Preliminary Stability and Settlement Analyses Subgrade Improvements MSE Wall Support Third Runway Project

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Prepared for
HNTB

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AR 045402

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PRELIMINARY STABILITY AND SETTLEMENT ANALYSES
SUBGRADE IMPROVEMENTS FOR MSE WALL SUPPORT
THIRD RUNWAY PROJECT

This report presents the results of Hart Crowser's preliminary global stability and settlement analyses for the proposed Mechanically Stabilized Earth (MSE) walls for the Third Runway project at Seattle Tacoma International Airport, Washington. Figure 1 shows the locations of the North Safety Area (NSA), West, and South MSE walls.

The purpose of the preliminary global stability discussed herein is to define the need for, and extent of, subgrade improvement below the proposed MSE walls. This report presents recommendations for subgrade improvement to achieve acceptable factors of safety. As part of this study, we also developed estimates of post-construction settlements with and without the subgrade improvements. The report includes conceptual design information to provide a basis for HNTB to evaluate subgrade improvement alternatives from the perspective of constructability, cost and schedule impacts.

The following section presents a summary of our findings and key recommendations. More detailed discussions are provided for each MSE wall in the subsequent sections that include:

- Subgrade Improvements Needed for Stability;
- Settlements With and Without Subgrade Improvement; and
- Recommended Subgrade Improvement Alternatives.

Detailed information regarding global stability design assumptions, criteria, analysis methods and input soil parameters is provided in Appendix A.

Appendix B presents schematic cross sections that generally illustrate the soil conditions used for analyses of each wall. These cross sections illustrate the variation in wall geometry and existing subgrade soils along the three walls. Groundwater conditions (not shown in Appendix B) vary along each section, as presented in detail in data reports prepared by Hart Crowser for each wall.

Appendix C provides a summary of our research on available subgrade improvement alternatives.

Finally, Appendix D presents additional details on dewatering systems that can be used to evaluate cost, schedule and constructability of the two recommended subgrade improvement alternatives.

SUMMARY

Hart Crowser analyzed 6 to 8 representative cross sections for each of the NSA, West, and South Walls for the global stability analyses. As illustrated in Appendix B, these cross sections vary in wall height, wall configuration, and subgrade soil conditions.

Subgrade Improvement will be Needed for all Three Walls

We found that improvement of subgrade soils will typically be needed to depths on the order of 10 to 25 feet below various parts of all three MSE walls to satisfy one or more of the following objectives:

- **Removal of low shear strength, highly compressible peat.** Peat and other organic-rich soils were found below portions of the NSA Wall (and adjacent right of way for the relocated 156th Street) and below part of the West Wall. In our opinion, the peat cannot be improved in place to provide acceptable strength and compressibility, and will therefore need to be removed.

- **Prevention of seismic liquefaction.** Loose to medium dense, slightly silty to non-silty sands that are susceptible to liquefaction, were encountered below all three walls. These soils can be removed and replaced with compacted fill or improved in place to prevent liquefaction.

- **Removal or improvement of soft to medium stiff silt and clay soils.** Fine-grained soils below part of the NSA Wall and West Wall were found to be problematic because of low shear strength and/or high compressibility, and/or because the rate of construction would need to be limited in these areas to avoid instability related to development of excess pore pressures. These soils can be removed and replaced with compacted fill or improved in place.

Figures 2, 3, and 4 show the areas where subgrade improvements are needed for the three walls.

Subgrade Improvement Alternatives

Hart Crowser evaluated eight general methods for ground improvement (as discussed in Appendix C) and found two alternatives that could provide the necessary improvement in strength, compressibility or prevention of excess pore pressures needed to provide stability at all three wall sites. These alternatives consist of a) over-excavation and replacement of the unsuitable native soil with compacted structural fill, and b) installation of stone columns.

Additional detail is provided later in this report to support selection of a preferred alternative for final design. Both alternatives have pros and cons, including the following:

- Overexcavation and replacement with structural fill would require dewatering at all three wall sites to maintain dry conditions and avoid disturbance of subgrade integrity, thus both alternatives would need to involve specialty contractors.
- Compared to conventional fill control, verification of subgrade improvement during installation of stone columns will require drilling or other *in situ* methods (such as the cone penetrometer or load tests) at the time of construction.
- Installation of stone columns may displace some groundwater, but can typically be accomplished with less risk of perception of off-site environmental impacts compared to extraction and disposal of groundwater by pumping.

We estimate the total dewatering flow rate for overexcavation and replacement with structural fill at each of the wall sites would be approximately as shown below:

Location	Dewatering Flow Rate	Likely Method
NSA Wall	30 to 120 gallons per minute	Combination of pumped wells, vacuum wellpoints and sumps.
West Wall	20 to 80 gallons per minute	Combination of vacuum wellpoints and sumps.
South Wall	25 to 100 gallons per minute	Combination of pumped wells and sumps

Figures 2, 3, and 4 show the areas where dewatering is anticipated to be needed to enable overexcavation.

Special Embankment Fill is Required Below the MSE Walls

In addition to subgrade improvement of the native soils that support the MSE walls, our stability analyses also indicated slopes of the "common embankment" fill alone will not provide adequate support for the MSE walls. The analyses indicate that the embankment fill slopes (2H:1V) which support the MSE walls will typically need to be constructed of high strength fill material (such as a compacted rock fill) or a reinforced soil fill, to provide the strength necessary for seismic stability. Our analyses have not defined the extent of this reinforced zone within the embankment fill, because it will depend on the geometry of the reinforcing zone behind the wall, embedment of the wall into the sloping fill, and selection of the seismic basis of design. We anticipate that Hart Crowser, HNTB and the wall designer will jointly develop requirements for the fill zone below the wall.

Our analyses did not show that any reinforcement would be required for the sloped embankment above the MSE walls, although the length of the reinforcement in the MSE wall will need to account for this surcharge. This finding is generally applicable to the South and West walls, as well as the NSA wall.

Following design of the wall reinforcing, and the reinforced zone below the walls (by others), we anticipate that Hart Crowser will verify global stability, compound stability and acceptable deformations for the final wall and slope geometry at all three walls.

SUBGRADE IMPROVEMENTS NEEDED FOR STABILITY

This section summarizes the extent of subgrade improvements needed for each wall, based on results of the preliminary global stability analyses. Refer to Appendices A and B for discussion of these analyses and to review the cross-sections. Figure 5 shows the conceptual extent of subgrade improvement for a representative wall section.

NSA Wall

Stability of the NSA Wall is influenced by the presence of peat, potential excess pore water pressures in silt and clay soils, and mitigation of liquefaction in sandy soils. We assumed that the peat will need to be removed and thus dewatering will be needed even if stone columns are used to accomplish the other subgrade improvement objectives for the NSA Wall.

The following discussion refers to cross sections identified by projection to the proposed road stationing along relocation of 156th Street, see Figure 2 and Appendix B.

Potential stability problems related to development of excess pore pressures depend on the rate of fill placement relative to the rate at which pore pressures reach equilibrium under the embankment load. As discussed in Appendix A, the excess pore-water pressure buildup is governed by the thickness of silt/clay, the permeability of soils, and the rate of fill placement.

Based on a nominal rate of vertical fill placement of 1.75 feet per day, the maximum excess pore-water pressures of subgrade soils below the NSA Wall area were calculated to be 615 psf for soils typical for cross sections from stations 101+20 to 107+19; and 2,700 psf for the soils at sections 110+47 and 114+10. These values correspond to the maximum silt/clay thickness of 5 and 8 feet, respectively. The maximum thickness of silt/clay was determined by taking into account the sand lenses disclosed by some of the CPT probes. Considering the length of the proposed NSA Wall it appears that these pore pressures are unlikely to be exceeded, based on a comment by a wall designer at our May 2 meeting, that a goal for MSE wall construction is typically up to about 1,500 square feet of wall face per day, but that less is often achieved.

As shown on Figure 2, the preliminary global stability results indicate that subgrade improvement would be needed in the majority of the NSA Wall area to achieve the required factors of safety. As noted below, subgrade improvement is needed for different reasons along different portions of the wall.

- The west half of the wall (typified by sections 105+20 to 107+19) will require overexcavation to remove surficial peat to about 10 feet below grade. This zone extends westerly from about 50 feet east (or south) of the wall, and includes peat removal below the new road as far south as Station 101+20.

- The middle portion of the wall (sections 105+20 to 110+47, i.e., overlapping the area described above) will require subgrade improvement to about 20 to 25 feet in depth for liquefaction mitigation. The width of the improved zone is approximately 50 feet on each side of the wall. These liquefiable sands are interbedded with low-strength silt and clay.

- The east flank of the wall (Section 114+10) will require approximately 10 feet of subgrade improvement to remove or improve surficial, liquefiable soils. The presence of thick, fine-grained soils below 10 feet will cause excessive post-construction settlements (discussed later in this report) and potentially the buildup of excess pore-water pressures during rapid placement of fill. Additional subgrade improvement to a depth of about 20 feet will likely be needed for settlement control and to avoid limiting the rate of fill placement to assure stability. This subgrade improvement would mitigate liquefiable soils in the upper 10 feet, and would increase shear strength and reduce settlements for the fine-grained soils below 10 feet. The width of the improved zone is approximately 50 feet on each side of the wall.

The presence of silt or clay interbedded with and/or below the liquefiable sands in the east portion of the wall (sections 110+47 and 114+10) would require limiting the rate of fill placement to allow dissipation of potential excess pore-water pressures during construction unless the proposed subgrade improvement extends to a depth of 20 feet. While the rate of wall construction (1,500 square feet per day) may not be constraining, the same limitation would also be needed for fill placement below the wall, which could constrain the contractor. Subgrade improvement from 10 to about 20 feet deep would either remove the silt/clay or shorten the drainage path to allow excess pore-water pressures to dissipate rapidly.

West Wall

Like the NSA Wall, stability of the West Wall is influenced by the presence of peat, liquefiable sands and the potential for excess pore pressures in silt and clay soils. Typically the presence of these soil constraints varied along the length of the wall as indicated by the section in Appendix B. These sections are identified by projection to the proposed runway stationing.

The maximum excess pore-water pressures of the subgrade soils in the West Wall area were calculated to be 615 psf for Section 184+50 and 2,700 psf for sections 173+65 to 180+05 (Figure 2). Similar to the NSA Wall, these values correspond to the maximum silt/clay thickness of 5 and 8 feet, respectively. Hydrostatic pore-water pressures were assumed for sections 181+90 to 186+20 because there is less than 1 foot of silt/clay in the subgrade.

As shown on Figure 3, the results of our analyses indicate that subgrade improvements are generally required throughout the West Wall area. Specifically:

- In the central part of the wall (sections 178+60 to 180+05), the wall height is generally in excess of about 135 feet. This high wall requires subgrade improvement to remove up to about 5 feet of peat and to mitigate liquefiable soils to depths of about 10 to 15 feet. The width of the improved zone extends from approximately 50 feet west of the toe of the wall to about 50 to 100 feet behind the top of the wall.

- The south portion of the wall (sections 173+65 and 176+40) requires subgrade improvement to remove up to about 5 feet of organic soils and mitigate liquefiable soils to depths of about 10 to 20 feet. The width of the improved zone extends from approximately 100 feet west of the toe of the wall to about 30 to 50 feet behind the top of the wall.

- The north portion of the wall (sections 181+90 and 183+08) requires subgrade improvement about 10 to 15 feet in depth to remove up to about

5 feet of organic soils and mitigate liquefiable soils. The improvement depth may gradually increase to about 20 feet toward the north end of the wall (sections 184+50 to 186+20). The width of the improved zone extends from approximately 150 feet west of the toe of the wall to about 50 feet behind the top of the wall.

The presence of thick, low-permeability silt or clay in the south portion of the wall (section 173+65 to 180+05) could require controlling the rate of fill placement to allow dissipation of pore-water pressure during construction. However, the proposed subgrade improvement would either remove the silt/clay or shorten the drainage path to allow excess pore-water pressures to dissipate rapidly, so controlling the fill placement rate would not be necessary.

South Wall

Unlike the other wall areas, the subgrade soils in the South Wall area are primarily granular. No peat and no excess pore-water pressure due to construction are anticipated, although soil liquefaction remains a significant issue. The sections discussed below are identified by projection to the proposed runway stationing.

Artesian groundwater was encountered at a depth of about 34 feet during drilling near Section 142+72, with the groundwater level about 2 feet above existing ground surface. This should not present any problem since Hart Crowser's stability analyses assumed hydrostatic conditions below the top of the embankment underdrain. However, potential slope erosion due to seepage west of the wall needs to be further evaluated and could affect recommended subgrade improvement in the South Wall area.

As shown on Figure 4, the results of our analyses indicate that subgrade improvement would be needed along a major portion of the South Wall to achieve the required factors of safety for each section. Specifically,

- The area between sections 141+78 and 147+50 will require subgrade improvement to remove or improve surficial, liquefiable soils to about 10 feet below grade. The width of the subgrade improvement zone generally extends from the crest of the embankment slope above the MSE wall to about 30 feet west from the toe of the wall.

Our analyses at Section 142+72 indicate that soil shear strength for the improved subgrade zone may need to be greater in this area compared with necessary subgrade improvements at other sections in order to meet the factor of safety criteria, (friction angle as high as 40° compared with 35° elsewhere).

This result appears to be the combined result of the ground slope in front of the wall and the presence of adversely inclined subgrade stratigraphy, and will need to be further evaluated during final design.

WALL SETTLEMENTS WITH AND WITHOUT SUBGRADE IMPROVEMENTS

We anticipate that settlements resulting from compression of fill soils in the embankment and behind the MSE walls will occur rapidly during the construction of embankments and walls. Post-construction settlements along the MSE wall profile primarily will be governed by the presence and thickness of silt or clay layers in the subgrade. The compressible silt or clay soils could be removed or reinforced, depending upon the type of subgrade improvement used in each section of the wall.

Based on the subgrade conditions along the wall profile, the laboratory consolidation test results, and the proposed ground improvement from the stability analyses discussed above, we provide preliminary settlement estimates for each of the NSA, West, and South walls as follows:

- **NSA Wall.** We estimate that the native subgrade could experience up to 12 inches of post-construction settlement with a maximum differential settlement of about 1/50. The recommended subgrade improvements would reduce the total post-construction settlement to a maximum of about 6 inches, with a maximum differential settlement of no greater than 1/200. This includes extending the depth of subgrade improvements to about 20 feet deep in the east flank (vicinity of sections 110+47 to 114+10).

- **West Wall.** We estimate that the native subgrade could experience up to 20 inches of post-construction settlement with a maximum differential settlement of about 1/50. The recommended subgrade improvements would remove or reinforce most of the compressible soils and would reduce the total settlement to about 6 inches maximum, with a maximum differential settlement of about 1/100 to 1/200.

- **South Wall.** Very limited fine-grained soils were encountered in our explorations along the South wall profile. Up to about 3 inches of settlement may be anticipated in the area between sections 141+78 and 142+72. Post-construction settlements in other areas along the South Wall profile would be smaller.

RECOMMENDED SUBGRADE IMPROVEMENT ALTERNATIVES

Appendix C presents the results of hart Crowser's literature review on the feasibility of various subgrade improvement techniques for this site. Our review concluded that overexcavation/backfill and stone columns installed by vibro-replacement are the two most feasible alternatives. For comparison purposes, we provide the following preliminary design recommendations for these alternatives.

Overexcavation and Replacement with Structural Fill

This alternative basically consists of removal of the unsuitable subgrade soils and replacement with compacted structural fill.

- Depending on the time of year when construction is accomplished, we anticipate the structural fill could typically be group 2, 1A or 1B soils compacted to at least 92 percent of maximum modified Proctor density. Group 3 and 4 soils are not recommended because they would tend to be less permeable overall than the predominantly sandy or interbedded sand, silt and clay soils they would replace.

- Once dewatering is accomplished, placement and compaction of the structural fill would be accomplished in much the same way as for other portions of the embankment.

One of the major considerations for overexcavation in the wall support areas is the need for temporary dewatering until backfilling is complete. Temporary construction measures such as dewatering are typically the responsibility of the contractor, with the contract documents providing the owner's expectations for performance and the conditions to be encountered. Based on our analysis of the soil and groundwater conditions, we conclude that dewatering would be accomplished with a combination of these of three techniques:

1) Drilled dewatering wells, installed in a curtain around the excavation to depressurize the subgrade and to intercept and remove potential groundwater inflow in advance of the excavation.

2) Drilled or jetted vacuum wellpoints installed with header pipes around the periphery of excavations.

3) Drainage trenches, with sumps, installed ahead of the main excavations.

In some areas, such as the NSA, dewatering and overexcavation would benefit from installation of sheet piling to limit the amount of water to be pumped or to provide ground support and limit the effect of the excavation slopes on adjacent wetlands.

Each of the methods listed above has merits and disadvantages that vary for different areas of the site, due to the wide range of subsurface conditions encountered. These considerations are discussed in Appendix D.

NSA Wall Support Zone

Figure 2 shows a conceptual dewatering system layout to enable over-excavation and replacement for the NSA Wall. This conceptual approach utilizes pumped wells, wellpoints, and trench drains with sumps because of the wide range in conditions in that area. We recommend using this approach to evaluate the feasibility of overexcavation and replacement although a dewatering contractor may try a simpler system first and then supplement it based on results obtained.

West Wall Support Zone

Figure 3 shows a conceptual dewatering system layout to enable over-excavation and replacement for the West Wall. This conceptual approach utilizes vacuum wellpoints, and trench drains with sumps, to handle the range in conditions anticipated in that area.

South Wall Support Zone

Figure 4 shows a conceptual dewatering system to enable over-excavation and replacement for the South Wall. This approach utilizes pumped wells and trench drains with sumps to handle the range in conditions anticipated in that area.

Use of Stone Columns for Ground Improvement

The vibro-replacement method of ground improvement ("stone columns") provides densification of native soils, increases shear strength and stiffness of the composite mass, and provides drainage a path for the dissipation of excess pore-water pressures.

As an alternative to overexcavation and backfill, the installation of stone columns would mitigate soil liquefaction. This would improve the subgrade, to satisfy global stability requirements, and would reduce post-construction settlements of the MSE walls. Due to the high fines content of some of the native subgrade soils, the vibro-replacement stone columns would be more effective than the sand-backfilled vibro-compaction technique in most of the NSA wall zone. Nonetheless, the less expensive vibro-compaction approach may be applicable in the South Wall area where the subgrade soils are mostly granular. We understand that mitigation of soil liquefaction at the South Wall with vibro-compaction could be accomplished using the same equipment as would be used for installation of stone columns by vibro-replacement at the West and NSA walls. Figure 7 illustrates the construction of stone columns for subgrade improvement.

Hart Crowser recommends that nominal 42-inch-diameter stone columns, with 8-foot center-to-center spacing and installed in a triangular pattern, be used as the basis for evaluating this alternative. This preliminary design will provide an area replacement ratio of about 17 percent, which we estimate will provide an acceptable strength improvement. Hart Crowser will produce a more detailed design if stone columns are selected as the preferred alternative.

These stone columns should be installed to the refusal (dense to very dense glacial soils) or about the depth as shown in Figures 2 and 3.

Stone columns are not expected to change the hydrologic behavior of the subgrade significantly. Although the stone columns will provide vertical drainage pathways, the presence of dense glacial soils (especially till) at the base of each column will prevent downward drainage. There will be some limited potential for upward drainage of any excess pore pressures into the embankment drainage layer, which will extend across the top of any areas improved using stone columns.

The gradation of the stone used in stone columns is typically designed to meet both the drainage and filter criteria, and is governed by the gradation of native soils. For preliminary cost estimate purposes, we recommend using AASHTO No. 57 aggregate for stone columns and drainage sand as per WSDOT 9-03.13 standards for vibro-compaction.

Stone columns should be installed using the dry, bottom-feed method. If the wet method is required in some areas to penetrate dense or hard soil layers, a containment berm/ditch system with sumps and sediment settling ponds/tanks should be provided to prevent discharge to the adjacent wetlands or other environmentally sensitive areas.

For the vibro-compaction stone column alternative at the South Wall, we preliminarily estimate that 42-inch-diameter sand columns with an 8-foot center-to-center spacing should be installed in a triangular pattern. This design will provide an area replacement ratio of about 17 percent. In practice, these stone columns will likely be installed to refusal (top of dense glacial soils), or about the depth as shown on Figure 4.

THE 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 preliminary 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 completed to date provide a reasonable basis for showing the proposed extent of subgrade improvements on the construction plans. As previously discussed, Hart Crowser recommends that subgrade conditions within the wetlands be further assessed with test trenches when access for such work is permitted. Note 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.

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Vicinity Map

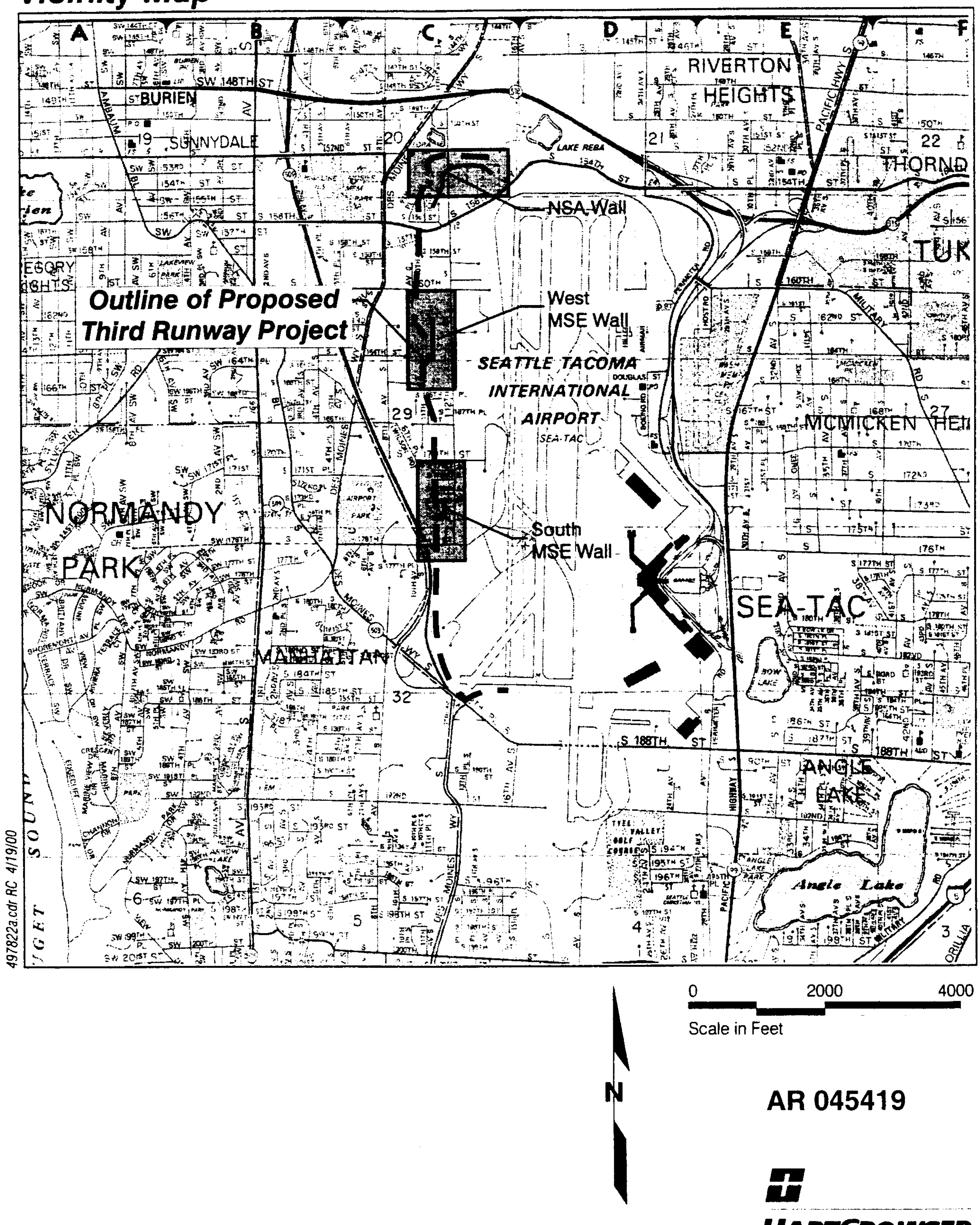

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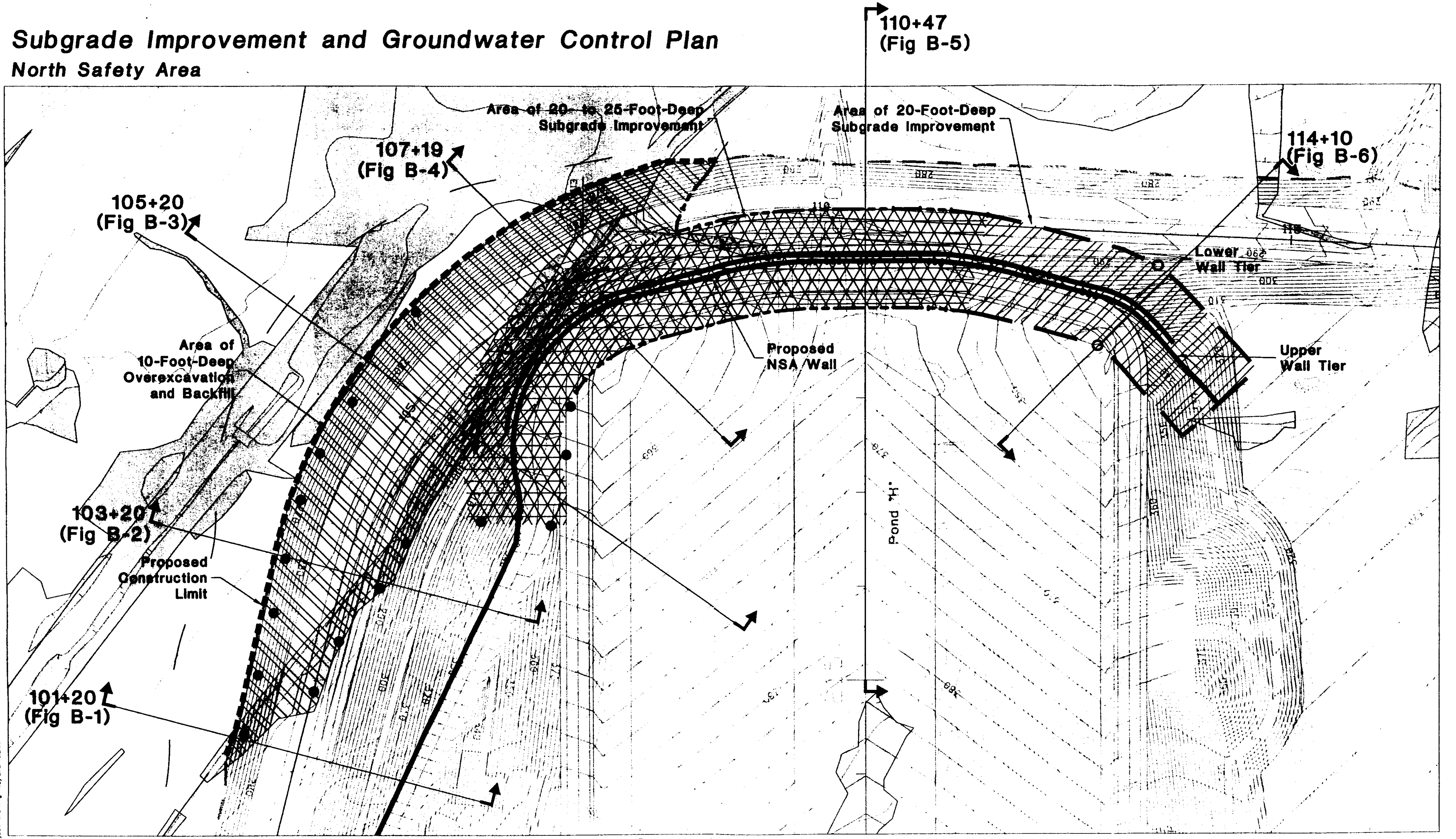
Figure 1

Subgrade Improvement and Groundwater Control Plan

North Safety 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_020800.dwg", dated February 8, 2000.

Proposed Groundwater Control Components

- Wells
- Wellpoints
- Drains and Sumps
- Sheet Pile

103+20 Cross Section Location and Designation

0 100 200
Scale in Feet

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Figure 2

DTN 6/21/00 1=100 (xref)see drawing file/charlie.pc2
4978209.dwg

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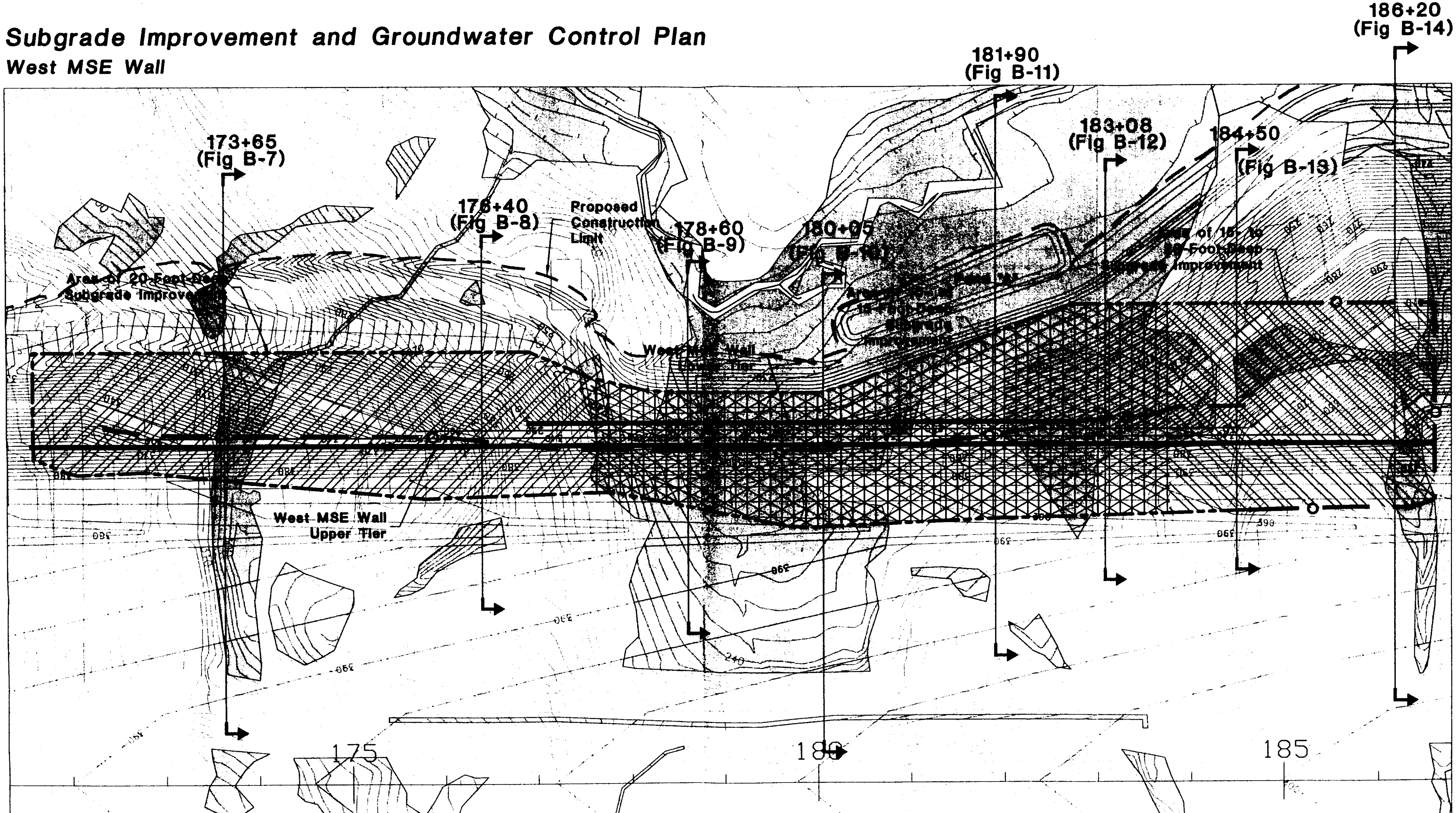
Subgrade Improvement and Groundwater Control Plan

West MSE Wall



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_020800.dwg", dated February 8, 2000.

Proposed Groundwater Control Components

Wellpoints

Drains and Sumps

180+05 Cross Section Location and Designation

N

0 100 200

Scale in Feet

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Figure 3

DTN 6/21/00 1=100 (xref)see drawing file/charlie.pc2
49782208.dwg

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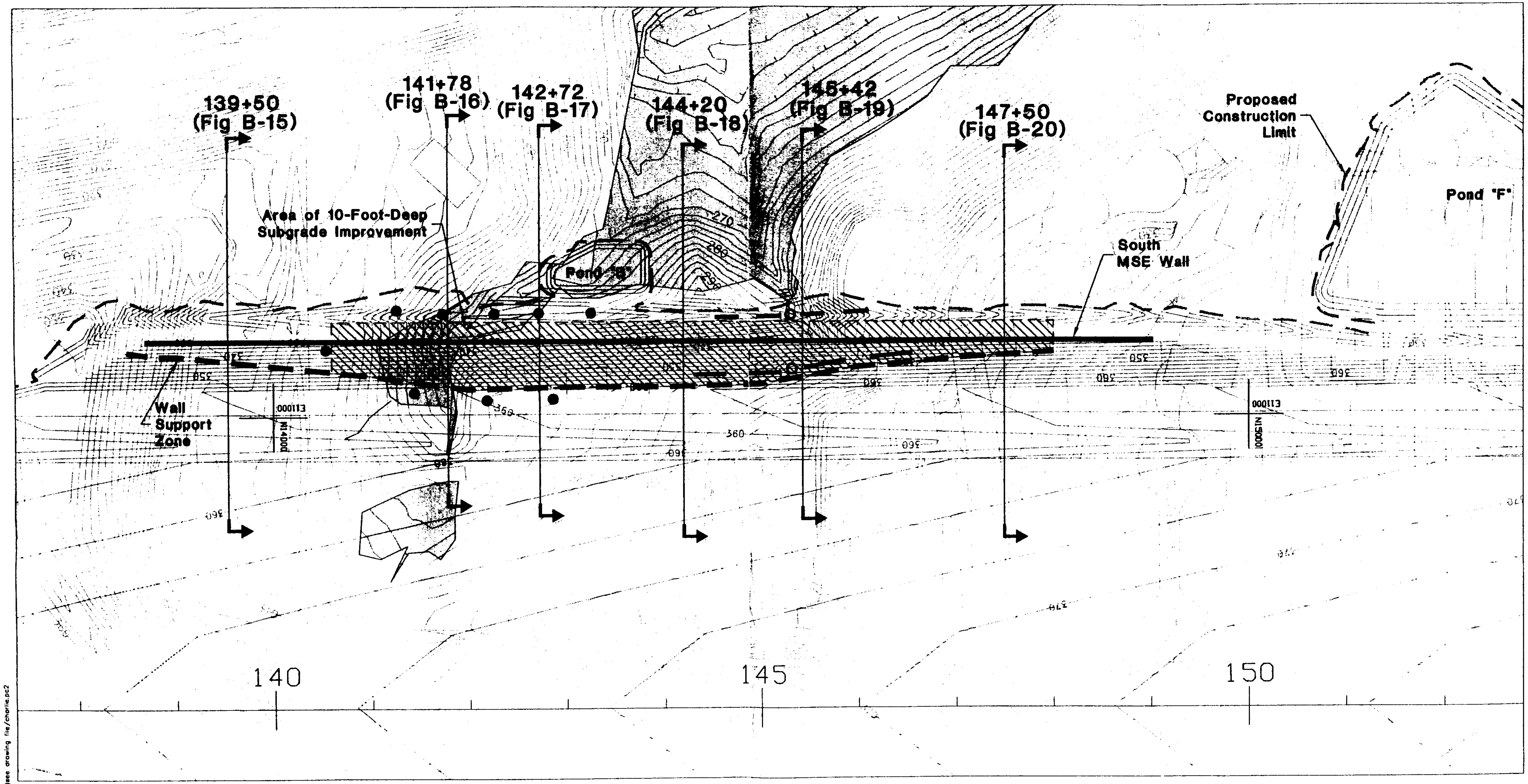
Subgrade Improvement and Groundwater Control Plan

South MSE Wall



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_020800.dwg", dated February 8, 2000.

Proposed Groundwater Control Components

- ● Wells
- —o— Drains and Sumps

145+42 — Cross Section Location and Designation

N

0 100 200

Scale in Feet

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Figure 4

DTN 6/21/00 1=100 (xref)see drawing file/charlie.pc2
4978207.dwg

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DTN 6/21/00 1=1 Woodsick.pc2 49782210.dwg

Extent of Subgrade Improvement to Mitigate Liquefaction for a West Wall Section

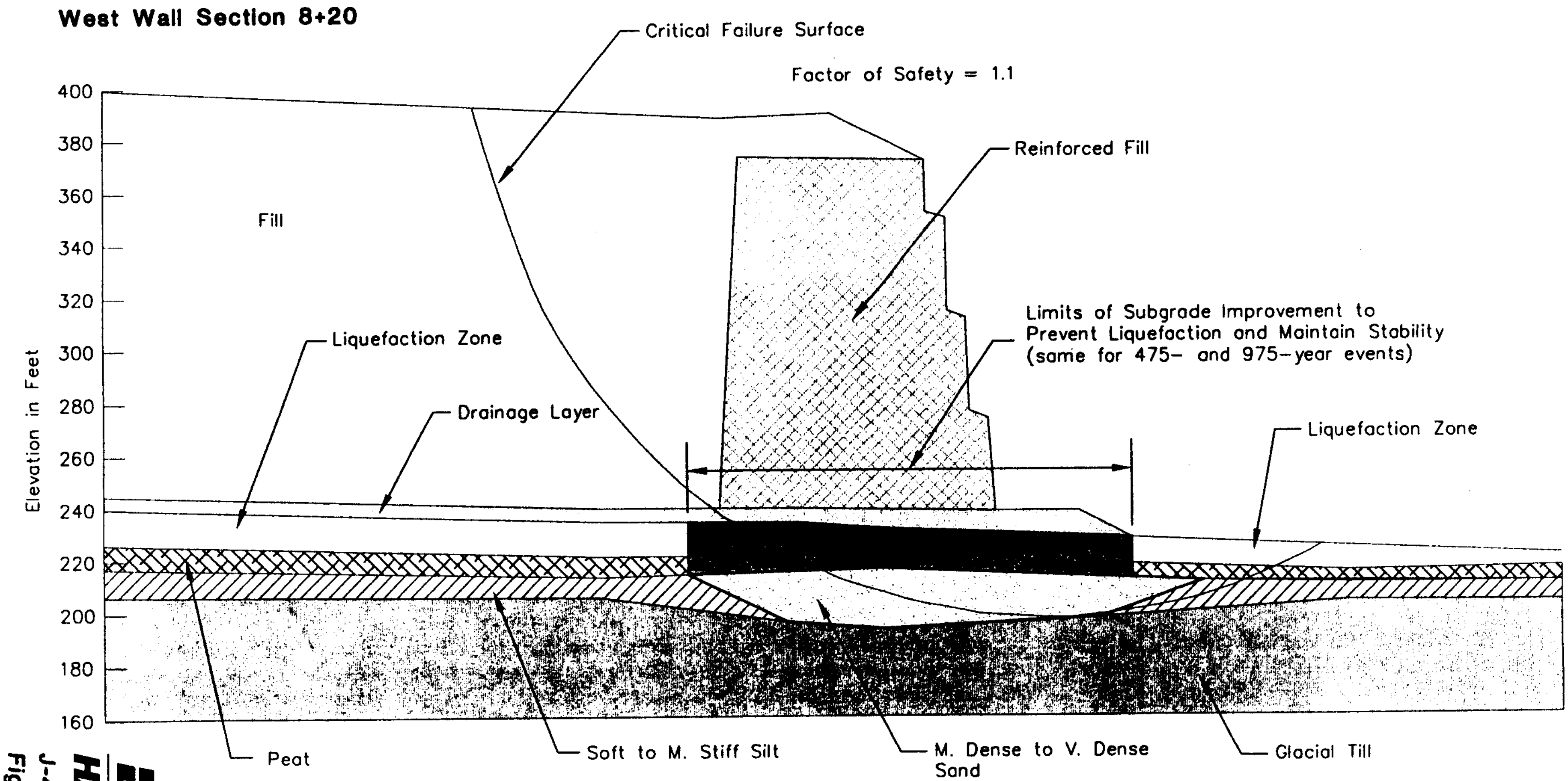

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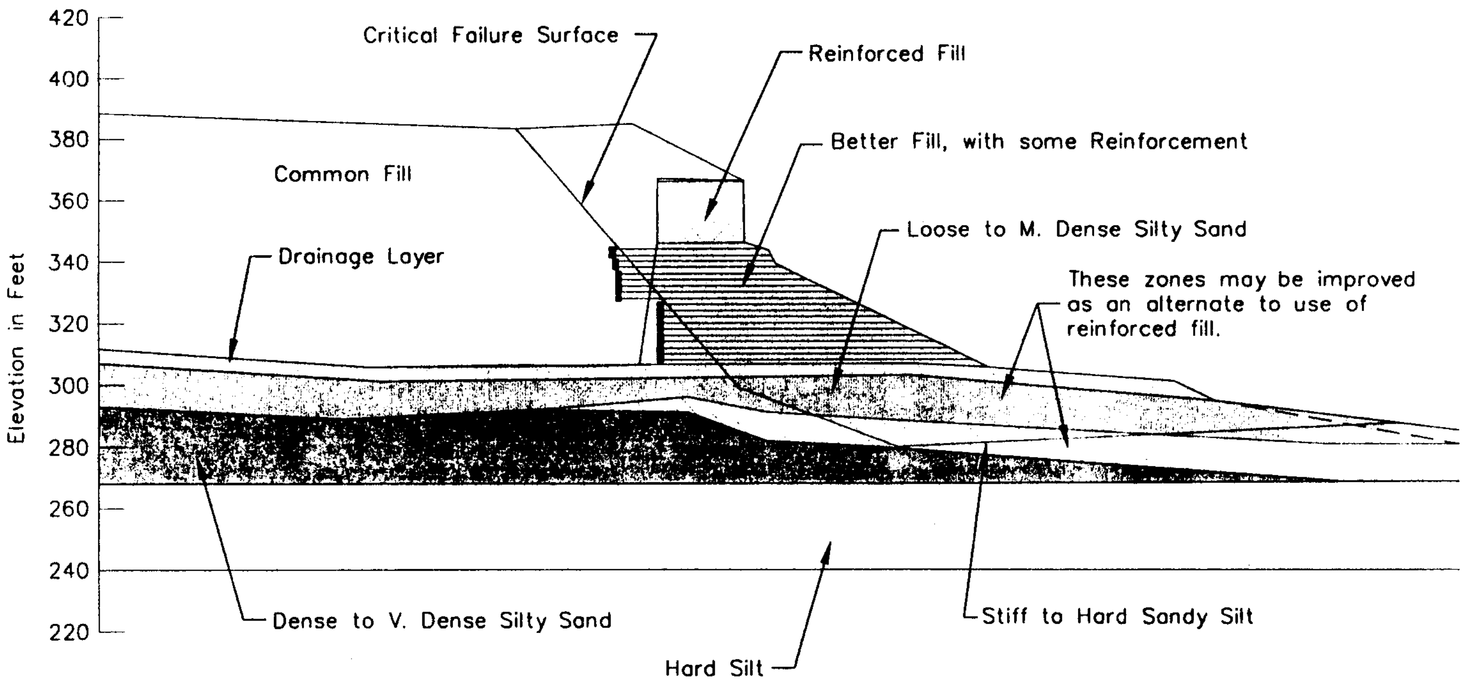
Figure 5

DTN 6/21/00 1=1 Woodstck.pc2 49782211.dwg

Fill Reinforcement to Mitigate Potential Failure Surface for a West Wall Section

West Wall Section 1+82



Seismic Event	Geotextile Strength	Factor of Safety
475–year	0	1.0
475–year	4000 lb/ft	1.1
975–year	0	0.9
975–year	4000 lb/ft	1.0
975–year	7500 lb/ft	1.1

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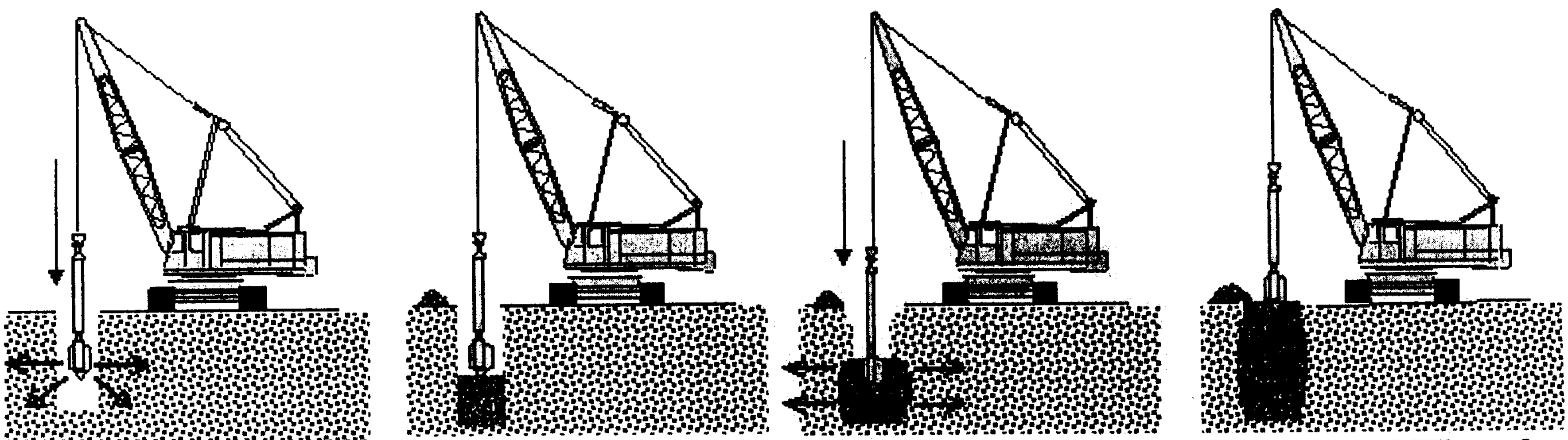
Figure 6

Stone Column Installation for Subgrade Improvement



Borehole is formed by sinking vibrator into the ground

Backfill material is placed into the borehole in stages and compacted

Alternating backfilling and compacting form continuous stone column

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Figure 7

AR 045425

Corel\497822\impact122 (charlie)

APPENDIX A
STABILITY ANALYSES - DESIGN ASSUMPTIONS, CRITERIA, METHODS, AND INPUT SOIL PARAMETERS

APPENDIX A
STABILITY ANALYSES - DESIGN ASSUMPTIONS, CRITERIA, METHODS, AND INPUT SOIL PARAMETERS

The main objective of the analyses discussed in this report is to preliminarily examine the global stability of the proposed MSE walls. We anticipate that the compound stability will be governed by the final design of the reinforcement details provided by the MSE vendor. Hart Crowser plans to verify the global stability and accomplish compound stability analyses when the final design is available to us. The following sections present the design assumptions, stability criteria, analysis methods, input soil parameters, and the results of stability analyses.

Design Assumptions

Subsurface conditions in the MSE wall areas, including exploration logs and laboratory test results, are documented in Hart Crowser's draft Data Reports for NSA Wall (dated March 20, 2000), West Wall (dated June 2000), and South Wall (dated April 7, 2000). Subsurface profiles and some of the representative cross sections are presented in these data reports. We also reviewed information from our Subsurface Conditions Data Report for 404 Permit Support submitted in July 1999, seismic design memoranda prepared by Hart Crowser, and previous reports prepared by others for this project.

Our preliminary global stability analyses of MSE wall embankments include the following cases: (1) End of Construction (EOC), (2) partially drained EOC, (3) steady state, (4) pseudo-static, and (4) liquefaction conditions. These design scenarios are briefly discussed as follows:

- The EOC condition assumes that fine-grained foundation soils will exhibit undrained behavior due to rapid placement of fill. The undrained shear strength of silt/clay is determined from laboratory consolidated undrained (CU) and unconsolidated undrained (UU) triaxial compression tests. We used drained shear strength for granular soils assuming no excess pore-water pressure would develop.

- The partially drained EOC condition assumes that excess pore-water pressure induced by the placement of fill will dissipate during the course of construction. An effective stress approach is adopted to account for the pore pressure dissipation. The effective friction angle determined from CU triaxial tests is used in the analyses. Based on laboratory consolidation test results, we estimate a coefficient of consolidation of c_v = 2.5 square feet per day for calculating excess pore-water pressures at the end of construction.

The pore pressure calculation assumes double drainage and a maximum fill placement rate of 1.75 feet per day. We used drained shear strength for granular soils assuming no excess pore-water pressure would develop.

- The steady-state condition assumes that fine-grained foundation soils will exhibit drained behavior in the long-term condition. No excess pore-water pressure is anticipated in the steady-state condition. A drained friction angle of soils determined from CU triaxial tests was used in the analysis.

- The pseudo-static condition simulates the earthquake force by incorporating a seismic coefficient that is about one half of the peak horizontal ground acceleration. We typically apply the seismic coefficient to the most critical failure surface identified in the steady-state condition.

- Soils have a tendency to loose a significant portion of shear strength when they become liquefied during an earthquake. Our analyses used an average residual undrained strength for the liquefaction-prone soils. Other than this undrained residual strength, the liquefaction analyses assumed the same soil parameters as the steady-state and did not include the seismic coefficient used in the pseudo-static analyses.

Other assumptions used in our stability analyses include:

- All peat and organic-rich soils will be overexcavated to avoid long-term secondary settlements.

- A 4-foot-thick drainage layer will be provided at the base of the MSE wall and adjacent embankment. All MSE wall and embankment fills are relatively granular soils and no excess pore-water pressure will develop.

- The drainage layer has full water flow in the worst condition and the hydrostatic piezometric water level for the subgrade soils is at the top of the underdrain, with no areas of saturation or positive pore-water pressure within the embankment fill.

- Reinforcement length extends from the toe of the wall to 80 percent of total wall height behind the wall.

Stability Criteria

The minimum factors-of-safety criteria selected for our preliminary stability analyses are 1.3 (EOC, EOC partially drained, and steady state conditions) and

1.1 (pseudo-static and liquefaction conditions). Subgrade improvement is proposed where analyses indicate factors of safety below these values.

In general, the seismic (pseudo-static and liquefaction) conditions control the need for subgrade improvement. Once the subgrade improvement produces a factor of safety of 1.1 for the seismic conditions, the static factor of safety is typically greater than 1.4. We recommend verifying the static factor of safety for the steady-state condition to be greater than 1.5 when the detailed design is completed.

Based on the result of a Probabilistic Seismic Hazard Analysis (PSHA), provided in our memorandum dated October 8, 1999, we selected peak ground horizontal accelerations of 0.36 g for the 475-year event and 0.47 g for the 975-year event. (However, we found the extent of subgrade improvement needed to mitigate liquefaction to be essentially the same for both of these events - see Hart Crowser's April 10, 2000 memorandum).

Methods Used in Analyses

Hart Crowser used the computer program SLOPE/W for our global stability analyses. The program employs limit equilibrium theory-based methods by Janbu, Bishop, Spencer, and Morgernstern-Price to search for the most critical failure plane, including both circular and wedge surfaces. Spencer and Morgernstern-Price methods satisfy both force and moment equilibrium. In general, we used the Spencer method for determining the most critical factor of safety, and occasionally used the Morgernstern-Price method for more complicated user-defined failure surfaces.

Our global stability analyses generally searched for failure planes extending into the subgrade and beyond the MSE wall reinforcement zone. For this stage of design, we did not examine any failure planes cutting through the MSE wall reinforcement zone. The analysis sequence for each section generally starts with the EOC condition, followed by EOC partially drained, steady-state, pseudo-static, and liquefaction conditions. If in any case the factor of safety did not meet the minimum criteria, subgrade preparation or ground improvement was then incorporated into the analyses. For example, if the factor of safety was less than 1.1 for the liquefaction condition, the liquefied soil zone within the failure plane was modeled as if it were overexcavated and backfilled with structural fill or improved by stone columns. The extent of simulated subgrade improvement was adjusted until the factor of safety criteria was satisfied. All other cases were then re-examined with the improved subgrade to check the minimum factor-of-safety criteria.

AR 045429

To address the uncertainties in estimating the residual shear strength of liquefiable soils, we used the Monte Carlo Probability analysis feature in the SLOPE/W. The probabilistic distribution of factors of safety for each case was reviewed for evaluation of its failure potential.

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. A summary of unit weight and shear strength of soils is listed in Table A-1.

Table A-1 - Summary of Input Soil Parameters

Soil Type	Unit Weight	Drained Strength		Undrained Strength	
		c′	ϕ'	c	ϕ
	(pcf)	(psf)	(deg.)	(psf)	(deg.)
Loose to Medium Dense Sand	125	0	32	-	-
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	115	0	32	1000	0
Stiff to Hard Silt/Clay	115	0	32	4000	0
Structural Backfill	135	0	35	-	-
Crushed Rock Backfill	145	0	40	-	-
Embankment Fill	135	0	35	-	-
Reinforced Embankment Fill	140	0	40	-	-
Drainage Blanket	140	0	37	-	-
Improved Subgrade	135	0	35	-	-

We tentatively assumed that removal of unsatisfactory subgrade soils and replacement with compacted fill, or the use of stone columns, will be used for deep ground improvements. Table A-1 shows that a drained friction angle of 35° was selected for our preliminary stability analyses. For stone columns this fiction angle was calculated based on an area replacement ratio of about 15 to 20 percent, and a typical stress ratio assuming that the compacted crushed rock column would have a friction angle of 42° and the surrounding soils would have a friction angle of 32°. We tentatively estimate that 42-inch-diameter stone columns with 8-foot center-to-center spacing should be installed in a triangular pattern. This design will provide an area replacement ratio of about 17 percent.

Soil liquefaction is a major seismic hazard for this project because of the presence of loose to medium-dense sand in the foundation subgrade and the high groundwater table. We evaluated the trigger liquefaction using field measurements, such as SPT-N values and CPT penetration resistance, from all available explorations. Our study also included the overburden effect of the MSE wall and embankment. Following our trigger liquefaction analyses, we determined the residual undrained shear strength of the liquefied soil from the SPT-N value using an empirical relationship published by Seed and Harder (1990). This was followed by a statistical analysis of residual strength from all the soil borings to determine the average and standard deviation values for the NSA, West, and South walls. The results indicate that the average residual strength of liquefied soils ranges from 850 to 900 psf, with a range of standard deviations from 450 to 550 psf.

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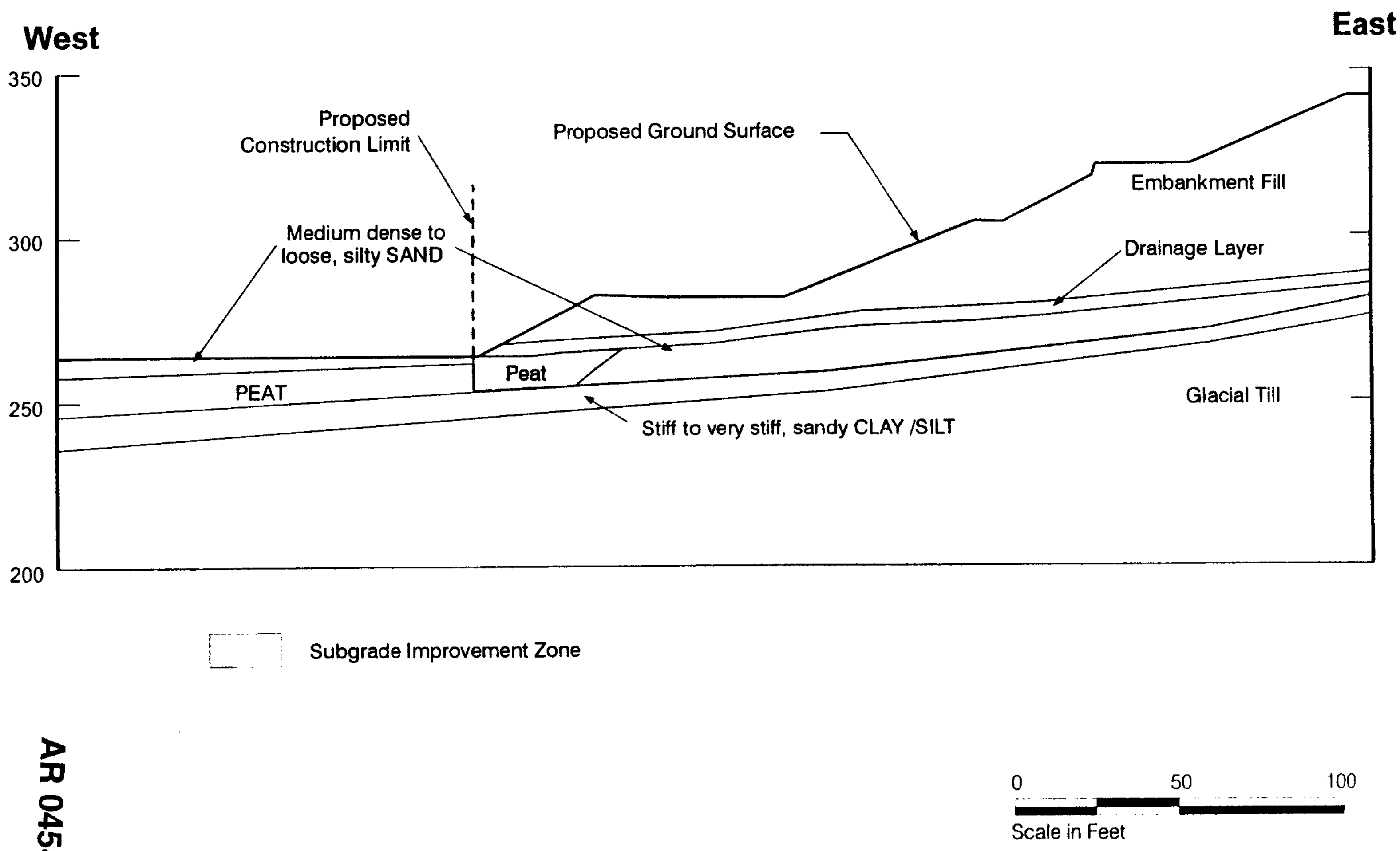
APPENDIX B
MSE WALL CROSS SECTIONS

North Safety Area MSE Wall
Section 101+20



AR 045433

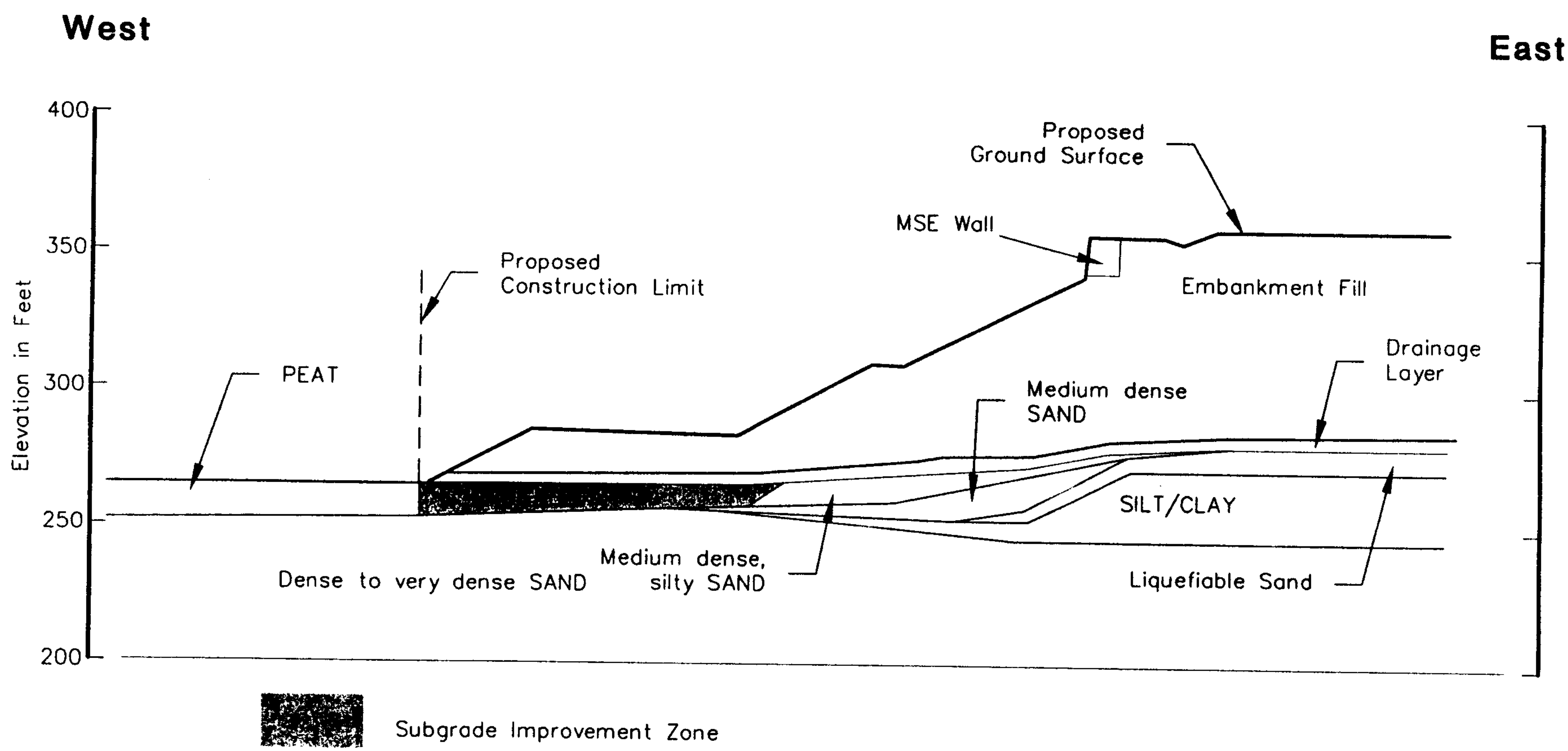
DTN 6/21/00 497822C.cdr

HARTCROWSER
J-4978-22 5/00
Figure B-1

DTN 6/21/00 1=60 charlie-8.pc2 49782212.dwg

North Safety Area MSE Wall

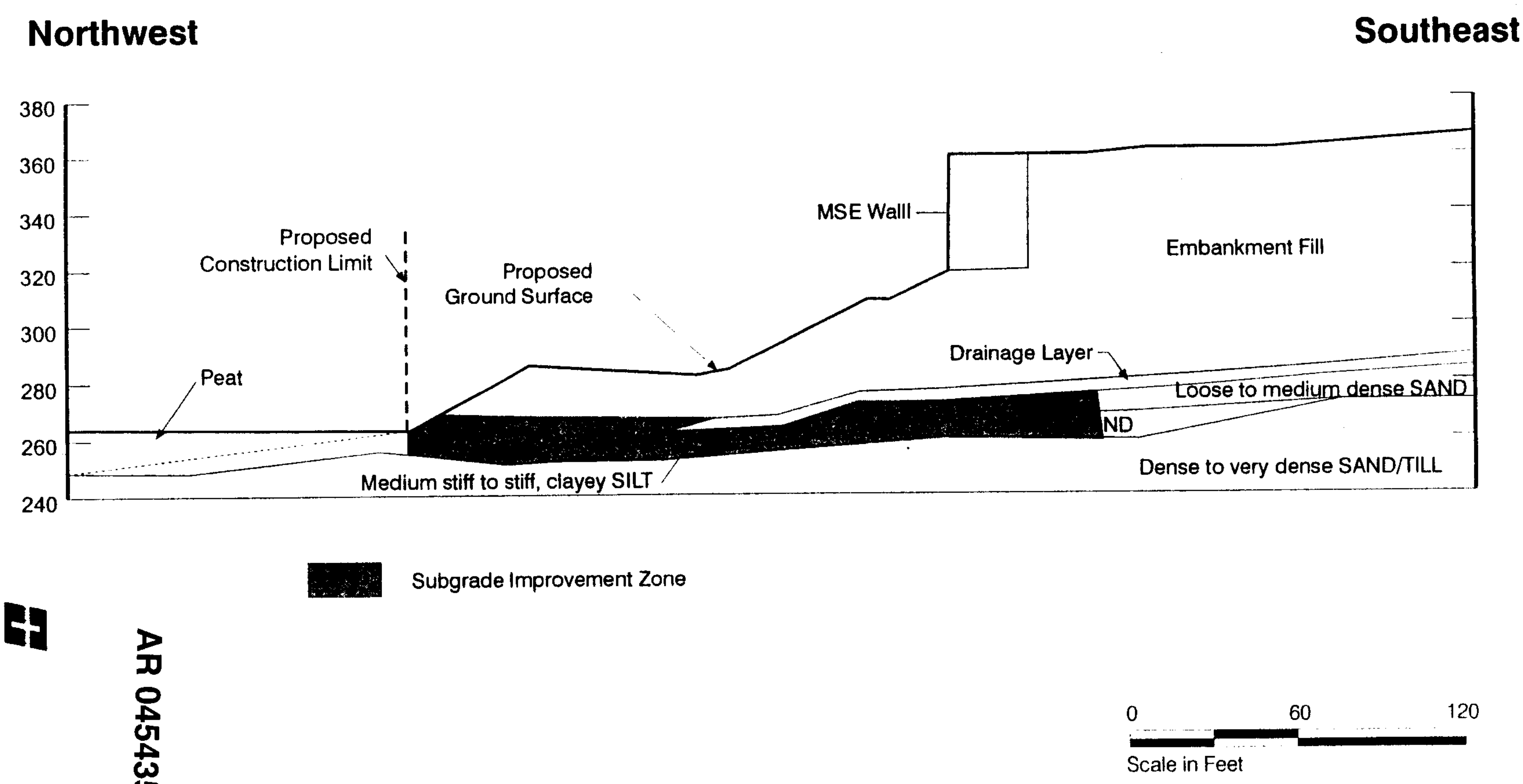
Section 103+20



AR 045434

HARTCROWSER
J-4978-22 5/00
Figure B-2

North Safety Area MSE Wall
Section 105+20



DTN 6/21/00 497822D.cdr

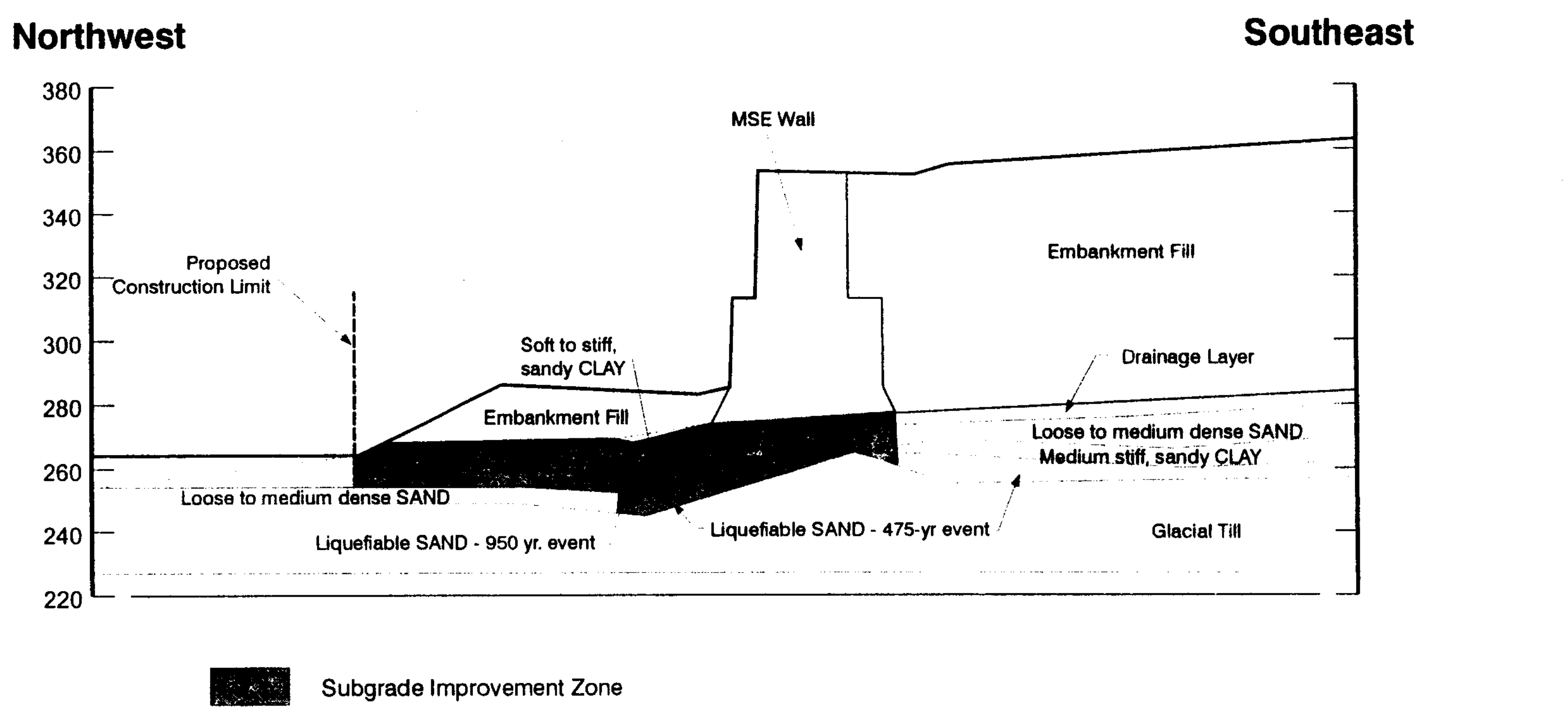
AR 045435

HARTCROWSER

J-4978-22 5/00

Figure B-3

North Safety Area MSE Wall
Section 107+19



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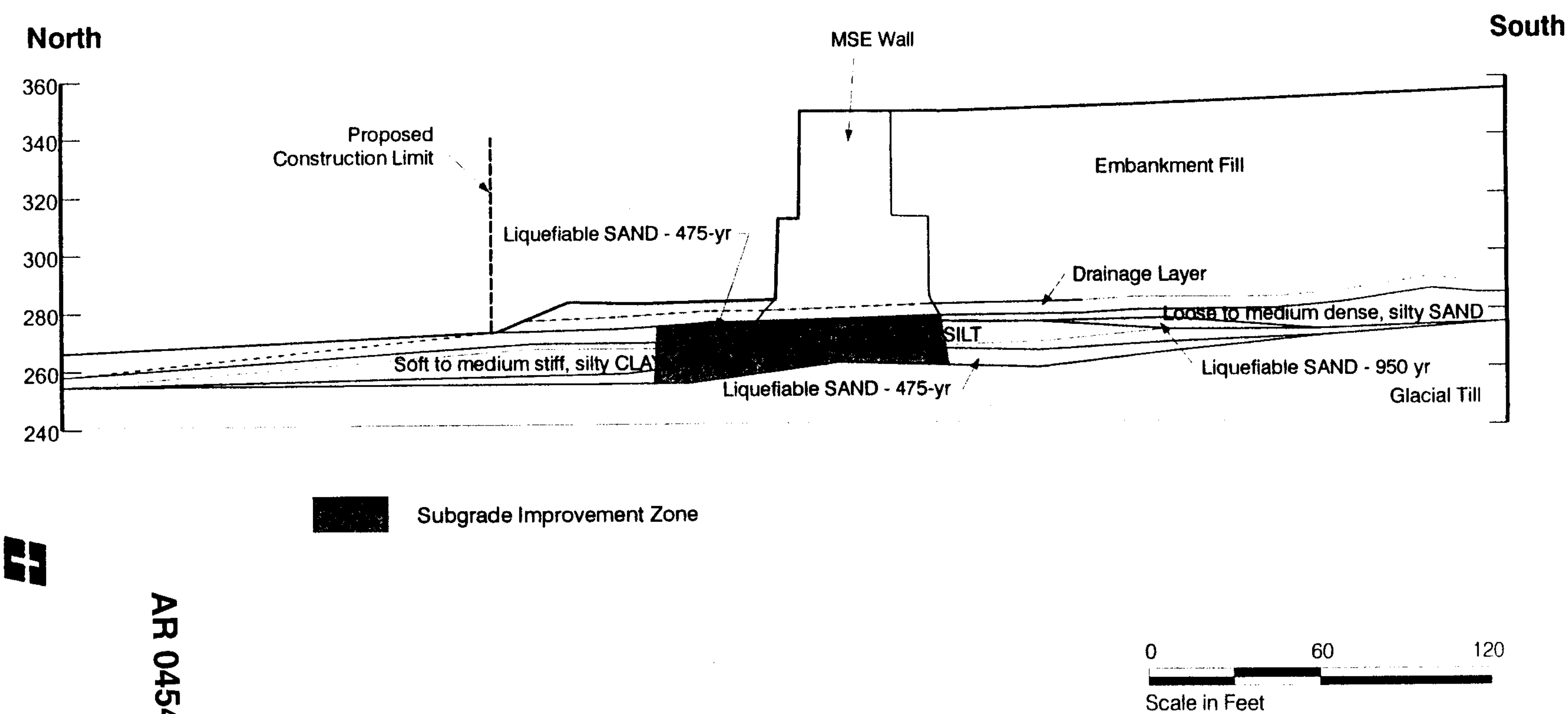
AR 045436

HARTCROWSER

J-4978-22 5/00

Figure B-4

North Safety Area MSE Wall
Section 110+47

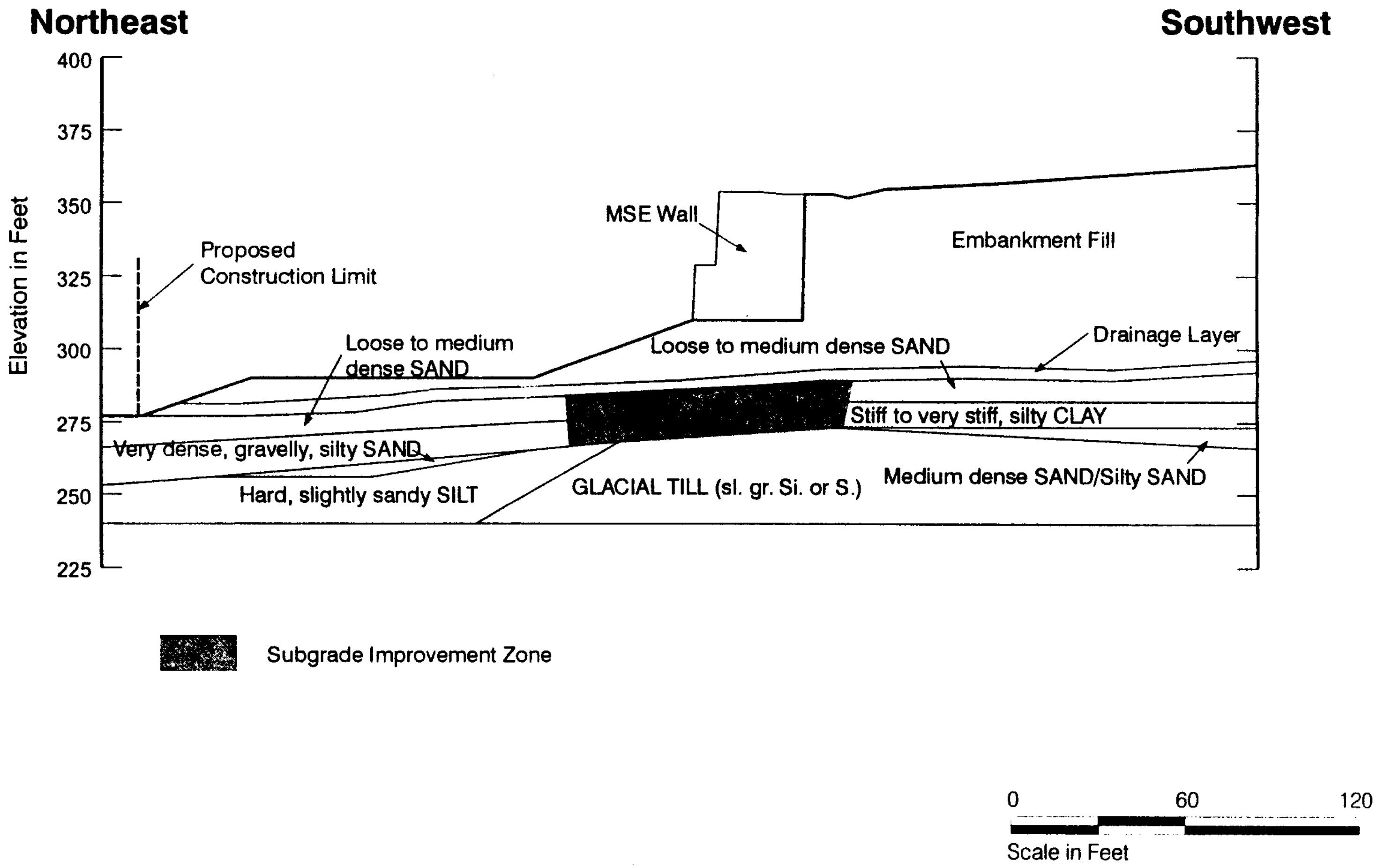


DTN 6/21/00 497822G.cdr

AR 045437

HARTCROWSER
J-4978-22 5/00
Figure B-5

North Safety Area MSE Wall
Section 114+10



DTN 6/21/00 497822H.cdr

AR 045438

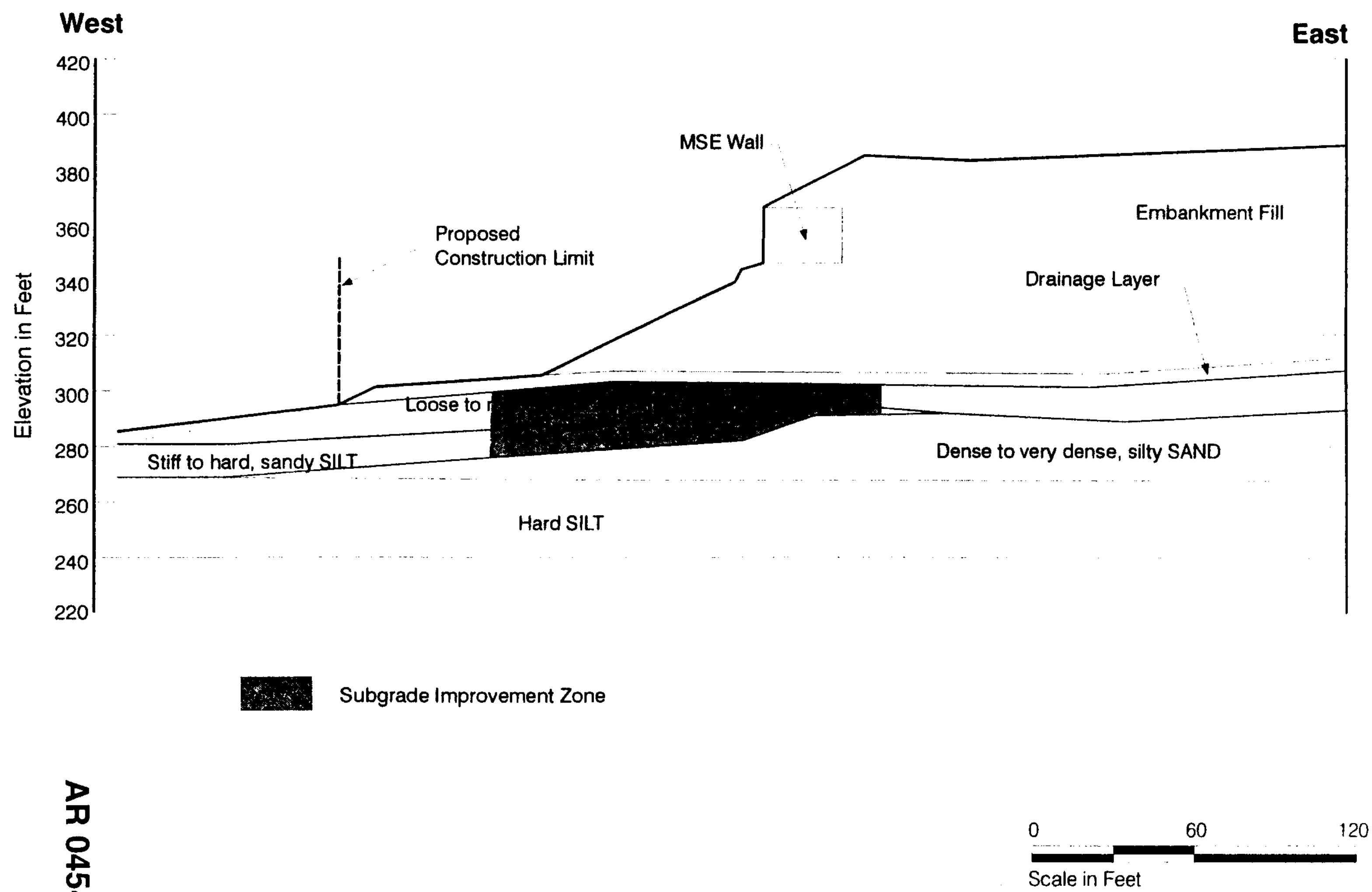
HARTCROWSER

J-4978-22 5/00

Figure B-6

West MSE Wall

Section 173+65



DTN 6/22/00 497822J.cdr

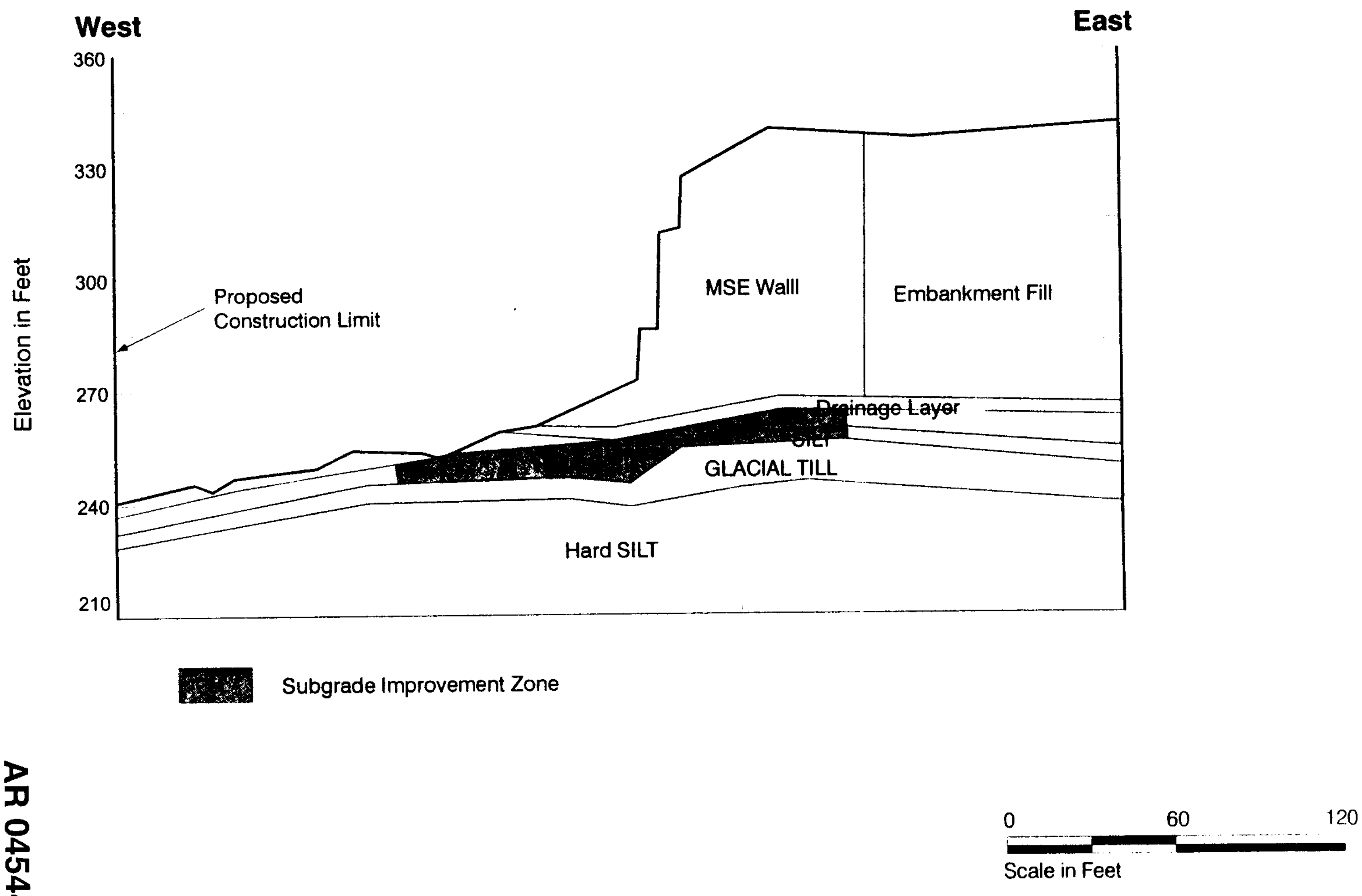
AR 045439

HARTCROWSER

J-4978-22 5/00

Figure B-7

West MSE Wall
Section 176+40



DTN 6/22/00 497822P.cdr

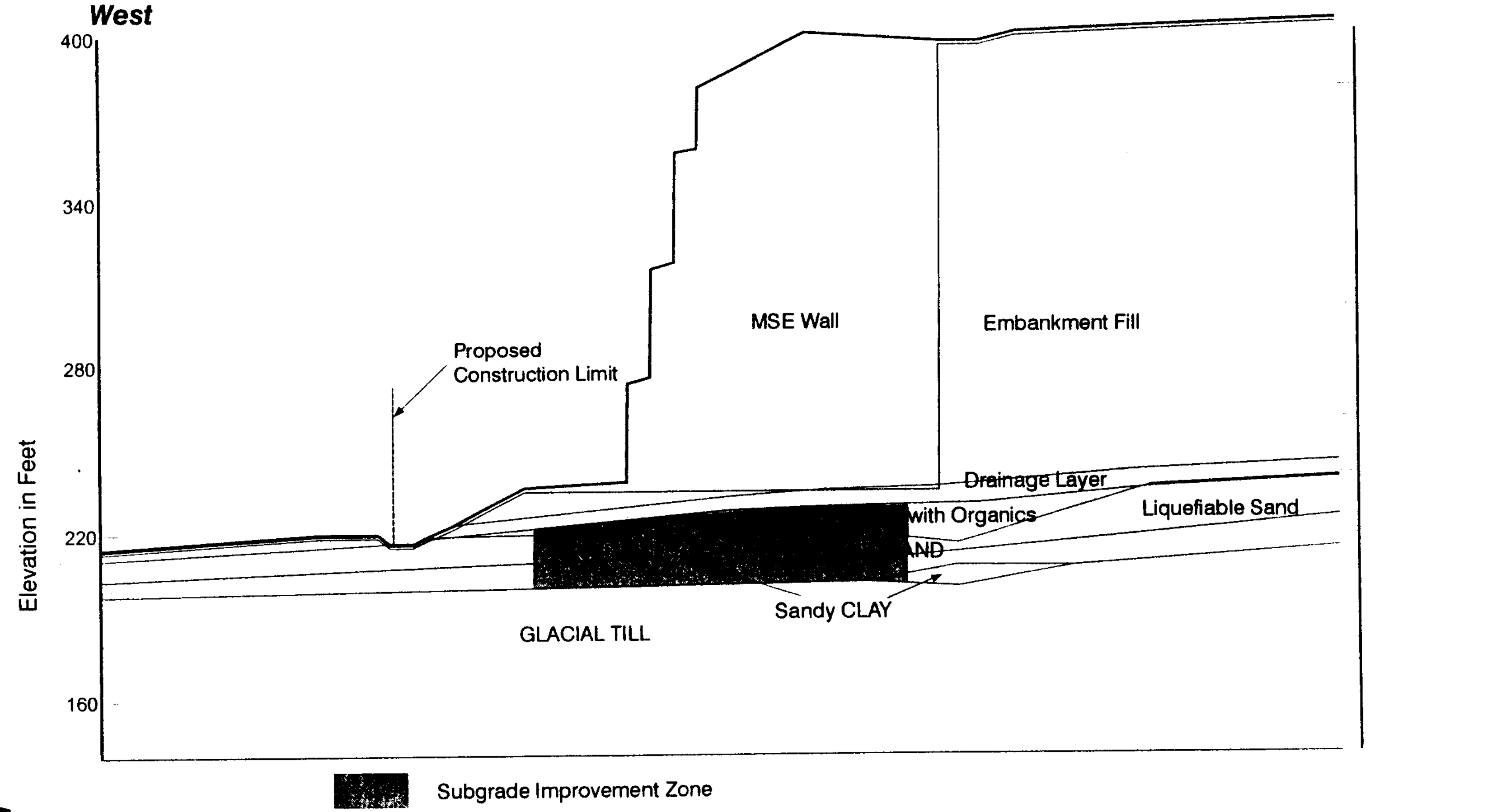
AR 045440

HARTCROWSER

J-4978-22 5/00

Figure B-8

West MSE Wall
Section 178+60



DTN 6/22/00 497822Q.cdr

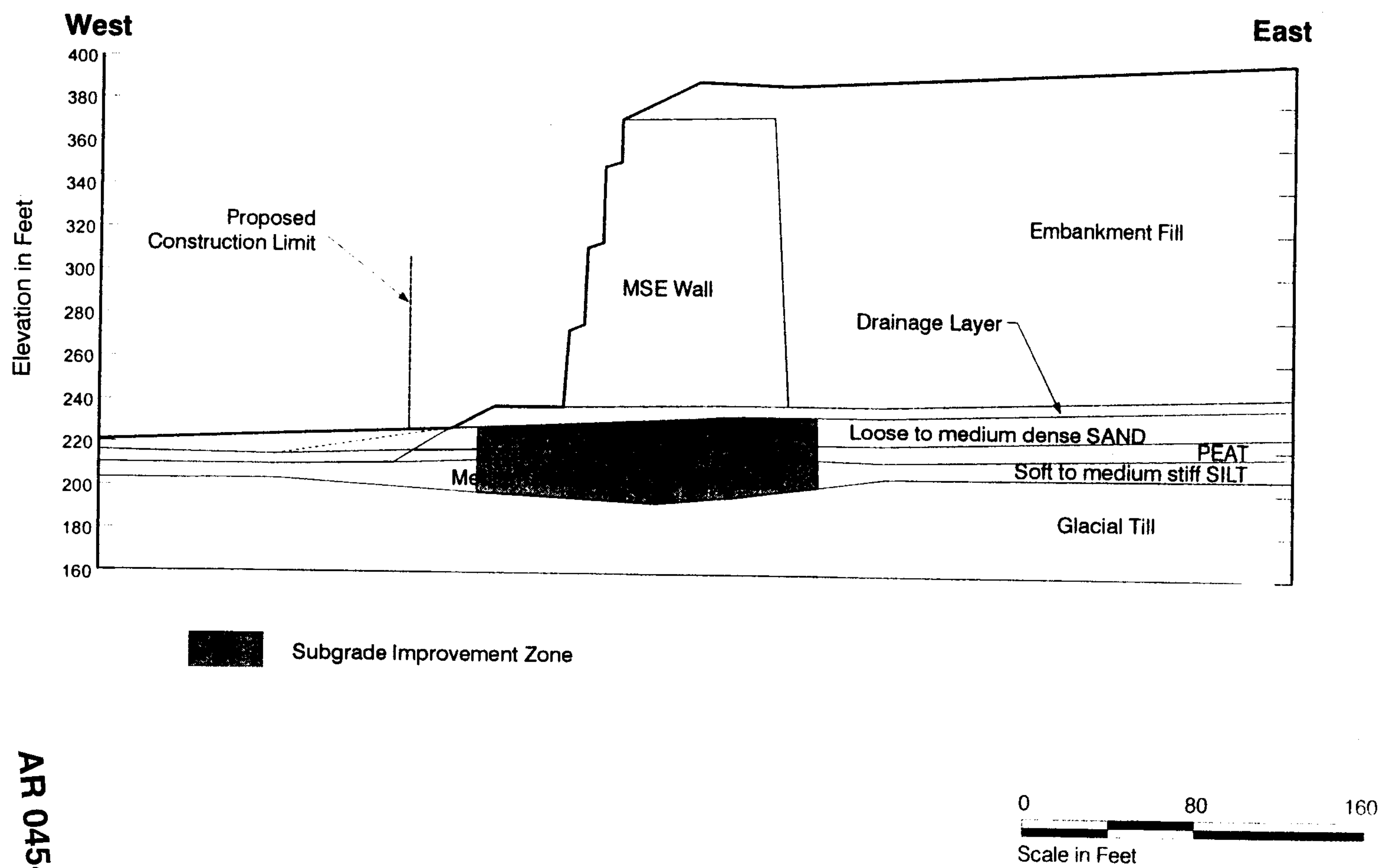
AR 045441

HARTCROWSER

J-4978-22 5/00

Figure B-9

West MSE Wall
Section 180+05

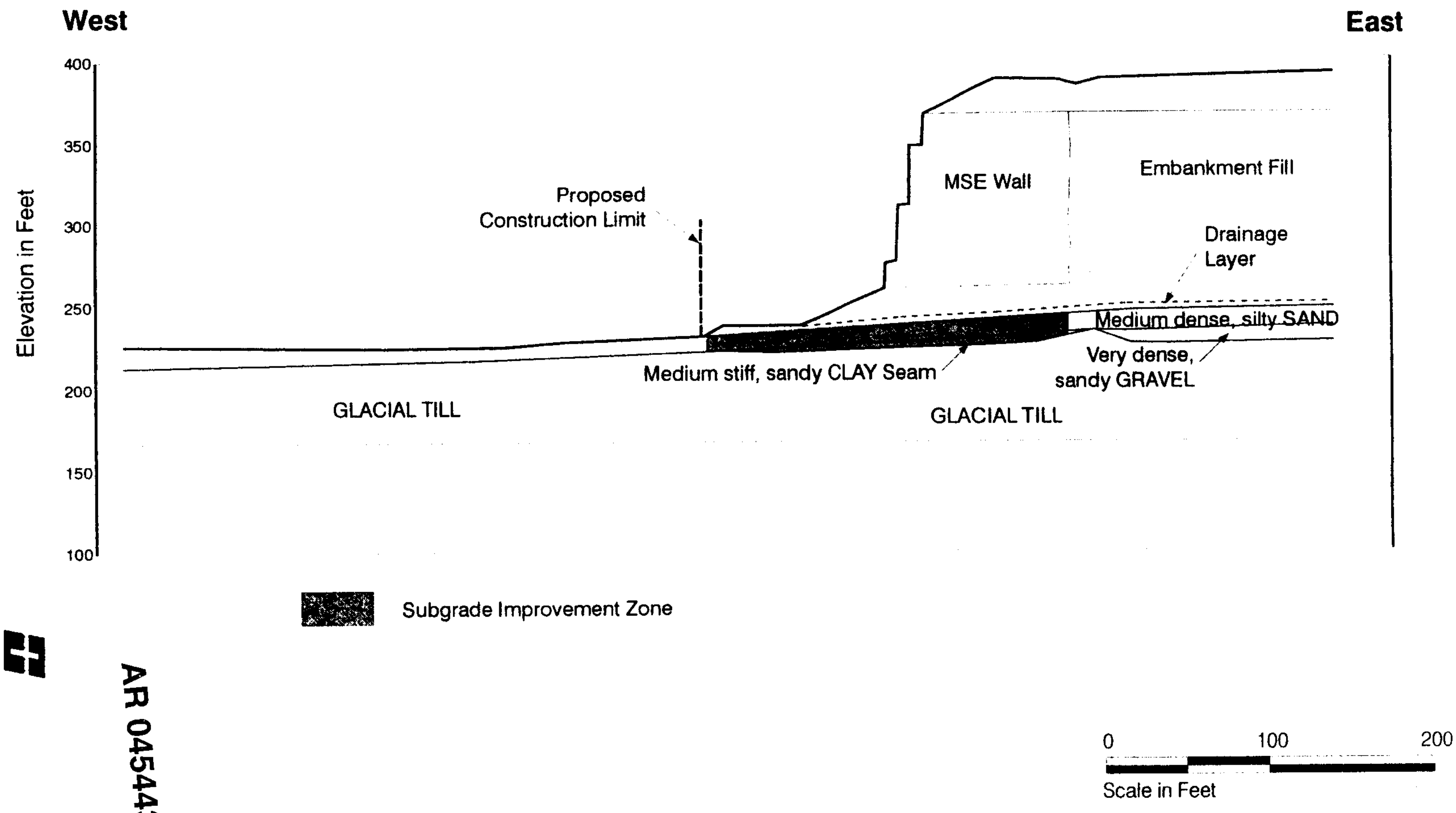


DTN 6/22/00 497822L.cdr

AR 045442

HARTCROWSER
J-4978-22 5/00
Figure B-10

West MSE Wall
Section 181+90



AR 045443

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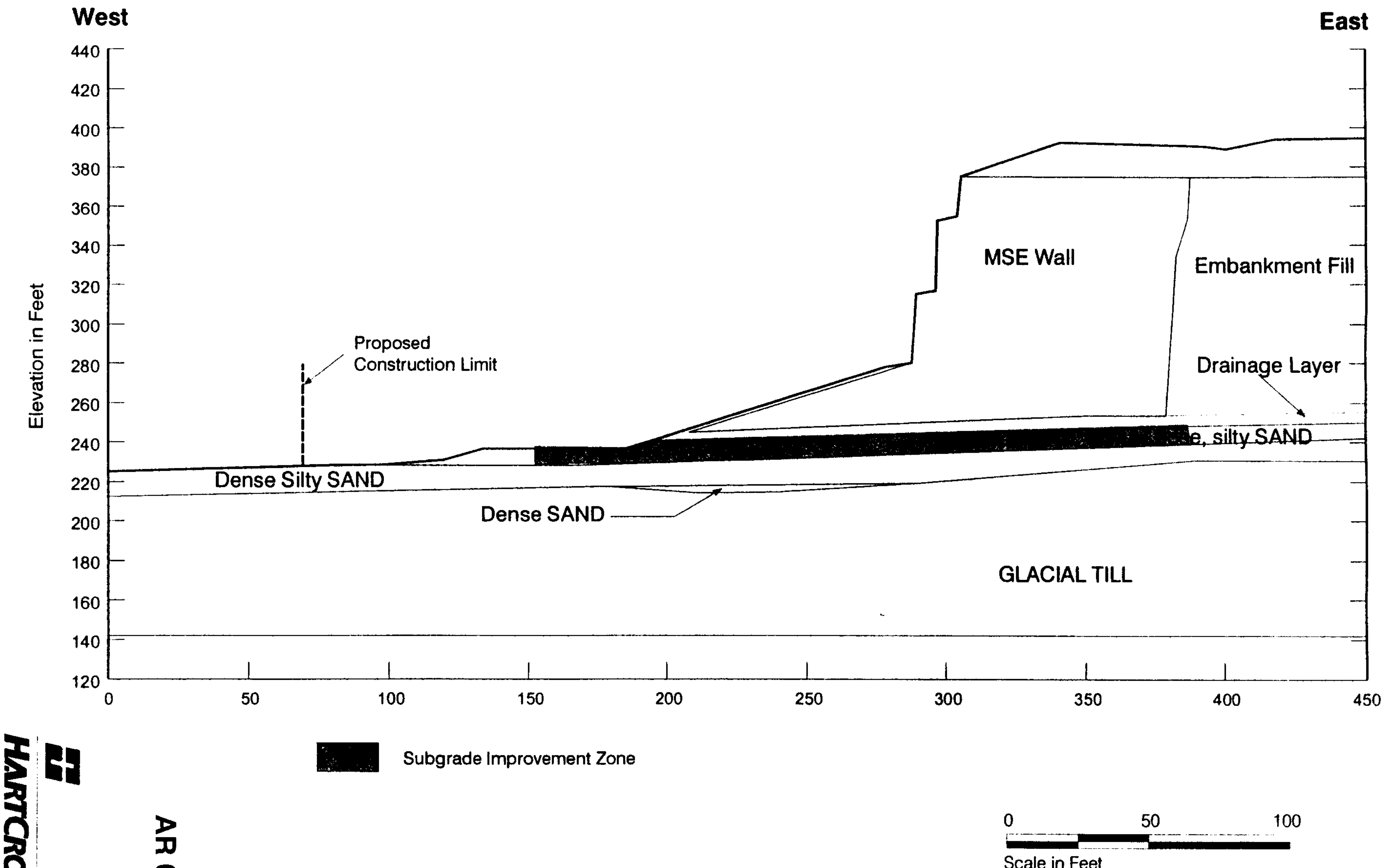
J-4978-22 5/00

Figure B-11

DTN 6/22/00 4978220.cdr

West MSE Wall
Section 183+08



DTN 6/22/00 497822R.cdr

AR 045444

HARTCROWSER

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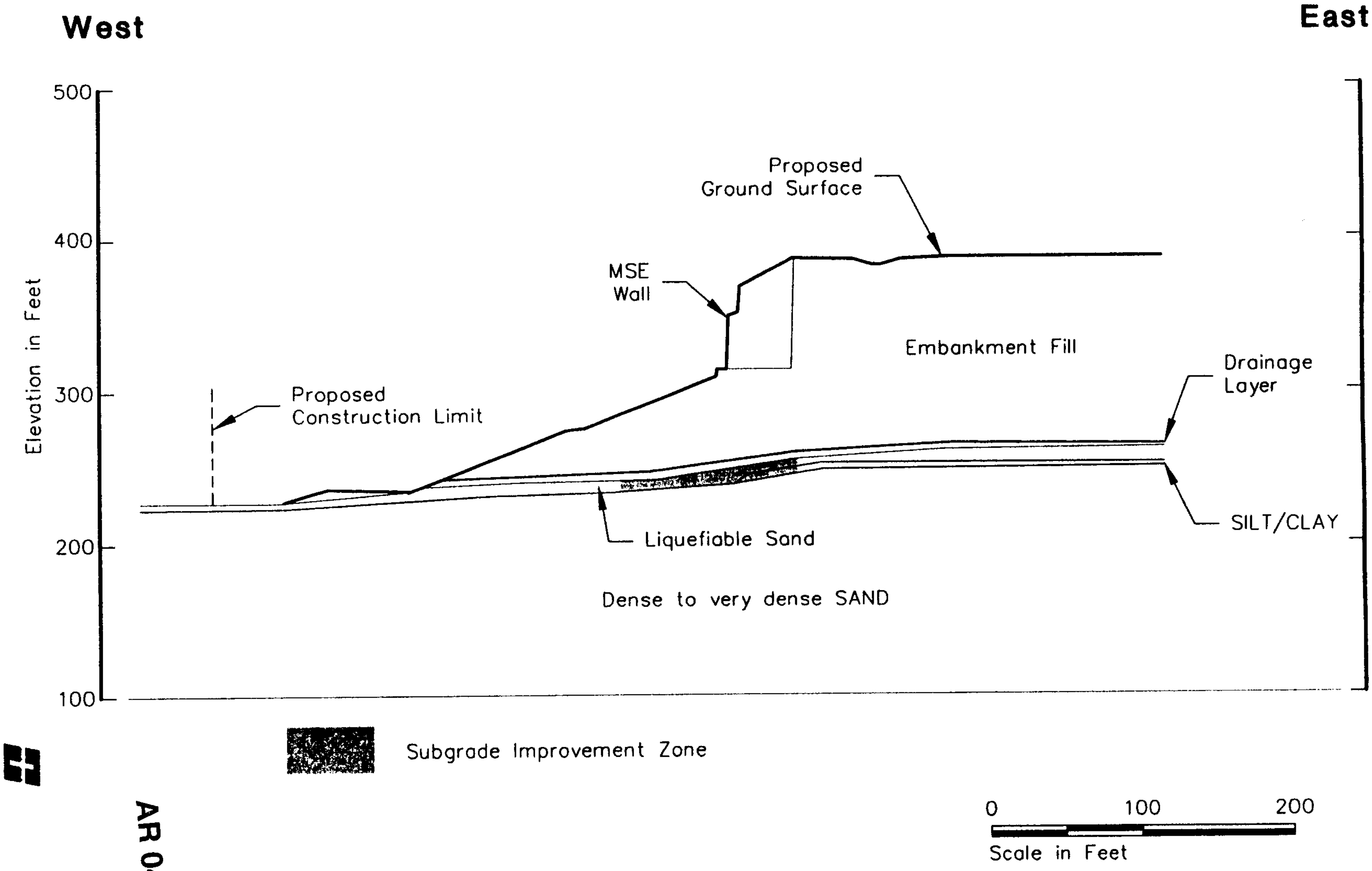
Figure B-12

DTN 6/22/00 1=100 charlie-8.pc2 49782212.dwg

West MSE Wall

Section 184+50



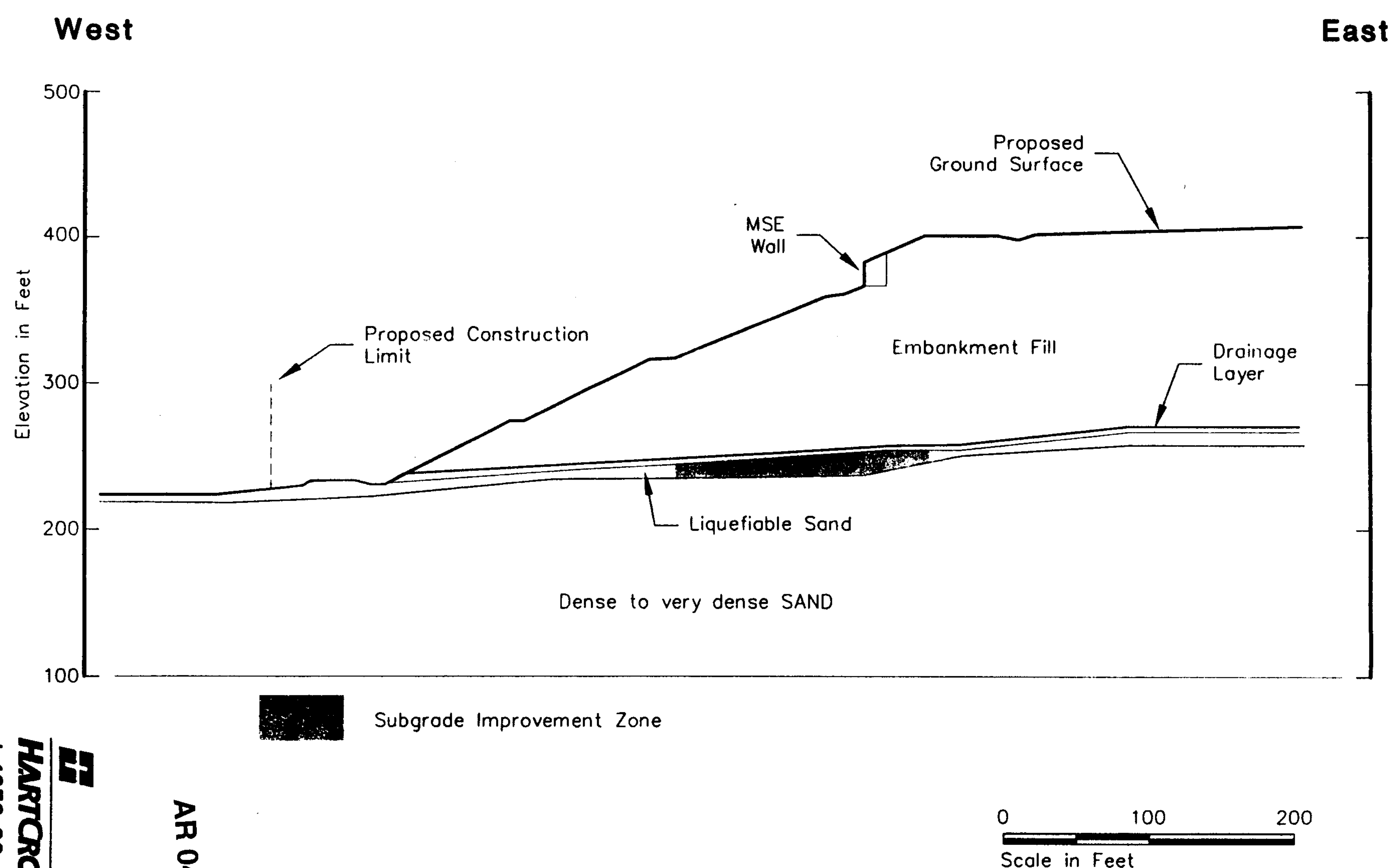
AR 045445

HARTCROWSER
J-4978-22 5/00
Figure B-13

DTN 6/22/00 1=100 charlie-B.pc2 49782212.dwg

West MSE Wall

Section 186+20



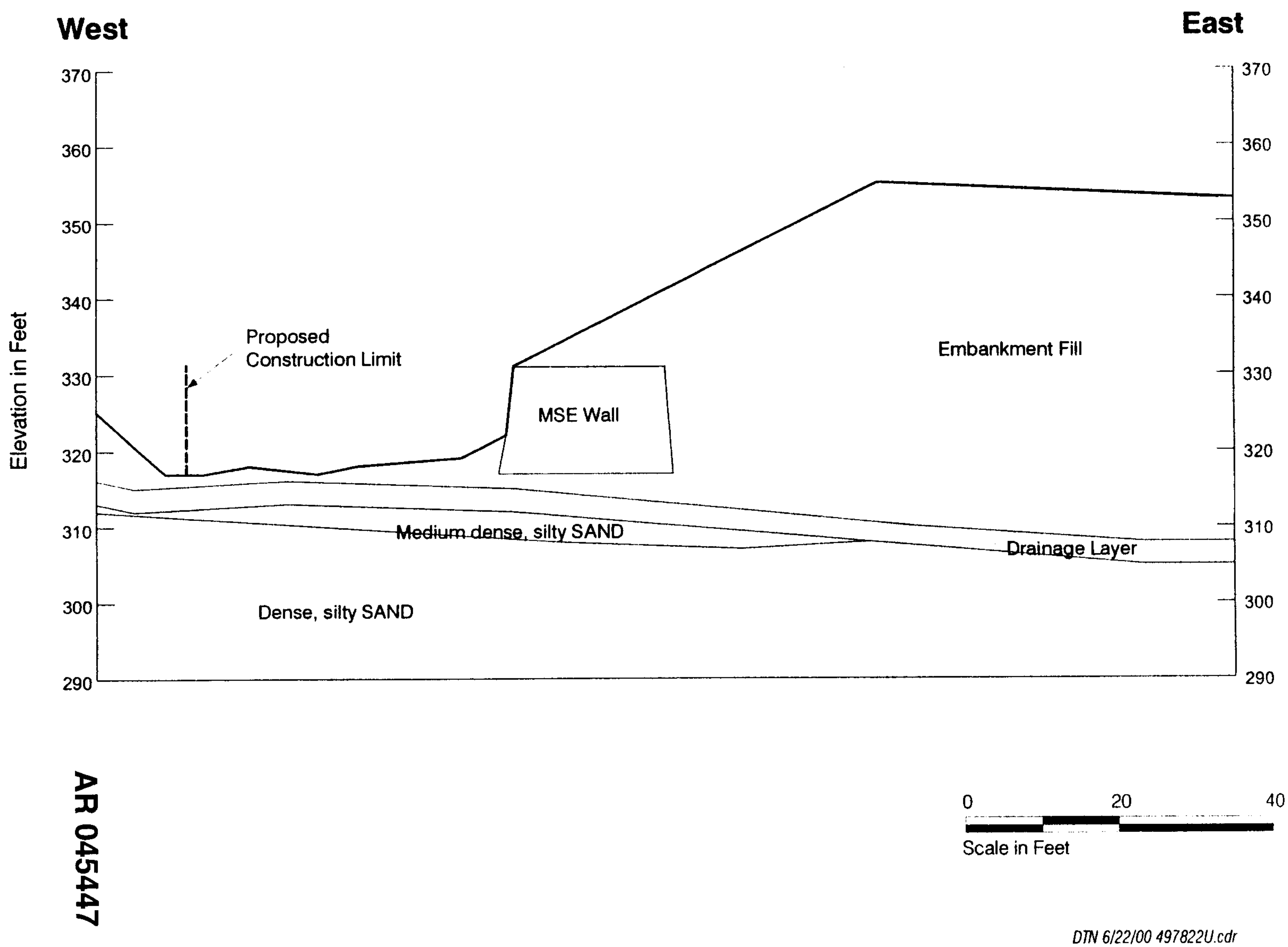
AR 045446

HARTCROWSER

J-4978-22 5/00

Figure B-14

South MSE Wall
Section 139+50



AR 045447

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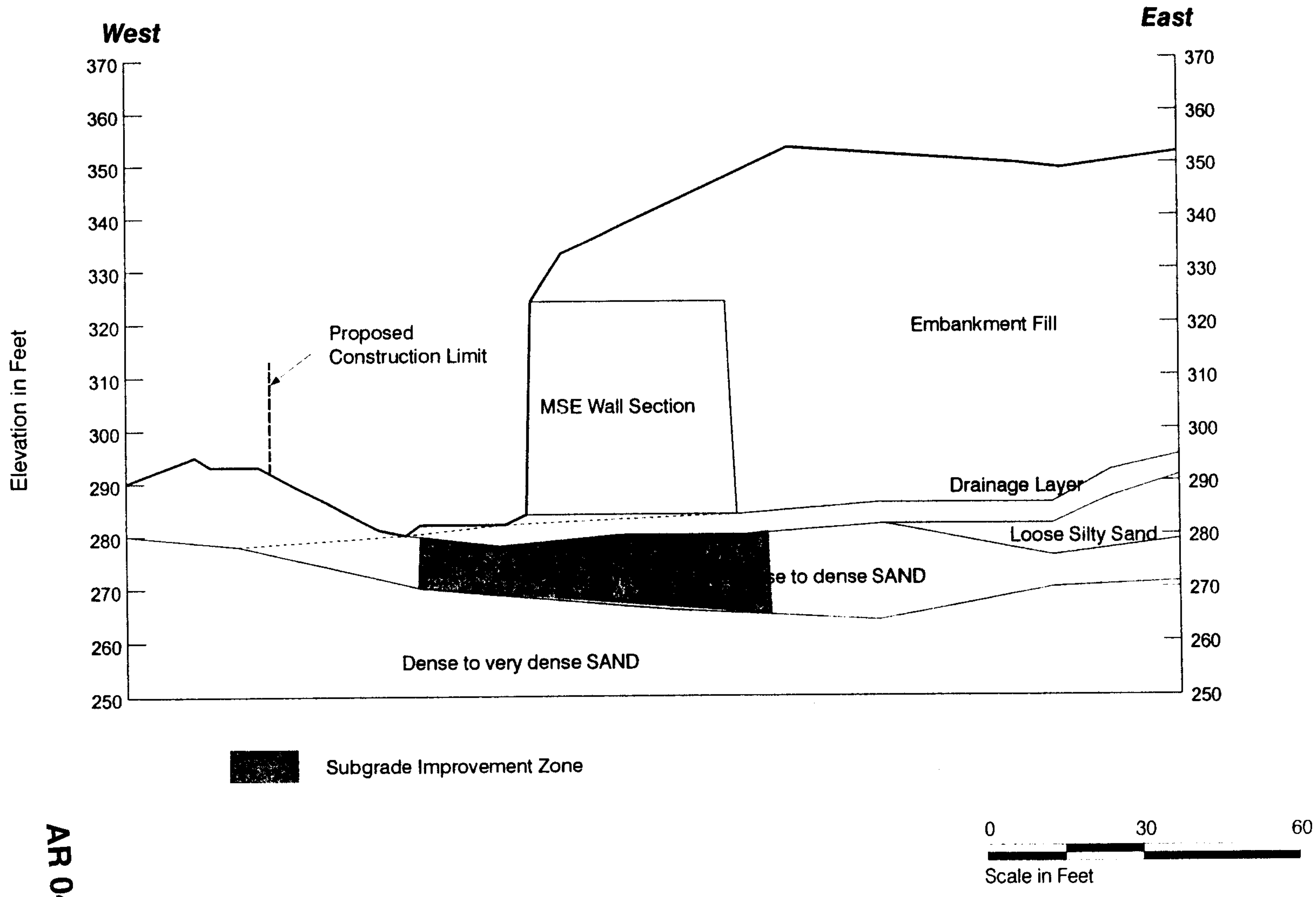
J-4978-22 5/00
Figure B-15

South MSE Wall
Section 141+78



DTN 6/22/00 497822V.cdr

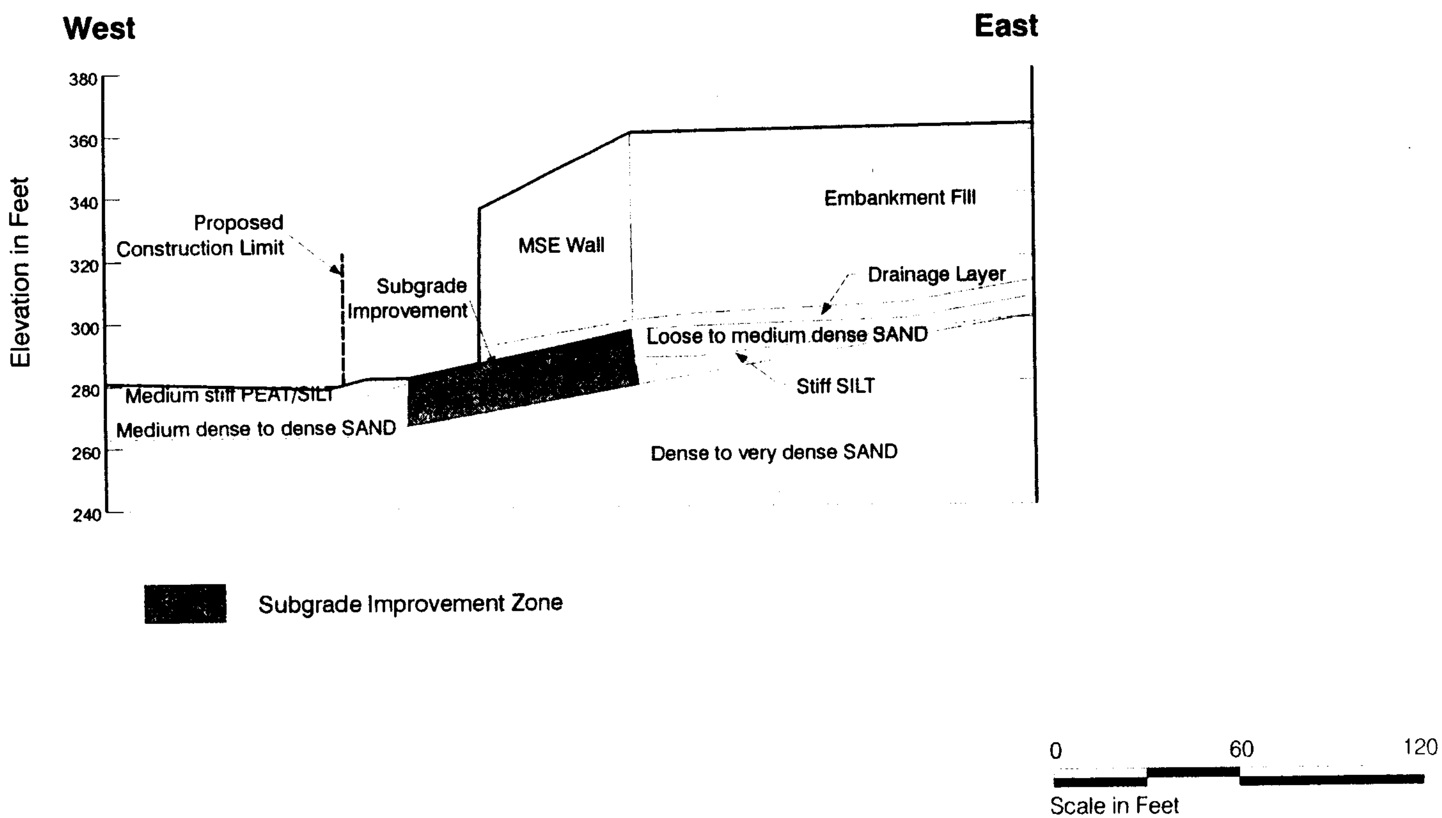
AR 045448

HARTCROWSER

J-4978-22 5/00

Figure B-16

South MSE Wall
Section 142+72

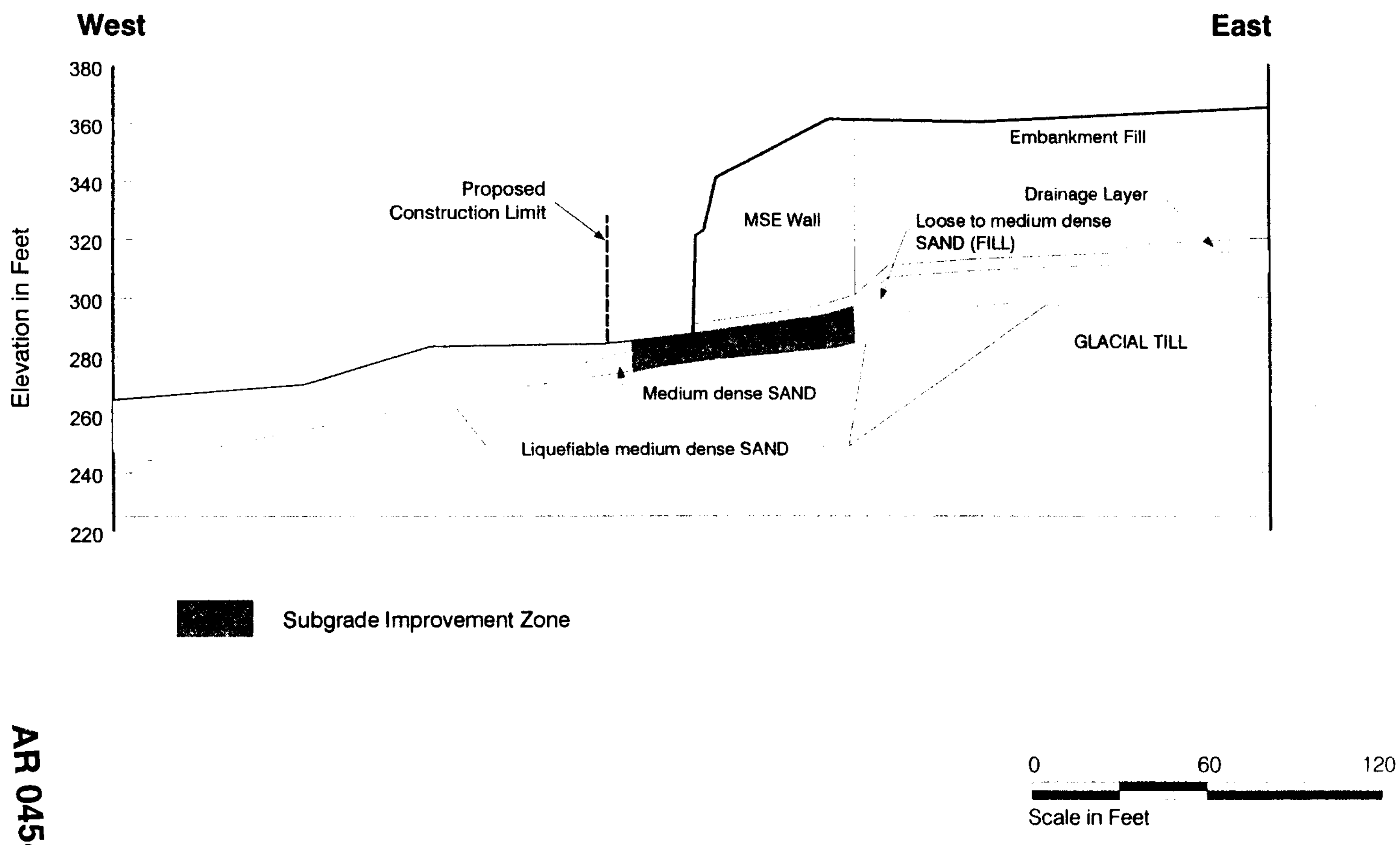


DTN 6/22/00 497822M.cdr

AR 045449

HARTCROWSER
J-4978-22 5/00
Figure B-17

South MSE Wall
Section 144+20



DTN 6/22/00 497822N.cdr

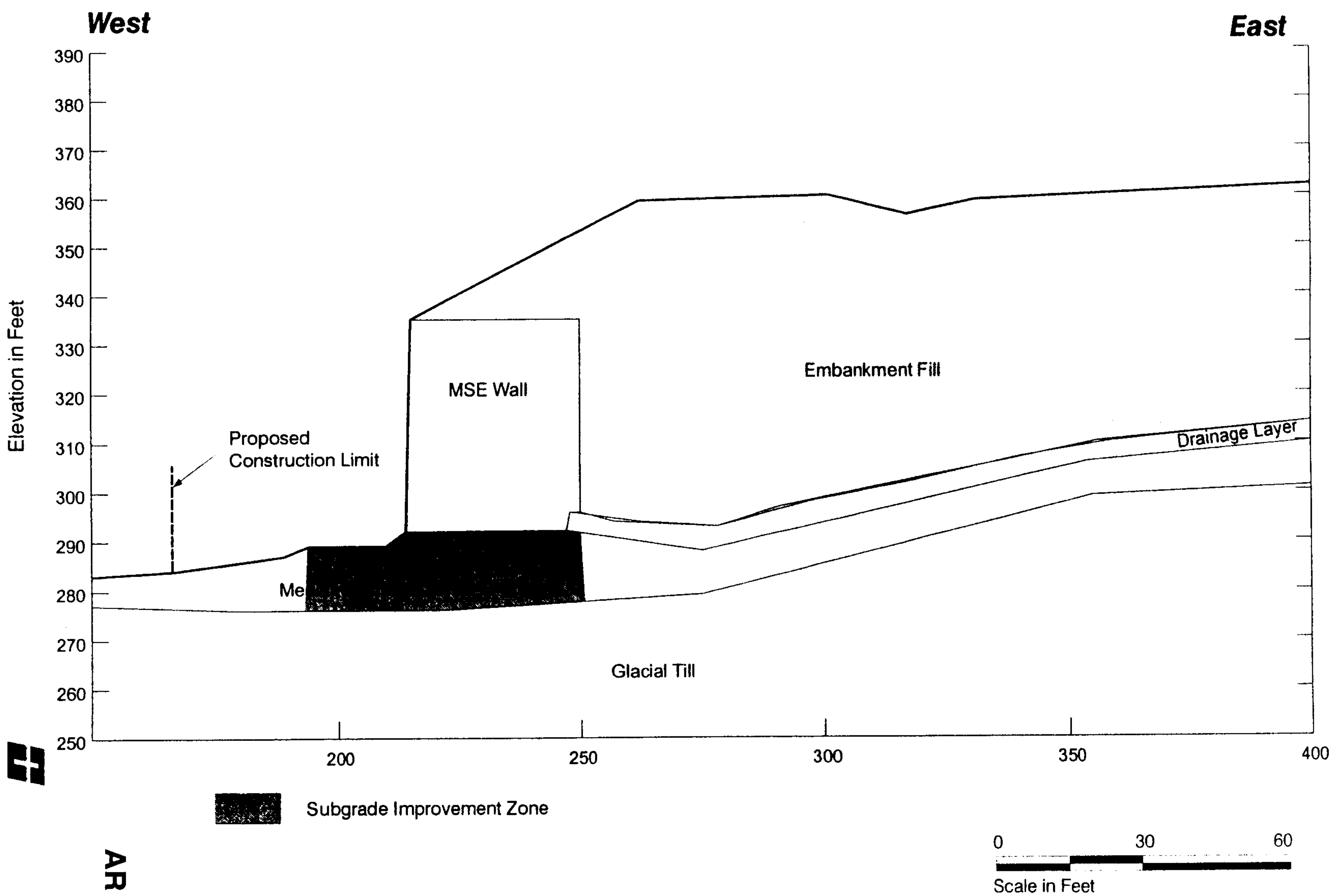
AR 045450

HARTCROWSER

J-4978-22 5/00

Figure B-18

South MSE Wall
Section 145+42



DTN 6/22/00 497822S.cdr

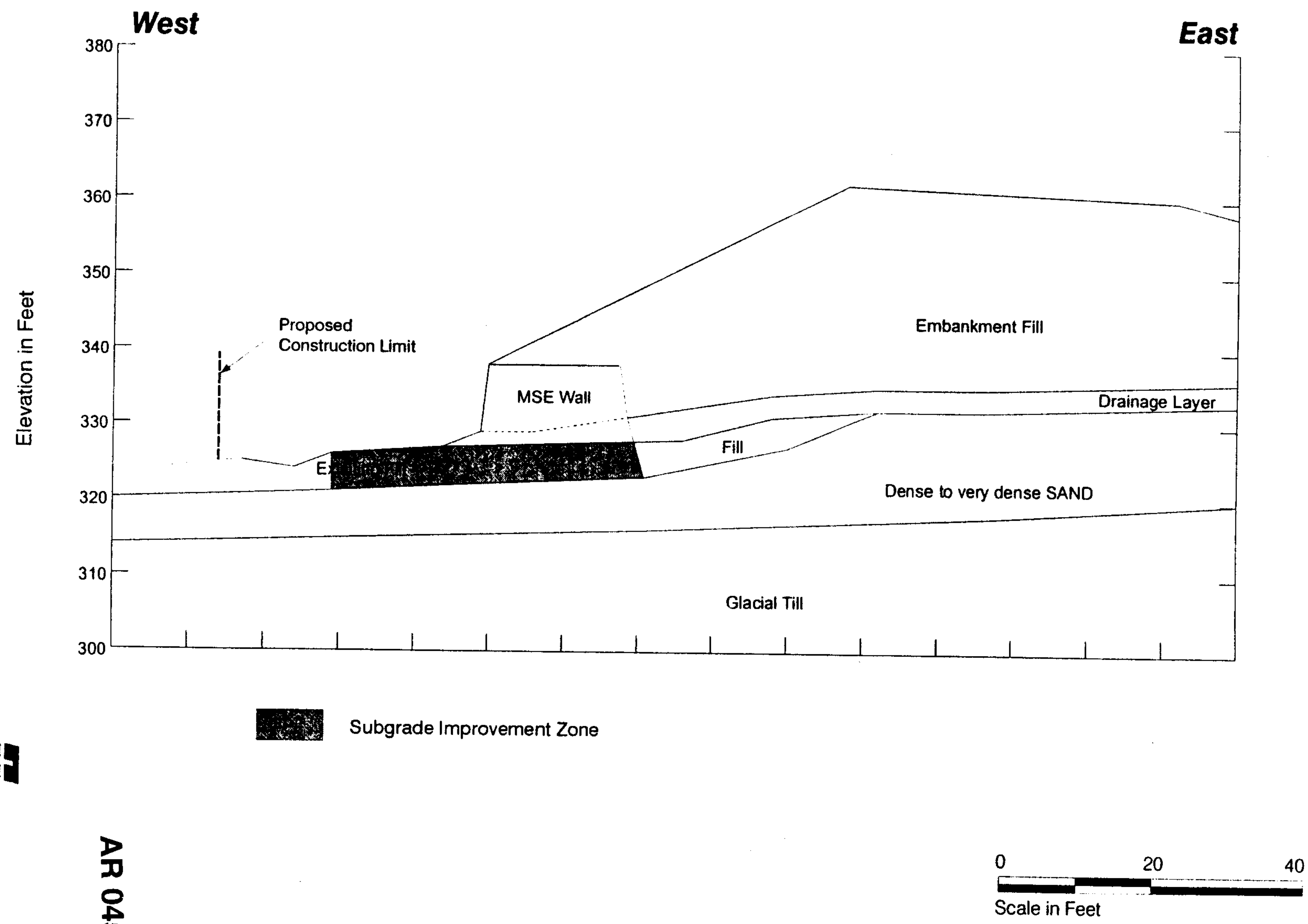
AR 045451

HARTCROWSER

J-4978-22 5/00

Figure B-19

South MSE Wall
Section 147+50



DTN 6/22/00 497822T.cdr

AR 045452

HARTCROWSER
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Figure B-20

APPENDIX C
SUMMARY OF SUBGRADE IMPROVEMENT ALTERNATIVES

APPENDIX C
SUMMARY OF SUBGRADE IMPROVEMENT ALTERNATIVES

This report summarizes information obtained from a focused review of geotechnical engineering literature on grade improvement alternatives for conditions at the Third Runway site. Sources of information are listed at the end of this Appendix.

We assumed the steps necessary for determining what subgrade improvement method (or cost effective combination of) should be used at Third Runway are as follows:

1. The first step is to determine which methods are feasible based on existing subsurface information. (This appendix.)
2. The second step is to determine how much strength gain is necessary for the required factor of safety at various areas of the site (see Appendix A).
3. The third step is to determine which feasible alternatives from Step 1 can achieve the strength gain necessary from Step 2, and meet any other project constraints.

Introduction of Methods

Table C-1 provides a summary of subgrade improvement methods and their general applicability to the soil conditions at the Third Runway MSE walls.

AR 045454

Table C-1 - Summary of Techniques – Generally from Rollings, 1996

Technique	Suitable Soils	Comments	Feasible	Process
Overexcavate and replace	All	Requires dewatering	Y	Soil replacement
Pre-compression	Normally consolidated fine-grained soils, fills, organics		N	Water removal, not effective for granular soils.
Vertical Drains	Typically fine-grained	Promotes strength gain by acceleration of consolidation	N	Water removal; not proven effective for avoiding liquefaction.
Dynamic Compaction; Heavy Tamping	Cohesionless soils		N	Soil strengthening by densification; not effective for cohesive soils.
Vibro-compaction; Compaction Piles; vibro-flotation	Cohesionless with <20% fines; <20-25 % [Mitchell]; <10-15 % [Terra]	Causes a reduction in void ratio and compressibility, and an increase in the shear strength	Y	Soil strengthening technique
Vibro-replacement (Stone Column)	Soft to medium stiff cohesive Soils and loose to medium-dense granular soils.	Displaces soil laterally to form a hole that is then filled with gravel	Y	Soil strengthening technique
Grouting	Course and fine-grained soils		N	Stabilization by densification and replacement, or permeation with cementing agent. Eliminated due to reduction in permeability of improved zone.
Deep Soil Mixing	Soft soils		N	Mechanically mixes soil with cementing agent. Eliminated due to reduction in permeability of improved zone.
Pinch Piles	Soft to stiff cohesive soils, loose to medium dense granular soils.	Increases soil density by displacement, increases shear strength by densification and replacement.	N	Soil strengthening technique. Eliminated because reduced permeability would impede groundwater flow; likely to be relatively high cost.

Each of these methods is briefly discussed below.

AR 045455

Alternative 1. Overexcavated and Replacement

Design Considerations

Soil Type Categorization

Overexcavation and replacement is applicable to all native soil types, particularly where the area to be improved is large enough to permit high production earth moving operations with large equipment. Replacement fill should be relatively non-silty (less than 12 percent fines overall) to avoid reducing permeability below that of adjacent native soils, and to avoid possible impacts to shallow base flow to adjacent wetlands.

Site Constraints

Dewatering is anticipated to be the main site constraint at the Third Runway. The technology to deal with groundwater for conditions at the wall sites is readily available but typically would require involvement of a specialty subcontractor. Dewatering can be accomplished at any time, but would best be accomplished during the summer construction season to minimize the volume of water handled (and to maximize availability of TESC facilities if needed to treat discharge water).

Sheet piling may be needed to reduce the volume of dewatering and/or to limit the extent of construction effects in peat soils.

Strength Gain

Strength gain will be a function of the replacement soil type and its degree of compaction. Reinforcing can be used to increase the strength of naturally occurring soil materials if needed.

Constructability

Overexcavation and replacement is highly constructible. Use of a specialty subcontractor for dewatering is common and unlikely to be a disincentive to prospective bidders.

Third Runway Feasibility

Overall the use of overexcavation and replacement is considered to be highly feasible for the Third Runway MSE wall subgrade improvements.

AR 045456

Alternative 2. Pre-Compression

Design Considerations

Soil Type Categorization

Typically limited to soft, normally consolidated fine-grained soils (silts and clays). Not applicable to densification of liquefaction-susceptible loose to medium-dense sands.

Site Constraints

Time required for consolidation or access to place preload fills is not anticipated to be a constraint at this site.

Strength Gain

Strength gain depends on the magnitude and duration of preloading, and is generally greatest for very soft soils.

Constructability

Preloads are readily constructible, contractors are familiar with the technique and no special means or methods are needed beyond those used in conventional earthwork.

Third Runway Feasibility

Use of preloads is generally not considered feasible for the Third Runway MSE wall subgrade improvements because the areas of cohesive soils that could be improved in this manner are relatively small and immediately adjacent to areas where other subgrade improvement methods must be used (i.e., removal of peat and mitigation of liquefiable granular soils). There is no clear benefit to the use of preloads under these circumstances, and the use of other alternatives could be less disruptive to the overall project schedule.

Alternative 3. Vertical Drains

Design Considerations

AR 045457

Soil Type Categorization

Typically vertical drains are used to increase shear strength and accelerate settlements due to consolidation of fine-grained soils, and in some circumstances may be used to improve dissipation of seismically induced pore pressures and to avoid liquefaction of granular soils.

Site Constraints

The interbedded fine and coarse soils in the NSA results in drainage path lengths on the order of a few feet. For fine-grained soils in the NSA, the vertical drains would have to be exceptionally closely spaced to avoid the problem of excess pore pressures induced by construction, and would not be practical for this purpose.

Strength Gain

The magnitude of strength gained depends on the consolidation process, i.e., on the magnitude and duration of (pre)loading, and is generally greatest for very soft soils.

Installation of vertical drains may or may not affect density of granular soils, depending on the type of drain. Sand or gravel drains installed by displacement techniques are discussed below under Alternatives 5 and 6. Wick drains are unlikely to affect density and thus would not materially change strength of granular soils (and may not be effective in preventing liquefaction).

Constructability

Installation of vertical drains requires a specialty subcontractor but is not uncommon. The general approach is considered to be readily constructible, although means and methods would vary depending on details of the drain method.

Third Runway Feasibility

Generally use of vertical drainage by itself is not considered sufficient to provide the subgrade improvement needed for this project.

- Time of consolidation may be reduced and rate of strength gain increased by use of vertical drains in fine grained soils, especially when combined with preloads, but this is not likely to produce significant benefits for this site because of the interbedding noted above.

- Use of vertical drainage in combination with densification or replacement of weak soils with stronger soils (i.e., stone columns) is considered more beneficial for the conditions at this site (see Alternatives 5 and 6).

Alternative 4. Dynamic Compaction

Design Considerations

Soil Type Categorization

To determine whether dynamic compaction is feasible, the soil type can be categorized as favorable (Zone 1), intermediate (Zone 2) or unfavorable (Zone 3), based on gradation, plasticity index (PI) and permeability (k).

- Zone 1: $PI = 0$; $k > 10^{-3}$ cm/s
- Zone 2: $0 < PI < 8$; $k = 10^{-3}$ to 10^{-6} cm/s
- Zone 3: $PI > 8$; $k < 10^{-6}$ cm/s

Use of dynamic compaction is most practical with Zone 1 soils. It may be used even when the water table is near the surface. For Zone 2 soils, energy will need to be applied in multiple passes or phases, allowing time for excess pore-water dissipation. For Zone 3 soils, especially with a high water table, dynamic compaction may not be feasible. No examples were found for fine and coarse-grained soils closely interbedded, such as in the NSA. However, note that a report from the Japanese Geotechnical Society does not recommend dynamic compaction for soils with fines content greater than 20 percent.

Site Constraints

Ground vibrations or lateral ground displacement could have an affect on adjacent properties, structures, and utilities. There are recommended guidelines for assessing suitability of this method, but less information on what peak velocities or vibrations can be expected for various soils and other site conditions.

- It is preferable to have utilities at least 25 feet away from the area compacted;
- Utilities can typically tolerate 3 to 5 inches per second particle velocities;
- Maximum depth of improvement is 35-40 feet; and
- Frequency of ground vibrations is in the range of 6 to 10 hz.

AR 045459

Strength Gain

Reported experience in predominantly granular soils indicates a 300 to 500 percent increase in SPT blow count, up to a maximum N value of about 25. This would typically produce soils in the upper end of the medium-dense range and probably would prevent liquefaction.

Constructability

Dynamic compaction would require involvement of a specialty subcontractor, but is not uncommon. The method is somewhat uncommon in this region, but has been widely used elsewhere and is relatively simple to implement. Verification of increases in density would require drilling or cone penetrometer measurements during construction.

Dynamic compaction is typically the least expensive of all methods, given the depth of improvement (greater than 10 feet) required. However, applicability for the Runway site is limited by the presence of soils with high fines content, especially where the water table is near the ground surface.

Third Runway Feasibility

Preliminary evaluation of the site soils shows that dynamic compaction may be possible in some areas of the Third Runway site, but is overall not recommended because of the fines in some of the predominantly sandy soils, and interbedding of coarse and fine-grained soils.

Alternative 5. Vibro-compaction

Summary. Vibro-compaction is a method of deep densification of *in situ* granular soils by means of rearranging loose cohesionless grains into a denser array by insertion of a vibratory probe. The two systems available are: vibro-flotation and the vibratory hammer probe.

Design Considerations

Soil Type Categorization

Vibro-compaction methods are best suited for densification of non-silty granular soils. Experience has shown that these methods are generally ineffective when the percentage by weight of fines exceeds around 10 to 25 percent (Task Force 27 report says 20 to 25 percent, Terra Systems states 10 to 15 percent, Rollings says 20 percent). Permeability of the soil materials containing greater percentages of fines is too low to allow the rapid drainage of the pore water that is required for densification, following liquefaction under the action of the vibratory forces. Also the structure of silty or clayey sands may be more difficult to disrupt owing to the cohesion contributed by the fines.

AR 045460

Site Constraints

Interbedded fine and coarse soils in the NSA and the presence of peat, silt and clay in the NSA and West Wall areas limits applicability of this method.

Strength Gain

Reported increase in densification as indicated by increase in SPT blow count is on the order of 300 to 500 percent. Area treated per hole is 3 to 20 m^2 (Rollings); 1.5 to 3 m O.C (Mitchell 1981). The typical depth range for vibro-compaction is from 10 to 50 feet. (Task Force 27). Relative densities in excess of 85 percent can be achieved, with relative densities of 70 percent being common. Allowable bearing pressures of up to 8 ksf are not uncommon (Terra Systems). There is no potential increase in strength for predominantly fine-grained soils.

Constructability

Vibro-compaction would require involvement of a specialty subcontractor. The method is somewhat uncommon in this region but has been widely used elsewhere. Verification of increases in density would require drilling or cone penetrometer measurements during construction.

Third Runway Feasibility

Vibro-compaction is not recommended as a feasible alternative for the Third Runway site due to the high fines content typical in some of the soils that require improvement (i.e., silty to very silty sands and layers of silts and clays interbedded with non-silty sands).

Alternative 6. Stone Columns Installed by Vibro-replacement

Summary: Vibro-replacement involves use of a vibrating probe to displace cohesive or cohesionless soils laterally, increasing density, and placement of a gravel column in the ground as the probe is removed.

Design Considerations

Soil Type Categorization

Stone columns can be used in loose sands and fine-grained soils with minimum undrained shear strength as low as about 150 psf; the practical upper limit, based on resistance to vibrator and economic considerations, is 1,000 to 2,000 psf.

Site Constraints

Installation of stone columns may be a problem where surficial fill or interbedded dense or gravelly soil layers are present. Pre-augering may be necessary to penetrate fill containing rubble or interbedded dense or gravelly soils.

Organic layers such as the peat encountered in the NSA and West Wall areas may not provide enough lateral support; large vertical deflections of the columns may result because of the high compressibility of the peat. Stone columns are not recommended when the thickness of the organic layer is greater than 1 to 2 stone column diameters.

Design Requirements

Designers need to consider the bearing capacity below the columns, the potential for bulging (unlikely to be a problem), and local bearing failure.

Strength Gain

For projects employing the usual stone column construction equipment and spacing, the relative density of the improved subgrade reportedly has been on the order of 70 to 85 percent. Other reports indicate a 200 to 400 percent increase in the SPT blow count. Studies show that a composite friction angle can be determined using an area replacement ratio, which enables significant strength gain through adjustment of column spacing and use of high-friction-angle gravel backfill.

Constructability

Use of stone columns would require involvement of a specialty subcontractor. The method is somewhat uncommon in this region, but has been widely used elsewhere and is relatively simple to implement. Verification of increases in density would require drilling or *in situ* testing such as cone penetrometer or load test measurements during construction.

Both "wet" and "dry" hole installation methods are used. Preparation of specifications should involve discussion with experienced contractors as to the best way to specify performance without unnecessarily eliminating prospective bidders. The wet hole method would require control of effluent from construction and treatment of silty water in project TESC facilities. Some

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groundwater displacement would result with the "dry" hole, bottom displacement method, but water control would likely be less of a problem.

Third Runway Feasibility

Installation of stone columns is considered very feasible for conditions at the Third Runway site. Construction of stone columns would mitigate potential seismic liquefaction through soil densification and dissipation of excess pore pressures through radial drainage to the stone columns. The method would also improve soft to medium-stiff silt and clay soils through the additional shear strength and stiffness contributed by the stone columns.

Alternative 7. Grouting

Summary. Four basic categories of grouting were considered:

- Intrusion - Cementitious grout is injected under relatively high pressures to form lenses, "roots" and sheets of grout that increase total stress, fill voids, and possibly consolidate or compact the soil.
- Compaction (Displacement) - Very stiff, low mobility soil cement mixes are injected at high pressures at discrete locations to densify soft or loose disturbed soils.
- Permeation (Flow into existing pores) - Low viscosity grout is introduced into soil pores (displacing soil and water), without any essential change in the original soil volume and structure.
- Jet - Mechanical injection of a high pressure grout jet that mixes with the soil in place, creating a "soil cement."

Design Considerations

Soil Type Categorization

The feasibility of using intrusion and permeation grouts is highly dependent on gradation and density of the soil. Both work relatively well in uniform soil deposits and are considerably less effective in stratified soils. Permeation grouting is severely limited by the presence of fines but works well in predominantly sand or gravel soils.

- Compaction grouting is highly effective in loose granular soils, but due to the slow dissipation of pore water pressure, it is less efficient in cohesive soils.

- Jet grouting can be accomplished in all soil types.

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- All grouting methods reduce soil permeability to some degree, and grouting is typically used to reduce or eliminate seepage.

Site Constraints

Jet grouting is anticipated to be effective in the interbedded coarse and fine-grained soils as well as in the zones that are predominantly one or the other soil type in all three wall areas. Jet grouting is not as likely to be effective in peat soils and would probably not eliminate the need to remove these soils in the NSA and West Wall areas.

Design Requirements

N/A

Strength Gain

Strength gain varies widely depending on the type of grout and method of installation.

Constructability

All types of grouting would typically require involvement of a specialty subcontractor. Jet grouting is somewhat uncommon in this region but has been used on local transit and structural foundation projects. Verification of increases in density would require drilling or cone penetrometer measurements during construction.

Third Runway Feasibility

Intrusion, compaction and permeation grouting are not considered feasible for the Third Runway site based on the range in soils that require subgrade improvement.

Jet grouting is not recommended because of the reduction in permeability that is anticipated, and the consequent potential for effects on base flow to wetlands adjacent to the MSE walls. Jet grouting is also reported to be relatively expensive compared to other methods of ground improvement, although this was not evaluated in detail for this report.

Alternative 8. Deep Soil Mixing

Summary. Deep soil mixing uses a trencher, auger drill or other mechanical means to disturb soil in place and mix it with grout to form a soil cement, without excavation and replacement.

Design Considerations

Soil Type Categorization

Deep soil mixing has reportedly been used in virtually all types of soil for remediation of environmental contamination ("soil stabilization"). There is little reported experience available on its use as a structural ground improvement technique.

Site Constraints

N/A

Strength Gain

Limited information suggests post-treatment soil shear strength on the order of hundreds of pounds per square inch can be obtained.

Constructability

Deep soil mixing would require involvement of a specialty subcontractor. Deep soil mixing is not uncommon for environmental remediation and therefore presumed to be feasible for structural ground improvement.

Third Runway Feasibility

Deep soil mixing is not recommended because of the reduction in permeability that is anticipated, and the consequent potential for effects on base flow to wetlands adjacent to the MSE walls.

Alternative 9. Pinch Piles

Reinforcement with piles to improve the stability of soil below fills, and to mitigate liquefaction potential, is sometimes used for waterfront fills and has been reported as a means to improve slope stability.

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Design Considerations

Soil Type Categorization

This method can work in all soil types.

Site Constraints

None presented in these references.

Design Requirements

Design for mitigation of liquefaction is typically based on estimation of the increase in density resulting from compaction to the native soil resulting from displacement by the piles.

Design for shear strength improvement involves:

- Use of stability analyses to determine the increase in resistance along a potential slip surface that would be required to provide an adequate factor of safety;
- Checking the potential for structural failure of the pile due to loading from the soil mass; and
- Checking the potential for plastic failure (i.e., flow of soil around the pile).

Strength Gain

The strength gain is controlled by the number and type of piles, and should be treated as a wall design rather than as a simple soil strength increase.

Constructability

Installation of pinch piles requires a specialty subcontractor but involves techniques that are commonly used in other types of construction. The general approach is considered to be readily constructible.

Third Runway Feasibility

This method may be expensive compared to other alternatives. Pinch piles are also not recommended because the solid piles would result in a net reduction in permeability and increased risk of off-site impacts from changes in base flow conditions.

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Appendix C References

Texts

1. "*In situ Soil Improvement Techniques*", AASHTO Task Force 27 Report, SA-92-041

2. "*Ground Improvement, Ground Reinforcement, Ground Treatment: Developments 1987-1997,*" Proceedings: Committee on Soil Improvement and Geosynthetics of the Geo-Institute of the ASCE in conjunction with Geo Logan '97, Logan, Utah, July 1997, NY, NY ASCE

3. "*Application of the State of the Art of Stone Columns – Liquefaction, Local Bearing Failure, and Example Calculations,*" by Richard D. Barksdale, REMR-GT-7 (Army); AD-A191606

4. "*Drainage Capacity of Stone Columns or Gravel Drains for Mitigating Liquefaction Hazards,*" Boulanger, Idriss, Hashash, and Schmidt, Geotechnical Earthquake Engineering and Soil Dynamics III, ASCE, Seattle, WA 1998 GSP No. 75, Volume 1 pp 678-690.

5. "*Design and Construction of Stone Columns,*" FHWA, Volume 1 is RD-83-026 (NTIS# PB84-190024) and Volume 2 is RD-83-027 NTIS# PB85-215416/AS)

6. "*Geotechnical Circular Number 1, Dynamic Compaction,*" FHWA-SA-95-037

7. "*Engineering Principles of Ground Modification*" by Hausmann, M. R., McGraw Hill, NY 1990

8. "*Remedial Measures against Soil Liquefaction: from Investigation and Design to Implementation,*" edited by the Japanese Geotechnical Society, A.A. Balkema, 1998

9. "*Improvements of Liquefiable Foundation Conditions Beneath Existing Structures,*" Ledbetter, REMR-GT-2 (Army). AD-A160-695

10. "*Ground Control and Improvement*" by Xanthakos, P., Abramson, L.W., Bruce, D., Wiley, NY

11. "*Construction of Fills*" by Monahan, E.J., Wiley, NY 1994

Journal Articles

1. *"Mechanical Behavior of Stone Columns under Seismic Loading,"* R.R. Goughnour, T.M. Lok and J.M. Pestana, 2nd International Conference on Ground Improvement Techniques, 7-9 October 1998, Singapore

2. *"Ground Improvement Systems in Combination,"* R.R. Goughnour, J.S. Jones, prepared for Task Force Twenty Seven of FHWA, 1968.

3. *"Performance of a Stone Column Supported Embankment,"* R.D. Barksdale and R.R. Goughnour, International Conference on Case Histories Geotechnical Engineering, St. Louis, MO, May 6-11, 1984.

4. *"Use of Prefabricated Drains with Storage Capacity for Reduction of Liquefaction Potential,"* R.R. Goughnour, J.M. Pestana, C.E. Hunt, and A. Kammerer, PEER Report, University of California at Berkeley.

5. *"Case studies of Reinforced Ground with Micropiling and other Improvement Techniques,"* Kamon, M., Prediction versus Performance in Geotechnical Engineering, Rotterdam, 1994 Balkema

6. *"Embankment on Sand Columns: Comparison between model and field Performance"*, B.B. Huat and F.H. Ali, Prediction versus Performance in Geotechnical Engineering, Rotterdam, 1994 Balkema
"Case studies on liquefaction-induced Ground Displacement During Past Earthquakes in Japan", M. Hamada, Prediction versus Performance in Geotechnical Engineering, Rotterdam, 1994 Balkema

7. *"Reinforced Embankment Behavior: Lessons from a Number of Case Histories"*, by R.K. Rowe, Recent Developments in Soil and Pavement Mechanics

8. *Geotechnical Material in Construction*, by M.P. Rollings and R.S. Rollings, Parts of Chapter 7; McGraw-Hill, c1996.

9. *Building on Soft Soils*, A.A. Balkema, Table of Contents and part of Chapter 7

10. *"Soil Improvement - State of the Art Report,"* by J.K. Mitchell, Proceeding of the Tenth International Conference of Soil Mechanics and Foundation Engineering, Stockholm, 1981

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11. *"Soil Improvement - General Report,"* by J.K. Mitchell and R.K. Katti, Proceeding of the Tenth International Conference of Soil Mechanics and Foundation Engineering, Stockholm, 1981

12. *"Design Approach and Field Control for Stone Columns,"* by K.R. Datye and S.S. Nagaraju, Proceeding of the Tenth International Conference of Soil Mechanics and Foundation Engineering, Stockholm, 1981

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APPENDIX D
INFORMATION ON DEWATERING SYSTEMS

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INFORMATION ON DEWATERING SYSTEMS

This appendix discusses the basis for the selection of the proposed dewatering system components and provides additional information that can be used to evaluate the over excavation and replacement alternative for subgrade improvement below the MSE walls.

This appendix includes a discussion of where the different methods of dewatering are considered to be appropriate, followed by information on anticipated rates of dewatering, start-up schedule, rates of groundwater extraction and water disposal.

Dewatering Methods

Pumped Dewatering Wells. Dewatering wells would be appropriate only in areas where an appreciable thickness of permeable subsoils exists below the proposed depth of excavation needed for subgrade improvement. To function effectively, the dewatering wells must be able to draw continuous water inflow from adequately permeable ground below the excavation. This in turn will depressurize to subgrade, preventing the occurrence of heave or boiling that could otherwise compromise the integrity of the subgrade material. For this reason the application of dewatering wells is probably limited to portions of the NSA and/or the South Walls.

- **North Safety Area.** Western end, in the area of Sections 101+20 to 105+20. Wells could be located on approximately 50-foot centers around the excavation (total of 18), with an average depth of 25 feet including 15 feet of well screen. Well casings/screens would typically be at least 6 inches in diameter, and equipped with 0.5 HP submersible pumps.

- **South Wall.** Sandy subsurface conditions with a shallow water table occur at Sections 141+78 to 142+72. Ten dewatering wells (constructed as above) would be required in this area.

Wells are unlikely to be appropriate or successful in other parts of the three wall sites, primarily because these areas are underlain by dense, low-permeability materials (i.e., glacial till or similar).

Vacuum Wellpoints. The installation of vacuum wellpoints is likely to be more appropriate in areas where the base of the excavation is within 10 to 20 feet of the ground surface, and where the bottom of the excavation is immediately underlain by dense, low-permeability materials such as glacial till. These conditions pertain in the following areas of the site:

- **North Safety Area.** Between Sections 107+19 to 110+47 for about 400 linear feet along the south side, and 650 feet along the north side of the excavation. Wellpoints will typically need to be installed at 10-foot spacing to be effective in dewatering these soil materials. Wellpoints can be installed by jetting through the surficial soils to the top of the glacially overridden soils at depths of between 20 and 25 feet.

- **West Wall.** The entire improvement area at the West Wall probably would be most appropriately dewatered using vacuum wellpoints, with the exception of a small portion at the northern end near Section 186+20. The total length of vacuum headers and installed wellpoints would be around 3,000 linear feet. Wellpoints typically need to be installed at 10-foot spacing to be effective in dewatering these materials. Wellpoints can be installed by jetting through the surficial soils to depths as needed, up to about 20 to 25 feet.

Wellpoints could also be installed on an ad-hoc basis as wet conditions are encountered in the excavations. Inflow rates are not expected to be very high (tens of gallons per minute), giving the option to advance the excavation by trenching and sumps in many areas, with the further option of installing wellpoints (or drilled wells, if necessary) as the excavation proceeds. This approach, however, may affect the excavation schedule by one or two days in each instance where wellpoints or wells need to be added.

Trenches and Sumps. Parts of the excavation may be most amenable to passive dewatering from within the excavation using a system of interception trenches and sumps that are dug and installed in advance of the bulk excavation, and are deepened as necessary to deal with the groundwater inflow. This method will be particularly appropriate where excavations are shallow; where the depth of excavation below the water table is on the order of 1 to 3 feet; and where the soil conditions exhibit moderate to low permeability, such as in the following areas:

- **North Safety Area.** In the area of Section 114+10 approximately 600 feet of drainage trench would be required. Additional trenching (up to 300 feet) may be required in the center of the excavation around Sections 105+20 to 107+19, where the excavation is relatively wide.

- **West Wall.** Drainage trenches can be used at the north end of the west wall excavation (total length: 600 feet), and may be required along the middle of the excavation if vacuum wellpoints are ineffective in removing remnant water at the base of the excavation (total length: 1,000 ft).

- **South Wall.** Drains could be used in a small area around Section 145+42, where wet conditions will be encountered at the base of the excavation (total length: 400 feet).

For cost estimating purposes, assume the trenches would be around 4 feet deep, excavated by trackhoe in wet conditions, and partially backfilled with gravel (e.g., drain rock) to at least half depth. Sumps would be composed of perforated oil drums or similar, installed at low spots along each trench (assume 1 sump per 100 feet of trench). A trench box may be needed in loose sandy soils below the water table. The time for excavation and installation of trench drains should be added to the excavation schedule.

Sheet Piling. The temporary installation of sheet piling or other form of ground support may be required in addition to the above dewatering methods. Sheet piling probably will be required where the NSA excavation is immediately adjacent to wetlands, to limit the volume of water extracted and the extent of wetlands disturbance.

Hart Crowser assumed that sheet piles are likely to be needed in the NSA between Sections 101+20 to east of 107+19, on the northwest side of the excavation, because of constraints on the area that can be disturbed for construction. Total length of the sheet pile cut-off was anticipated to be around 900 lineal feet. Nominal penetration depth would be on the order of 30 feet below ground level to obtain the maximum cut-off benefit, but this usually would be limited locally by dense to very dense soils. Intermittent soldier piles or lateral bracing to the sheet piles probably would be required because of the anticipated difficulty in penetrating the dense to very dense soils below the bottom of the excavation. Based on discussions with HNTB, Hart Crowser will further evaluate an open cut alternative.

Flow Rates

Flow rates from the various parts of the system have been estimated based on anticipated ground conditions:

- **Dewatering Wells.** Flow rates of between 2 and 5 gallons per minute (gpm) per well are expected, with a maximum of 10 gpm.
- **Wellpoints.** Each wellpoint should produce 1 or at most 2 gpm.
- **Trenches and Sumps.** Inflow rates to trenches and sumps will vary from less than 1 gpm to as much as 10 gpm per 100 linear feet of trench. Flow rates

could increase further during storms if surface water runoff can also drain into the trenches.

Note that 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.

Start-Up Schedule

Project schedules should include the time required to install and commission dewatering measures, and to make them fully operational prior to, or in concert with, the start of excavations. The following information can be used as a guide for estimating system installation time (per rig or team for each location where a pump is required).

- **Well Drilling.** Assume 3 to 4 hours per 30-ft deep well, installed using a truck-mounted rig or bucket-auger, including moves and set-up time.
- **Wellpoints.** Assume jetting wellpoints at the rate of 1 per hour.
- **Trenches and Sumps.** Assume 4 hours per 100 feet of trench and one sump.
- **Discharge System.** Additional time should be allowed to set up discharge facilities. Allow 4 hours for installing and hooking up each 100 feet of header pipe, and extra time for straight pipe runs to the discharge area(s). Dewatering pipe would not need to be buried except where surface pipe would impede Contractor work access. Additional time is also needed for installation of erosion protection in land discharge areas.

Provided that wells and wellpoints are installed before the excavations proceed below the water table, the initial drawdown of the water table will occur within hours of setting each part of the system into operation and effective dewatering to the subgrade elevation will occur within a few days. There will therefore be no need to delay excavation any further once installation is complete and the dewatering systems are set into operation.

Water Disposal

A number of options exist for water disposal:

- Storm water drains;
- Direct stream discharge via project TESC facilities;
- Land application; and/or
- Re-injection.

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Development and start-up water from wells and wellpoints (assume the first day's discharge) is expected to contain fine sand or silt, and would likely exceed turbidity limits for discharge to surface water. This water would need to be conducted to the site's stormwater settlement ponds. Specifications should require the contractor to use sandpacks and suitable well development methods to limit the time this discharge must be treated, but in the worst case it may require treatment throughout construction.

Water from trenches and sumps probably will remain turbid throughout construction, regardless of construction method, and should be routed to the TESC ponds.

Storm water drains may or may not be available locally for direct discharge of relatively clear, sediment-free water from the wells and wellpoint systems. Local wastewater management requirements may prohibit this approach, particularly during wet weather. Water-quality sampling probably would be required to prove that the discharge does not contain unacceptable levels of contamination or turbidity.

Direct Stream Discharge should be possible for relatively clear, sediment-free water after treatment, although such a decision probably will require concurrence from both Ecology and the Department of Fisheries and Wildlife. We anticipate that water-quality sampling will be required, and NPDES permit issues may be triggered by this action. Diffuser pipes can be used to spread the discharge along the creek and avoid erosion problems associated with point discharges.

Land Application of both clean and potentially turbid discharges is generally anticipated to be practical, especially where wetlands exist beyond the downslope limit of each excavation. A diffuser pipe system or irrigation sprinklers could be laid out on the upslope side of the wetland, or on gently sloping meadowland or other areas where surface flows could be accepted. Increased evaporation from the land surface during the summer months might be a concern with respect to temporary effects on base flow.

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Artificial Recharge by Re-injection could be considered in the absence of all other methods of water disposal, but would be relatively costly and may be difficult to permit. This approach could require additional explorations to identify local areas amenable to recharge. Problems of bacterial floc clogging and the entrainment of air bubbles can reduce recharge efficiency, and water quality could be a concern for Ecology regulators.

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