DRAFT

Geotechnical Engineering Analyses and Recommendations
Third Runway Embankment, Phase 5
Sea-Tac International Airport
SeaTac, Washington

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Prepared for
HNTB

September 26, 2001
4978-28

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Prepared by
Hart Crowser, Inc.

DRAFT

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SEA-TAC INTERNATIONAL AIRPORT
SEATAC, WASHINGTON

INTRODUCTION

This report presents the results of Hart Crowser's geotechnical engineering analyses and recommendations for Phase 5 construction of the proposed embankment to support the new Third Runway at Seattle-Tacoma International Airport (STIA), in SeaTac, Washington.

This report provides geotechnical recommendations for design and construction for the Phase 5 work, including embankment construction, subgrade improvements, construction of Pond "A" for temporary erosion and sediment control (TESC), as well as modifications to Ponds D and F. Hart Crowser's geotechnical recommendations are based on results of subsurface explorations and tests that are presented in a companion report "Subsurface Conditions Data Report, Phase 5 Fill and Subgrade Improvement" (Hart Crowser 2001b).

SUMMARY

This section presents a summary of the major observations and recommendations presented in this report. At the end of the summary is a description of how this report is organized.

The remainder of this report presents important information that supports and expands on this summary, and the entire report should be reviewed and considered as a whole.

Site Preparation

Site preparation recommendations for the Phase 5 embankment are the same as previously recommended and implemented for prior Third Runway fill construction. Site preparation includes installation of TESC; abandonment of existing underground utilities by filling pipes with concrete; removal of surficial vegetation; proof rolling to disclose any soft or loose zones unsuitable to support the embankment; and removal of topsoil in a strip 50 feet wide along the toe of the new fill.

Subgrade Improvement

Phase 5 fill construction will extend over areas identified as wetlands, which include but are not limited to drainage swales created by erosion. Observations indicate these areas range from relatively competent soils to areas with soft sediment and organic soils (peat). Construction details developed by HNTB and Hart Crowser for Phase 4 construction are suitable for use in areas where thickness of the drain layer needs to be increased to accommodate existing seepage and/or stabilize soft subgrade.

In addition to subgrade improvement that may locally be needed below the current fill placement area, Phase 5 construction includes removal and replacement of soils that are not suitable to support fill and mechanically stabilized earth (MSE) retaining walls, which will be built in subsequent phases. Geotechnical recommendations for subgrade improvement include dewatering, temporary excavation slopes and sheet pile barrier, verification of subgrade suitability, and backfill placement and compaction.

Dewatering

Hart Crowser recommends that a system of vacuum well points installed around the perimeter of the subgrade improvements area, combined with sumps within the excavation, be used to dewater the subgrade improvement area. The dewatering system should be designed, installed, and operated by a specialty subcontractor in accordance with performance criteria specified by the Port.

Dewatering should be accomplished to eliminate water in the excavation so that suitability of the exposed subgrade can be verified, and to enable compaction of backfill to a very dense condition.

Temporary Excavation Slopes

Subgrade improvement is needed to remove soft or loose soils and replace them with very densely compacted structural fill suitable for support of the permanent embankment slopes and MSE walls. Depth of the subgrade improvement is anticipated to vary up to about 17 feet.

Stability of temporary cut slopes along the perimeter will depend in part on the success of dewatering. Where dewatering is accomplished to avoid seepage from the face or at the toe of the temporary cut slopes, we recommend that the sides of the excavation be no steeper than 1.75H:1V. While this slope is anticipated to have an overall factor of safety of about 1.3, some raveling or surficial sloughing may still occur due to local variation in soil strength.

Temporary Sheet Pile Barrier

Recommendations are provided for installation of a temporary sheet pile barrier shoring along the west side of the subgrade improvement excavation. This sheet pile barrier is intended to reduce risk of uncontrolled seepage from Miller Creek into the excavation, improve efficiency of the well point dewatering system, and improve stability of the adjacent temporary cut slope. Recommended extent and a typical cross section of the sheet pile barrier are shown on Figures 8 and 10, respectively.

Verification of Subgrade Suitability

Phase 5 contract documents show proposed base of subgrade improvements based on information from borings drilled in the area. While this provides a reasonable basis for bidding the excavation work, actual soil conditions vary between exploration locations. Suitability of the exposed subgrade and removal of unsuitable soils will need to be verified by an experienced inspector at the time of construction.

To verify subgrade suitability, all loose or soft soils will need to be removed, and there should be no standing water present at the time of the subgrade excavation. To prevent possible disturbance, particularly under moist conditions, traffic should be kept off the exposed subgrade prior to backfilling.

Backfill for Subgrade Improvements

Backfill for subgrade improvements should consist of very densely compacted structural fill. Because there may be a wet subgrade or some unavoidable seepage at the time of backfilling, Hart Crowser recommends backfill soil be limited to Type 1A or 1B, using the same gradation criteria that were used in Phase 4 (presented in Table 1 of this report).

Backfill will need to be very densely compacted to avoid potential liquefaction of the backfill if it is saturated at the time of a future earthquake. For bidding purposes, we recommend specifying that minimum compacted density exceed 98 percent of modified Proctor maximum density. This specification may be modified as discussed elsewhere in this report.

Embankment Fill Construction

Embankment fill materials and methods of construction are generally anticipated to be the same as were used for previous phases of construction for the Third

Runway embankment. Results to date indicate the specifications used for Phase 4 are generally well-suited for continued use.

At the time of preparing this report, fill moisture content limitations for Group 3 and Group 4 soils are being evaluated relative to anticipated Phase 5 construction rate and the embankment geometry. Observations during Phase 3 and Phase 4 construction indicate some potential for development of excess pore pressures, which could lead to instability. These pore pressures may need to be controlled by restricting moisture contents.

Fill placement rate is not anticipated to present any risk of instability except in one area near the center of the Phase 5 fill, typified by the cross section at runway station 179+50. In this area a thick layer of low strength clay and peat, of limited extent, is estimated to produce a minimum factor of safety of about 1.1 for a fill rate of 2 feet per day, or a factor of safety of about 1.2 for a fill rate of 1 foot per day. However, there are not many borings, and no test pits, in this area to confirm inferred soils conditions (the area is in a wetland). Hart Crowser recommends this area be further assessed when access is available under the 404 Permit.

TESC Ponds

Hart Crowser recommends that Pond A be surrounded by sheet piles embedded outside the limits of the cut slopes to provide stability. This is recommended because the pond will be excavated into soft soils and operation of the pond is anticipated to include frequent filling and drawdown as winter stormwater is collected and pumped to other parts of the project TESC system.

We recommend that Pond A side slopes be no steeper than 2H:1V, and the slopes should be protected with a combination of geotextile "filter fabric" and riprap. A shallow trench drain around the outside of the sheet piles needs to be provided to convey seepage to avoid disruption of shallow base flow to downgradient wetlands (Hart Crowser 2001a).

Design and construction recommendations for modification of TESC Ponds D and G are presented in a separate report (Hart Crowser 2001c). Construction of Pond F will be addressed when this report is finalized.

Organization of This Report

Following this summary is a brief discussion of subsurface conditions affecting geotechnical design and construction recommendations. The remaining text discusses specific geotechnical observations and recommendations for construction.

Figure 1 is a map showing the STIA vicinity and location key for maps that illustrate features discussed in this report. Figures 2 and 4 are Site and Exploration Plans for Phase 5 Work Areas West and South 2, respectively. Figure 3 is a groundwater elevation contour map for Work Area West, based on the highest recorded groundwater levels.

Figures 5 through 7 present a detailed plan and cross sections through TESC Pond A. Figures 8 through 10 present a detailed plan and cross sections for the subgrade improvements in Work Area West.

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

SUBSURFACE CONDITIONS

Information on subsurface conditions in the area covered by this report is based on field explorations and laboratory tests that are described in an accompanying data report (Hart Crowser 2001b). The Phase 5 data report is a compilation of exploration and test information from several previous reports prepared for different aspects of the Third Runway project.

Areas Addressed in this Report

This report addresses the Third Runway embankment and subgrade improvement areas generally within the area referred to as Work Area West, as shown on Figure 2. Embankment fill materials provided by the Port may be obtained from the stockpiles and excavations in Work Area South (see Figure 4) and Borrow Area 4 (see Hart Crowser 2001d).

Soils

Work Area West

Work Area West includes the proposed Phase 5 embankment, Pond A, and the subgrade improvement excavation area for the West MSE wall and adjacent embankment (see Figures 2, 3, 5, and 8).

Subgrade conditions in the Phase 5 embankment and subgrade improvement areas (Work Area West) typically consist of up to about 20 feet of relatively soft or loose surficial soils (consisting of interbedded silty sand, sandy silt, clays, peat, and fill) overlying dense or hard glacially overridden soils. The following soils from the ground surface downward were encountered in Work Area West:

Topsoil. Topsoil, consisting of a loose mixture of silt and sand with roots and other organic material was intermittently encountered in our explorations, ranging from about 1/2 to 1 foot thick where encountered.

Pre-Construction Fill. Existing fill, consisting of loose to medium dense, variable mixture of silty or clayey sand, and gravel was encountered intermittently in Work Area West, typically associated with prior site use including paved streets and residential housing. Fill is generally absent in the low-lying portions of the site, adjacent to wetlands. Most of the fill is less than 1 foot thick. The density and granular nature of the fill materials resemble the recessional outwash deposits, and the fill is sometimes difficult to distinguish from the outwash.

Alluvial Deposits Consisting of Interlayered Silt, Clay, Sand, and Peat. Alluvial deposits are sediments associated with Miller Creek. These soils occur mainly in the low-lying areas to depths of up to about 15 feet.

The consistency of the clay and silt deposits vary widely from soft to stiff, and these soils generally contain sand fractions ranging up to about 30 percent. The alluvial sands are generally loose to medium dense, and range from non-silty to very silty or clayey (i.e., up to about 50 percent fines).

Peat was encountered in the wetlands in the west central part of Work Area West, around Runway Station 180+00. Both surficial and shallow buried peat deposits were encountered in this area. Buried deposits tend to be medium stiff to stiff, whereas the surficial peat exhibited consistencies in the very soft to soft range. Buried peat deposits were encountered at depths ranging from 3.5 to 9.5 feet and varied in thickness between 1.5 and 5.5 feet. Peat deposits near the ground surface varied in thickness between a few inches to about 2 feet.

Colluvium and Recessional Outwash. These soils generally consist of medium dense to dense, slightly silty to silty, slightly gravelly to gravelly sand.

Recessional outwash overlies the glacial till, or advance outwash where the glacial till has been eroded. Thickness of the colluvium and recessional deposits varies over the site, but is generally less than 20 feet. These deposits are generally intermittent or may be absent where alluvial materials are located and dense to very dense glacial till or advance outwash sand and gravel underlies the alluvium.

Glacial Till. Glacial till soils observed at the site consist of dense to very dense, slightly gravelly to gravelly, silty to very silty sand. In general, glacial till differs from the overlying recessional soils by having a higher silt content and much higher density. The top of the glacial soils is generally within 10 to 20 feet of the ground surface.

Glacial till is generally encountered near the surface of the west-facing slope on the east side of Work Area West. The glacial till is absent, downslope to the west, where the advance outwash soils are exposed. Springs and seeps occur along the western edge of the glacial till from perched water and interflow above the glacial till horizon as well as seepage through the underlying advance sand.

Advance Outwash Soils. Underlying the glacial till are advance outwash deposits consisting of dense to very dense, slightly silty, slightly gravelly to gravelly sand. In general, but not always, the advance outwash can typically be distinguished from the glacial till by lower silt content.

In some locations, the advance outwash deposits include hard silt and clay soils. Below a depth of about 30 feet, these soils have been reported to be part of the Lawton Silt and Clay or "Pre-Vashon Deposition." These hard soils may be laminated or contain planes of separation (partings). These deposits are typically reported to be relatively plastic and are often slickensided (i.e., showing evidence of previous failure planes).

Shallow groundwater flows through the fill, colluvium, and alluvial soils, including seepage perched on the glacial till and silty or clayey zones within the soils noted above. Seepage varies seasonally. Figure 4 presents an elevation contour map of the highest measured groundwater levels in this area. Groundwater conditions are discussed later in this report.

Work Area South 1

The majority of the work in this area will consist of removing the existing fill Stockpile No. 52 and excavating below existing ground surface as indicated in Hart Crowser (2001b). The subsurface conditions in this area from the ground surface downward are generally as follows:

Topsoil. Typically topsoil up to about 1 to 1.5 feet thick was reported in some but not all of the previous explorations in this area, but was typically not described in the logs.

Pre-Construction Fill. Existing fill soils consist of very loose to very dense, silty sand with asphalt, and occasional gravel and organic material. Pre-construction fill consisting of silty sand with gravel, included some asphalt concrete and organic debris in two borings, but the spacing of the previous explorations did not allow the extent of asphalt or other debris to be quantified. Boring AT97-B14 was drilled 5 feet from boring AT97-B13 after the initial boring was abandoned as a result of the presence of debris. The borings are located at the very north end of this work area, adjacent to an access road. The fill depth at this location was 22 feet. No other information is available on the pre-construction fill in this area.

Existing Stockpile No. 52. No specific information was collected on soils from Stockpile No. 52 for this report. Hart Crowser understands the stockpile generally consists of silty sand and gravel from excavation of the IWS Lagoon No. 3 project, and other recent construction work at the airport. We understand this material is likely to be Type 2 soil, which is somewhat moisture-sensitive and suitable for compaction in fair weather.

The remainder of the native soils in Work Area South 1 consists of a sequence of Recessional Outwash (slightly silty sand), Glacial Till (silty sand with occasional gravel), and Advanced Outwash (sand with minor amounts of silt and gravel). These soil units are generally similar to the same units located within Work Area West, as described above.

Work Area South 2

Construction in Work Area South 2 is anticipated to consist of removing existing stockpiled fill for use in the embankment and excavation of existing native soils to grade. The location of the existing Stockpiles Nos. 1 and 2 is shown on Figure 3, along with explorations and anticipated cut depths. The subsurface conditions in this area from the ground surface downward are generally as follows:

Topsoil. Topsoil ranging up to about a foot in thickness was reported in some but not all of the previous explorations in this area, and was typically not described in the logs.

Pre-Construction Fill Consisting of Very Loose to Very Dense, Non-Silty to Silty Sand. This layer was encountered in <u>all</u> of the explorations in this work area.

The fill thickness ranges from about 10 to 48 feet (some explorations were not advanced deep enough to encounter natural soil deposits). The fill material appears to be predominantly glacial till-type soils, likely obtained during grading of portions of the airfield in 1961 or 1962 (AGI 1999). Occasional organic materials, and wood and concrete debris were also encountered.

Existing Stockpile Nos. 1 and 2. Eight test pits were completed to assess soils in Stockpile Nos. 1 and 2 located within Work Area South 2. Conversations with the field inspector who observed stockpile construction for the Port of Seattle indicates that Stockpile No. 1 consists mostly of non-silty gravely sand, with a separate area of silty sand and gravel (glacial till-type soils), while Stockpile No. 2 consists of a mixture of silty sand and gravel (glacial till-type soils). Visual classification of the test pit soils and soil gradation test results concur with this description. Gradation tests for the non-silty portion of Stockpile No. 1 indicate this material would be suitable for use within the embankment underdrain.

Below the existing pre-construction fill, Glacial Till (silty sand with some gravel and cobbles) and Advance Outwash soils (non-silty to slightly silty sand) were encountered discontinuously in borings previously accomplished in Work Area South 2.

Groundwater

Groundwater is typically encountered in discontinuous zones perched in surficial fill soils and colluvium above the glacial till, and within the alluvial and advance outwash soils. The alluvium and advance outwash, also known as the Shallow Regional Aquifer, discharges to Miller and Des Moines Creeks, and via underflow to Puget Sound and the Green River valley (AGI 1996). The following sections summarize water level data and hydraulic conductivity data collected in the three Phase 5 Work Areas. Water levels observed in open borings at the time of drilling (ATD) and seepage observed in test pits are shown in Table 1 and on the exploration logs in Appendix A of the Phase 5 Data Report (Hart Crowser, 2001b).

Work Area West

Elevation contours for the highest measured groundwater levels are shown on Figure 4. We recommend these elevations be used for design of dewatering and other construction planning.

Typically up to about 3 feet of seasonal fluctuation have been observed in wells close to Miller Creek, with less groundwater fluctuation in wells located in upland areas east of the creek.

Dewatering will be required during excavation of unsuitable subgrade soils, to enable removal of soft or loose soils within the designated area, and backfill with compacted fill. Magnitude and rate of flow will vary locally due to changes in gradation and density of the soils.

Seepage and wet soils are typically observed at the surface in wetlands in some areas where fill placement or subgrade improvement (overexcavation and replacement) is anticipated. Artesian conditions were observed in two wells (AT94A-B3 and AT96-B4), east of the former 12th Avenue South alignment. Artesian pressures are likely sustained by recharge occurring in higher elevation areas of the existing airport area to the east.

Hydraulic Conductivity Testing

Slug testing was performed in four wells (HC99-B37 through HC99-B40) within or adjacent to Work Area West. The mean hydraulic conductivity was 1.1×10^{-4} cm/sec.

Work Area South 1

Water level data from wells AT97-B8 and AT97-B14 are presented in Table 3 of Hart Crowser 2001b. AGI and Hart Crowser advanced several other explorations (borings and test pits) in this area to depths ranging from 8 to 19.5 feet. Only a few of these explorations encountered groundwater. At the time of drilling/excavation, groundwater was encountered at elevations ranging from 356 to 367.5 feet, generally at least 8 feet below the proposed ground surface. Bottom elevations of the explorations that did not encounter groundwater were generally at least 7 feet below the final proposed ground surface.

Based on available data, cuts within most of Work Area South 1 will likely not encounter groundwater. However, groundwater might be encountered near the wetland area near boring/well AT97-B8, where high water levels measured in this well are near the final proposed ground surface elevation. Note that water levels vary with time and may rise above elevations reported herein.

Work Area South 2

There are no monitoring wells located in Work Area South 2. Therefore, no long-term water level readings are available for this area.

Several borings in this area were advanced to depths ranging from 14.5 to 34.5 feet. Only the deepest boring (AT97-B18) encountered groundwater at the time of drilling at elevation 329 feet. Bottom of boring elevations range from 301 to

359.5 feet and are generally below proposed final ground surface elevations. Water level observations at the time of drilling may not accurately represent water table conditions and may vary over time.

Test pit AT94b-TP16 encountered water at elevation 363 feet, approximately 3 feet above the proposed final ground surface elevation at this location. Based on observation in the other explorations in this area, this seepage probably represents a local perched water zone of limited extent.

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 for Phase 4 embankment construction (Hart Crowser 2000b) are applicable to the proposed Phase 5. These recommendations are reiterated and/or modified herein.

Hart Crowser provides recommendations for construction in the following areas:

- Embankment site preparation;
- Subgrade improvement;
- Embankment construction; and
- TESC Pond A construction.

Hart Crowser's recommendations for modification of TESC Ponds D and G, and for development of Borrow Area 4, are presented Hart Crowser (2001c and 2001d, respectively).

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, and/or local overexcavation and replacement, of unsuitable soils; and (5) abandonment of wells and buried pipelines by grouting. Note that item (4) is separate and distinct from the subgrade improvement located to the west of the Phase 5 embankment fill area

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; and

- Rake and remove loose organic debris resulting from clearing and mowing activities. Grubbing to remove roots and stumps, 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 5 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. We recommend there be some provision in the contract documents 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 an 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 construction 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 the future airfield pavement will be located above part of the Phase 5 fill. In our opinion, the risk of future settlements resulting from organic decomposition causing potential long-term pavement damage is probably quite 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: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 treated with one of the following alternatives:

1) Leave the soft soils in-place and fill over with the embankment drain layer, provided the subgrade is sufficiently firm that satisfactory compaction of the underdrain can be achieved;

2) Excavate and replace the soft soils with compacted drainage layer material where needed to achieve firm subgrade support; or

3) The subgrade support can be improved by placement of quarry spalls followed by a geotextile fabric below the underdrain.

The first 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 simply 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. Our previous analysis for Phase 4 indicated a woven geotextile such as the product Mirafi Filterweave 700 (previously specified for this purpose) is satisfactory.

Depending on condition of the subgrade, it may be necessary to increase the thickness of the first lift of underdrain 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

Occurrence of Problematic Subgrade Conditions Impacts Permanent Slope Stability

While most of the surficial soils and all of the underlying glacially overridden soils below the embankment will provide good support, there are extensive areas along the perimeter of the embankment and MSE walls where near-surface soils will not provide good embankment support. These soils include compressible clays and silts, peat, and liquefiable granular soils. A more complete summary of these potentially problematic subgrade conditions is provided in other reports (see Hart Crowser, 2000a). The recommended ground improvement alternative is to overexcavate the problematic soils and replace them with very densely compacted structural fill.

Anticipated limits of subgrade improvement in Work Area West are shown on Figure 8. Hart Crowser has accomplished an extensive assessment of available soils information to identify location of problematic conditions and to verify adequacy of the proposed subgrade improvements. Stability analyses used to define the limits of subgrade improvement are discussed in Appendix A. Some of these analyses are ongoing at the time of preparing this report as a draft, and limits of anticipated subgrade improvement could vary as a result.

Removal and replacement of soft or loose soils will involve dewatering, excavation with temporary cut slopes, use of a sheet pile barrier to seepage near Miller Creek, and backfill with select fill material.

Dewatering

Hart Crowser recommends that dewatering be accomplished by well points installed continuously around the perimeter of the subgrade improvement area in Work Area West. The perimeter well point system should be supplemented as needed by interior well points or trenches and sumps to completely remove groundwater from the excavation.

We recommend that the dewatering system be designed by a specialty-dewatering contractor with at least 10 years experience in this area. The Contractor should be required to provide a dewatering system design submittal to the Port for acceptance, prior to installation. We also recommend monitoring effectiveness of the dewatering system as the work proceeds. Hart Crowser is submitting recommended draft specifications to HNTB.

We recommend that the specifications require that a perimeter well point system be used so that bidders do not assume that groundwater can be controlled solely with trenches and sumps or pumped wells. Interception of groundwater with a continuous well point system is recommended to prevent seepage from the temporary cut slope face and enable these slopes to be cut as described below. Also, dewatering with well points in advance of the excavation would reduce soil moisture and improve suitability for reuse of the excavated granular soil as embankment fill (the excavated soil that is predominantly silt or clay and peat, would still need to be disposed of).

In our opinion, the entire subgrade improvement area would be most appropriately dewatered using vacuum well points, with the possible exception of a small portion at the northern end near Section 186+20, where pumped wells could be used in predominantly sandy soils (Hart Crowser, 2000a). The total length of vacuum headers and installed well points would be around 3,000 linear feet. Well points typically need to be installed at 10-foot spacing to be effective in dewatering these materials. Well points can be installed by jetting through the surficial soils to depths as needed, up to about 20 to 25 feet.

Hart Crowser estimates that dewatering flow rate would probably not exceed about 80 gallons per minute (gpm). Individual well points are anticipated to produce up to 1 to 2 gpm, but maybe much less. Inflow rate to trenches and sumps in the interior of the excavation is estimated to produce on the order of less than 1 gpm up to about 10 gpm per 100 linear feet of trench. Flow rates would increase further during storms if surface water runoff can also drain into the trenches.

Flow rates vary in direct proportion to soil permeability, which can vary by orders of magnitude in heterogeneous glacial soil deposits such as the soils at the Third Runway site. The estimates provided above are for general guidance only, and actual conditions could vary considerably during construction.

Temporary Excavation Slopes

Surficial soil in the subgrade improvement area should be excavated to competent, dense or hard, glacially deposited and overridden soils. Soils to be removed include loose to medium dense sands and silty sands and soft to medium stiff silts, clays, and peat. Depth of overexcavation in the areas will vary from about 10 to 20 feet, as shown on Figure 8.

Hart Crowser accomplished stability analyses for representative cross sections using the range of soil conditions and excavation depths anticipated. Assuming the groundwater will be temporarily drawn down below the base of the excavation to decrease hydrostatic pressure on the face of the cut slope, we concluded that temporary excavation slopes of 1.75H:1V slopes are feasible.

Our analyses used a target factor of safety of 1.3 for temporary construction conditions. Slopes of 1.75H:1V are near the "angle of repose" for some of the soils anticipated, and there may be areas where peat or other low strength soils are thicker than indicated by our explorations. The Contractor should be prepared to deal with some raveling or surficial sloughing.

Slope maintenance work may be required in some areas, and may become progressively worse the longer the excavation is left open. Slope maintenance will likely consist of removal of the failed material, but may include a need to place compacted fill berms to stop failure progression, depending on the overall rate of backfilling.

Temporary Sheet Pile Barrier

Based on discussions with HNTB, Hart Crowser recommends installation of a temporary sheet pile barrier along the west side of the subgrade improvement area that is closest to Miller Creek, in the area shown on Figure 8. The sheet pile barrier serves the following purposes: 1) reduce potential for slope instability, if any, from affecting the creek; and 2) improve effectiveness of the dewatering system, by reducing the amount of water from the creek that may enter the dewatering system.

Subsurface conditions used for analysis of the sheet pile barrier are shown on Figure 9. Recommendations for construction are summarized on Figure 10; we recommend that Figure 10 be reproduced in the construction drawings.

Hart Crowser analyzed lateral stability of the sheet pile barrier assuming the excavation and well point geometry shown on Figure 10. Lateral earth pressure and groundwater conditions are not anticipated to control design of the sheet piles. Driving resistance is not likely to be significant in the surficial soils, but will increase quickly when the tip of the sheet piles reaches the glacially overridden soils at the level of the bottom of the subgrade improvement excavation. Hart Crowser recommends a minimum embedment of 2 feet into the very dense soils, primarily to provide a seepage cutoff; as a practical matter, this depth could be reduced if exceptionally difficult driving is encountered. We estimated that minimum section modulus for the sheet piles would need to be around 60 cubic inches per foot, to drive the sheets into the underlying subgrade, but recommend that selection of the sheet pile section and driving equipment be left to the Contractor.

Verification of Subgrade Suitability

After the subgrade improvement excavation is completed, suitability of the exposed subgrade should be verified by an experienced inspector working for the Port. The Contractor should be required to avoid disturbance of the exposed subgrade. Hart Crowser has provided recommended specification language to HNTB to cover this.

Backfill of Subgrade Improvement Area

For the replacement fill material, we recommend using Group 1A or Group 1B material compacted to at least 98 percent of the maximum modified Proctor density (ASTM D 1557). The purpose of limiting the fines content (i.e., avoiding Type 2 soils) is to reduce potential for compaction problems, particularly if there is some excess moisture at the base of the excavation at the time backfilling begins.

The high degree of minimum compaction (98 percent) was proposed by the Embankment Technical Review Board as a convenient way to specify a density that would not allow the backfill to liquefy during a seismic event. Prevention of liquefaction likely occurs at around 80 percent "relative density." This relative density is different from the modified Proctor density. Relative density can be determined from a laboratory procedure after the fill source is determined. It may be that a comparison of 80 percent relative density with the maximum modified Proctor density could indicate that something less than 98 percent maximum modified Proctor is acceptable; however, this would need to be determined at the time of construction.

Although 98 percent of modified Proctor density is higher than we have previously specified for the Third Runway project, it is reasonable to expect that the Contractor can achieve this degree of compaction for the specified fill materials.

Embankment Construction

Embankment Underdrain

Hart Crowser recommends that an embankment underdrain, consisting of a minimum 3-foot-thick layer of free-draining fill (Type 1A) be placed under the 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. Recommendations for identifying areas where the underdrain could be omitted are discussed below.

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 our stability analysis showed potential for initiation of shallow instability associated with the toe of the fill. This condition could be aggravated by erosion of 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. This recommendation specifically covers the area where the fill crosses the former 12th Avenue South Right of Way, but could also be applicable elsewhere.

Where existing seepage occurs in peat areas that will be stabilized with quarry spalls, Hart Crowser recommends excavation of finger drains that can be backfilled with Type 1A 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 Group 1A 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 1A soils for the underdrain and addressing alternate materials through submittal review in the event this is required.

Hart Crowser recognizes that underdrains are not typically used in construction of embankments for roadways and some other types of earth fill. Underdrains are commonly used in construction of earth dams and for backfill behind some retaining walls. Underdrains are appropriate to control identified or anticipated seepage problems. However, in the absence of identified or anticipated seepage problems, our opinion is that it would be reasonable to omit the underdrain in areas of the Third Runway Embankment that meet the following criteria:

1. Maximum fill height not more than 50 feet;

2. Average 2H:1V embankment slope (average slope including benches, if used);

3. Use of stormwater control swales or other improvements on the top of slope to avoid ponding of stormwater;

4. Absence of any existing seepage or topographic swales below the fill that may preferentially channel seepage due to infiltration through the embankment; and

5. Absence of any upslope embankment areas that have already been constructed, where the previously constructed fill has an underdrain.

While the 50-foot height criterion is somewhat arbitrary, it limits the magnitude of potential problem if instability should result, such as might occur if there is some build-up of pore pressures over time.

Where the above criteria are met except item 4, it may be possible to use a limited underdrain to convey seepage through the backfilled swale.

These recommended criteria do no apply to any area where MSE walls are used.

In our opinion, the use of the 20-foot-wide "shell" of Type 1B fill used on the outside of the embankment in Phase 4 to control erosion due to perched seepage within the fill is a good practice, and should be continued in the area where the underdrain is omitted.

Fill Materials

Hart Crowser recommends continuing to use the fill material gradation criteria used in the Phase 4 construction specifications.

Placement and Compaction of Embankment Fill

Hart Crowser has observed fill construction intermittently and reviewed field reports and test data provided by Terra Associates for the embankment construction from 1998 to date. We continue to believe that the construction is going generally as intended and that conscientious inspection and regular testing is the best way to assure conformance to the specifications.

Additional recommendations for field inspection are being considered and will be submitted when this draft report is completed.

Excess Pore Pressures Due to Construction

Two general types of excess pore pressure related to construction need to be considered as part of design. These are related to 1) silt and clay subgrade soils; and 2) embankment construction involving fine-grained fill soil that is wet of the optimum moisture content for compaction.

Excess Pore Pressures in Subgrade Soils. Potential embankment stability problems resulting from 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 resulting from daily fill placement with continuous pore pressure dissipation from consolidation. Rate of fill placement was adjusted to determine limiting values.

Compressible soils are anticipated near the center and north edge of the Phase 5 construction footprint. Soft and medium stiff to stiff silt, clay, and peat subgrade soils in the work areas of Phase 5 were evaluated to assess the potential for construction-induced pore pressures to reduce soil shear strength below acceptable values. Potential for occurrence of this problem below the temporary fill slopes was based on density and thickness of silts and clays, and proposed fill height.

A section along runway Station 179+50 was analyzed and was determined to be the "worst-case." This section typifies a limited area that includes soft to medium stiff silts and clays with a thickness of about 7 to 10 feet; below a proposed fill height of up to 113 feet. Based on preliminary discussions with HNTB, Hart Crowser analyzed this section with two different fill geometries:

A) For the toe of fill beginning at the east side of the former 12th Avenue South, we estimate minimum Factor of Safety is about 1.1 for a fill rate of 2 vertical feet per day.

B) For the toe of fill beginning about 150 feet west of former 12th Avenue South Hart Crowser also estimated the minimum factor of safety during construction would be about 1.1 for a fill placement rate of 2 feet per day, and about 1.25 for a fill placement rate of 1 foot per day.

Our analysis indicates that maximum excess pore pressure and risk of instability are likely to occur within the first 10 feet of filling, and will be relatively constant over the duration of fill placement in this area.

The above analysis relies on a very conservative interpretation of soil conditions. Available explorations indicate that this condition is likely very limited in extent, since the same thickness of low strength poorly drained soils is not persistent from one exploration to the next. In addition, thin sand layers (commonly observed in test pits in this area) would improve rate of consolidation of the clay and prevent excess pore pressure build-up. We recommended further test pits to assess this condition after the 404 Permit is granted. If the test pits confirm thickness of the clay soils inferred from the borings, mitigation to avoid instability could be consist of either limiting the rate of fill placement to about a foot per day or less, or local overexcavation and replacement of the soft clay.

Excess Pore Pressures in Embankment Fill. Review of field test results indicates problems with compaction below specified minimum density are infrequent, but that when they occur they are typically related to fill material that is wet of specified limits. In addition, observation of "pumping soils" suggests that moisture contents may need to be limited to avoid development of excess pore pressures that could lead to instability, as discussed below.

Hart Crowser is continuing to evaluate the potential need to change the maximum allowable moisture content for the Group 3 and Group 4 soils, to avoid potential risk of excess pore pressure development within the fill. As discussed with HNTB, this evaluation and recommendations will be completed after review of the 90 percent Phase 5 contract documents and further discussion with HNTB on the rate of filling anticipated for Type 2 soils.

Slope Face Treatment

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

Overbuilding 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.

We also concur with the practice implemented by HNTB of using Type 1B fill for the outer 20 feet of the embankment, to facilitate drainage and limit potential for piping of infiltration that may becomes perched on finer grained soil layers within the embankment.

TESC Pond A

Pond A is discussed in a prior Hart Crowser technical memorandum related to potential impacts on seepage to Miller Creek (Hart Crowser 2001a). To avoid the potential that pumping from the pond might remove groundwater from adjacent wetlands, Hart Crowser recommended that Pond A be surrounded by a sheet pile barrier extending down to the top of very dense underlying soils as shown on Figures 5 and 7.

Sheet Pile Barrier

As previously discussed, the purpose of this sheet pile barrier is to limit groundwater movement away from wetlands during cycling of the pond and, during the short term, to avoid any need for the subgrade improvement dewatering system to handle seepage from the pond.

In addition, the sheet pile barrier would limit any instability associated with sloughing of pond side walls. However, recommended minimum sheet pile embedment is not intended to assure stability of a cantilever sheet pile (i.e., with no soil support on the pond side), and some maintenance of the pond side slopes should be anticipated in the event sloughing does occur.

Hart Crowser makes the following recommendations for sheet pile construction:

- Install the perimeter French drain entirely around the proposed pond prior to any sheet pile installation. This will assure adequate access for construction on the west side of the pond without any wetland encroachment and avoid any interruption of groundwater seepage as the sheet piles are installed.

- Install sheet piles on the west, north, and south sides of the pond (i.e., the sides closest to Miller Creek) prior to excavation. This will enable the piles to protect the creek in the event there is any excavation sloughing during pond construction.

- Drive piles to refusal or at least 2 feet into the top of the glacial till soils. The Port's contract documents should state that "jetting" shall not be used to aid driving.

- Prior to construction, the Contractor should provide the Port with a submittal that describes pile driving equipment and sequence of construction. During construction, the Port should verify that minimum embedment criteria are met.

Figure 5 shows the area where the temporary sheet pile barrier from the subgrade improvements will abut the Pond A sheet pile barrier. In our opinion, it is unnecessary to have these two sections of pile connect, and it is unlikely that any significant quantity or rate of seepage will occur through or below the Pond A perimeter drain at that location.

Perimeter Drain

To avoid concerns about the sheet pile barrier having an adverse impact on base flow to the wetlands during the time Pond A is in use, Hart Crowser recommends installation of a perimeter French drain as shown on Figures 5 and 7.

Groundwater flow would be maintained around the sheet pile barrier by conventional French drain consisting of a gravel-filled trench with a perforated drain pipe located within the gravel. The gravel-filled trench provides for relatively uniform seepage into the French drain and from the French drain into the adjacent undisturbed soil. The pipe enables effective transmission of water around the sheet piled area with relatively little loss of head. A geotextile filter fabric around the gravel will prevent migration of fine soil particles and potential clogging that might otherwise diminish effectiveness over the one to two year operating life of the system. Dimensions and details of the system are shown on Figure 7.

The trench will collect shallow groundwater on the upstream (eastern) side of Pond A and convey it to the soils on the downstream (western) side of the pond. Flow can occur around both the southern and northern ends of the pond. Groundwater that seeps into the upgradient side of the drain will be available to reinfiltrate back into the shallow soils on the western side of Pond A, thus maintaining groundwater levels in the wetland.

USE OF THIS REPORT

Hart Crowser prepared this report for the exclusive use of HNTB Corporation and the Port of Seattle for specific application to the site and project discussed herein. 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 geotechnical explorations completed provide a reasonable basis for design and preparation of construction contract documents, including subgrade improvements. Note, however, that the explorations performed for this study reveal subsurface conditions only at discrete locations and that actual conditions in other locations could vary. Furthermore, the nature and extent of any such variations may not become evident until construction. If significant variations are observed during construction, Hart Crowser should be notified so that we may observe such conditions and modify or verify our conclusions and recommendations as needed.

Please call if you have any questions.

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AGI 1996. Appendix Q-A, Baseline Groundwater Study, Final Environmental Impact Statement, Proposed Master Plan Updates, Sea-Tac International Airport, SeaTac, WA. January 3, 1996.

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

Hart Crowser 2000b. DRAFT Geotechnical Engineering Report, Phase 4 Fill, Third Runway Embankment, Sea-Tac International Airport. December 4, 2000.

Hart Crowser 2001a. Memorandum: Avoidance of Wetland Impacts, Temporary Stormwater Pond A, Sea-Tac Third Runway. June 18, 2001

Hart Crowser 2001b. Subsurface Conditions Data Report, Phase 5 Fill and Subgrade Improvement, Third Runway Embankment, Sea-Tac International Airport, SeaTac, Washington. September 2001.

Hart Crowser 2001c. DRAFT Geotechnical Engineering Report, Detention Ponds D and G, Third Runway Embankment, Sea-Tac International Airport. Draft report pending, September 2001.

Hart Crowser 2001d. DRAFT Basis of Design Summary Report, Site Development and Reclamation, Third Runway Borrow Areas 3 and 4, SeaTac Washington. Draft report pending, September 2001.

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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
Group 5	
6-inch	100
U.S. No. 200(1, 2)	0 to 6

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.

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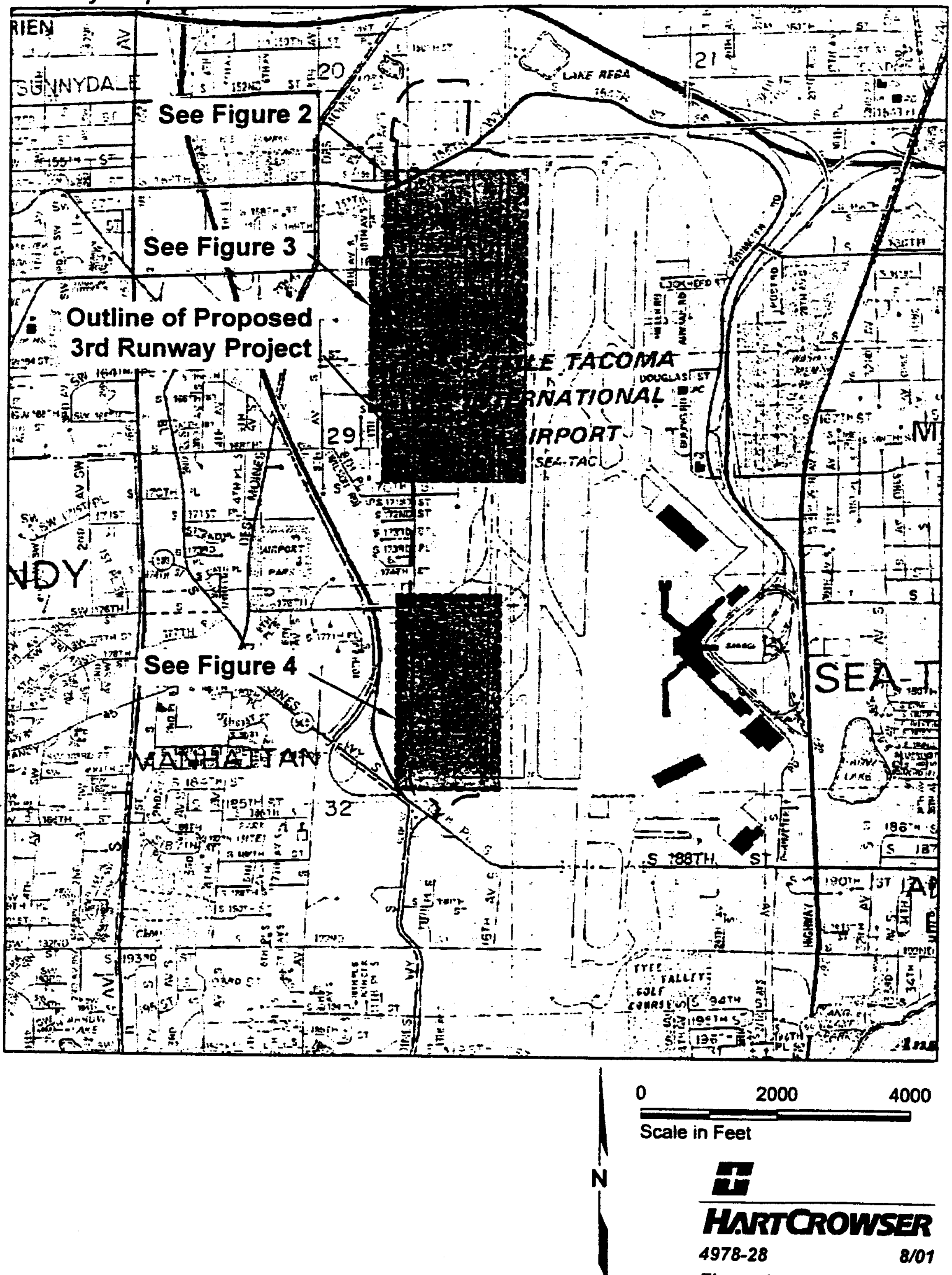
Vicinity Map



HARTCROWSER

4978-28 8/01

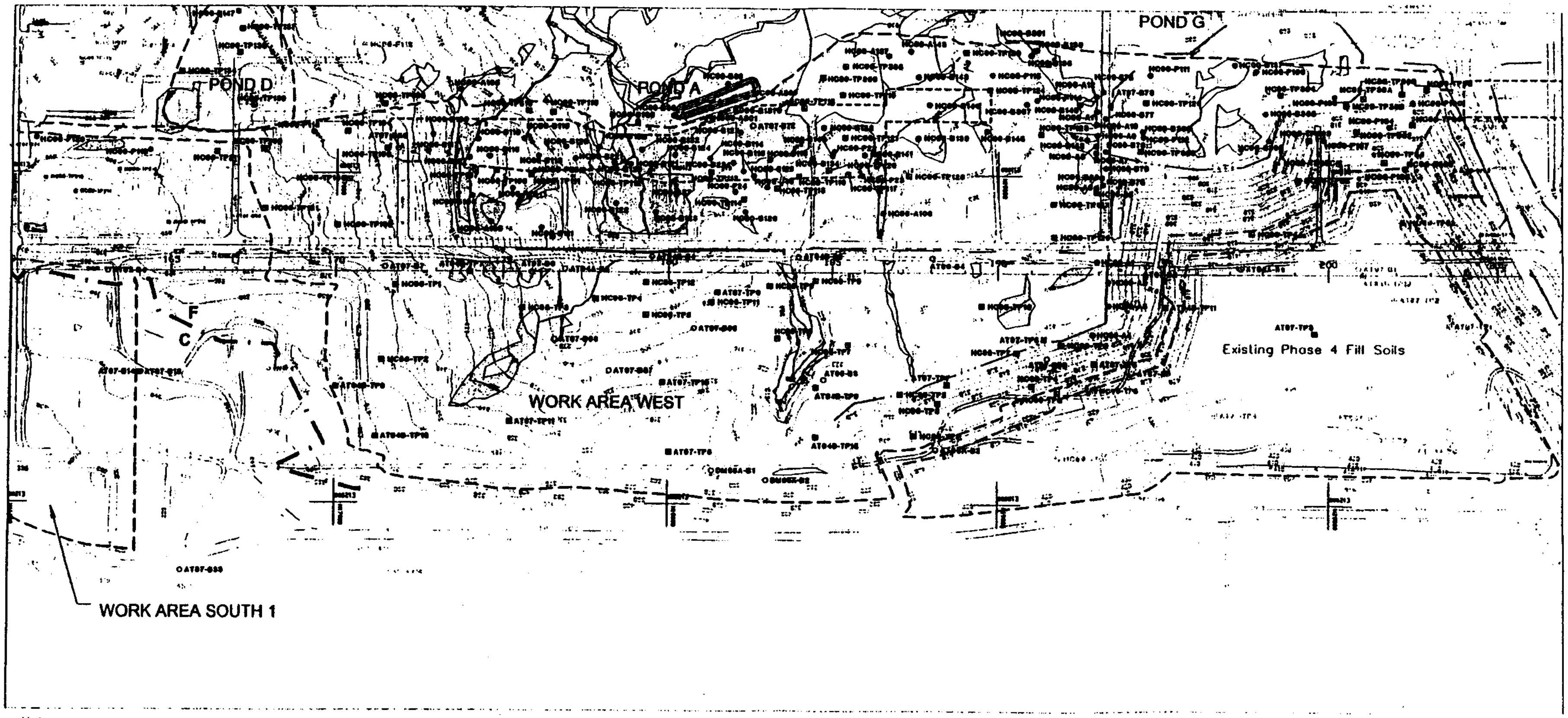
Figure 1

4978285O.dwg RC 1:2000 9/26/01 charlie.pc3

AR 052076

Site and Exploration Plan

Work Area West



Notes:
1) Base map prepared from drawing provided by HNTB. Wetlands delineations prepared from drawing provided by Parametrix.

2) Phase 5 Fill limits based on drawing provided by HNTB, June 2001.

F / C — Limit of On-Site Cut and Fill

HC00-TP0 Exploration Location and Designation

Anticipated Work Area Limit

Limit of Subgrade Improvement (See Figure 8)

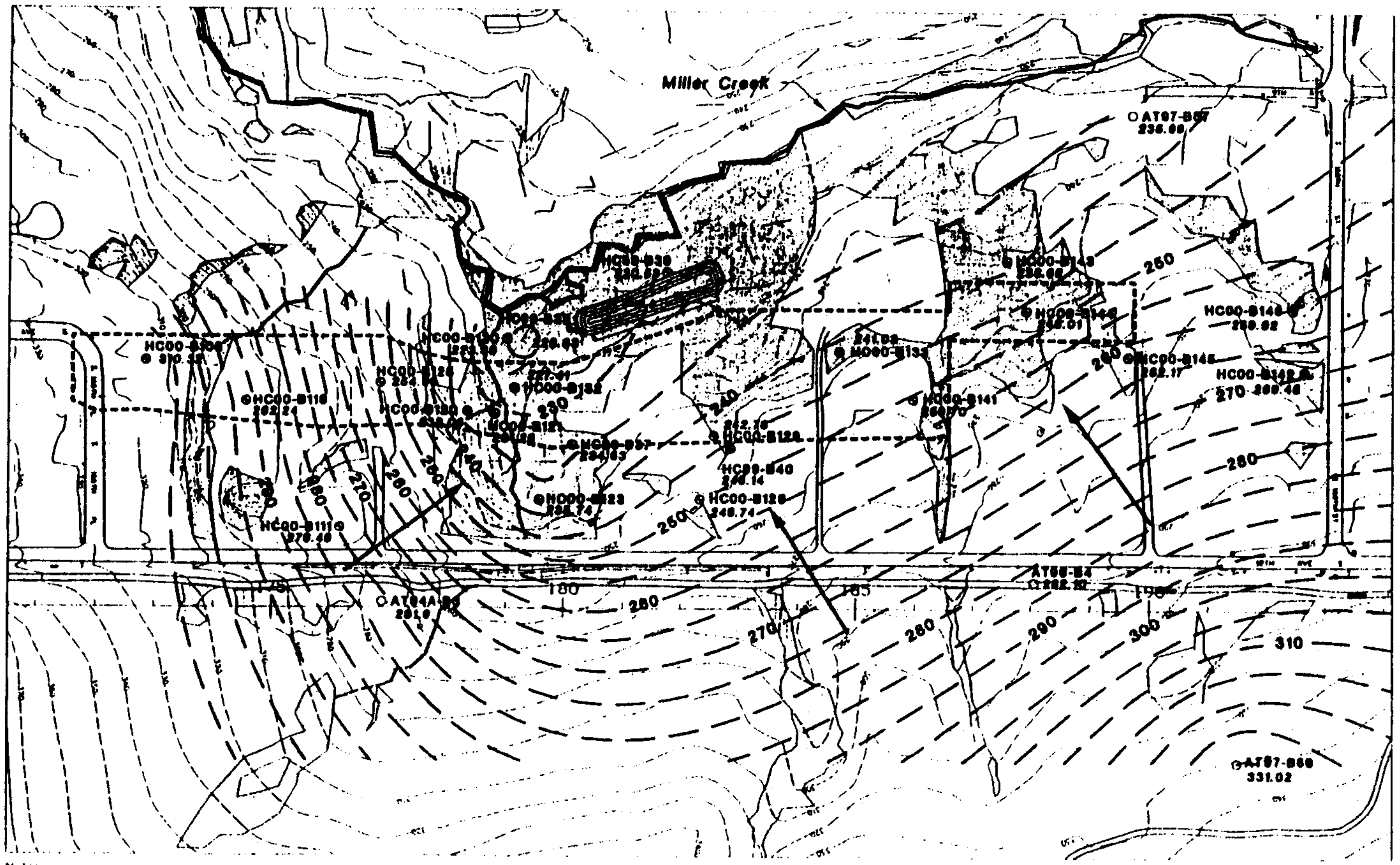
Wetland

Z

0 300 600
Scale in Feet

HARTCROWSER
J-4978-28 9/01
Figure 2

Groundwater Elevation Contour Map
Work Area West



Notes.
1) Base map prepared from drawing provided by HNTB entitled, "SlhBase dwg", dated August, 1998. Wetland locations based on drawing provided by Parametrix entitled, "w_022201 dwg," dated February 22, 2001

2) Groundwater elevation contours generated using Surfer 6.02 Surface Modeling Program based on highest water levels measured in monitoring wells.

— 250 —	Groundwater Elevation Contour in Feet
	Existing Elevation Contour in Feet
⊕HC99-B36	Monitoring Well Location and Number
281.9	Groundwater Elevation in Feet
⟵	Inferred Groundwater Flow Direction
175	Runway Stationing
	Wetland Location
-----------	Limit of Subgrade Improvement

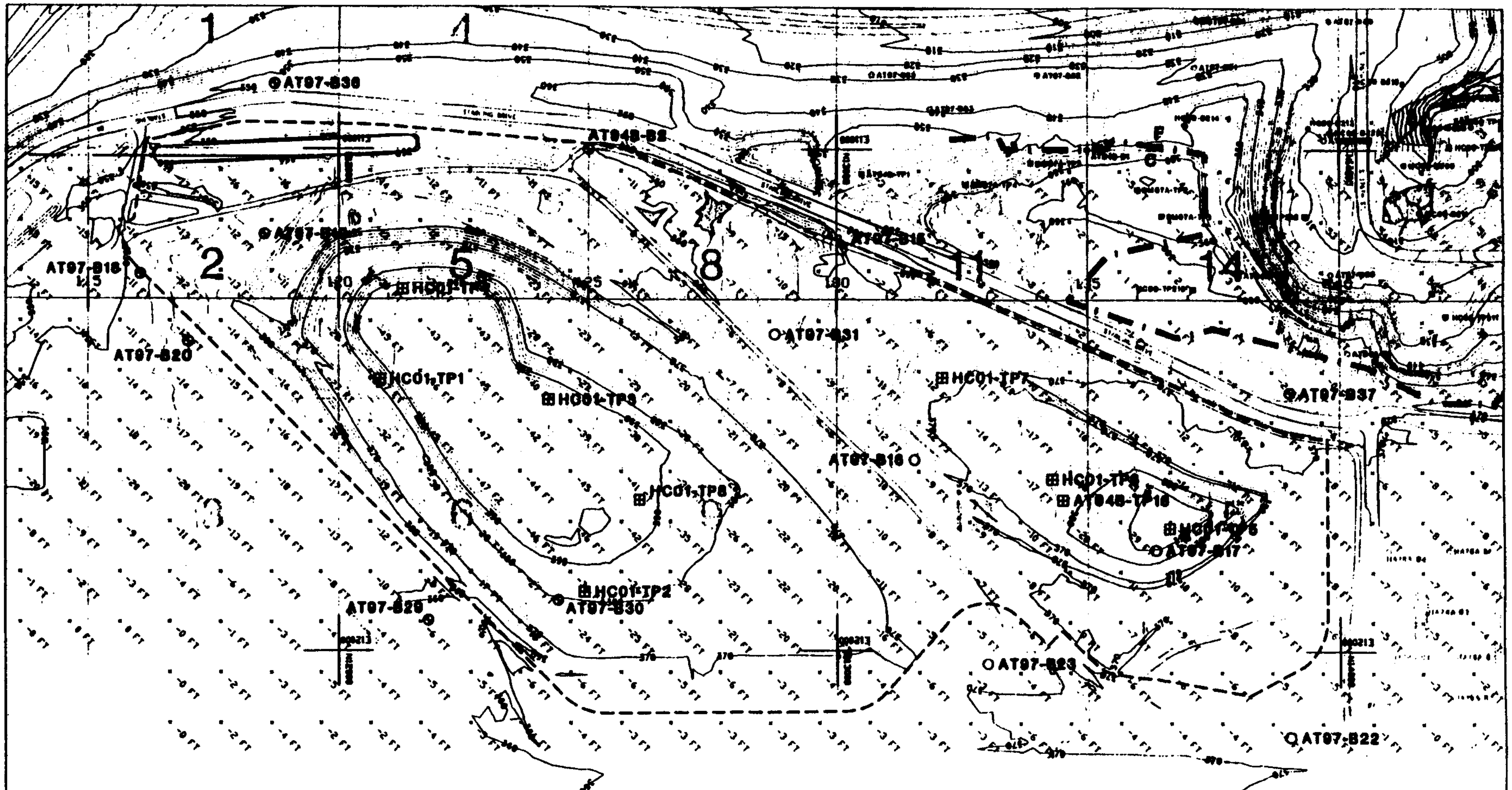
N

0 200 400
Scale in Feet

HARTCROWSER
J-4978-28 9/01
Figure 3

Site and Exploration Plan

Work Area South 2



Note:
1) Base map prepared from drawing provided by HNTB entitled "X_Topo.dwg", dated April, 2001.
2) Negative number at node within work area limits indicates approximate depth of cut and positive number indicates approximate height of fill.

See reference list for explorations not included in this report.

F / C — Limit of On-Site Cut and Fill

HC99-B80 ⊕ Exploration Location and Designation

Anticipated Work Area Limit

• -3 FT Anticipated Cut Depth at Spot Location

N

0 200 400

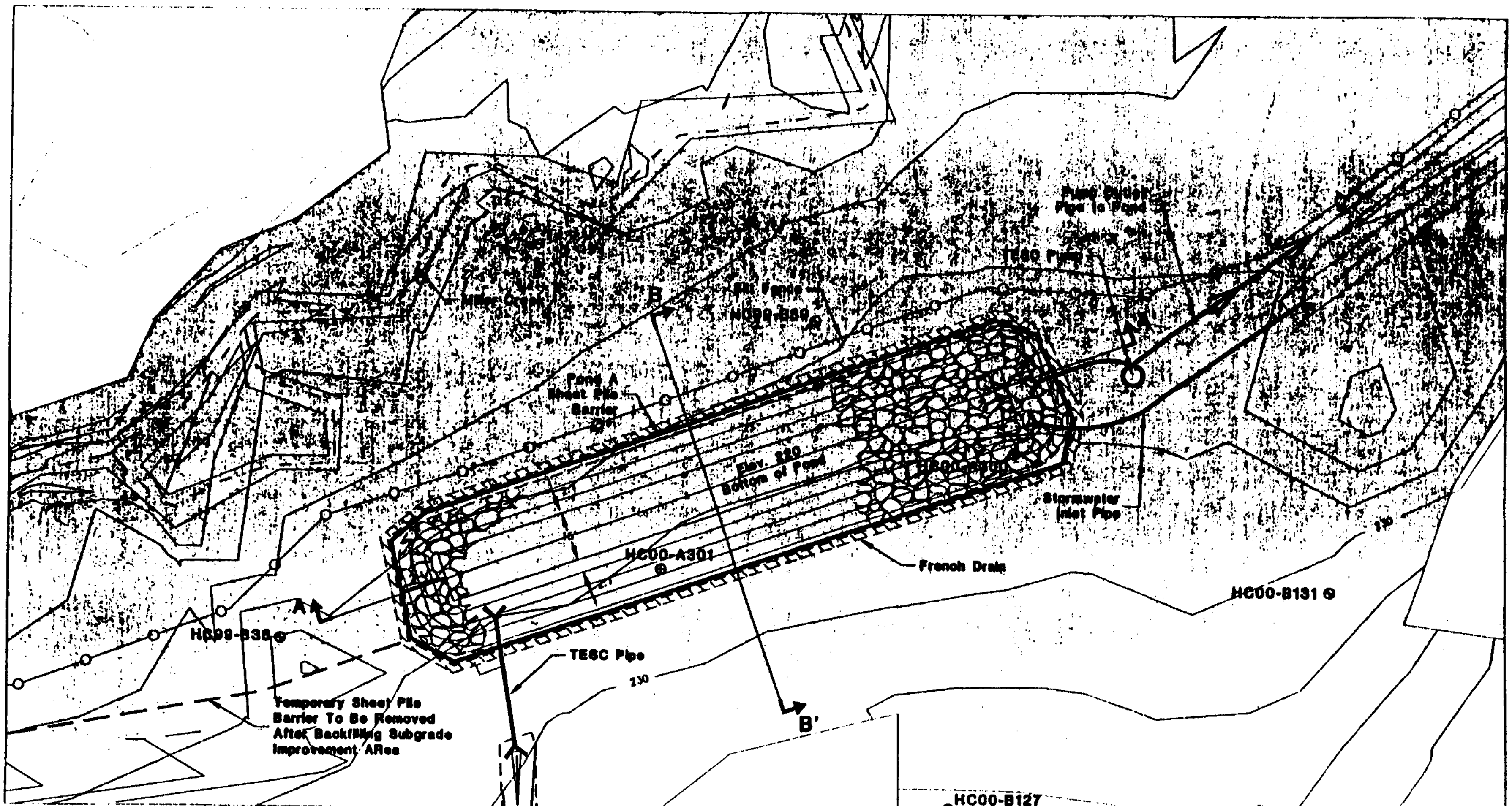
Scale in Feet

HARTCROWSER

J-4978-28 9/01

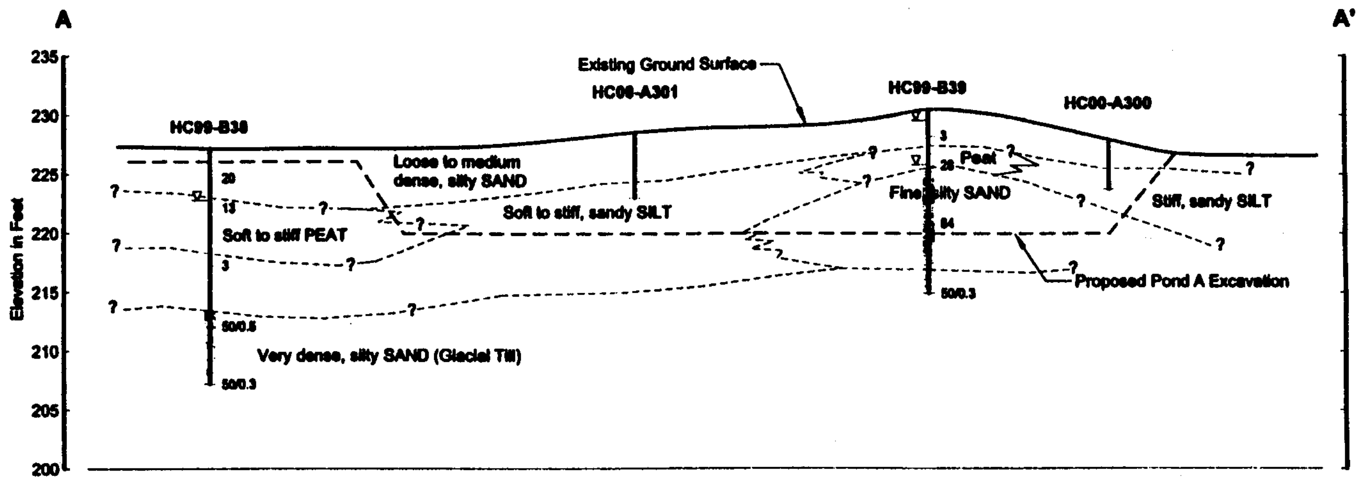
Figure 4

TESC Pond A Plan



Note: Base map prepared from drawing provided by HNTB entitled "X_topo.dwg," dated April 4, 2001. Wetland delineation prepared from drawing provided by Parametrix entitled "W_022201.dwg," dated February 22, 2001.

TESC Pond A
Cross Section A-A'



Note: Contacts between soil units are based upon interpolation between borings and represent our interpretation of subsurface conditions based on currently available data.

HC00-A301 Exploration Number

Exploration Location

Groundwater Level

50/0.3 Standard Penetration Resistance in Blows per Foot

Monitoring Well Screen Section

------- Inferred Geologic Contact

Horizontal Scale in Feet
0 30 60
0 10 20
Vertical Scale in Feet
Vertical Exaggeration x 3

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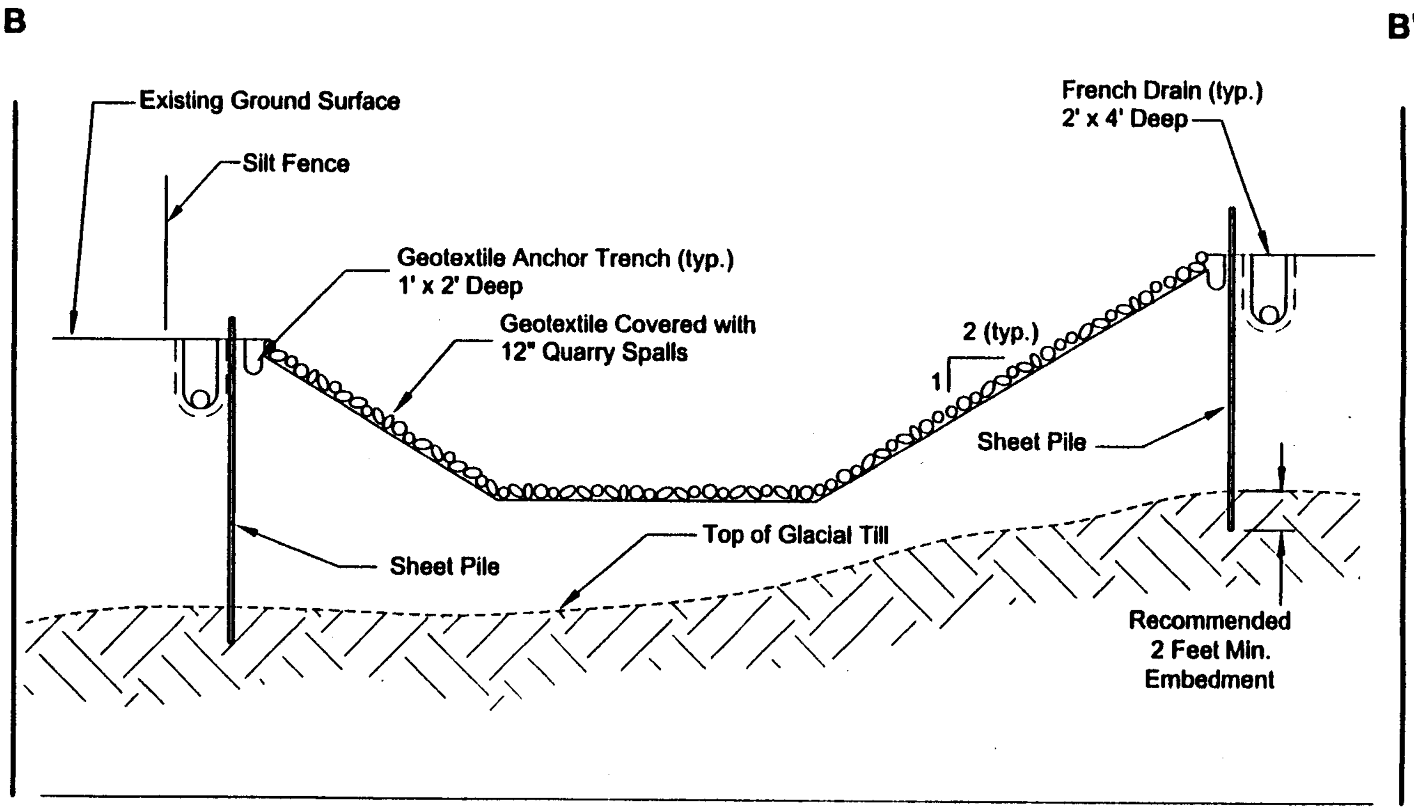
J-4978-28
Figure 6

DIN 9/26/01 1=1 charlie-8 pc2 49/82844

TESC Pond A
Cross Section B-B'

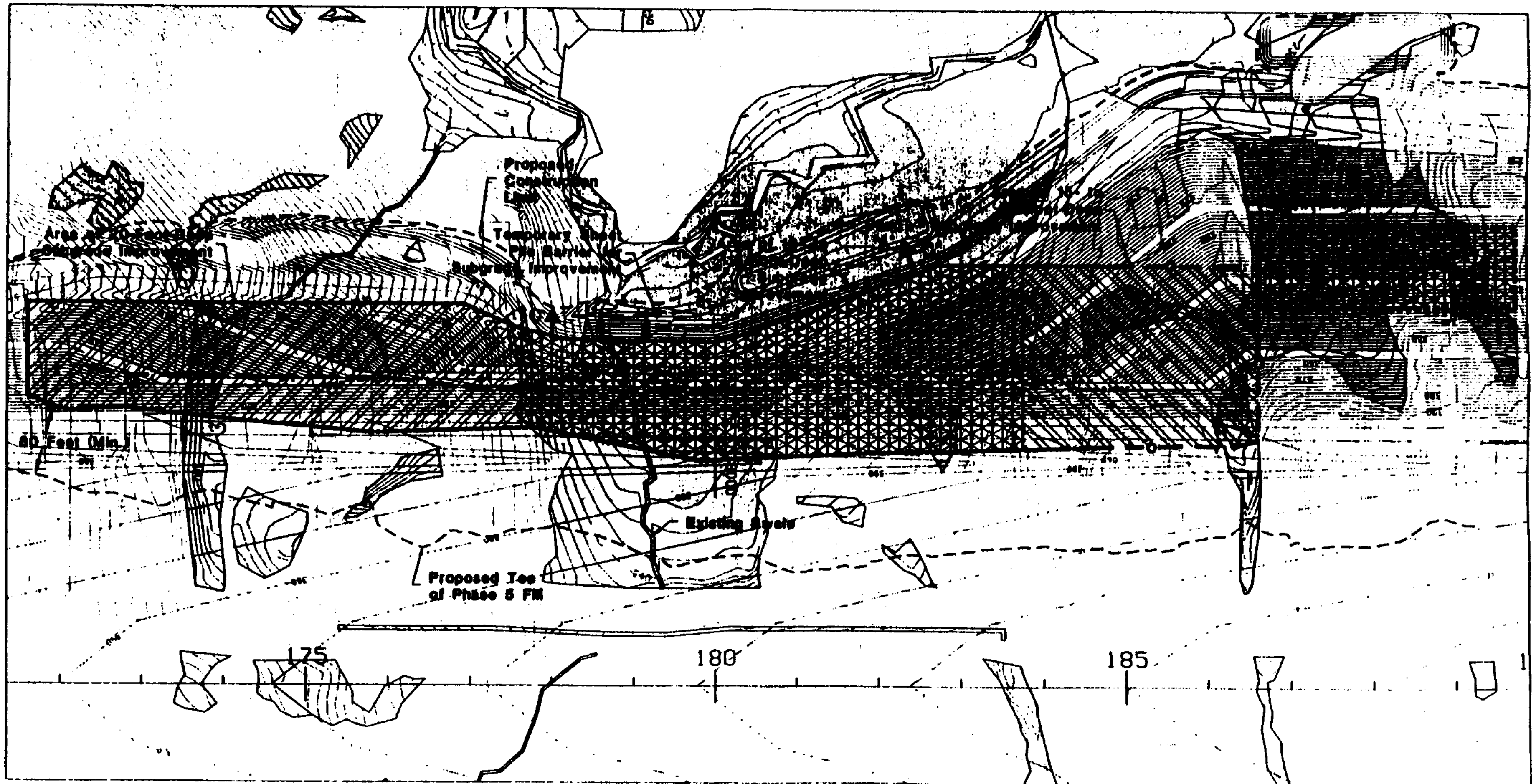


Note: Contacts between soil units are based upon interpolation between borings and represent our interpretation of subsurface conditions based on currently available data.

Not to Scale

HARTCROWSER
J-4978-28 9/01
Figure 7

Subgrade Improvement
Work Area West



Notes:
1) 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_020800.dwg", dated February 8, 2000.

2) Depth of subgrade improvements shown is typical, actual depth will vary.

Proposed Groundwater Control Components
- Well Points
- Drains and Sumps

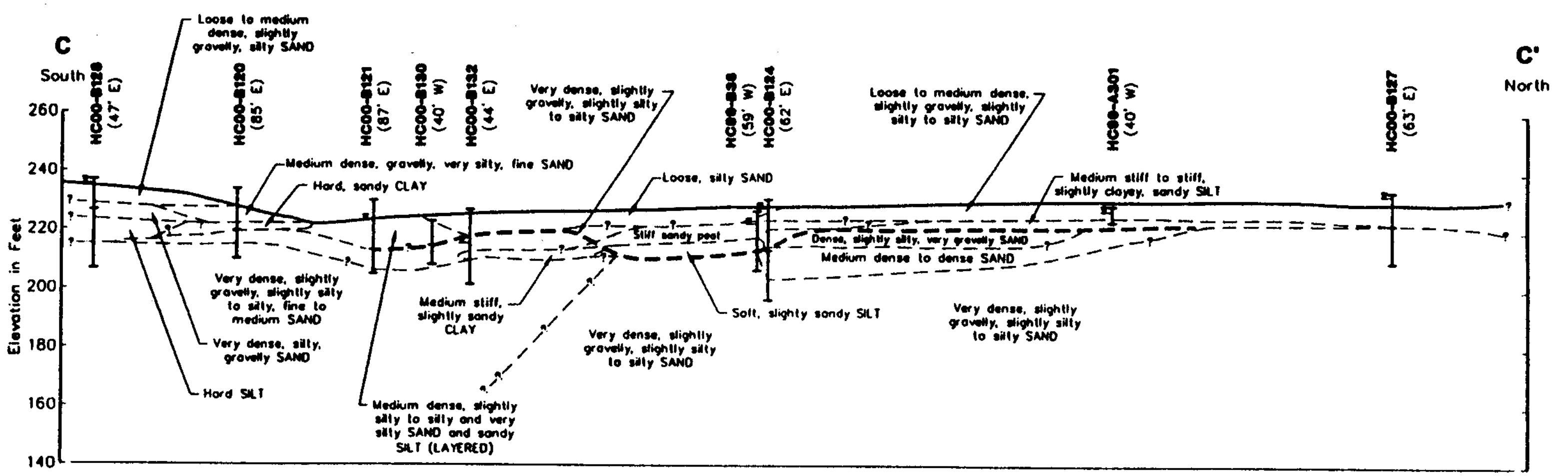
C C' Cross Section Location and Designation

N

0 120 240
Scale in Feet

HARTCROWSER
J-4978-28 9/01
Figure 8

Cross Section C-C'



Note: Contacts between soil units are based upon interpolation between borings and represent our interpretation of subsurface conditions based on currently available data.

HC00-B128 (2.18' W) — Exploration Number (Offset Distance and Direction)

ATD — Water Level

Geologic Contact (Approximate)

Bottom of Subgrade Improvement Zone (Approximate)

0 40 80
Scale in Feet

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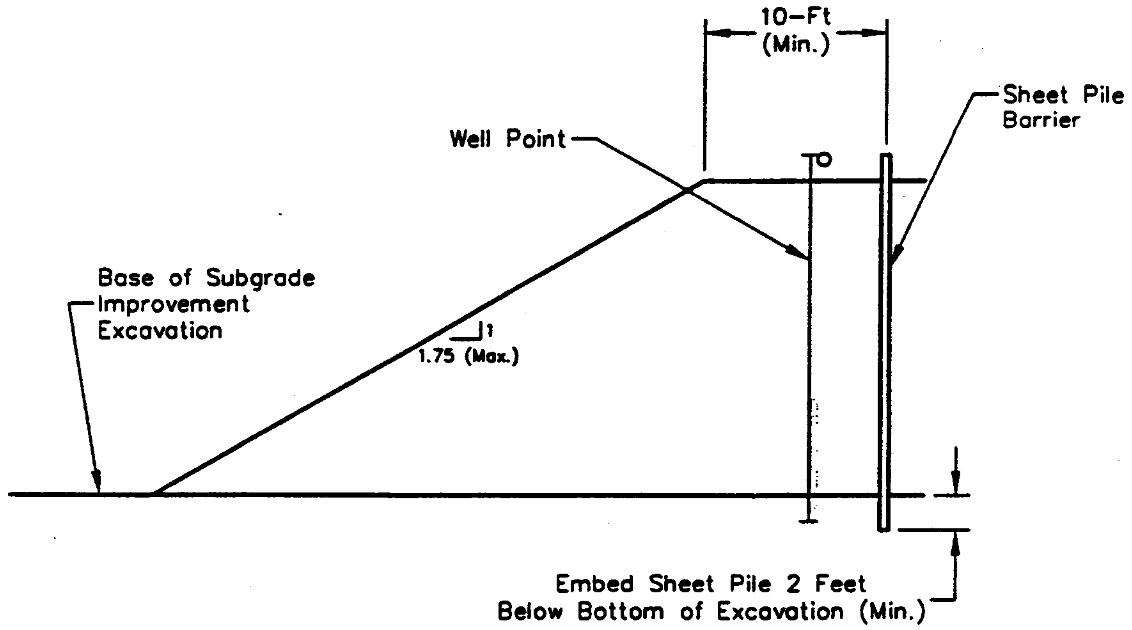
J-4978-28 9/01
Figure 9

AR 052084

Typical Sheet Pile Barrier



NOT TO SCALE

HARTCROWSER
J-4978-28 9/01
Figure 10

AR 052085

APPENDIX A
SLOPE STABILITY ANALYSES

APPENDIX A
SLOPE STABILITY ANALYSES

Introduction

This appendix provides a geotechnical description of the stability analyses used for design of the temporary embankment slopes within the current work area for the Third Runway. This discussion includes the design assumptions, methods of analyses, design criteria, and input soil parameters. The analyses consisted of:

- Limit equilibrium analyses accomplished with various computer codes that include the methods of analysis developed by Spencer, Bishop, and Janbu.
- Force and moment equilibrium analyses accomplished for sheet pile sections, as discussed in Hart Crowser (2001a).

The objectives of our stability analysis was to verify stability of the proposed temporary cuts for overexcavation and replacement of problematic soils, and temporary fill slopes.

Slope Stability Analyses

Temporary cut slope and temporary fill slope designs were based on slope stability. Limit equilibrium analyses of slope stability were accomplished for representative cross sections to verify the proposed slopes met target values for factor of safety. Various drainage conditions were assessed as discussed herein, in accordance with conventional geotechnical engineering practice.

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

- Verify that temporary cut slopes, constructed while overexcavating problematic soils, would be stable for duration of construction activity;
- Discuss areas where possible mitigation measures may be necessary to stabilize cut slopes; and
- Verify that temporary fill slopes, and the underlying subgrade, for the interior work areas would remain stable during construction.

Limit Equilibrium Slope Stability Analyses

Limit equilibrium analyses were accomplished with the computer program SLOPE/W.

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 (2000a and 2000b).

Table A-1 - Limit Equilibrium Analyses

Analysis Condition	Target FS
Undrained EOC	> 1.3
Partially drained EOC	> 1.3
Steady state	> 1.5

Hart Crowser analyzed slope cross sections developed in the previous phase of analyses, as noted in Hart Crowser (2000a), and generalized cross sections for the interior fill areas.

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 (silt/clay) (see Hart Crowser, 2001a and 2001b).

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 fill is placed so rapidly that the load is transferred to the pore water. The soft to medium stiff silt/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 worst-case section 179+50, where EOC stability was below the target value of factor of safety for undrained conditions, 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 build-up 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.

Other Limit Equilibrium Assumptions

Other assumptions used in our stability analyses include:

- Laterally extensive areas of peat would be removed, if within the subgrade improvement zones. Outside of these zones, surficial layers may be left in-place and 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.
- Soil shear strength of subgrade improvement consisting of overexcavation and replacement is represented by phi = 35 degrees.
- 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.

Summaries of unit weight and shear strength parameters for the analyses are listed in Table A-2.

Table A-2 - Summary of Input Soil Parameters

Soil Type	Unit Weight In pcf	Drained Strength		Undrained Strength Parameters	
		c' in psf	ϕ' in Degrees	S_u/σ_v' [a]	ϕ' in Degrees
Existing Subgrade Soils					
Loose to Medium Dense Sand	125	0	32	-	32
Medium Dense to Dense Sand	130	0	35	-	35
Dense to Very Dense Sand	135	0	37	-	37
Glacial Till	130	250	40	-	40
Soft Peat or Organic Silt [c]	110	0	7 to 15	0.30	0
Medium Stiff Silt/Clay [b]	115	0	30	0.30	0
Stiff to Hard Silt/Clay [b]	115	0	30	0.30	0

Soil Type	Unit Weight In pcf	Drained Strength		Undrained Strength Parameters	
		c' in psf	φ' in Degrees	$S_u/\sigma_v'^{(a)}$	φ' in Degrees
Post Construction Soils					
Embankment Fill	135	0	35	-	35
Drainage Blanket	140	0	37	-	37
Improved Subgrade	135	0	35	-	35

(a) Undrained strength ratios were used for fine-grained soils based on CU triaxial results and are a function of confining pressure (σ_v'). Additionally, a 30% increase in shear strength is allowed during transient loading conditions (pseudo-static).

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

(c) Drained friction angle for the peat was 15 degrees except at low confining pressure where a value of 7 degrees was used, see Hart Crowser (2001b).

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