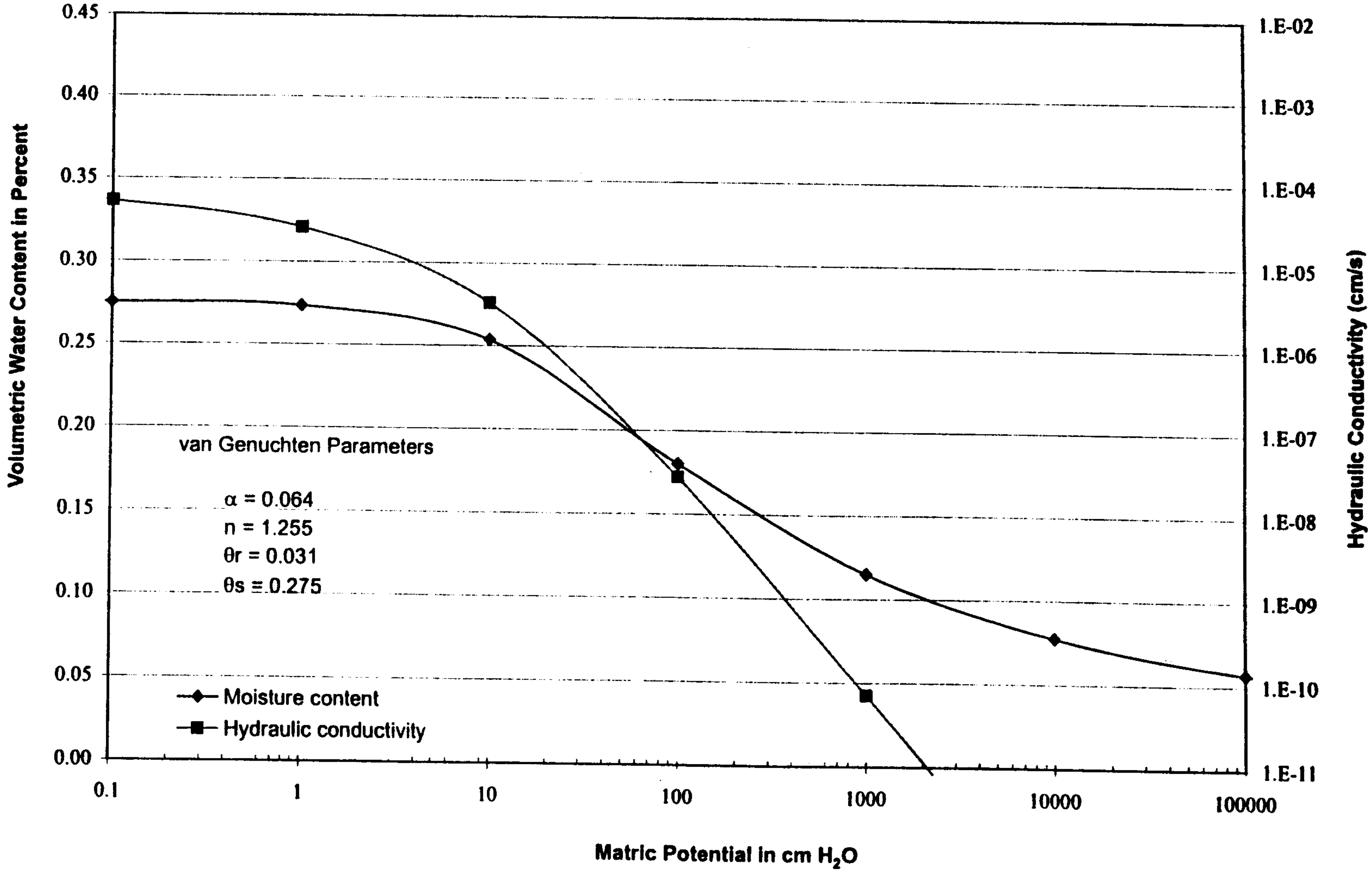


DJH 10/12/00 4978287.cdr

HARTCROWSER
J-4978-28 10/00
Figure C-10

AR 049753

Soil Moisture/Conductivity Characteristic Curves for Group 4 Fill Material



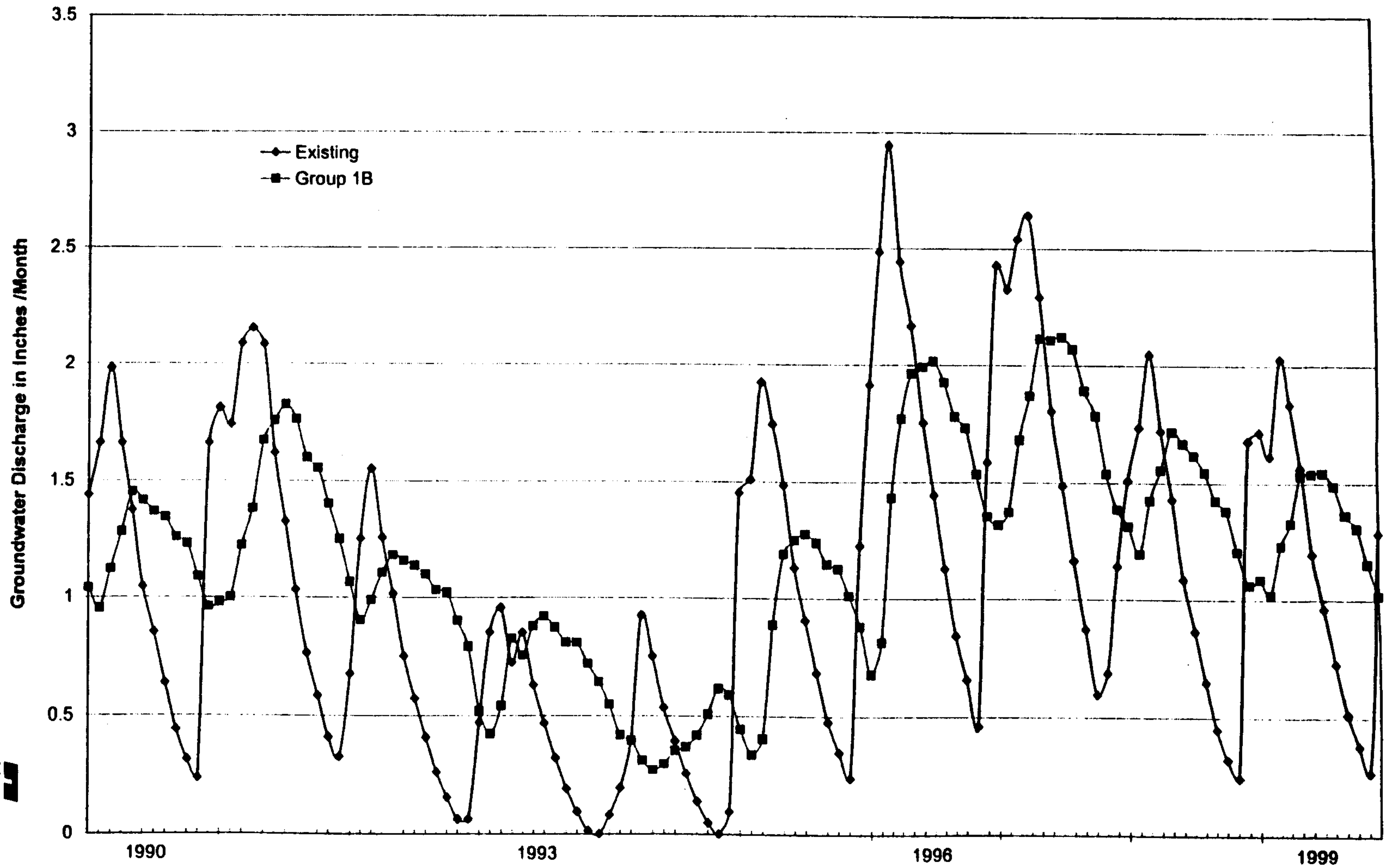
DJH 10/12/00 497828RR.cdr

HARTCROWSER

J-4978-28 10/00

Figure C-11

Simulated Groundwater Discharge Rates for Existing Conditions and Group 1B Fill



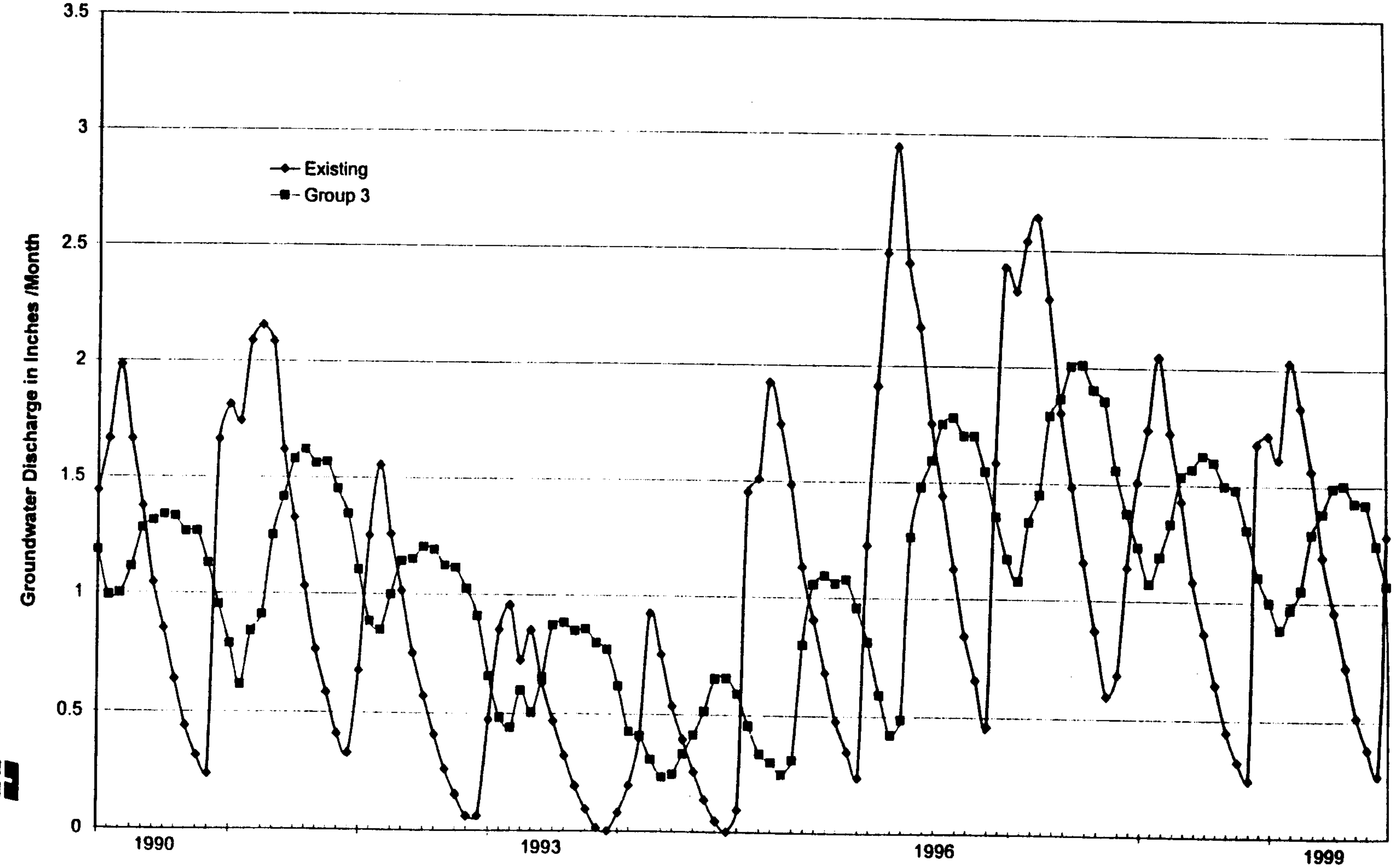
DJH 10/12/00 497828AA.cdr Sheet 1 of 4

HARTCROWSER

J-4978-28 10/00

Figure C-12

Simulated Groundwater Discharge Rates for Existing Conditions and Group 3 Fill



DJH 10/12/00 497828AA.cdr Sheet 2 of 4

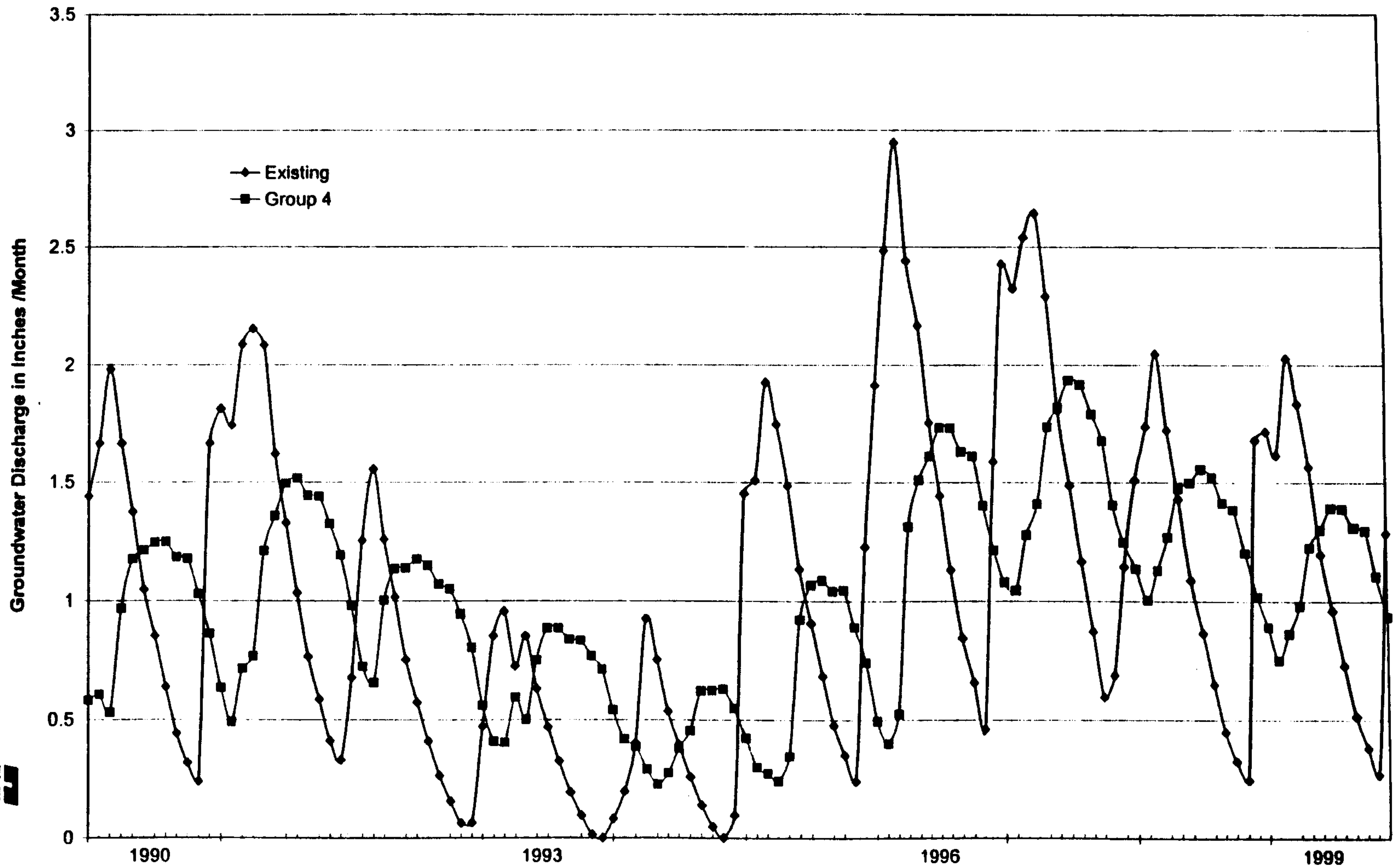
HARTCROWSER

J-4978-28 10/00

Figure C-13

AR 049756

Simulated Groundwater Discharge Rates for Existing Conditions and Group 4 Fill



DJH 10/12/00 497828AA.cdr Sheet 3 of 4

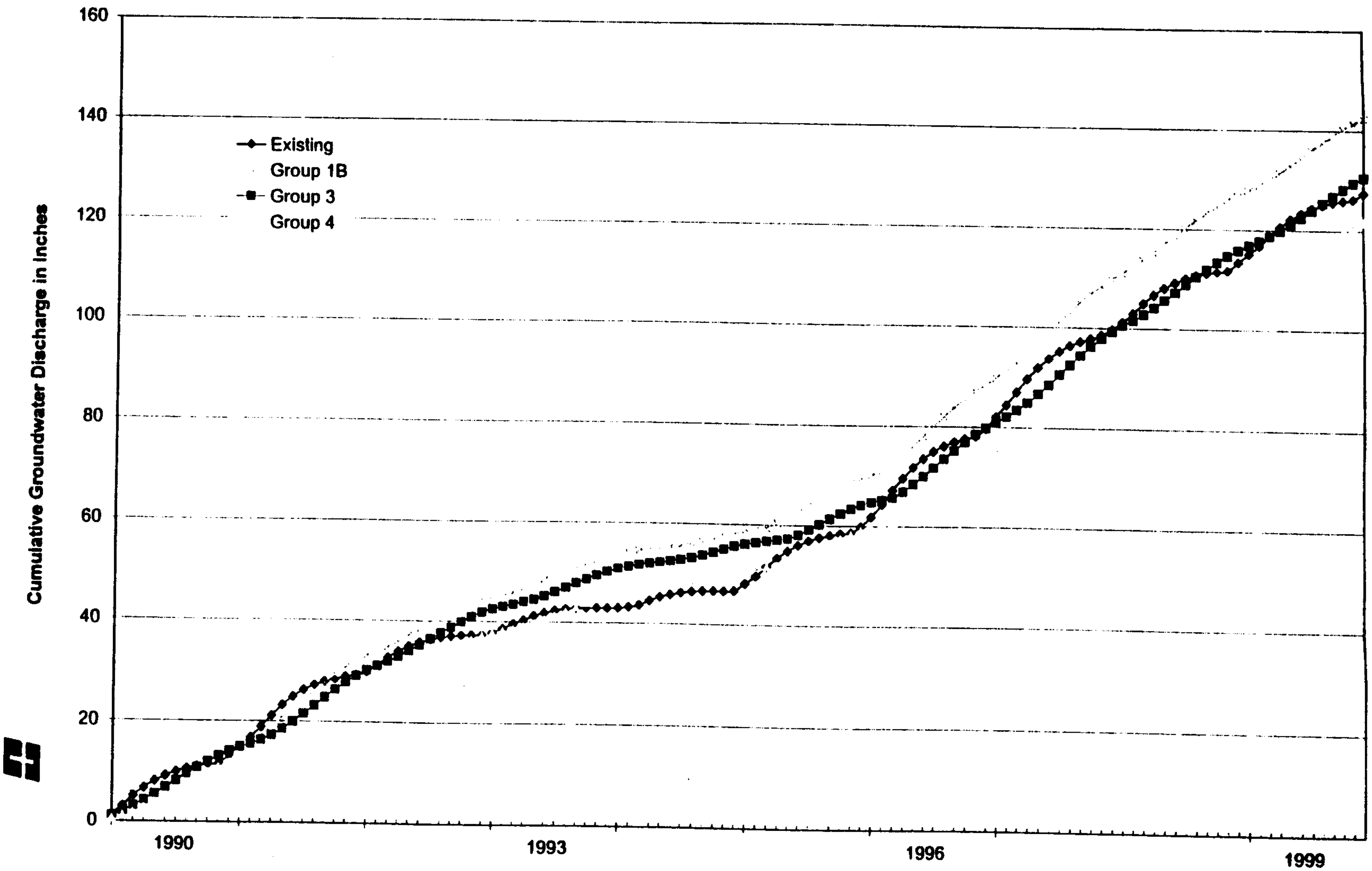
HARTCROWSER

J-4978-28 10/00

Figure C-14

AR 049757

Cumulative Plot of Simulated Groundwater Discharge Rates



HARTCROWSER

J-4978-28 10/00

Figure C-15

AR 049758

DJH 19/12/00 497828AA.cdr Sheet 4 of 4

APPENDIX D
ALTERNATIVE ACCEPTANCE CRITERIA FOR EMBANKMENT FILL MATERIALS

www.hartcrowser.com

HARTCROWSER
Delivering smarter solutions

MEMORANDUM

Anchorage Boston Chicago Denver Fairbanks Jersey City Juneau Long Beach Portland Seattle

DATE: September 27, 2000

TO: Jim Thompson, HNTB

FROM: Michael J. Bailey, P.E., and John P. Laplante, Hart Crowser, Inc.

RE: **Alternative Acceptance Criteria for Embankment fill Materials**
DRAFT Specification: Direct Shear Testing for Fill Material Substitution
Third Runway Project
SeaTac, Washington
J-4978-25

As we have discussed, there are a number of potentially acceptable fill sources that do not fall within the currently specified gradation ranges for Groups 1A through 4 for the Third Runway embankment. It is generally necessary to specify gradation and density, and some other characteristics, to provide assurance that the resultant fill will have acceptable strength and deformation characteristics. This is relatively easy to do by limiting gradation, as we have done to date. The approach presented herein is to specify some additional testing and acceptance criteria that could be used by the Port and its Contractors to assess suitability of alternative fill materials.

This memorandum and attachments provide our recommendations for specifying use of the "Direct Shear Test" along with other information already part of Item P-152, Excavation and Embankment, of the Project Manual. This approach is somewhat uncommon because 1) strength tests are typically limited by the maximum size of soil particles that can be tested, and 2) there is contradictory information on how to adjust "small scale" test results for actual gradations used in construction. We have adopted a conservative approach herein, based on a review of geotechnical literature and the practices used by other engineering organizations.

Background

The effect of fill gradation on strength of compacted embankments is often not considered explicitly. Many organizations constructing earth fills select fill material, side slope, and fill compaction criteria simply because they have worked well in the past. Typically this is an

1910 Fairview Avenue East
Seattle, Washington 98102-3699
Fax 206.328.5581
Tel 206.324.9530

acceptable approach, particularly where strength to resist seismic loads is not an explicit design consideration, or where the cost of using "well-graded" fill material to conservatively achieve minimum strength required, is acceptable. (A "well-graded" fill is soil that has a mixture of particles of different sizes, which promotes densification, and provides higher strength and less compressibility compared to "poorly graded" or more uniform fill materials).

To date the Third Runway specifications for different fill material groups have allowed only relatively well-graded soils. This approach could be continued and would produce satisfactory fill, but might not be the most cost-effective way to produce satisfactory fill. For example, soil from Taxiway C was successfully placed in 1999 in an area that will become the interior of the embankment. This soil is too uniform (silty fine sand) than our current specs would allow, and Hart Crowser would not recommend use of this soil for permanent exterior slopes subject to seismic loads, without test results to verify strength of the compacted fill.

Strength tests on soils such as the Taxiway C material are relatively easy to use. However, strength tests on gravelly soils are much more variable and are difficult to interpret. The goal of the approach presented herein is to provide a relatively simple procedure that can be used by the Contractor's testing lab, similar to the kind of tests now required for verification that proposed fill materials meet criteria in Item P-152.

Strength Test Requirements

To screen potential fill materials that do not meet the existing gradation requirements (Groups 1A through 4), we recommend the Port require the Contractor to complete a testing program to determine if the proposed soil can be sufficiently compacted to provide the strength needed for adequate embankment stability.

Strength for compacted soil specimens can be measured using "Direct Shear" testing, which is a commonly available technique. Almost all soils laboratories have capability to accomplish "direct shear testing" (ASTM D 3080) but many labs are not equipped to accomplish more sophisticated "triaxial shear" tests (ASTM D 4767, D 2850, etc.).

Most soils laboratories are limited to testing fill samples with a maximum particle size considerably less than 1 inch, regardless of what test methods are used. For comparison of strength test results to actual soil fills, a conventional rule of thumb is that increasing the portion of the soil larger soil particles will increase, or at least not decrease, the measured strength. However, several research studies have shown this is not always true, and there are a number of explanations why.

To address the uncertainty of particle size effects, ASTM requires the minimum Direct Shear sample diameter be at least 2 inches, or 10 times the maximum particle diameter, whichever is greater. Typical commercially available Direct Shear test equipment allows samples sizes from about 2 inches up to a maximum of about 12 inches. Even for a 12-inch shear box, the maximum particle size that can be tested (on the order of 1-1/4 inch if ASTM standards are used) is much smaller than the maximum particle size (6 inches) allowed for the import fill material at the Third Runway.

Published research is inconsistent on the relationship between particle size and the strength of a soil sample, apparently because no simple, direct correlation exists. Table 1 lists the references we checked and summarizes the general conclusions from each study.

Some research suggests that laboratory samples, which have had larger particles screened out for testing, will exhibit lower strength than their parent soils. However, more of the published test results suggest the opposite. That is to say, testing a portion of a soil (screened sample) that contains only the finer fraction of the parent soil should yield strengths that are actually higher than the parent material would exhibit. The available research further predicts that this decrease in strength is linear with increasing particle size. As a result, mathematical relations have been developed that allow the reduced strength of the parent material to be estimated based on the laboratory strength test results of screened samples.

In developing the enclosed draft specification, Hart Crowser assumed that soil strength decreases with increasing particle size according to the most current research we located. Thus, smaller test samples must demonstrate higher strengths to assure acceptable strengths are achieved during construction. The enclosed draft specification includes different minimum strength acceptance criteria to enable the Contractor to use any commercial soils testing laboratory to evaluate potential fill sources.

Implementation

Upon acceptance by HNTB and the Port, the enclosed draft special provision can be included under Item P-152 (or possibly as part of the Substitutions clause, Section 01630), in the Project Manual for Third Runway Embankment Construction. Completion of a direct shear test program would provide a way for the Contractor to support the use of an alternative fill material that does not conform to the gradation requirements for soil Groups listed in P-125.

If a Contractor wants to propose use of an alternative or "non-conforming" fill material, the submittal for this testing program can be prepared by any test laboratory with Direct Shear equipment, and can be easily reviewed for acceptance by the Port of Seattle. We estimate that each series of Direct Shear tests would cost the Contractor about $1,000.

One question that needs careful consideration is how many strength tests to require per submittal, or what variation to tolerate in an approved fill material before considering it as a "new" material. We propose to require Direct Shear tests at a rate of 1 test per 50,000 tons of proposed import fill material. Ideally, tests would be performed on a pre-approval basis, with the test lab certifying that the entire source was examined and found to be consistent enough such that the samples tested were representative of the whole. As you know, there have been some problems with this approach but overall it seems to be working.

At present, changes in soil gradation for a particular source require the Port to obtain additional Proctor test results, but do not put any cost burden on the Contractor. We propose to use the same approach with the stipulation that changes in fill gradation sufficient to require a new Proctor test may be the basis for requiring the Contractor to submit new Direct Shear strength test results or for rejection of the material.

We would like to get your comments on the enclosed draft specification, and particularly how you might propose to modify this approach based on your experience. Please contact us if you have any questions.

F:\docs\jobs\497825\3r direct shear spec.doc

Attachments:
  Table 1 - Summary of Supporting Research
  List of References
Enclosure:
  Draft Specification: Direct Shear Testing for Fill Material Substitution

Table 1 - Summary of Supporting Research

Source	Maximum Particle Size, Type of Test	Conclusion
Marachi et al. (1)	6 in., Triaxial Shear	ϕ increases as particle size decreases.
Bishop (2)	1-1/4 in., Direct Shear	Particle size does not affect ϕ.
Lewis (3)	1-1/4 in., Direct Shear	ϕ decreases as particle size decreases.
Vallerga, et al. (4)	0.2 in., Triaxial Shear	Particle size does not affect ϕ.
Rowe (5)	1 mm, Sliding Friction	ϕ_u increases as particle size decreases.
Leslie (6)	3 in., Triaxial Shear	ϕ increases as particle size decreases.
Kirkpatrick (7)	2 mm, Triaxial Shear	ϕ increases as particle size decreases.
Marsal (8)	8 in., Triaxial Shear	ϕ increases as particle size decreases.
Lee et al. (9)	3/4 in., Triaxial Compression	Compression increases as particle size increases.
Fumagalli (10)	260 mm, Triaxial Compression	Compression increases as particle size increases.
Thiers and Donovan (11)	2-1/2 in., Triaxial	ϕ increases as particle size decreases.
Knodel, P.C. US Dept. of Interior (12)	Varying gravel contents	Gravel content below 50% has little affect on ϕ.
Holtz & Kovacs (13)		Particle size, at constant void ratio, does not influence ϕ significantly.
Hirschfeld & Poulos (14)		For some materials gradation might not be an important factor.
Lambe & Whitman (15)		Samples with same uniformity coefficient, but different avg. particle sizes have much the same ϕ; A better distribution of particle sizes will have a higher ϕ.

F:\docs\jobs\497825\3r direct shear spec.doc

List of References

(1) Marachi, N.D., C.K. Chan, and H.B. Seed, 1972. "Evaluation of Properties of Rockfill Materials," *Journal of Soil Mechanics and Foundation Division,* ASCE, January 1972, pp. 95-114.

(2) Bishop, A.W., 1948. "A Large Shear Box for Testing Sands and Gravels," *Proceedings for the 2nd International Conference of Soil Mechanics and Foundations Engineering,* Vol. 1, pp. 207-211.

(3) Lewis, J.G., 1956. "Shear Strength of Rockfill," *Proceedings of the 2nd Australia-New Zealand Conference on Soil Mechanics and Foundation Engineering,* pp. 50-52.

(4) Vallerga, B.A., et al., 1957. "Effect of Shape, Size and Surface Roughness of Aggregate Particles on the Strength of Granular Materials," *Special Technical Publication No 212,* ASTM.

(5) Rowe, P.W., 1962. "The Stress-Dilatancy Relation for Static Equilibrium of an Assembly of Particles in Contact," *Proceedings of the Royal Society of London,* Vol. 269, pp. 500-527.

(6) Leslie, D.E., 1963. "Large Scale Triaxial Tests on Gravelly Soils." *Proceedings, 2nd Pan Am Conference on Soil Mechanics and Foundation Engineering,* pp. 181-202.

(7) Kirkpatrick, W.M., 1965. Effects of Grain Size and Grading on the Shearing Behavior of Granular Materials," *Proceedings, 6th International Conference on Soil Mechanics and Foundation Engineering,* Vol. 1, pp. 273-277.

(8) Marsal, R.J., 1965. *Discussion, Proceedings, 6th International Conference on Soil Mechanics and Foundation Engineering,* Vol. 3, pp. 310-316.

(9) Lee, K.L. and I. Farhoomand, 1967. "Compressibility and Crushing of Granular Soils in Anisotropic Compression," *Canadian Geotechnical Journal,* Vol. 4, No. 1, pp 68-86.

(10) Fumagalli, E., 1969. "Tests on Cohesionless Materials for Rockfill Dams," *Journal of Soil Mechanics and Foundations Division,* ASCE, Vol. 95, No. SM1, Proc. Paper 6353, January 1969, pp 313-332.

(11) Thiers, G.R. and T.D. Donovan, 1981. "Field Density, Gradation, and Triaxial Testing of Large-Size Rockfill for Little Blue Run Dam," *Laboratory Shear Strength of Soil,* ASTM STP 740, R.N. Young and F.C. Townsend, Eds., American Society for Testing and Materials, pp. 315-325.

(12) Knodel, P.C., 1966. United States Department of the Interior; Bureau of Reclamation, "Summary of Large Triaxial Shear Tests for Silty Gravels Earth Research Studies," Report No. EM-731, June 1966, Denver.

(13) Holtz, R.D., and W.D. Kovacs, 1981. "An Introduction to Geotechnical Engineering," Prentice Hall, Englewood Cliffs, NJ, pp 516-517.

(14) Hirschfeld, R.C. and S.J. Poulos, Eds., 1973. "Embankment-Dam Engineering," John Wiley & Sons, New York, pp.15-159.

(15) Lambe, W.T. and R.V. Whitman, 1969. "Soil Mechanics," John Wiley & Sons, New York, pp. 146- 149.

The following materials were reviewed, but direct reference to the influence of particle size on the shear strength of soils was not discussed:

(16) Sherard, J.L., R.J. Woodward, S.F. Gizienski, and W.A. Clevenger, 1963. "Earth and Earth-Rock Dams," John Wiley & Sons, New York.

(17) United States Department of the Interior; Bureau of Reclamation, 1997. "Design of Small Dams," A water Resources Special Publication, U.S. Government Printing Office, Washington.

(18) Terzaghi, K., R.P. Peck, and G. Mesri, 1996. "Soil Mechanics in Engineering Practice," John Wiley & Sons, New York.

F:\docs\jobs\497825\3r direct shear spec.doc

Draft Specification: Direct Shear Testing for Fill Material Substitution

The following specification should be inserted at the end of Part B of Item P-152-1.1:

6. Direct Shear test for soil strength in accordance with ASTM D 3080.

All soil tests shall be accomplished in conformance with ASTM D 3740: Minimum Requirements for Agencies Engaged in the Testing and/or Inspection of Soil and Rock as Used in Engineering Design and Construction.

The following specification should be inserted at the end of Item P-152-1.2

G. Alternative Approval Process for Non-conforming Fill Borrow Material: Fill borrow material that meets all other requirements of P152-1.2 except gradation requirements of Part A, may be approved by the Engineer as a substituted fill material based on the submittal of Direct Shear test results which demonstrate the proposed non-conforming fill meets the minimum strength criteria specified herein.

1. General Requirements

1.1. Applicability

This special provision covers testing and acceptance requirements for proposed import soils that do not meet the gradation requirements specified for soil groups in Section 152-1.2. All other provisions of Section 151-1.2 apply to the proposed import soils.

The Contractor's Independent Testing Laboratory (ref. Section 01451) shall accomplish all tests required for submittals as specified herein. The Testing Laboratory shall collect the test samples of the proposed non-conforming fill material, and certify that tests were completed on representative portions thereof.

1.2 Testing Frequency

For each non-conforming soil type proposed by the Contractor, at least one series of Direct Shear tests shall be required per 50,000 tons of proposed import soil. Where more than 100,000 tons of a particular soil are available, a maximum of 3 series of Direct Shear tests shall be required for that soil type, provided the Engineer observes that gradation and maximum dry density ($\gamma_{d\,max}$) of the soil placed and compacted are consistent. Change in soil gradation or maximum dry density shall be grounds for rejecting further use of an approved non-conforming soil type, or for requiring additional Direct Shear tests to support continued use.

2. Sample Preparation

2.1. Samples for each series of Direct Shear tests shall be prepared in general accordance with the procedures specified in ASTM D 3080. At least 3 samples shall be compacted and tested for each series.

2.2. Maximum test sample particle size shall be subject to the following limitations, based on the equipment being used to perform the Direct Shear test.

Size of Shear Box*	Maximum Particle Size in Sample
2.5-inch	1/4 inch
4-inch	3/8 inch
12-inch	1-1/4 inch

* Consult the engineer for shear box sizes other than those listed herein.

2.3. Each test sample shall be prepared by removing oversize particles to the limits specified above, while taking care to maintain gradation of the test sample as nearly parallel as possible to the grain size curve of the original material.

2.4. Each test sample shall be compacted to a minimum of 92 percent of maximum dry density, with a moisture content that is within ± 2.0 % of optimum moisture content, as determined by ASTM D 1550. Higher percentages of maximum dry density may be used for the test samples to obtain the minimum shear strength specified herein, provided the Contractor can consistently achieve those same higher compaction levels during construction.

Compact each test sample directly into the Direct Shear test apparatus shear box, or use a mold that facilitates transfer of the compacted sample into the shear box without disturbance.

All three samples within a Direct Shear test series shall be compacted to within ±1.5% of the same dry density of one another.

3. Direct Shear Tests

3.1 Test Methods and Standards

Direct Shear testing shall be performed in conformance with requirements of ASTM D-3740 and ASTM D 3080.

A series of tests shall consist of 3 samples, tested at normal loads of 5, 10, and 15 kips per square foot (ksf). Analyze and report results in accordance with ASTM D 3080.

Maximum dry density of each test sample shall be measured in accordance with ASTM D 1557.

4. Submittals

For each type of proposed non-conforming fill borrow material, submit results of each Direct Shear test series including the following:

4.1 Location of material source, and name or designation provided to uniquely identify the proposed import soil.

4.2. Sieve analysis and existing moisture content of proposed import soil at the time of field sampling.

4.3 Sieve analysis of test samples prepared for Direct Shear tests.

4.4 Moisture/density relationship of proposed import soil, per ASTM D 1557.

4.5 Dry density and moisture content of all three samples used for the series of Direct Shear tests.

4.6 Test results, including plots of shear stress and shear strain for each test sample; and normal load vs. shear stress at failure, with the interpreted soil strength friction angle for the test series..

4.7 Certification from a licensed Professional Engineer that the submitted soil test data accurately represent the proposed non-conforming fill material from the designated source site.

5. Acceptance Criteria

The proposed non-conforming fill material may be accepted by the Engineer provided 1) the soil friction angle from Direct Shear testing meets or exceeds the soil strength requirements in the following table; and 2) the soil conforms to all other requirements of this specification except gradation.

Shear Box Size*	Minimum Soil Strength Expressed as Friction Angle in Degrees
2.5-inch	41
4-inch	39
12-inch	37

* Consult engineer for shear box sizes not listed herein.

Acceptance of a non-conforming fill material shall further be contingent upon verification that fill compaction by the Contractor meets or exceeds both

1. The percent compaction that was required for the Direct Shear test series to meet the specified acceptance criteria; and

2. Minimum percent compaction for the Zone in which it is placed, specified in section 152-2.3.

The contractor shall not propose an import material that requires higher laboratory compaction than can be achieved in the field.
(end of specification insert)

F:\docs\jobs\497825\3r direct shear spec.doc