Appendix G
Site Geotechnical Reports

GEOTECHNICAL ENGINEERING SYUDY
BOW LAKE TRANSFER STATION IMPROVEMENTS
FACILITIES MASTER PLAN
KING COUNTY, WASHINGTON

November 16, 1993

Project No. 93112

Prepared for:

R.W. Beck and Associates

HONG WEST
& ASSOCIATES, INC.

HWA Project No. 93112
November 16, 1993

Mr. Karl J. Hufnagel, P.E.
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Subject: GEOTECHNICAL ENGINEERING STUDY
BOW LAKE TRANSFER STATION IMPROVEMENTS
FACILITY MASTER PLAN
KING COUNTY, WASHINGTON

Dear Mr. Hufnagel:

In accordance with your request, Hong West and Associates, Inc. has completed a geotechnical investigation for the proposed improvements associated with the Bow Lake Transfer Station Facility Master Plan Study, King County, Washington. Results of our investigation are presented in the accompanying report.

We appreciate the opportunity to provide geotechnical services on this project. Should you have any questions or comments, or if we may be of further service, please do not hesitate to call.

Sincerely,

HONG WEST & ASSOCIATES, INC.

Robert C. Metcalfe
Geotechnical Engineer

Sa H. Hong, P.E.
President

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BOW LAKE TRANSFER STATION IMPROVEMENTS
FACILITY MASTER PLAN
KING COUNTY, WASHINGTON

1.0 INTRODUCTION

Hong West & Associates, Inc. (HWA) has completed Task 1 of a two-tasked geotechnical investigation for potential improvements associated with the Bow Lake Transfer Station Facility Master Plan, King County, Washington. The purpose of this geotechnical engineering study was to evaluate the subsurface conditions at the site based on existing information and to provide preliminary geotechnical recommendations for the proposed building foundations and other aspects of site improvements.

Based on the results of our study, expansion for the proposed improvements is considered feasible from a geotechnical prospective provided the recommendations presented in this report are implemented during design and construction.

1.1 PROJECT DESCRIPTION

The project site is located on Orillia Road (Exit 152) east of and adjacent to Interstate Highway 5, (Vicinity Map, Figure 1). The existing facilities include a steel framed covered transfer station facility, trailer parking areas, loading pit ramps, weigh station, and access roads. Included under the covered roof area are tipping pads, trailer loading bay, office building, and refuse storage pit. The existing facilities are built over a solid waste landfill and the structures are supported on pile foundations and spread footings. The existing facilities and site conditions are shown on the Site and Exploration Plan (Figure 2).

We understand that potential improvements may include (1) constructing compactor units and conveyors on the north side of the site, (2) constructing trailer loading and parking areas on the north side of the site, (3) providing additional space for trailer parking on the south end of the site, (4) widening the east access road, (5) constructing a recycling area on the south end of the site, (6) constructing a new office building on the southeast side of the existing office building, and (7) installing a sewer line down the east fill slope. The widening of the east access road and the additional parking area on the south end of the site will require construction of a "sliver" fill on the existing slope. The recycling area will require some minor grading of the area and construction of cut and fill slopes. It is our understanding that filling of the existing trailer pit loading ramps on the north end of the existing building is also being considered.

1.2 AUTHORIZATION

A proposal for the performance of this geotechnical investigation was submitted by HWA on August 26, 1993. Authorization to proceed was received in a subconsultant agreement with R. W. Beck, dated September 17, 1993.

1.3 PURPOSE AND SCOPE OF WORK

The purpose of this geotechnical engineering study was to evaluate the existing conditions of the transfer station site with respect to the potential future improvements. In particular, emphasis was given to evaluating geotechnical constraints with respect to potential settlement, slope stability, and probable foundation types.

Our scope of work was divided into two tasks. Task 1, includes the results of our study based on previous subsurface investigations in the vicinity of the existing transfer station facility. Task 2 will be performed at a later date and will include additional explorations to provide information not available from the previous investigations. Construction of the additional trailer parking and a recycling area on the south side of the site will be addressed during Task 2 of our work.

The scope of work for this part of the project (Task 1) included the following:

1) Gather, review, and compile information from previous geotechnical and geologic reports pertaining to the proposed improvements.

2) Develop profiles based on the existing data. The profiles were drawn to describe the existing conditions of the soil and solid waste stratigraphy.

3) Evaluate the data compiled from the previous reports and perform preliminary engineering analyses with respect to the proposed improvements.

4) Prepare this report containing the results of our geotechnical engineering study, including descriptions of surface and subsurface site conditions, and a site plan showing locations of the previous investigations. The results of our engineering evaluation and analyses and our geotechnical engineering recommendations pertaining to the following items are presented:

a) Presence and effect of existing solid waste and fill soils, and their impact on design and construction of the proposed improvements,

b) Recommendations for proposed fill slopes. This includes conducting landslide and stability analyses for the potential expansion on the south side of the site,

c) Active and passive lateral earth pressures,

d) Foundation types for the proposed structures,

e) General site drainage considerations, and

f) Recommendations for geotechnically-related construction issues.

2.0 BACKGROUND

The Bow Lake Transfer Station Facility is located in King County approximately 14 miles south of Seattle, Washington, and about 1.5 miles east of the south end of Sea-Tac International Airport. The existing transfer station occupies approximately 8 acres and is built over an old landfill site. The site is situated near the top of the west Duwamish Valley slope approximately 200 feet above the Green River floodplain.

Elevations on site vary from about 270 feet mean sea level (MSL) in the southwest to about 242 feet MSL on the north. The south east corner of the property boundary is at an elevation of approximately 80 feet MSL. The eastern limits of the refuse may extend down the slope to at least an elevation of 160 feet MSL, although this has not been confirmed. The east property slope is generally at an inclination of about 3H:1V (Horizontal:Vertical). Refuse and fill material is approximately 45 feet thick on the east side of the site while no refuse was encountered in some explorations on the west end of the site, west of the west access road.

The following sections describe the information obtained from our review of available literature.

2.1 AERIAL PHOTOGRAPH REVIEW

Based on aerial photographs and previous reports, the site was originally an old burn dump as far back as the late 1930's or early 1940's. The site later served as a nonburning dump. Aerial photographs taken in 1936 show the site as a wooded hill side with no development.

However, photographs taken in 1946 show a substantial amount of refuse already placed. Refuse material was initially placed on the northwest side of the site and proceeded east to its present configuration. Based on the aerial photographs, the east expansion of the landfill appears to be separated and down slope from the original northwestern dump area. The landfill was closed in the mid 1960's, at about the time the original transfer station facility was constructed. The original transfer station facility was located on the east side of the existing facility and consisted of a timber supported transfer pit structure. The existing facility was constructed in the late 1970's. Aerial photographs appear to indicate that the landfill was constructed in a swale on the slope and directly on the original ground surface.

2.2 PREVIOUS SUBSURFACE INVESTIGATIONS

There have been several previous geotechnical investigations performed at the site. We have acquired seven geotechnical reports which document subsurface explorations at various locations across the site. These reports include: Dames & Moore (1965), Shannon & Wilson (1976 and 1977), Hong Consulting Engineers (1986), Hong Consulting Engineers (1987), Hong Consulting Engineers (1988), and Golder Associates (1992).

The subject of these previous reports as well as relevant borings and data are summarized below. The approximate location of the borings from each report are shown on Figures 2 and 3. Copies of the exploration logs are presented in Appendices A through F.

2.2.1 Dames & Moore (1965)

Dames & Moore conducted a geotechnical investigation in 1965 to explore the subsurface conditions at the site and to advise on the best location for siting a new transfer station structure, as well as to provide recommendations for support of the proposed structure. The investigation included five exploratory borings, two borings near the old dump bay area (east side of existing facility) and three borings 120 to 220 feet west of the old dump bay (southwest area of existing facility). The two borings drilled near the pit area ranged in depth from 41 to 47 feet below the ground surface, and the three borings west of the existing structure ranged in depth from 30 to 32 feet below the ground surface.

Dames & Moore concluded that the refuse and fill material encountered in the borings on the west side of the site ranged from 17 to 19 feet thick, and was 31 to 41 feet thick in the area of the pit. Groundwater was observed in one boring and it appeared to be a locally perched zone. Dames & Moore suggested that a new facility be built in the vicinity of the west borings and that the structure be supported by timber pile foundations, except for the pit area which could be supported on the underlying native soil deposits.

2.2.2 Shannon & Wilson (1976 and 1977)

Shannon & Wilson conducted a soil investigation in 1976 to explore the subsurface conditions and provide recommendations for use in design and construction of the existing transfer station facility. The 1977 report is a revised version of the 1976 report and reflects changes which were made regarding planned facility construction after the 1976 report was issued. The investigation included seven exploratory borings, six near the existing structure and one approximately 280 feet southwest of the existing structure at the location of the originally proposed weigh station (not constructed).

The six borings located near the existing structure encountered refuse material and fill which ranged from about 12 to 36 feet thick. The boring located southwest of the existing facility encountered approximately 5 feet of fill material. No consistent groundwater was observed, however they encountered occasional perched water zones within the fill materials. They concluded that the structure should be supported on pile foundations (steel, concrete, timber, or auger-cast) and/or spread footings depending on the location of the final grades for the structure. They expected the south wall, roof support columns, tipping pads, and storage pit floor to be supported on piles while the north wall and north roof support columns could be supported on spread footings bearing on dense native sand.

2.2.3 Hong Consulting Engineers (1986)

Hong Consulting Engineers (now Hong West & Associates, Inc.) conducted a geotechnical investigation in 1986 in support of planned underpinning of the southeast roof column support damaged due to settlement of the pile foundations. The investigation included four exploratory borings near the southeast corner of the building.

The borings were drilled to depths ranging from 39 to 59 feet below the ground surface. Boring 3 encountered an obstruction at 26.5 feet below the ground surface, and therefore is not shown on Figures 2 and 3. Fill materials encountered in the borings ranged from 24 to 28.5 feet thick. A groundwater observation well was installed in Boring 4 and a water table, possibly perched, was measured about 50 feet below the ground surface 24 hours after drilling. No measurable change was observed one week later.

Hong Consulting Engineers concluded that the pile foundations supporting the southeast roof column were not driven into the recommended bearing stratum and were terminated in the overlying fill soil. Settlement of the column occurred because the pile tip elevations were underlain by loose refuse materials. Recommendations were made to underpin the column with composite steel pipe pile and H-pile sections to minimize future settlements.

2.2.4 Hong Consulting Engineers (1987)

Hong Consulting Engineers performed pile driving inspections and additional subsurface explorations in 1987 for the underpinning of the southeast roof support column, pile support for the east tipping pad, and pile support for the expansion of the storage pit floor. As part of the southern storage pit floor extension project, three exploratory borings were drilled to determine subsurface conditions in the area.

The exploratory borings were drilled to depths ranging from 39 to 40 feet below the ground surface. No groundwater, refuse, or fill soils were encountered in these borings. All pile locations were predrilled about 25 feet prior to driving piles. At these locations the subsurface materials were logged based on soil cuttings obtained during predrilling.

Reported pile tip elevations for the southeast roof column support ranged from 215 feet to 217.5 feet. Pile tip elevations for the east tipping pad ranged from 208.5 feet to 216.5 feet. In the area of the pit floor extension on the south end of the storage pit, the pile tip elevations ranged from 211.5 feet to 224.5 feet.

2.2.5 Hong Consulting Engineers (1988)

Hong Consulting Engineers performed a geotechnical investigation in 1988 to determine the subsurface conditions and to provide recommendations for improvements to the storm drain system and expansion of the north trailer parking area. The improvements included new storm drain lines along the north, south, and east sides of the transfer station structure and a storm water holding tank on the east side. The trailer parking area north of the transfer station was to be expanded, regraded and repaved. The subsurface investigation included five exploratory borings drilled along the alignment of the proposed storm drain.

The two borings drilled in the north trailer parking area encountered between 12 and 18 feet of fill and refuse material. However, the three borings which were drilled to depths ranging from 39 to 49 feet below the ground surface, along the top of the east slope, encountered fill and refuse material ranging between 30 and 46 feet thick. Perched groundwater was encountered in some borings.

Hong Consulting Engineers concluded that the storm drain pipes would experience up to12 inches of additional settlement over the following 10 year period due to settlement of the loose fill and refuse materials.

2.2.6 Golder Associates (1992)

Golder Associates performed a subsurface investigation in 1992 to provide recommendations for the proposed relocation of an 8-inch water main on the west side of the transfer station facility. Golder provided recommendations with respect to extent of fill materials on the west half of the site, alignment of the proposed water main, and pipe foundation support. The subsurface investigation included five test pit excavations, three hand auger borings, and numerous ground penetrating radar traverses.

Golder Associates performed backhoe test pits and hand auger borings to better define the west and south limits of the refuse materials. The maximum depth explored was about 15 feet below the ground surface. Hand auger borings were all less than 4.5 feet deep. Perched water within the refuse material was encountered in one test pit excavation.

Golder Associates concluded that it would be feasible to relocate the water main along the west edge of the site and that it could be constructed in a trench 2 to 4 feet deep provided that some settlement could be tolerated. Golder also suggested that overexcavating the fill materials would further reduce settlements.

2.3 SETTLEMENT HISTORY

The landfill has settled considerably since it was closed in the 1960's. Shannon & Wilson reported that the King County Solid Waste Disposal Division (SWDD) monitored more than 3.6 feet of settlement between 1966 and 1976. The report also indicated that since 1975 landfill settlement was increasing at a maximum rate of about 0.02 feet per month or about 0.24 feet per year, if assumed constant. The settlement can be attributed to (1) loose placement of refuse, (2) decomposition of refuse materials, and (3) increased loading on the landfill refuse due to traffic and structures.

Hong Consulting Engineers (1986) reported structural damage caused by landfill settlement and found that the floor slabs in the vicinity of the southeast corner of the existing facility had settled up to 15 inches. The southeast column support piling had also settled up to 12 inches thus requiring underpinning to prevent future settlement.

In addition to the distressed conditions observed by Hong Consulting Engineers, Shannon & Wilson (1976) and Golder Associates (1992) reported substantial cracks in the pavement on the west side of the facility. The SWDD observed lateral movements on the order of 0.1 feet per year in the pavement on the west side of the site. The cracks were attributed to landfill settlement. Golder also reported long cracks in the pavement on the west, south, and north sides of the site.

Apparently it is regular practice to add fill and/or repave portions of the site to mitigate settlement problems. The north trailer parking lot was apparently repaved in 1990 to rehabilitate the pavement structure.

3.0 SITE CONDITIONS

3.1 GENERAL GEOLOGIC CONDITIONS

The project site lies along the west rim of the north-south trending Duwamish valley which occupies part of the Puget Lowland, a major linear depression trending northward between the Olympic Mountains on the west and the Cascade Range on the east. The Duwamish Valley lies within a glacially carved trough sculpted by glacial advances and retreats. The transfer station facility is located along the upper part of a moderately steep, east facing slope overlooking the Duwamish Valley. This slope is on the eastern side of a north-south trending elevated plain which separates the Duwamish Valley from the Puget Sound. The slope on which the transfer station facility is situated, as well as the elevated plain and valley, are relics of past glacial activity in the Puget Sound area (Waldron, 1962).

Kame Terrace deposits make up most of the Duwamish Valley west wall. These deposits were laid down by ice-marginal streams flowing between higher ground on one side and glacial ice on the other. Kame Terrace deposits primarily consist of silty sand and pebble to cobble gravel, but may locally consist of lenses and pods of till and lenses and beds of silt, sand, and clay. The Kame Terrace deposits reach a maximum elevation of approximately 400 feet near Angle Lake, about one mile west of the site. Glacial till, recessional outwash, and local areas of advance outwash make up most of the soil deposits on the elevated plain between Puget Sound and the Duwamish Valley.

Erosion since the last glaciation has modified the upland slopes and elevated plain somewhat. Surface drainage on the uplands is poorly integrated, local undrained depressions are numerous, and few streams have incised appreciably into the landscape. Angle and Bow Lakes are depressions apparently fed by local runoff only. A minor ravine incised by a small stream is located on the south side of the project area.

Numerous sand and gravel pits were mined along the west slope above the Duwamish Valley within the Kame Terrace deposits. Sand and gravel from these pits have been used extensively as aggregate and fill.

3.2 SURFACE CONDITIONS

The topography in the area has been extensively modified by previous landfill operations and construction of Interstate Highway 5. Based on review of aerial photographs, the landfill was constructed near the top of the west valley slope. Construction of I-5, west of the site, leveled the topography and waste fill was placed in areas northwest of the transfer station facility.

The slope on which the site is situated is at an inclination of about 8H:1V. However, the slopes on the east side of the transfer station facility are much steeper, about 3H:1V between elevation 160 and 240 feet (refuse fill slope), and about 2H:1V below elevation 160 feet. Local areas along the slope have inclinations near or steeper than 1H:1V.

A HWA geologist visited the site and noted several areas which exhibited settlement. The most obvious indications of recent settlement were visible in the pavement. Four areas with significant pavement cracks were noted during our site visit as listed below:

1. A large settlement crack, approximately 260 feet long, is oriented roughly parallel along the middle to south end of the west access road. The crack is generally ¼ to ½ inch wide and the east side of the crack has approximately 2 to 4 inches of relative downward movement.

2. Two cracks, totaling about 25 feet in length, are oriented subparallel to the south commercial tipping pad entrance road, approximately 120 feet south of the southwest corner of transfer station structure. The cracks are about ¼ inch wide and roughly 1 inch of relative vertical movement has occurred across the cracks.

3. Two cracks, totaling about 50 feet in length, are oriented roughly north-south near the top of the west trailer loading pit access ramp. The south end of the cracks terminates in a fatigue cracked pavement area.

4. Four areas along the east access road have significant settlement cracks. The cracks extend from east of the southeast side of the building northward to the east side of the north trailer pit loading exit ramp. These four areas typically consist of a series of smaller cracks oriented in a linear pattern roughly parallel to the west access road. The cracks are typically ¼ to ½ inch wide and up to 1 inch of relative vertical displacement was observed.

The cracks described, described as Items 1 and 3 above were also noted by Golder Associates (1992), however the cracks appear to have increased in length. Golder did not mention the cracks described as Items 2 and 4.

3.3 SUBSURFACE CONDITIONS

The subsurface conditions encountered during the previous soil investigations consisted of three general material types; fill soil, refuse, and Kame Terrace deposits, as described below.

3.3.1 Fill Soil

Fill soil was typically encountered at and within a few feet of the ground surface across the site. The fill soil encountered at the ground surface was probably part of the fill cover over the old landfill as well as new fill placed during construction of the transfer station facilities. The fill soil typically consists of loose to medium dense, brown, medium to fine sand, with gravel and silt. Some gravelly sand zones were also encountered. Fill deposits were encountered locally at depth within the refuse materials. The fill soil within the refuse was probably daily cover material placed during the old landfill operation, or fill soil from demolition or other activities.

3.3.2 Refuse

Refuse materials were encountered in most of the borings. The refuse deposit thickens from west to east across the site towards the east slope, with maximum measured refuse depth of 46 feet. The refuse typically consisted of varying amounts of paper, glass, plastic, metal, asphalt fragments, construction debris, and organic debris. Soil content varied from refuse with no soil, to refuse containing predominantly soil.

As part of our study, we estimated the bottom contours of the landfill, as well as the lateral extent of the refuse materials. The estimated lateral and vertical extent of the refuse materials is shown on Figure 3. These estimates were based on aerial photographs and the previous subsurface investigations. Bottom contours of the refuse material were only estimated in the areas where subsurface information was available. The vertical extent of the refuse and fill materials are also shown on the Generalized Geologic Cross Sections, Figures 4 through 9.

The limit of the refuse is generally bound by the west access road, the south fill slope, and the bottom of the east fill slope. The northern refuse limits extend beyond the north property boundary and subsurface information was not available in that area. Based on aerial photograph interpretations, the northern limits of the refuse may extend a few hundred feet beyond the north property line. The original landfill operation, northwest of the project boundary, is not shown on Figure 3 and was not included as part of this study.

The fill and refuse materials are typically 15 to 35 feet thick beneath the site. The west access road is underlain by up to 15 feet of material. The south access road is underlain by as much as 36 feet of fill and refuse material near the eastern end. The western end of the south access road is underlain by little or no refuse material, although up to 5 feet of fill soil was encountered. The central and west end of the north trailer parking area is generally underlain by 12 to 15 feet of loose material, while the east end is underlain by up to 30 feet of fill and refuse. The thickest refuse and fill deposits were encountered on the east side of the facility, under the east access road. In this area up to 46 feet of loose material was encountered near the middle of the east access road. The north and south ends of the east access road are underlain by up to 35 feet of fill and refuse.

The transfer station structure is underlain by thick deposits of fill in some areas, while other areas are underlain by thick deposits of refuse. As shown on Figure 8, the east side of the facility is generally underlain by about 20 feet of fill soil over approximately 10 to 12 feet of refuse. The thicker fill on this side of the facility is apparently associated with the original transfer station facility. The west side of the transfer station facility is typically underlain by less than 5 feet of fill soil over 15 to 20 feet of refuse, as shown on Figure 9. On the south side of the storage pit glacial deposits were encountered at higher elevations than other areas. The glacial deposits were encountered at about elevation 240 feet near the bottom of the storage pit.

3.3.3 Kame Terrace Deposits

Glacial deposits were encountered in all the subsurface explorations which penetrated through the fill and refuse materials. The glacial deposits may be identified as Kame Terrace deposits, however several of the previous reports identify them as glacial outwash and/or till deposits. The glacial deposits typically consisted of medium dense to very dense, gray, medium to fine sand. Some of the sand deposits also had varying amounts of silt and gravel. Generally the upper 5 to 10 feet of the glacial deposits varies from medium dense to dense, while the deposits below this are typically dense to very dense.

From the reported subsurface information for each boring we were able to estimate the contour lines for the bottom surface of refuse materials. The contours of the lower limits of refuse are shown in red on the Refuse Limits and Cross-section Location Plan, Figure 3. Six subsurface profiles, Figures 4 through 9, were constructed across the site based on information from the previous reports. The locations of the cross-sections are also shown on Figure 3.

3.4 GROUNDWATER

Most of the reports indicate that local perched groundwater zones exist throughout the fill and refuse material. As discussed previously, Hong Consulting Engineers (1986) Boring No. 4, encountered a water table which was measured about 50 feet below the ground surface 24 hours after drilling and no measurable change was observed one week later. However, this may also be a perched groundwater system.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 SETTLEMENT

In our opinion, and based on site observations, substantial future settlements should be expected and measures should be taken to accommodate these settlements. Pavements (existing and new) over refuse areas will probably continue to experience distress. Buildings or structures sensitive to settlement should be founded on the dense glacial deposits. Less sensitive structures and some utilities which can tolerate larger settlements may be placed on or within the loose fill or refuse materials.

4.2 FOUNDATIONS

Based on the final configuration of the planned improvements, new structures and buildings should be supported on shallow spread footings or pile foundations. We anticipate that construction of a new office building near the southeast corner of the site will require the use of pile foundations due to the depth of refuse and fill materials. Construction of new facilities and/or expansion of the existing facilities on the north end of the existing transfer station structure may require the use of pile foundations or both pile foundations and spread footings.

4.2.1 Spread Footings

Spread footings may be used in areas where shallow excavations extending to dense glacial deposits are feasible. In the vicinity of the trailer loading pit on the north end of the building, native soils may be located at shallow depths relative to the elevation of the existing trailer loading area. The native soils may be encountered at elevations between 224 and 235 feet. The fill and refuse deposits become deeper along the east side of the north end of the existing structure. Refuse in this area may be encountered at approximately elevation 216 to 220 feet.

In areas where native glacial deposits exist at shallow depths, it may be possible to support structures and retaining walls on spread footings. If the footings are founded in undisturbed dense glacial deposits, we anticipate that maximum allowable soil bearing pressures between 2,500 pounds per square foot (psf) and 4,000 psf may be appropriate. The footings should be a minimum of 12 inches wide and embedded a minimum of 18 inches below the lowest adjacent grade.

4.2.2 Deep Foundation Alternatives

Pile foundations will be required in areas underlain by thicker deposits of refuse and/or loose fill soils. We anticipate that pile foundations in conjunction with grade beams will be necessary to support new structures located near the southeast corner of the existing structure and on the north end of the existing facility. The pile foundations should be embedded at least 5 to 10 feet into the dense glacial deposits.

Based on the existing information, we anticipate that pile tip elevations in vicinity of the southeast corner of the existing structure should be at elevations between approximately 210 and 215 feet. The top of the glacial deposits varies between elevations of about 220 and 230 feet. The glacial deposits are encountered at higher elevations, approximately 240 feet, near the center of the south side of the structure. Shallower pile tip elevations may be sufficient, depending on the configuration of potential buildings in this area.

Structures located north of the existing transfer station structure (north trailer parking area) should also be founded on pile foundations. The glacial deposits were encountered at elevations between approximately 215 and 230 feet in this area. The glacial deposits contact appears near elevation 230 feet on the west side of the north parking area and remains relatively horizontal eastward towards the middle of the parking area before dropping off to elevation 215 feet on the east side of the parking area. Pile tip elevations will likely range between 205 and 220 feet in this area.

Downdrag loads will develop along the sides of piles due to continuous settlements within the refuse and fill soil. The allowable load carrying capacity of the piles will be reduced by the downdrag loading. We recommend that downdrag loads be subtracted from the pile capacities prior to applying a factor of safety to determine the allowable pile capacity. If the existing trailer loading pit access ramps are to be filled as part of potential site improvements, we recommend that the fill be placed prior to installing pile foundations. Lateral loading of piles, due to lateral movements of the landfill materials after pile installation, should also be considered in design.

Obstructions may be encountered in the landfill refuse during pile driving or placement. Obstructions were encountered in some of the previously reported subsurface explorations.

If obstructions are encountered during driving or placement of piles, then the pile locations should be moved and additional piles driven, or the pile locations should be predrilled. Piles which meet refusal prior to reaching their recommended pile tip elevations should not be used as foundation support.

Piles types may consist of driven timber or precast concrete piles, driven steel pipe or H-piles, auger-cast piles, or drilled caissons. The foundation support for the existing structures originally consisted of treated timber piles, however the southeast roof support column was later underpinned with steel pipe piles. Where the storage pit area was extended southward, the structure was supported by steel pipe piles.

Driven Piles

Treated timber piles typically have allowable capacities between 10 and 30 tons per pile. Obstructions may cause difficulties when installing timber piles because high driving stresses may be induced and damage to piles may result. Timber piles are probably the most economical driven pile type. We recommend that timber piles be considered due to their economy.

Precast concrete piles typically have allowable capacities between 30 and 50 tons. It is possible to damage precast concrete piles during driving due to high stress concentrations. This pile type is not recommended for the site conditions, but could probably be installed with pre-boring techniques.

Steel pipe and H-piles typically have allowable capacities between 40 and 80 tons, depending on size, depth, and soil conditions. Steel piles can be subjected to higher driving stresses and therefore light to moderate obstructions may be penetrated. However, steel is vulnerable to corrosion and therefore we recommend that steel piles should not be used for the design.

Auger-Cast Piles

Auger-cast piles typically have allowable capacities between 20 and 50 tons, depending on diameter. The piles can be reinforced with steel H-sections or rebar cages. When large voids exist in the solid waste or when the refuse is porous, grout can migrate into the waste and create high negative friction along the pile. However, auger-cast piles are probably the most economical pile type, therefore we recommend that auger-cast piles be considered.

Drilled Caissons

Drilled caissons are subject to sloughing of side wall materials and are required to be cased in order to clean the bottom for end-bearing piles. Perched water and gases within the refuse and fill materials can also present major construction problems. Therefore, we do not recommend use of drilled caissons or any other type of bored piles.

4.3 RETAINING WALLS

Where retaining walls are used on the site, they should be founded upon structural fill, glacial deposits, or piles. Retaining walls should be provided with drainage systems to prevent buildup of hydrostatic pressure. The walls should be designed using lateral earth pressures appropriate for the materials retained.

It may be possible to construct smaller noncritical retaining walls, which can accommodate a significant amount of settlement, directly on the refuse deposits.

4.4 CONCRETE SLABS-ON-GRADE

Concrete slabs-on-grade will undergo potentially severe settlements if constructed directly on refuse and loose fill materials. In areas where thick refuse and loose fill deposits exist, the concrete slabs should be pile supported with grade beams or structurally supported by building foundations.

In areas where the slabs-on-grade can be supported by properly compacted fill over glacial deposits, then prior to constructing concrete slabs-on-grade, surficial soils should be scarified and properly compacted. Scarification and compaction will not be required if floor slabs are to be placed directly on recently-placed compacted fill.

4.5 SLOPES

Fill slopes constructed over refuse and loose fill materials will increase the overburden pressure on the underlying compressible materials and increased settlement will likely occur. Potential widening of the east access road may require several feet of additional fill on the east side of the road (Figure 4). This additional fill placement will likely increase settlement in the immediate vicinity of the fill and differential settlement between the new section of the road and the original section may occur. Settlement can be reduced by minimizing fill heights.

We evaluated the potential widening of the east access road by 20 feet, and in our opinion, the slope should be relatively stable when considering overall (gross) stability. However, lateral movement induced by fill placement may occur. In our opinion, the lateral movement should not be detrimental to the transfer station facility. Some maintenance of the access road should be anticipated including possible re-paving and/or re-grading at periodic intervals. Planned fill slopes should be thoroughly evaluated once schematic designs are finalized.

Fill should be properly placed and compacted, and the slope should be properly benched prior to placing fill as part of the road widening.

4.6 FILL AREAS

Areas which may receive fill as part of the potential improvements will also promote increased settlement. Fill areas should be kept to minimal heights if possible. Areas which may receive substantial amounts of fill should be evaluated carefully to determine the effects of additional settlement with respect to the existing structures.

If the loading pit access ramps are backfilled with fill soils as part of the improvements on the north side of the existing transfer station facility, then settlement should be expected. Differential settlement between the fill soils and adjacent refuse materials may occur. In areas where fill soils may overly the access road fill slopes, substantial settlement and an increased rate of settlement of the refuse material may result.

4.7 SEWER LINE

It is feasible to construct a sewer line down the east slope provided appropriate geotechnical recommendations are followed. Part of the sewer line will be embedded in solid waste fill. Not only will the landfill continue to settle, but eastward lateral movements near the top of the slope can potentially separate pipe joints if not accounted for during design. Therefore, it should be designed to be as flexible as possible to accommodate potential future settlements on the east slope. The sewer pipe should also be designed to accommodate some lateral movements near the top of the embankment by employing telescoping joints. Periodic pressure testing may also be necessary.

4.8 EARTHWORK

Site preparation should begin with the removal of all deleterious matter, asphalt, concrete, and vegetation, and exportation of the debris from the construction area. We recommend that the upper 2 feet below pavement sections consist of compacted granular structural fill.

We anticipate that the on-site materials can be excavated with light to moderate effort using heavy duty construction equipment. It is anticipated that perched groundwater will be encountered randomly throughout the refuse materials.

4.9 DRAINAGE PROVISIONS

Adequate drainage provisions, both short and long term, should be incorporated into project design and construction. Measures should be taken to avoid ponding of surface water during construction. If possible, it is recommended that grading operations be performed during the drier summer months.

Water should not be allowed to pond on pavements or adjacent to building foundations. Existing cracks in the pavement should be repaired to prevent drainage of surface water into the landfill which can promote increased settlement, as well as additional leachate generation. Adequate surface gradients and drainage systems should be incorporated into the final design to conduct surface runoff away from structures and pavements and into swales or other controlled drainage devices. Vegetative erosion protection should be established. Drainage systems should be maintained in the future by the owner.

4.10 GAS VENTING

Organic waste materials deposited in the landfill will undergo anaerobic decomposition soon after placement. Methane and other gases are commonly generated as a by product from the decomposition process. High concentrations of methane gas can accumulate in voids under structures and buildings and if trapped, explosions and fires can result. Therefore, we recommend that appropriate measures be included in the design for venting gases which may accumulate beneath structures and buildings constructed over refuse materials.

5.0 UNCERTAINTY AND LIMITATIONS

This geotechnical literature search and review was planned and conducted in accordance with generally accepted engineering standards presently practiced within this geographic area. The conclusions, recommendations and opinions presented herein are (1) based upon our evaluation and interpretation of previous findings by others, (2) based upon an interpolation of subsurface conditions between the previous exploratory borings, and (3) based upon our understanding or potential future site improvements as described herein. The recommendations contained in this report are also based on the assumption that the soil and refuse conditions, as depicted in the previous reports, are representative of actual conditions throughout the subject site. This report should be used for planning purposes only. A comprehensive geotechnical engineering investigation which addresses specific future improvements should be used for final design.

Experience has shown that subsurface soil and groundwater conditions can vary radically over small distances. Inconsistent conditions can occur between explorations and not be detected by a geotechnical study.

The findings and recommendations of this report were prepared in accordance with generally accepted professional principles and practices in the fields of geotechnical engineering and engineering geology. No warranty, expressed or implied, is made.

6.0 REFERENCES

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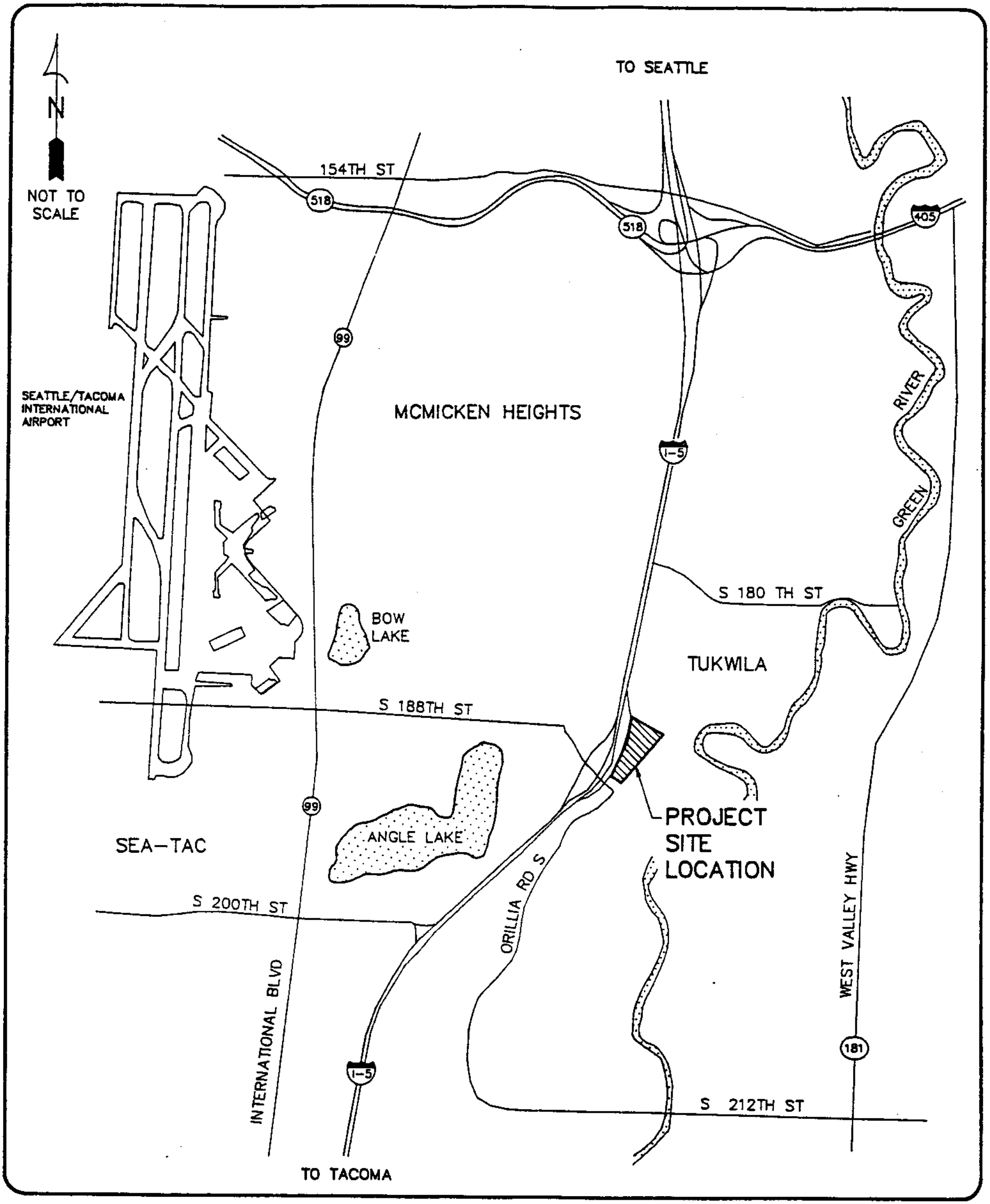


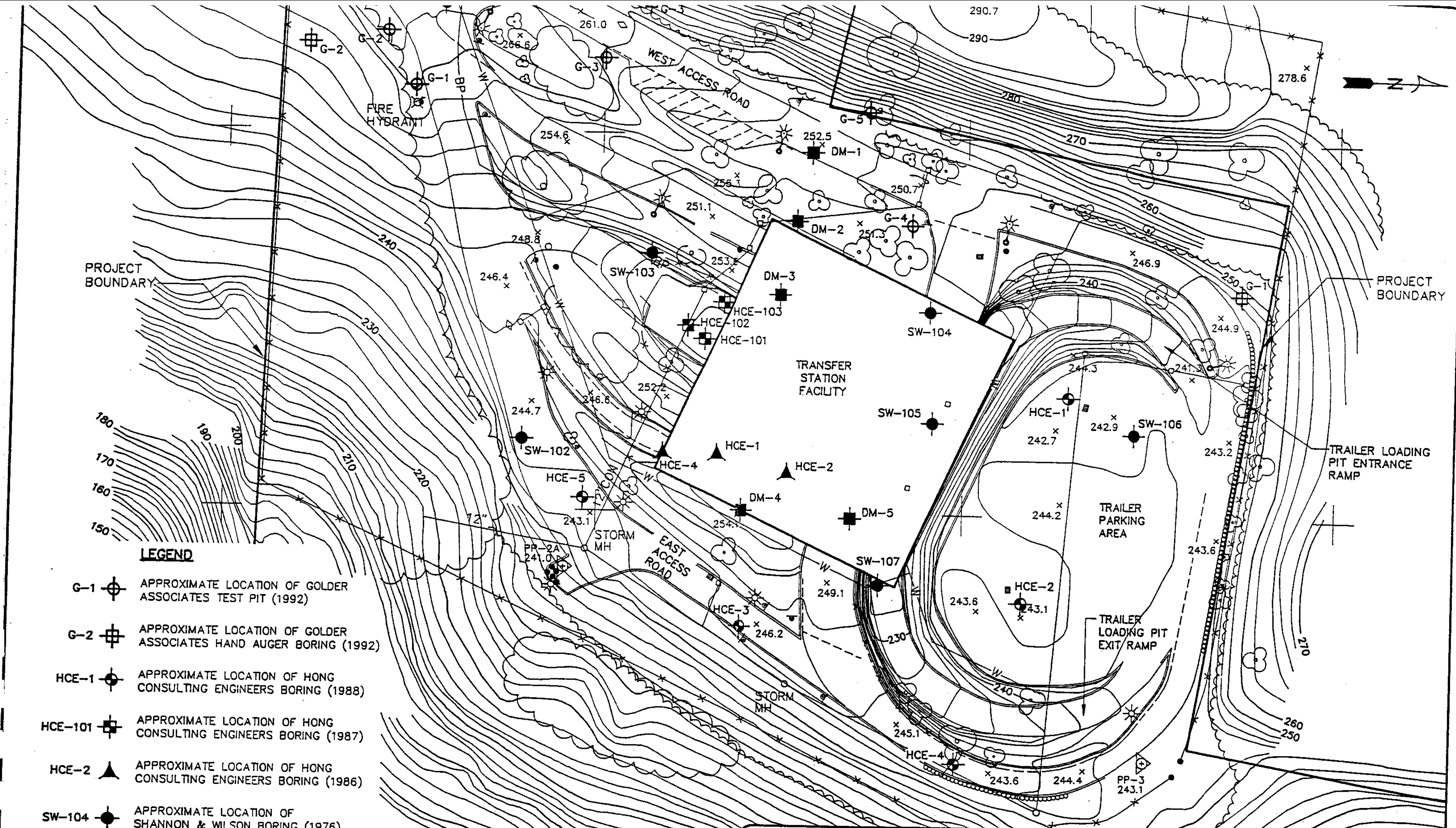
HONGWEST & ASSOCIATES, INC.

BOW LAKE TRANSFER STATION IMPROVEMENTS

VICINITY MAP

PROJECT NO.: 93112 FIGURE: 1

93112-A-004.1



LEGEND

G-1	APPROXIMATE LOCATION OF GOLDER ASSOCIATES TEST PIT (1992)
G-2	APPROXIMATE LOCATION OF GOLDER ASSOCIATES HAND AUGER BORING (1992)
HCE-1	APPROXIMATE LOCATION OF HONG CONSULTING ENGINEERS BORING (1988)
HCE-101	APPROXIMATE LOCATION OF HONG CONSULTING ENGINEERS BORING (1987)
HCE-2	APPROXIMATE LOCATION OF HONG CONSULTING ENGINEERS BORING (1986)
SW-104	APPROXIMATE LOCATION OF SHANNON & WILSON BORING (1976)
DM-3	APPROXIMATE LOCATION OF DAMES & MOORE BORING (1965)

0 30 60 120

SCALE: 1"=60'

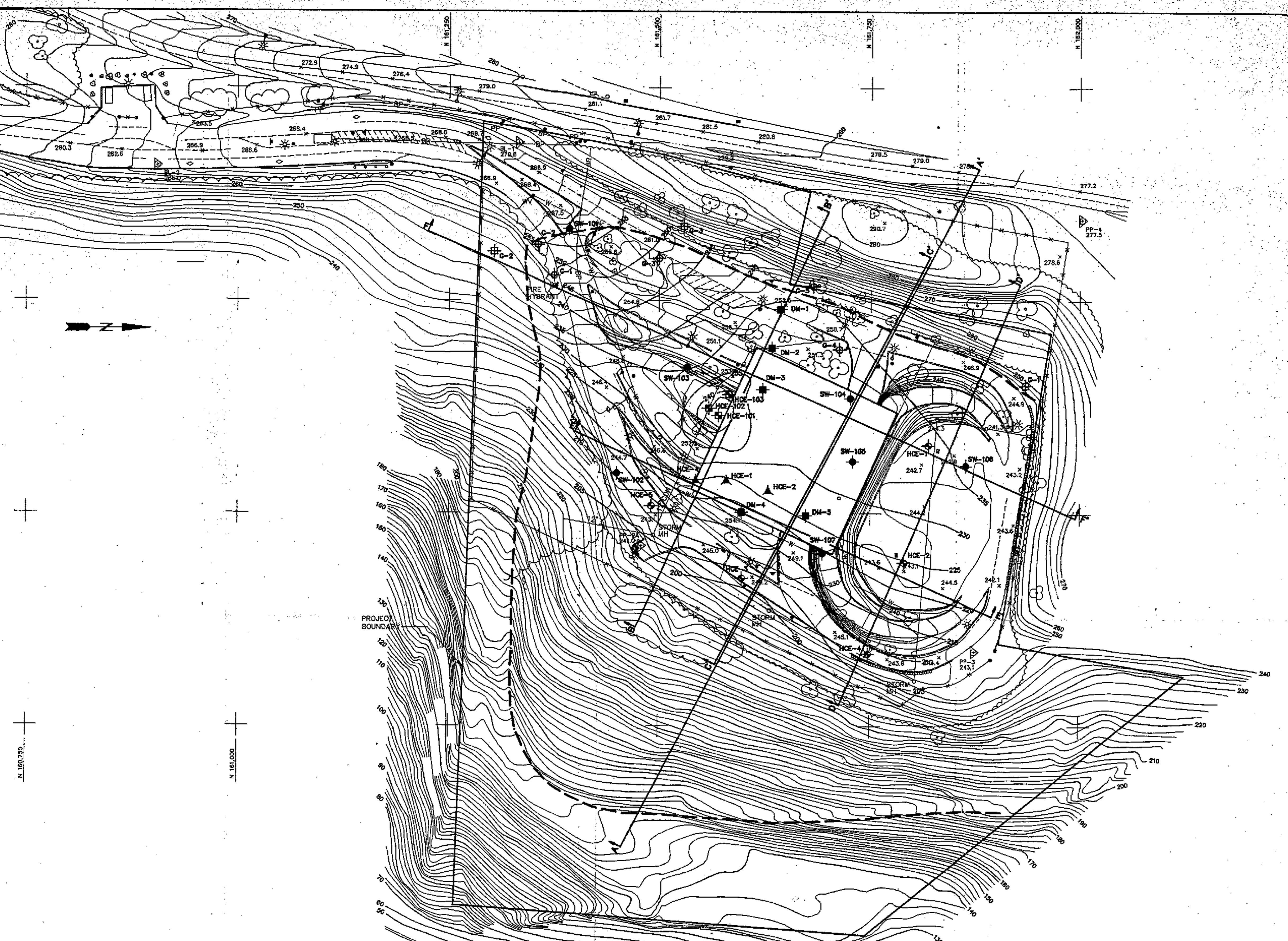
93112-8-011.1

HONG WEST & ASSOCIATES, INC.

BOW LAKE TRANSFER STATION IMPROVEMENTS

SITE AND EXPLORATION PLAN

PROJECT NO.: 93112 FIGURE: 2



LEGEND

G-1 — APPROXIMATE LOCATION OF GOLDER ASSOCIATES TEST PIT (1992)

G-2 — APPROXIMATE LOCATION OF GOLDER ASSOCIATES HAND AUGER BORING (1992)

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HCE-101 — APPROXIMATE LOCATION OF HONG CONSULTING ENGINEERS BORING (1987)

HCE-2 — APPROXIMATE LOCATION OF HONG CONSULTING ENGINEERS BORING (1986)

SW-104 — APPROXIMATE LOCATION OF SHANNON & WILSON BORING (1976)

DM-3 — APPROXIMATE LOCATION OF DAMES & MOORE BORING (1965)

A — A' CROSS-SECTION LOCATION

255 GENERALIZED SOLID WASTE FILL BOTTOM CONTOURS

ESTIMATED LIMITS OF REFUSE

0 30 60 120
SCALE: 1"=60'

HWA
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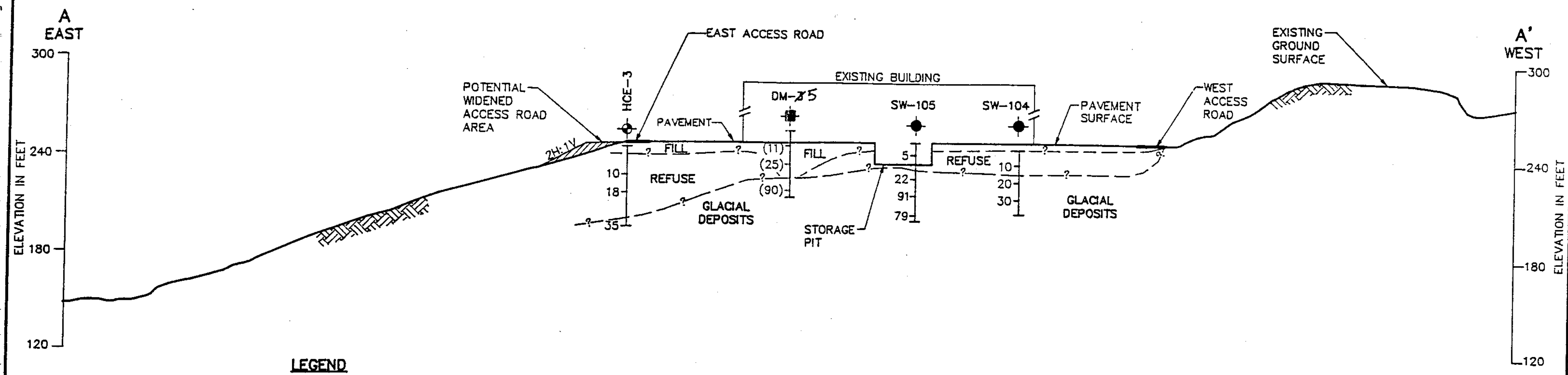
BOW LAKE TRANSFER STATION IMPROVEMENTS

REFUSE LIMITS AND CROSS SECTION LOCATION PLAN

PROJECT NO.: 93112 FIGURE: 3

93112-B-001.1



LEGEND

APPROXIMATE LOCATION OF EXPLORATION

10 — STANDARD PENETRATION TEST (SPT) WITH N-VALUE AT DEPTH INDICATED

(18) — BLOW COUNT USING 3.25" O.D. SPLIT BARREL SAMPLER DRIVEN WITH 260 lb HAMMER FALLING 24".

— —?— — ASSUMED GEOLOGIC CONTACT FOR AREA BETWEEN EXPLORATIONS (FOR GRAPHICAL PURPOSES ONLY)

POTENTIAL FUTURE FILL AREA

SCALE: 1"=60'

NOTE:
THIS GENERALIZED SOIL PROFILE WAS COMPILED FROM AVAILABLE SUBSURFACE INFORMATION IN THE VICINTY. IT IS INTERPRETIVE IN NATURE; ACTUAL SOIL CONDITIONS BETWEEN BORINGS MAY VARY FROM THOSE SHOWN.

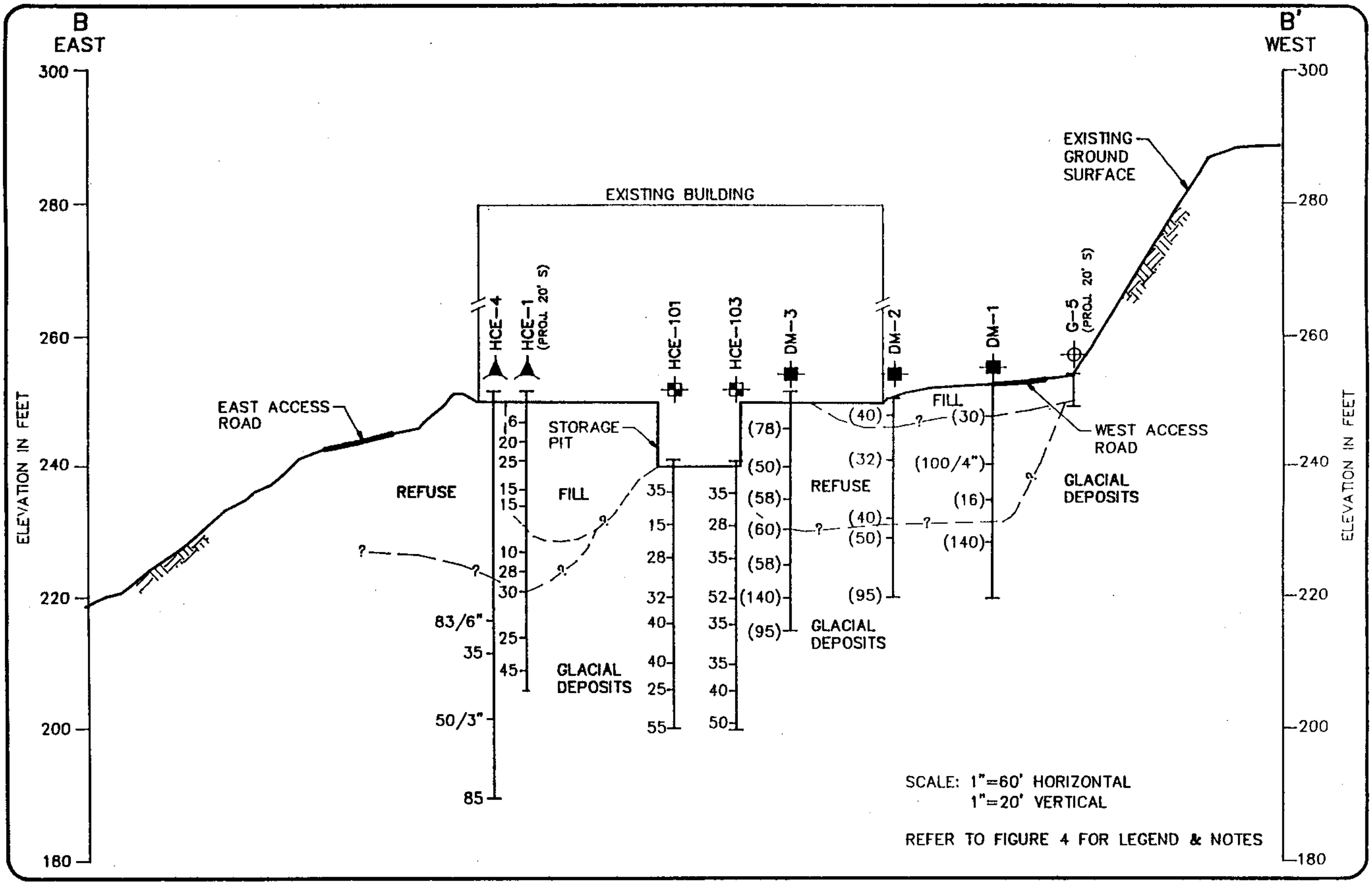
HWA HONGWEST & ASSOCIATES, INC.

BOW LAKE TRANSFER STATION IMPROVEMENTS

GENERALIZED GEOLOGIC CROSS SECTION A–A'

PROJECT NO.: 93112 FIGURE: 4

93112-8-010.2



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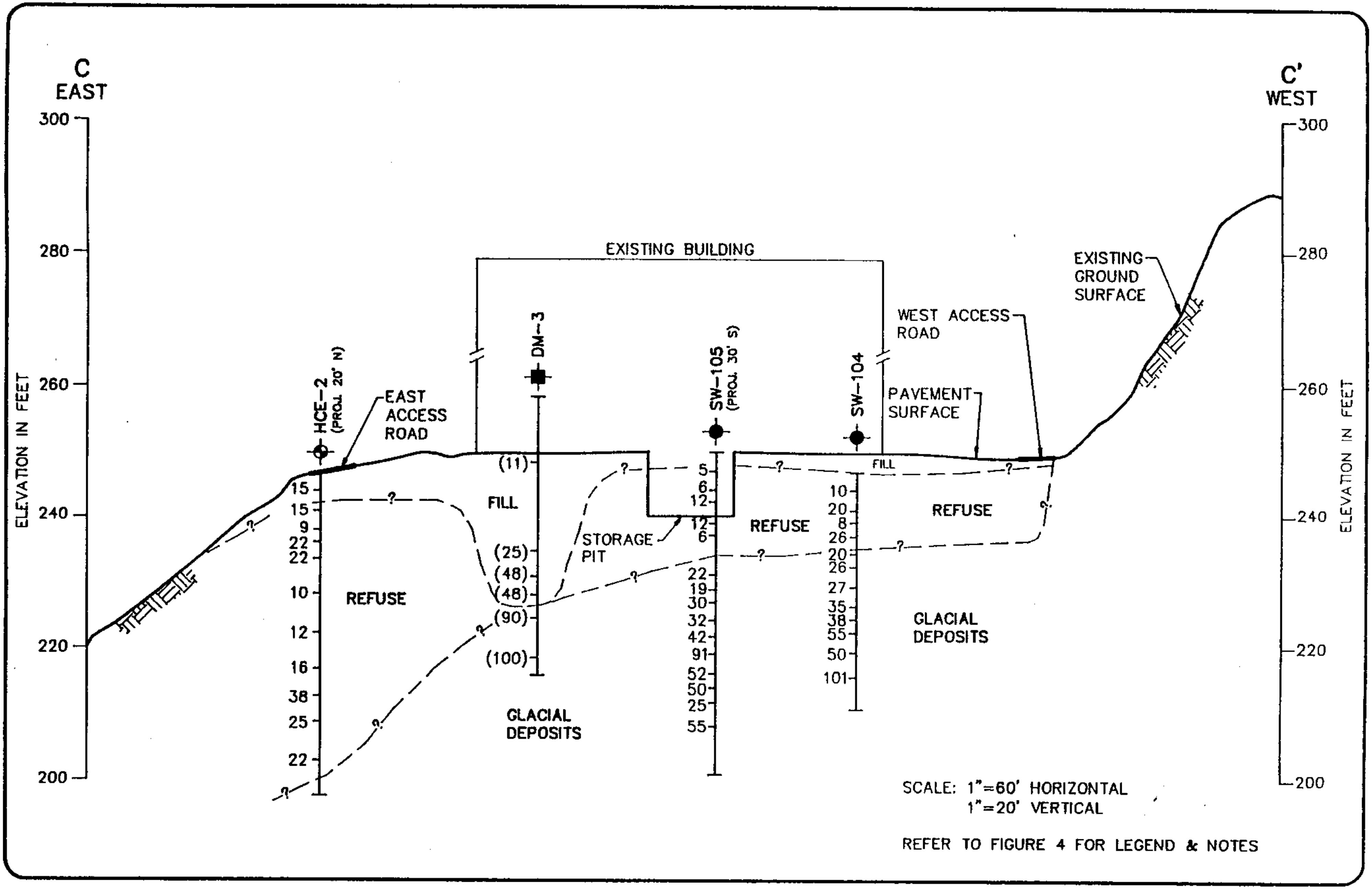
GENERALIZED GEOLOGIC CROSS SECTION B–B'

PROJECT NO.: 93112 FIGURE: 5

93112-A-005.2



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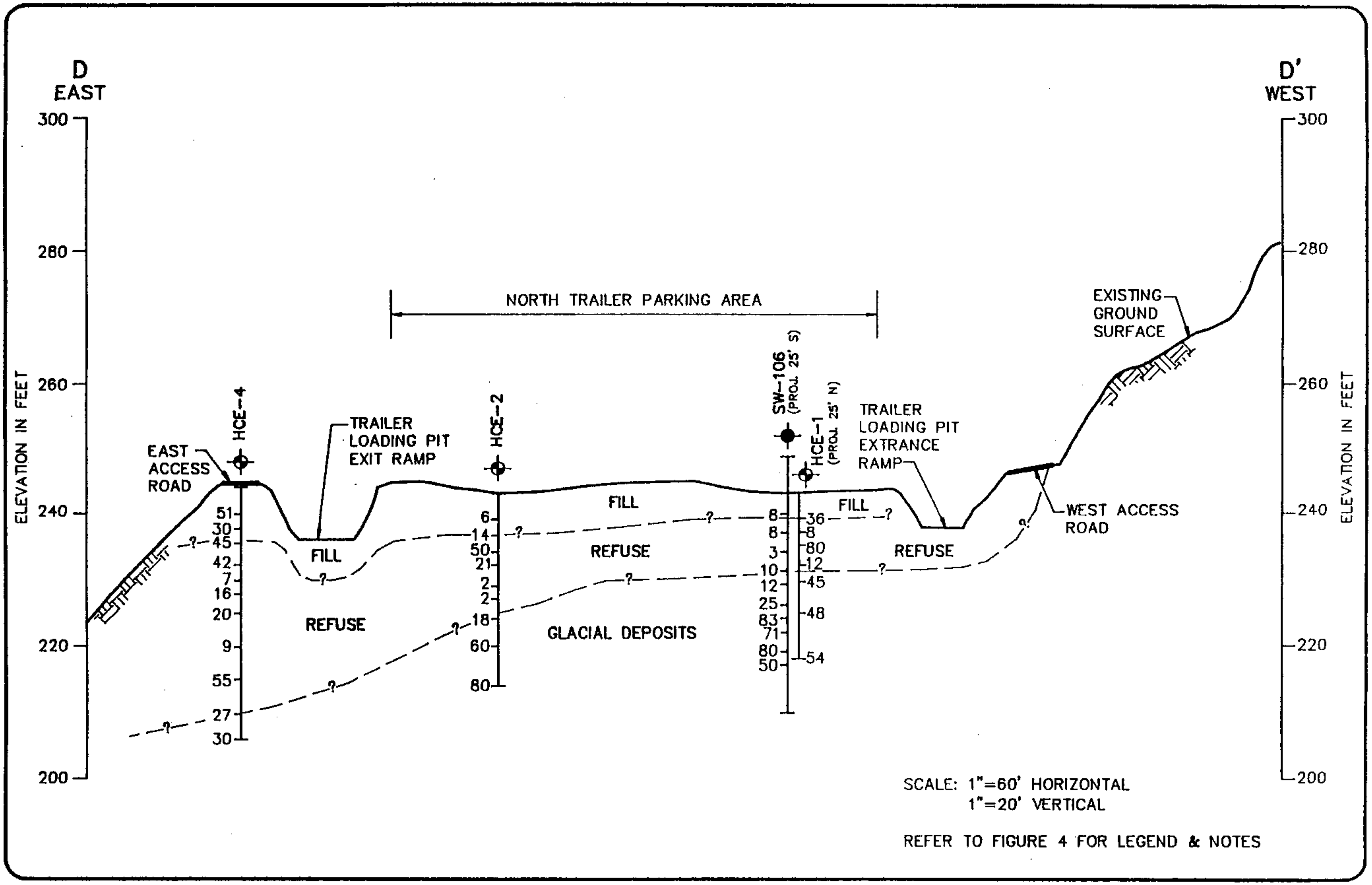
GENERALIZED GEOLOGIC CROSS SECTION C–C'

PROJECT NO.: 93112 FIGURE: 6

93112-A-006.1



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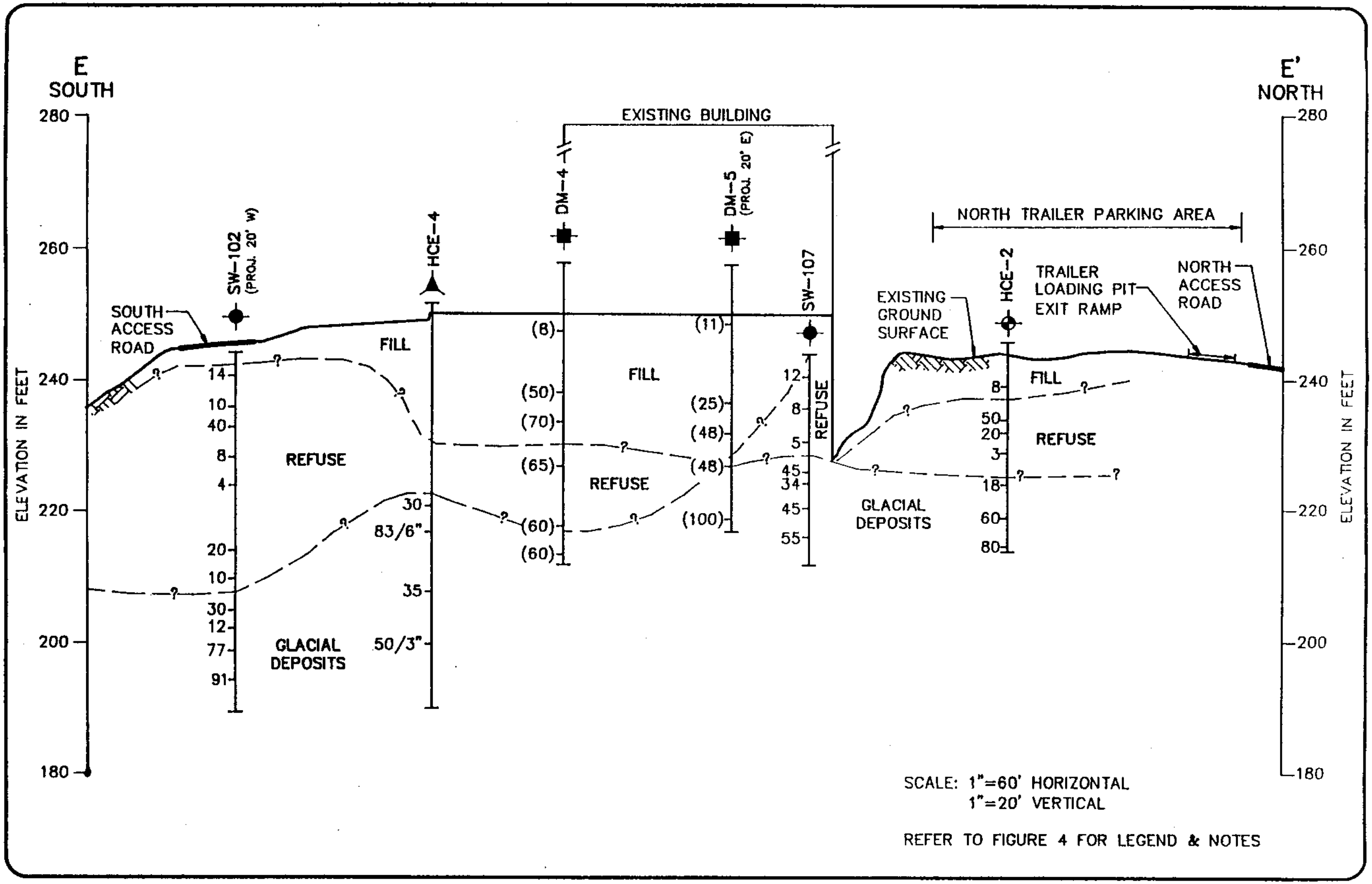
GENERALIZED GEOLOGIC CROSS SECTION D-D'

PROJECT NO.: 93112 FIGURE: 7

93112-A-007.1



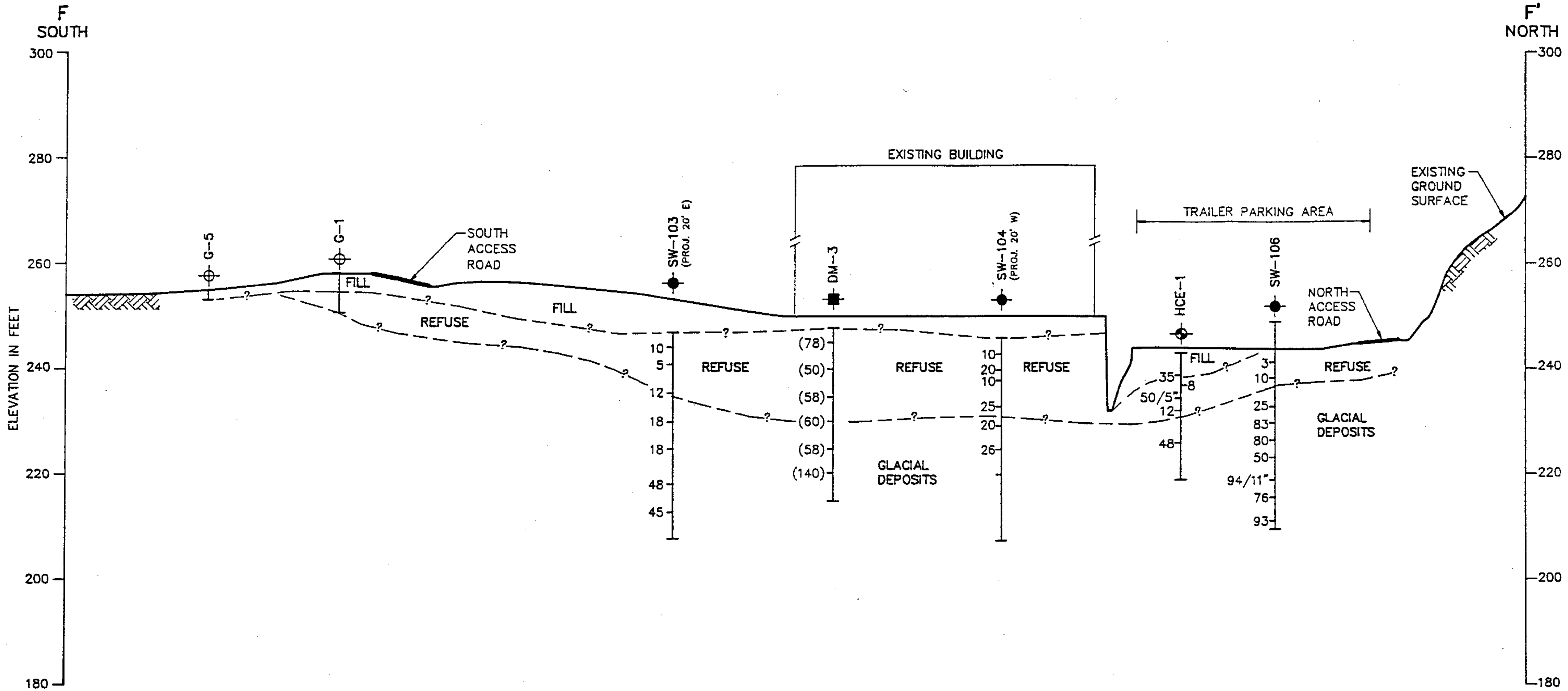
HONGWEST & ASSOCIATES, INC.

BOW LAKE
TRANSFER STATION
IMPROVEMENTS

GENERALIZED GEOLOGIC CROSS SECTION E–E'

PROJECT NO.: 93112 FIGURE: 8

93112-A-009.1



SCALE: 1"=60' HORIZONTAL
1"=20' VERTICAL

REFER TO FIGURE 4 FOR LEGEND & NOTES

HONGWEST & ASSOCIATES, INC.

BOW LAKE TRANSFER STATION IMPROVEMENTS

GENERALIZED GEOLOGIC CROSS SECTION F–F'

PROJECT NO.: 93112 FIGURE: 9

93112-B-008.1

DRAFT GEOTECHNICAL EVALUATION REPORT
WSDOT PROPERTY
BOW LAKE TRANSFER STATION/RECYCLING FACILITY
KING COUNTY, WASHINGTON

HWA Project No. 2003008-21

Contract No. E23001E

January 16, 2004

DRAFT

Prepared for:

R.W. Beck

HWA GEOSCIENCES INC.

January 16, 2004
HWA Project No. 2003-008-21

R.W. Beck
1001 Fourth Avenue, Suite 2500
Seattle, WA 98154-1004

Attention: Mr. Karl Hufnagel, P.E.

SUBJECT: **Draft Geotechnical Evaluation Report**
WSDOT Property
Bow Lake Transfer Station/Recycling Facility
King County, Washington

Dear Sir:

As authorized, HWA GeoSciences Inc. (HWA) has completed a preliminary geotechnical evaluation of the WSDOT property located immediately north of the Bow Lake Transfer Station. This investigation was undertaken as part of the Facility Master Plan Update and Implementation, specifically addressing general geotechnical conditions on the WSDOT property, and identifies geotechnical constraints that may impact King County's deliberations in respect to purchasing the property for expanding the existing facility.

We appreciate the opportunity to provide geotechnical services on this project. Please review and comment on the attached draft report, and call if we can be of further service.

Sincerely,

HWA GEOSCIENCES INC.

Brian E. Hall, P.E.
Senior Geotechnical Engineer

BEH:SHH:beh
Enclosure: Geotechnical Report

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Appendix A: Field Explorations

Appendix B: Laboratory Test Results

Appendix C: Previous Field Explorations

DRAFT GEOTECHNICAL EVALUATION REPORT
WSDOT PROPERTY
BOW LAKE TRANSFER STATION/RECYCLING FACILITY
KING COUNTY, WASHINGTON

1.0 INTRODUCTION

1.1 GENERAL

This report presents the results of a geotechnical evaluation of the WSDOT property located immediately north of the Bow Lake Transfer Station. The evaluation was undertaken as part of the Facility Master Plan Update and Implementation. It specifically addresses general geotechnical conditions on the WSDOT property, and identifies geotechnical constraints that may impact King County's deliberations in respect to purchasing the property for expanding the existing facility.

This work was performed ~~under King County Contract No. E23001E~~, Task 2 - Data Collection, Evaluation and Development.

1.2 PROJECT UNDERSTANDING

The site vicinity is shown in Figure 1, and site layout is shown in Figure 2. At the time of preparing this geotechnical evaluation report, only very preliminary concepts had been developed for using the WSDOT property. We understand that future uses of the property may include placing the facility entrance and scale plaza on the north of the property, routing the entrance road around the west, north and east sides, and using the central portion of the site for transfer trailer parking and maneuvering. Common to all future development plans for the property is the need to remove a large amount of existing fill to provide grade elevations compatible with those in the existing facility. For this evaluation an average final grade of El. 250 feet has been used to determine potential geotechnical impacts. This final grade requires the excavation and removal of up to 55 feet of fill material. Excavated fill material must be disposed off-site because very little additional fill is required for future development of the facility.

HWA GeoSciences Inc. (HWA) previously undertook a geotechnical study of the existing facility and prepared a geotechnical report entitled *"Geotechnical Engineering Study, Bow Lake Transfer Station Improvements, Facilities Master Plan, King County, Washington"*. This previous study covered the existing facility, but a single boring (BH-1) was located on the WSDOT property. The soil profile log was presented in a Technical Memorandum dated March 4, 1994, and is also included for reference in Appendix C.

2.0 FIELD EXPLORATION AND LABORATORY TESTING

2.1 FIELD EXPLORATION

HWA performed subsurface explorations at the site on October 9 and 10, 2003. Drilling was undertaken by Holocene Drilling, Inc. of Fife, Washington, under subcontract to HWA. The explorations located in easily accessible areas (BH-2 and BH-5) were undertaken with a Mobile B-61 truck-mounted rig, while difficult to access locations on sloping ground (borings BH-3 and BH-4) were drilled with a Simcoe 4000 tracked drill rig. All four borings were advanced under the full time supervision and were logged by an engineering geologist from HWA. During the field investigation, soil samples were classified in the field and pertinent information, including sample depths, stratigraphy, soil engineering characteristics and ground water occurrence was recorded. Representative soil samples were returned to our Lynnwood, Washington, laboratory for further examination and laboratory testing.

Approximate boring locations are shown on the Site and Exploration Plan (Figure 2), which is an extract from a topographical site survey provided to HWA by R.W. Beck on December 16, 2003. It should be noted that the boring locations are approximate because of discrepancies between the survey co-ordinates on the survey drawing and exploration location co-ordinates supplied by surveyors on October 10, 2003.

The borings were advanced using a 4-inch inside diameter, continuous flight, hollow-stem auger. At intervals of 5 feet within each boring, a Standard Penetration Test (SPT) was undertaken using a 140-pound hammer. In the SPT, a sample is obtained by driving a 1.5-inch O.D. sampler 18 inches into the soil with the hammer free-falling 30 inches. The number of blows required for each 6 inches of penetration is recorded. If more than 50 blows is recorded for a single 6-inch interval, the test is terminated, and the blow count is recorded as 50 blows for the number of inches penetrated. This resistance, or N-value, provides an indication of the relative density of granular soils and the consistency of cohesive soils.

A legend of the terms and symbols used on HWA exploration logs is given in Appendix A, (see Figure A-1). The summary boring logs for BH-2 through BH-5 are also included in Appendix A (see Figures A-2 through A-5). The soil boundaries indicated on the logs as distinct lines are interpreted between sample intervals and, accordingly, may not be precisely where indicated. Moreover, soil contact boundaries are often transitional in nature and not distinct as implied from the log representation. The soil and ground water conditions depicted on the exploration logs are also only for the specific dates and locations reported and, therefore, are not necessarily representative of other locations and times.

2.2 Laboratory Testing

Laboratory testing of samples retrieved from the borings included moisture content, particle size analysis, Atterberg limits, moisture/density, ash and organic content, and resistivity. In addition, TCLP testing was undertaken on a composite sample of "refuse-like" material to determine disposal requirements. Details of the laboratory test methods used and a summary of the test results are presented in Appendix B.

3.0 SITE CONDITIONS

3.1 Site Description

The WSDOT property is located immediately north of the existing Bow Lake Transfer Station against the eastern side of northbound I-5 traffic lanes (Figure 2). The eastern side of the site is part of a long down-gradient slope extending to the Duwamish River valley and Southcenter Parkway. The average slope angle is about 45% (about 2.2H:1V). On the north side of the WSDOT property, a ravine drains down into the valley. Total topographic relief ranges from El. 314 feet at the top of the stockpile, to El. 25 feet at the valley floor.

The property is dominated by a large fill stockpile that was placed sometime after BH-1 was drilled by HWA in February 1994. No historical information was provided on the stockpile material, except WSDOT has stated that the stockpile contains fill material and "there should be no surprises". However, we observed cobbles, boulder and concrete fragments of up to about 1.5 feet in size in the sides and top surface of the stockpile.

Figure 3 shows an east-west profile drawn through the site. Existing site elevations vary from an average top of stockpile elevation of about 304 feet (there is a local high on the north of the stockpile of 314 feet) to about 276 feet against northbound I-5 and about 230 feet along the eastern property line. The dimensions of the top of the stockpile are about 300 feet by 220 feet. The side slopes are relatively steep as can be seen in Figure 3 and amount to about 60% (about 1.7H:1V). A small roadway provides access from I-5 to the top of the stockpile.

Vegetation on the property consists of grass on the side slopes of the stockpile, and mostly Himalayan blackberry, Scots broom, and alders along the lower slopes. Such vegetation is typically associated with recently disturbed sites. The very dense blackberry thickets along the lower parts of the site contributed to limited site access. The top of the fill was sparsely vegetated.

No evidence of fill instability was observed, even along the slope crests where sloughing typically occurs in uncompacted fills.

3.2 GENERAL GEOLOGY

The geology of the Puget Sound region includes a thick sequence of over-consolidated glacial and unconsolidated non-glacial soils overlying bedrock. Glacial deposits were formed by ice originating in the mountains of British Columbia (Cordilleran ice sheet) and from alpine glaciers which descended from the Olympic and Cascade Mountains. These ice sheets invaded the Puget Lowland at least four times during the early to late Pleistocene Epoch (approximately 150,000 to 10,000 years before present). The southern extent of these glacial advances was near Olympia, Washington. During periods between these glacial advances and after the last glaciation, portions of the Puget Lowland filled with alluvial sediments deposited by rivers draining the western slopes of the Cascades and the eastern slopes of the Olympics. The most recent glacial advance, the Fraser Glaciation, included the Vashon Stade, during which the Puget Lobe of the Cordilleran ice sheet advanced and retreated through the Puget Sound Basin. Existing topography, surficial geology and hydrogeology in the project area were heavily influenced by the advance and retreat of the Vashon ice sheet.

Surficial geological information for the site area was obtained partly from the published geological map; "*Geologic Map of the Des Moines Quadrangle, King County, Washington.*" *(Waldron, 1962).* The map indicates that the plateau west of the site, upon which SeaTac International Airport, and the cities of SeaTac, Burien, and Des Moines reside, is predominantly mantled by Vashon till. This material was deposited as a discontinuous mantle of ground moraine beneath glacial ice on the eroded surface of older deposits. Soils defined as Vashon till consist of an unsorted, heterogeneous mass of silt, gravel, and sand in varied proportions. The till is of high density/strength due to glacial over-consolidation, and typically has low permeability.

The surficial geology of the slope forming the side of the river valley, which includes the subject site, is mapped as kame-terrace deposits. This material consists of stratified sand and gravel that was deposited by meltwater streams flowing from receding glacial ice, and is deposited against or close to the ice as Ice-Contact Stratified Drift. Inclusions of till are common, typically discontinuous and of limited thickness. In the past, these kame-terrace deposits were frequently mined for sand and gravel pits.

3.3 SUBSURFACE CONDITIONS

During an onsite meeting held with R.W. Beck to select boring locations, it was decided to limit the length of boring through the recent fill because of the presence of potential obstructions and because most of this material would be removed and not used in the future development. Based on this meeting, two borings (BH-2 and BH-5) were drilled on the western side of the property adjacent and at similar grade to the wide gravel pullout next to I-5. These borings encountered 2 to 3 feet of recent fill over native soils. The other two borings (BH-3 and BH-4) were drilled near the toe of the recent stockpile; BH-3 on the south side near the existing transfer station, and BH-4 on the eastern slope. Both of these borings encountered a few feet of recent fill over 28 to 33 feet of old fill with refuse, above native soils. The subsurface conditions encountered in BH-3 and BH-4 were similar to those encountered in BH-1 (drilled in 1994).

Following are brief descriptions of the soil deposits encountered in our explorations, in the order of stratigraphic sequence by which they were deposited or placed, with the youngest unit described first:

- **Recent Fill** – Each of the borings encountered non-organic silty gravel with sand to silty sand with gravel (Unified Soil Classification of GM to SM) at the ground surface and extending to variable depths. In the sample of recent fill from boring BH-2, the fines content was 38%. It should be noted that cobble, boulder and concrete rubble inclusions were also encountered in this unit.

- **Older Fill with Refuse** – The older fill is recognized by a blackish-brown color. Older fill was encountered in BH-3 and BH-4, and consisted of silty and sandy gravel to silty gravelly sand (Unified Soil Classification of SM to GM) with variable amounts of glass and metal, and rarely plastic inclusions. The blackish-brown color likely reflects its organic content (ranges from 1.9 to 5.6% based on laboratory tests) and from burning of the refuse. The site history of the adjacent transfer station property indicates that much of the refuse on that site was disposed of by burning.

- **Outwash/Ice-Contact Stratified Drift** – Each of the borings were advanced through recent and older fill into native glacial soils, consisting of stratified clean sand (Unified Soil Classification of SP), and till and silty sand (Unified Soil Classification of SM to GM), of variable density. The stratified character, varied texture, and variable density are consistent with an ice-marginal origin; e.g. at the edge of an ice-filled valley during glacial retreat. The deposits can also be classified as a kame-terrace deposit.

3.4 SUMMARY OF BORINGS

Table 1 summarizes details of the borings, and Figure 3 shows a geological profile through the property:

Table 1. Summary of Conditions Encountered in the Borings

Boring #	Ground Elevation (feet)	Depth to Base of Fill (Bottom Elevation) (feet)	Thickness of Fill Remaining below Final Grade (feet)*	Total Boring Depth (feet)	Description of Material below Final Grade
BH-1	273	29.5 (243.5)	6.5	44	Medium dense, fine to medium sand with silt and gravel
BH-2	279	3 (276)	No fill remaining	41.5	Weathered till grading to till over ice-contact drift
BH-3	275	35 (240)	10	44.5	Loose to medium dense, silty gravelly sand with refuse inclusions over dense, fine to clean sand, ice-contact drift
BH-4	286	35 (251)	1	49.5	Thin layer of medium dense fill over medium dense, clean fine to medium sand, ice-contact drift
BH-5	279	2 (277)	No fill remaining	41.5	Medium dense to very dense, fine to medium sand, ice-contact drift

.* Based on average final grade of El. 250 feet.

Figure 3 indicates that the surface of the native glacial material is deeper near the center of the property compared to the eastern side. While this could reflect inaccuracies in the boring locations and elevations, it may indicate that the property was originally a borrow pit that was subsequently filled. In the previous HWA geotechnical report on the adjacent transfer station site, it was suggested that the area was previously a swale before landfilling.

3.5 GROUND WATER

No ground water was encountered in any of the borings (both current and previous) at the time of drilling. Because of the need to excavate to substantial depth below existing ground surface, however, standpipe piezometers were installed in borings BH-4 and BH-5 to allow long-term monitoring, to determine whether elevated water tables could impact excavation and design of structures on the property.

No water was encountered in these piezometers shortly after installation. We recommend that the next set of readings be undertaken at the end of the winter when the water table should be at the highest elevation.

4.0 DISCUSSION AND CONCLUSIONS

4.1 GENERAL

This preliminary investigation shows that the site presently consists of a large stockpile of recently-placed fill (placed sometime since 1994) over successive layers of older fill containing refuse and ash. These fill deposits overlie native glacial deposits. No ground water was encountered within the depths investigated. The configuration of the upper surface of the native glacial deposits suggests that the area may previously have been used as a borrow pit prior to landfilling. The previous study undertaken by HWA on the existing transfer station site indicated that the transfer station site was originally a burn dump dating back to the late 1930s/early 1940s. Prior to 1936 the entire area was a wooded hillside.

It should be noted that this is a preliminary geotechnical assessment and further explorations will be necessary at a later stage, especially if development is planned along the eastern edge of the property. During the current investigation, drilling was not undertaken along the extreme eastern portions because of the steeply sloping ground. Also, as discussed previously, drilling through the recent fill was limited because of the presence of cobbles, boulders and concrete rubble. If the County decides to pursue the purchase of the property, test pitting should be undertaken to obtain a better indication of the characteristics of the recent fill (especially the amount of processing that may be required to remove oversize boulders and concrete rubble).

As shown in Figure 3, the planned excavations, to a final site grade of about El. 250 feet would result in the removal of all recent fill and most of the older fill. Depending on what is planned for areas where older fill will remain below site grade, additional excavation or in-situ treatment of material will be required. At the time of detailed design, additional investigation must be taken in areas underlain by older fill to determine whether gas venting is required below enclosed structures.

We understand that all excavated material must be disposed off site, because fill is not required for future development of the existing facility. The information obtained from the investigation shows that the recent stockpile material is suitable for use as structural fill provided it is placed in dry weather and careful compaction controls are followed. However, the older fill is of very variable quality and will likely be difficult to market to outside users.

The scope of work for this investigation did not include undertaking an environmental assessment for contaminated material. If the County decides to proceed with acquiring the property, then environmental testing would be required.

4.2 REUSE OF MATERIAL EXCAVATED FROM ABOVE FINAL GRADE

4.2.1 Recent Fill

The borings were located around the edge of the recent fill, and so do not provide complete information on the variability of the recent fill. If more detailed information is required on the recent fill, test pits should be excavated using a large tracked excavator. We further recommend that WSDOT be again asked whether they have any information on the material, especially source, when placed, presence of cobbles and boulders, and any test results, etc.

Where encountered, the recent fill consists of non-organic, silty gravel with sand to silty sand with gravel (Unified Soil Classification of GM to SM). In a sample from boring BH-2, the fines content was 38% indicating the material will be moisture sensitive and difficult to compact when wet. We anticipate, based on the presence of cobbles, boulders and concrete rubble on the fill surface, that the fill will require processing prior to use as structural fill. After processing, we anticipate the fill will meet the WSDOT requirements for Common Borrow (Clause 9-03.14(3), *WSDOT Standard Specifications for Road, Bridge and Municipal Construction, 2004*). Such material can be used as structural fill provided it is placed in dry conditions, and careful compaction controls are applied.

The following conditions apply to the use of this material as structural fill:

- The high fines content will make the material moisture-sensitive and unsuitable for use in wet conditions.
- Cobbles, boulders and concrete rubble are present in the fill. Screening is likely required to make the material suitable as backfill.

- The material should be compacted in layers of not greater than 8 inches (loose layer) thickness.

- Compaction should be undertaken to at least 95% of maximum dry density determined in accordance with ASTM D 1557 (Modified Proctor).

- Compaction moisture content should be within 2% of the Modified Proctor optimum moisture content for the material.

We did not detect evidence of soil contamination based on observations of soil color and smell. However, this was not an environmental investigation, and further investigation is required to confirm that the material is not contaminated.

4.2.2 Older Fill

Older fill was encountered in borings BH-1, BH-3 and BH-4 located in the central and eastern parts of the property. Older fill is easily recognized by a distinctive blackish-brown color. The older fill consists typically of silty, sandy gravel with variable amounts of glass and metal, and rarely plastic. The blackish-brown color is likely due to both organic content ranges from 1.9 to 5.6% [see Appendix B]), and ash resulting from refuse burning. Based on the presence of cinder, ash and melted glass (see BH-4 at 33 feet) recovered in the borings, and the previous site history of refuse burning on the adjacent transfer station property, it is likely that much of the older fill is burned refuse.

Most of the older fill material does not meet the WSDOT requirements for Common Borrow (Clause 9-03.14(3)) because of the presence of refuse inclusions and an organic content exceeding 3% in two of the samples tested.

The material could be used for non-structural fill, but use is dependent on the results of environmental testing for soil contamination. The TCLP testing (see Appendix B) for disposal of the boring cuttings showed that a small amount of metals leach from the soil. Consideration should be given to mixing the material with 3% cement and utilizing it in soil-cement structural fill. Detailed laboratory testing is necessary before this approach could be recommended because the presence of organic inclusions tends to inhibit setting of the cement.

We anticipate that it will be difficult to find a market for this material, even as non-structural fill.

4.2.3 Native Glacial Material

Along the western side of the property, excavation to depths of around 25 feet in glacial material could be required to provide site access. Excavations in this area would encounter native glacial materials below a thin layer of recent fill. The excavated material encountered in BH-2 is more silty than that encountered in BH-5. In BH-2, the material consists of layers of till and ice-contact drift and is mainly silty gravelly fine to coarse sand to silty fine to coarse gravel (mainly Unified Soil Classification of SM to GM); whereas, in BH-5, the ice-contact drift encountered consisted mainly of clean to slightly silty sand (Unified Soil Classification of SP/SM). Fines content varied from 18 to 25% in BH-2 to 15 to 17% in BH-5.

Some native material will meet the WSDOT requirements for Select Borrow [Clause 9-03.14(2)], but the material will mainly only be suitable as Common Borrow [Clause 9-03.14(3)].

We anticipate that the County will find sources for the disposal of this material without great difficulty provided it is demonstrated that the material has not been contaminated by leaching of contaminants from the mixed fill and refuse above. However, based on observations of soil color and smell, no evidence of soil contamination was apparent.

The material is suitable for use as structural fill provided it is placed and compacted as detailed for recent fill in section 4.2.1.

4.3 Excavatibility of Material above Final Grade

We anticipate that excavation of fill and native soils can be undertaken with conventional excavation equipment such as trackhoes. Some oversize material may be encountered but will not impact excavatibility. However, a range of unknown obstructions could be present in the older fill depending on the history of dumping previously occurring on the site. For example, in BH-3 a length of ½-inch metal pipe was encountered. Our previous experience is that large obstructions such as old car bodies, appliances, etc. could be present in such old fill deposits.

4.4 Development Issues

4.4.1 Critical Area Requirements and Slope Stability

Some of the adjoining slope angles shown on Figure 3 exceed 40%. Therefore, development of the property will need to adhere to Critical Area requirements for geologically hazardous steep slopes. Additional investigation will probably be required along the eastern property line to provide information for the Critical Area Review, especially if additional fill or retaining walls are placed in this area.

Currently, we observed no indication of slope instability. In addition, we are not aware of slope instability along the eastern side of the existing facility. An advantage of the plan to excavate the property to about El. 250 feet is that slope stability will be improved because most of the fill adjoining the top of slope will be removed.

Although we did not observe evidence of ground water in the borings during drilling, the following potential slope instability modes should be considered during detailed design:

- *Instability of the eastern slope of the older fill, if excavation does not extend down to El. 250 feet.* Figure 3 shows the outside (eastern) slope angle is around 60%, which is excessive if development is planned near the crest of the slope. The apparent reason for the stability of fill to date is that the drainage is good and water table is deep.

- *Slope instability related to seepage occurring above a silt layer.* If silt layers occur lower down in the slope profile, it is vital that fill placement should not obstruct any seepage that may be occurring along these less permeable layers. All fills should be constructed with suitable under-drainage to dissipate any potentially high pore pressures that could result.

- *Slope instability resulting from liquefaction occurring due to a design level earthquake.* Such a slide requires the presence of a water table. No water table was encountered during the current investigation.

4.4.2 Stability of Cut Slopes and Excavations

Maintenance of safe working conditions, including temporary excavation stability, is the responsibility of the contractor. Any excavations in excess of 4 feet in depth should be sloped in accordance with Part N of WAC (Washington Administrative Code) 296-155, or be suitably shored. The loose to medium dense fill classifies as Type C Soil.

Temporary excavations in Type C Soils may be inclined as steep as 1½H:1V. In lieu of excavations sloped to these requirements, trench boxes or other suitable shoring means may be used to permit work in trenches in excess of 4 feet in depth. Heavy construction equipment, construction materials, excavated soil, and vehicular traffic should not be allowed within a distance half the depth of the excavation, measured from the edge of the excavation, unless the shoring system has been designed for the additional lateral pressure.

With time and the presence of precipitation, the stability of temporary unsupported cut slopes can be significantly reduced. Therefore, all temporary slopes should be protected from erosion by installing a surface water diversion ditch or berm at the top of the slope, and by covering the cut face with well-anchored plastic sheeting. In addition, the contractor should monitor the stability of the temporary cut slopes and adjust the construction schedule and slope inclination accordingly.

For long-term stability, slopes should be cut no steeper than 2H:1V.

4.4.3 Foundations

Foundations will be placed either on native glacial material or on older fill after removal of the overlying recent and some of the older fill. For example, at BH-3, about 10 feet of older fill could remain after excavation to final design grade. Localized zones of thicker fill could be present elsewhere. In such cases, the allowable bearing pressures of footings should take into account the effective preloading caused by the thickness of fill removed.

***Foundations on Native Glacial Materials*:** Structures located on medium dense to dense native glacial materials, should be founded on shallow pad and strip footings designed for allowable bearing pressures of 3,000 psf, subject to minimum dimensions of 3 feet and 1.5 feet for pad and strip footings, respectively. External footings should be placed at least 1.5 feet below final adjoining ground surface for frost protection.

***Foundations on Older Fill Material where at least 10 feet of Fill was Removed*:** Structures located on older fill, where at least 10 feet of overlying fill (preload) was removed, may be founded on shallow pad and strip footings. The footings should be supported on a 3-foot thick pad of compacted structural fill placed over the fill. The footing should be designed for an allowable bearing pressure of 1,500 psf, subject to minimum dimensions of 3 feet and 1.5 feet for pad and strip footings, respectively. External footings should be placed at least 1.5 feet below final ground for frost protection.

***Foundations on Older Fill Material where less than 10 feet of Fill was Removed*:** If such cases exist on the property, site specific investigations should be undertaken. Foundation preparation would likely consist of excavation and replacement of fill, or supporting the building on piles or a mat foundation.

Differential settlement of footings designed as recommended above is not expected to exceed 1-inch. However, these are conceptual recommendations for typical buildings, and settlement estimates should be checked when the particulars of the structure are known.

4.4.4 Retaining Walls Supporting Cuts

Conventional concrete cantilever retaining walls or soil nail walls are considered suitable for support of cut slopes along the western side of the site. We recommend that any retaining walls be designed for a lateral earth pressure based on an equivalent fluid density of 55 pounds per cubic foot (pcf). This value assumes that backfill behind the walls is horizontal and is placed and compacted in accordance with our recommendations. This equivalent fluid pressure does not allow for traffic and construction loads. Such imposed loads should be included if they are imposed within a distance equivalent to the height of the wall. Fill within a distance of about 1-meter (3.3 feet) of the walls should be compacted with lightweight equipment. Care must be taken to avoid over-compaction near the walls, or excessive lateral pressures may develop.

Lateral forces may be resisted by a combination of sliding resistance of the footing on the underlying soil and passive earth pressure against the buried portions of the wall and footing. For design purposes, a coefficient of friction of 0.5 may be assumed between the base of the footing and native foundation soils or compacted structural fill. A passive earth pressure equivalent to a fluid weighing 260 pcf may be assumed for properly compacted fill placed against the buried portion of the wall foundation.

Positive drainage should be provided to prevent the buildup of hydrostatic pressures behind all retaining walls.

4.4.5 Retaining Walls Supporting Fill

If required, walls supporting fill along the eastern part of the site may consist of Structural Earth Walls (SEW), cantilever concrete, or gravity block walls. Final wall selection is dependent on the wall location, space available to accommodate wall construction, fill height, presence of suitable native bearing material at reasonable depth, and structural loading. It is necessary that walls along the extreme east of the site should be founded on native glacial material. If fill is present, the fill should be excavated and replaced, or consideration should be given to the use of a soldier pile and lagging wall if the fill is deep.

Reinforced soil slopes will also be suitable.

4.4.6 Drainage

No special drainage requirements, other than those typically provided, are necessary.

4.4.7 Subgrade Preparation for Roads

The native glacial material will provide a suitable subgrade for roads, but in areas of older fill, the road structure should be supported on at least 30 inches of structural fill placed over the fill. In areas where the subgrade is soft and yielding, the depth of structural fill should be increased to 40 inches and supported on woven separation grade geotextile (Clause 9.33.2 Table 3, WSDOT Specifications).

The design thickness of the overlying pavement and surfacing layers is dependent on design traffic and road performance requirements.

4.4.8 Soil Corrosiveness

The resistivity results given in Appendix B provide an indicator of the potential for soil corrosion of buried steel and concrete. Non corrosive soils typically have a resistivity in excess of 5,000 ohm-cm, and potentially corrosive soils have a resistivity of less than 2,000 ohm-cm. Soils with resistivities below 5,000 ohm-cm should be subject to more detailed chemical testing to evaluate the potential for corrosion. The results show the resistivity of the older fill is much lower than the underlying glacial materials, which indicates older fill is more corrosive. An exception is the sample of weathered drift/colluvium (BH-4, S-8) from immediately below the base of the fill that has a resistivity similar to that of the overlying older fill. This indicates that some migration of leachate from the overlying fill into the underlying native glacial till has occurred.

Based on these resistivity test results, we recommend that, at the time of detailed design, additional testing to determine the potential corrositivity of the soil be performed. We anticipate that all buried concrete and steel in the fill and near the surface of the native glacial materials should be designed assuming corrosive conditions.

4.5 Construction Aspects

4.5.1 Site Preparation

Site preparation for construction should begin with excavation of all unsuitable existing materials. Excavation for structures founded on older fill should be inspected by a geotechnical engineer to determine if the depth of excavation is sufficient. Pockets of poor materials may be present, and should be excavated and replaced with structural fill.

The exposed subgrade should be thoroughly proof-rolled with a heavy roller. All loose or soft areas that exhibit yielding should be replaced with structural fill materials, and compacted to a dense and unyielding condition in accordance with Section 2-03.3(14)C

(Compacting Earth Embankments) and/or Section 2-06.3(1) (Subgrade for Surfacing) of the 2004 WSDOT *Standard Specifications.*

4.5.2 Structural Fill and Compaction

For the purposes of this report, material used to raise site grades, placed directly under structures for support, or used as backfill behind below-grade structures such as catch basins or pipes, is classified as structural fill. Imported structural fill should consist of clean, non-plastic, free-draining sand and gravel free from organic matter or other deleterious materials. Such materials should contain particles of less than 3 inches maximum dimension, with less than 5 % fines (based on the ¾-inch fraction) as described in Section 9-03.14(1) or 9.03.17 (class B) of the 2004 WSDOT *Standard Specifications*.

Structural fill should be placed in loose, horizontal, lifts of not more than 8 inches in thickness and compacted to at least 95 % of the maximum dry density, as determined using test method ASTM D 1557 (Modified Proctor). At the time of placement, the moisture content of structural fill should be at or near optimum. The procedure required to achieve the specified minimum relative compaction depends on the size and type of compaction equipment, the number of passes, thickness of the layer being compacted, and the soil moisture-density properties.

When the first fill is placed in a given area, and/or anytime the fill material changes, the area should be considered a test section. The test section should be used to establish fill placement and compaction procedures required to achieve proper compaction. The geotechnical consultant should observe placement and compaction of the test section to assist in establishing an appropriate compaction procedure. Once a placement and compaction procedure is established, the contractor's operations should be monitored and periodic density tests performed to verify that proper compaction is being achieved.

5.0 CONDITIONS AND LIMITATIONS

We have prepared this report for R.W. Beck and King County Solid Waste Division for use in developing a Facility Master Plan and deciding whether to purchase the property. The conclusions and interpretations presented in this report should not, however, be construed as a warranty of the subsurface conditions at or influencing the site. Experience shows that soil and ground water conditions can vary significantly over small distances. Inconsistent conditions may occur between explorations that may not be detected by a geotechnical study of this scope and nature. If, during future site operations, subsurface conditions are encountered which vary appreciably from those described herein, HWA should be notified to review the recommendations made in this report, and revise, if necessary. If there is a substantial lapse of time between submission of this report and the start of construction, or if conditions change due to construction operations, it is recommended that this report be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

This report is issued with the understanding that it is the responsibility of the owner, or the owners' representative, to ensure that the information and recommendations are brought to the attention of the appropriate design team personnel and incorporated into the project plans and specifications, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

We recommend HWA GeoSciences Inc. be retained to undertake further follow-up investigations, as may be necessary for future site development, and monitor construction to evaluate soil and ground water conditions as they are exposed, and verify that subgrade preparation, backfilling, and compaction are accomplished in accordance with the specifications.

Within the limitations of scope, schedule and budget, HWA attempted to execute these services in accordance with generally accepted professional principles and practices in the fields of geotechnical engineering and engineering geology at the time the report was prepared. No warranty, express or implied, is made

This firm does not practice or consult in the field of safety engineering. We do not direct the contractor's operations, and cannot be responsible for the safety of personnel other than our own on the site. As such, the safety of others is the responsibility of the contractor. The contractor should notify the owner if any of the recommended actions presented herein are considered unsafe.

O • O

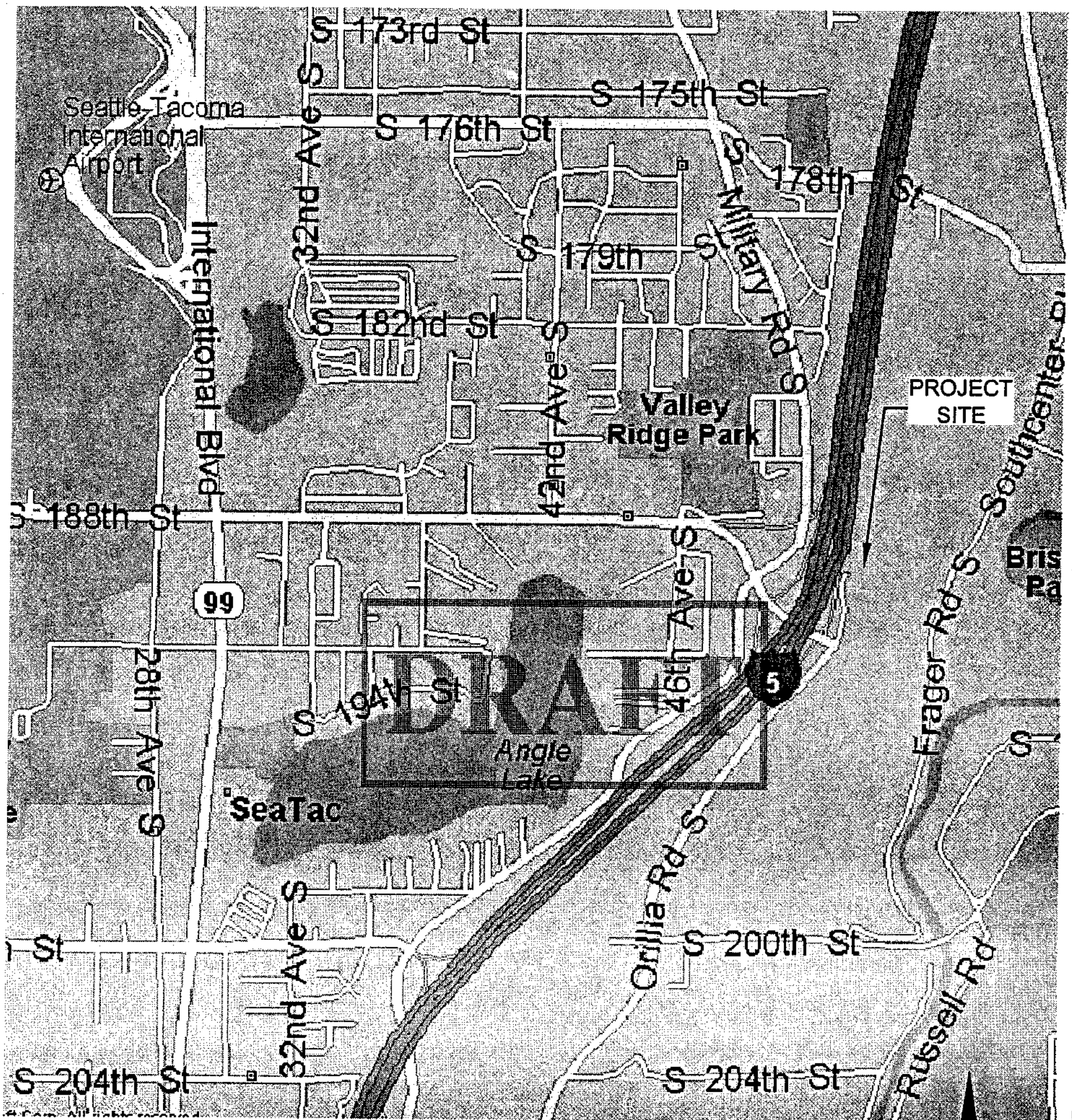
We appreciate this opportunity to be of service.

Sincerely,

HWA GEOSCIENCES INC.

Brian E. Hall, P.E. Senior Geotechnical Engineer	Sa H. Hong, P.E. Principal Geotechnical Engineer

BEH:SHH:shh



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NOT TO SCALE

HWA GEOSCIENCES INC.

VICINITY MAP

BOW LAKE TRANSFER STATION
MASTER PLAN UPDATE

KING COUNTY, WASHINGTON

DRAWN BY	KS
CHECKED BY	KS
DATE	12.31.03
FIGURE NO.	1
PROJECT NO.	2003-008

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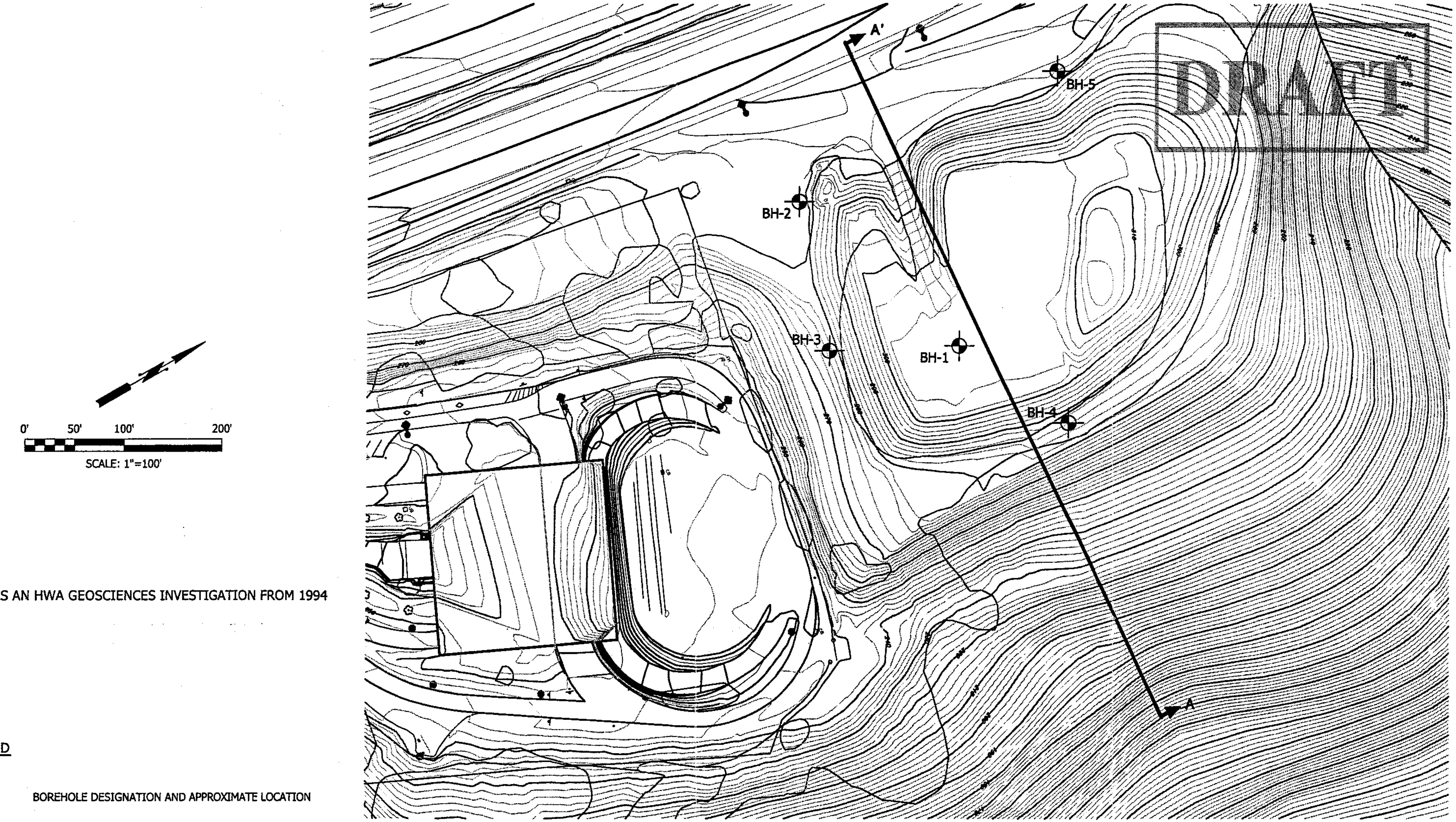
REV 00 KLS 12/31/03



NOTE

BH-1 IS AN HWA GEOSCIENCES INVESTIGATION FROM 1994

LEGEND

BH-1 BOREHOLE DESIGNATION AND APPROXIMATE LOCATION

A A' CROSS SECTION DESIGNATION AND LOCATION

HWA GEOSCIENCES INC.

BOW LAKE TRANSFER STATION
MASTER PLAN UPDATE

KING COUNTY, WASHINGTON

SITE AND EXPLORATION PLAN

DRAWN BY KS	FIGURE NO. 2
CHECKED BY BH	
DATE 12.15.03	PROJECT NO. 2003-008

REV 02 KLS 1/07/04

BASE MAP PROVIDED BY:
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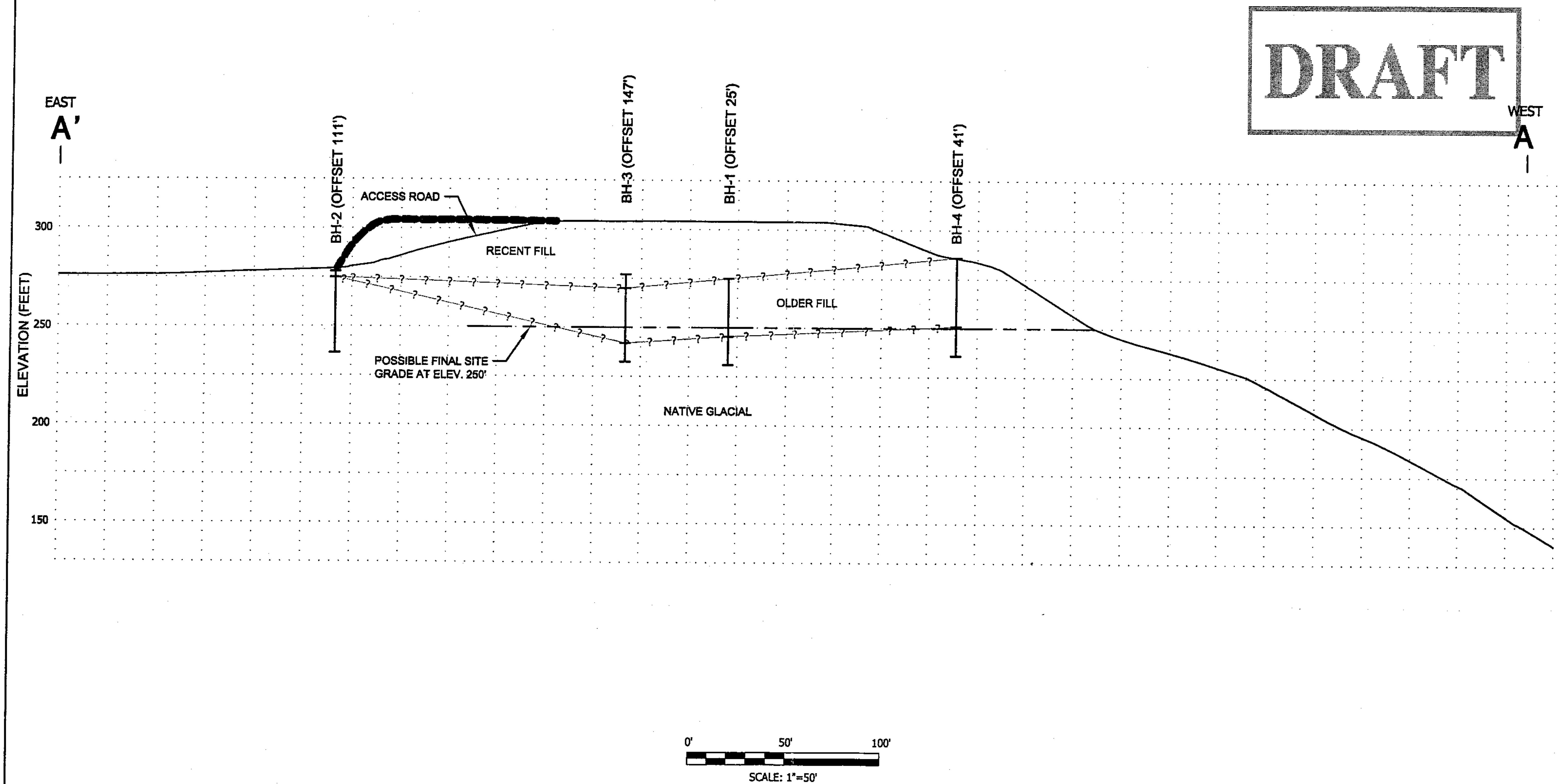

HWA GEOSCIENCES INC.

BOW LAKE TRANSFER STATION
MASTER PLAN UPDATE

KING COUNTY, WASHINGTON

SITE PROFILE
WEST - EAST

DRAWN BY KS	FIGURE NO. 3
CHECKED BY BH	
DATE 12.15.03	PROJECT NO. 2003-008

REV 02 KLS 1/07/04

BASE MAP PROVIDED BY:
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APPENDIX A

FIELD EXPLORATION

DRAFT

RELATIVE DENSITY OR CONSISTENCY VERSUS SPT N-VALUE

COHESIONLESS SOILS			COHESIVE SOILS		
Density	N (blows/ft)	Approximate Relative Density(%)	Consistency	N (blows/ft)	Approximate Undrained Shear Strength (psf)
Very Loose	0 to 4	0 - 15	Very Soft	0 to 2	<250
Loose	4 to 10	15 - 35	Soft	2 to 4	250 - 500
Medium Dense	10 to 30	35 - 65	Medium Stiff	4 to 8	500 - 1000
Dense	30 to 50	65 - 85	Stiff	8 to 15	1000 - 2000
Very Dense	over 50	85 - 100	Very Stiff	15 to 30	2000 - 4000
			Hard	over 30	>4000

USCS SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP DESCRIPTIONS	
Coarse Grained Soils	Gravel and Gravelly Soils	Clean Gravel (little or no fines)	GW	Well-graded GRAVEL
	More than 50% of Coarse Fraction Retained on No. 4 Sieve		GP	Poorly-graded GRAVEL
		Gravel with Fines (appreciable amount of fines)	GM	Silty GRAVEL
			GC	Clayey GRAVEL
More than 50% Retained on No. 200 Sieve Size	Sand and Sandy Soils	Clean Sand (little or no fines)	SW	Well-graded SAND
	50% or More of Coarse Fraction Passing No. 4 Sieve		SP	Poorly-graded SAND
		Sand with Fines (appreciable amount of fines)	SM	Silty SAND
			SC	Clayey SAND
Fine Grained Soils	Silt and Clay	Liquid Limit Less than 50%	ML	SILT
			CL	Lean CLAY
			OL	Organic SILT/Organic CLAY
50% or More Passing No. 200 Sieve Size	Silt and Clay	Liquid Limit 50% or More	MH	Elastic SILT
			CH	Fat CLAY
			OH	Organic SILT/Organic CLAY
Highly Organic Soils			PT	PEAT

TEST SYMBOLS

%F	Percent Fines
AL	Atterberg Limits: PL = Plastic Limit, LL = Liquid Limit
CBR	California Bearing Ratio
CN	Consolidation
DD	Dry Density (pcf)
DS	Direct Shear
GS	Grain Size Distribution
K	Permeability
MD	Moisture/Density Relationship (Proctor)
MR	Resilient Modulus
PID	Photoionization Device Reading
PP	Pocket Penetrometer Approx. Compressive Strength (tsf)
SG	Specific Gravity
TC	Triaxial Compression
TV	Torvane Approx. Shear Strength (tsf)
UC	Unconfined Compression

SAMPLE TYPE SYMBOLS

- 2.0" OD Split Spoon (SPT) (140 lb. hammer with 30 in. drop)
- Shelby Tube
- 3-1/4" OD Split Spoon with Brass Rings
- Small Bag Sample
- Large Bag (Bulk) Sample
- Core Run
- Non-standard Penetration Test (3.0" OD split spoon)

GROUNDWATER SYMBOLS

- Groundwater Level (measured at time of drilling)
- Groundwater Level (measured in well or open hole after water level stabilized)

COMPONENT DEFINITIONS

COMPONENT	SIZE RANGE
Boulders	Larger than 12 in
Cobbles	3 in to 12 in
Gravel	3 in to No 4 (4.5mm)
Coarse gravel	3 in to 3/4 in
Fine gravel	3/4 in to No 4 (4.5mm)
Sand	No. 4 (4.5 mm) to No. 200 (0.074 mm)
Coarse sand	No. 4 (4.5 mm) to No. 10 (2.0 mm)
Medium sand	No. 10 (2.0 mm) to No. 40 (0.42 mm)
Fine sand	No. 40 (0.42 mm) to No. 200 (0.074 mm)
Silt and Clay	Smaller than No. 200 (0.074mm)

COMPONENT PROPORTIONS

PROPORTION RANGE	DESCRIPTIVE TERMS
< 5%	Clean
5 - 12%	Slightly (Clayey, Silty, Sandy)
12 - 30%	Clayey, Silty, Sandy, Gravelly
30 - 50%	Very (Clayey, Silty, Sandy, Gravelly)

Components are arranged in order of increasing quantities.

NOTES: Soil classifications presented on exploration logs are based on visual and laboratory observation. Soil descriptions are presented in the following general order:

Density/consistency, color, modifier (if any) GROUP NAME, additions to group name (if any), moisture content. Proportion, gradation, and angularity of constituents, additional comments. (GEOLOGIC INTERPRETATION)

Please refer to the discussion in the report text as well as the exploration logs for a more complete description of subsurface conditions.

MOISTURE CONTENT

DRY	Absence of moisture, dusty, dry to the touch.
MOIST	Damp but no visible water.
WET	Visible free water, usually soil is below water table.

HWA GEOSCIENCES INC.

BOW LAKE TRANSFER STATION
FMP UPDATE AND IMPLEMENTATION
TUKWILA, WASHINGTON

LEGEND OF TERMS AND SYMBOLS USED ON EXPLORATION LOGS

PROJECT NO.: 2003-008 FIGURE: A1

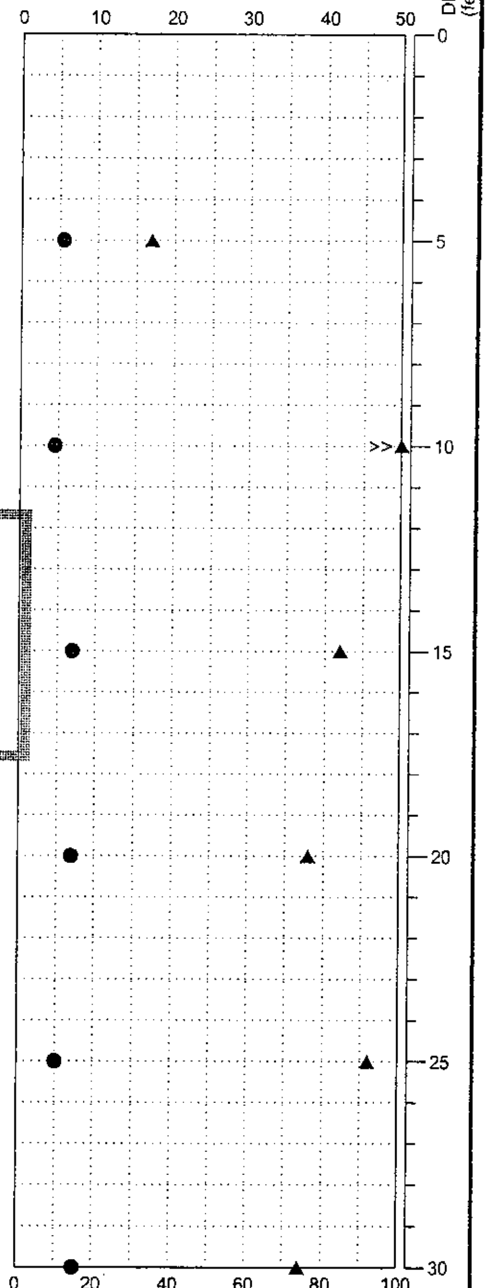
DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Hollow-Stem Auger, Mobile B-61 truck rig
SAMPLING METHOD: SPT w/ Autohammer
LOCATION: See Site & Exploration Plan, Figure 2

SURFACE ELEVATION: 279.00 ± feet

DATE STARTED: 10/9/2003
DATE COMPLETED: 10/9/2003
LOGGED BY: B. Thurber

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
0	GM	Medium dense, olive-brown, silty, sandy, fine to coarse GRAVEL, moist. **(RECENT FILL)**				
	SM	Cuttings: orange-brown, silty, fine to medium SAND, dry.				
5		Medium dense, rust- and gray-mottled light brown, silty, gravelly, fine to medium SAND, damp. **(WEATHERED TILL)**	S-1	5-7-10		
	GM	Gravelly drill action, 7 to 15 feet.				
10		Very dense, olive-brown, silty, fine to coarse sandy, fine to coarse GRAVEL, moist to wet. **(TILL)**	S-2	30-50/2"		
15	SM	Dense, olive-brown, silty, gravelly, fine to coarse SAND, moist; with approx. 2-inch stratified sand bed at 16 feet. **(ICE-CONTACT STRATIFIED DRIFT)**	S-3	8-17-25	GS pH/R	
20		Dense, olive-brown, silty to clean, fine gravelly, fine to medium SAND, moist. Stratified.	S-4	10-17-21		
	SP	Dense, light olive-brown to gray, clean, fine to medium SAND, damp.				
25		Dense, light gray, clean, fine to medium SAND, damp to moist.	S-5	10-21-25	GS pH/R	
30						

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot



For a proper understanding of the nature of subsurface conditions, this exploration log should be read in conjunction with the text of the geotechnical report.

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

BOW LAKE TRANSFER STATION
FMP UPDATE AND IMPLEMENTATION
TUKWILA, WASHINGTON

BORING:
BH-2

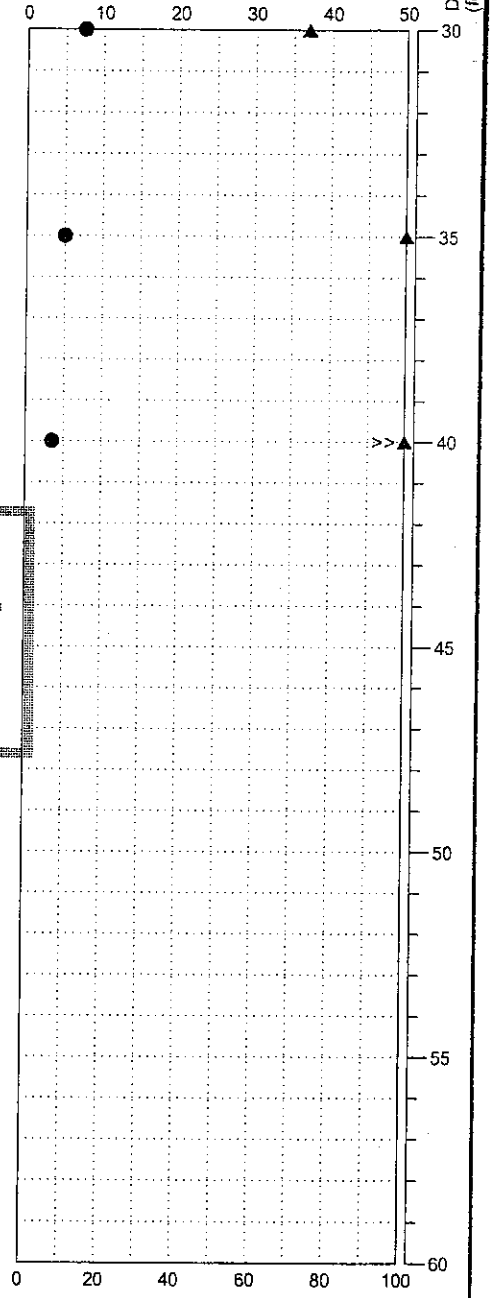
PROJECT NO.: 2003-008 FIGURE: A2

DRILLING COMPANY: Holocene Drilling
SURFACE ELEVATION: 279.00 ± feet
DRILLING METHOD: Hollow-Stem Auger, Mobile B-61 truck rig
SAMPLING METHOD: SPT w/ Autohammer
LOCATION: See Site & Exploration Plan, Figure 2

DATE STARTED: 10/9/2003
DATE COMPLETED: 10/9/2003
LOGGED BY: B. Thurber

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
30			Dense, light gray, clean to slightly silty, fine to medium SAND, moist. Minor silty sand lenses, 1-5 mm thick.		S-6	10-18-19		
35			Dense, light gray, clean, fine to medium SAND, damp. Appears massive.		S-7	14-24-26		
40			Very dense, light gray, clean, fine to medium SAND, damp. With lenses of slightly silty sand, 3-5 mm thick.		S-8	12-27-39		

Borehole terminated at 41.5 feet.
No ground water seepage encountered during drilling.
Borehole abandoned with bentonite chips.



For a proper understanding of the nature of subsurface conditions, this exploration log should be read in conjunction with the text of the geotechnical report.

NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

BOW LAKE TRANSFER STATION
FMP UPDATE AND IMPLEMENTATION
TUKWILA, WASHINGTON

BORING:
BH-2

PROJECT NO.: 2003-008 FIGURE: A2

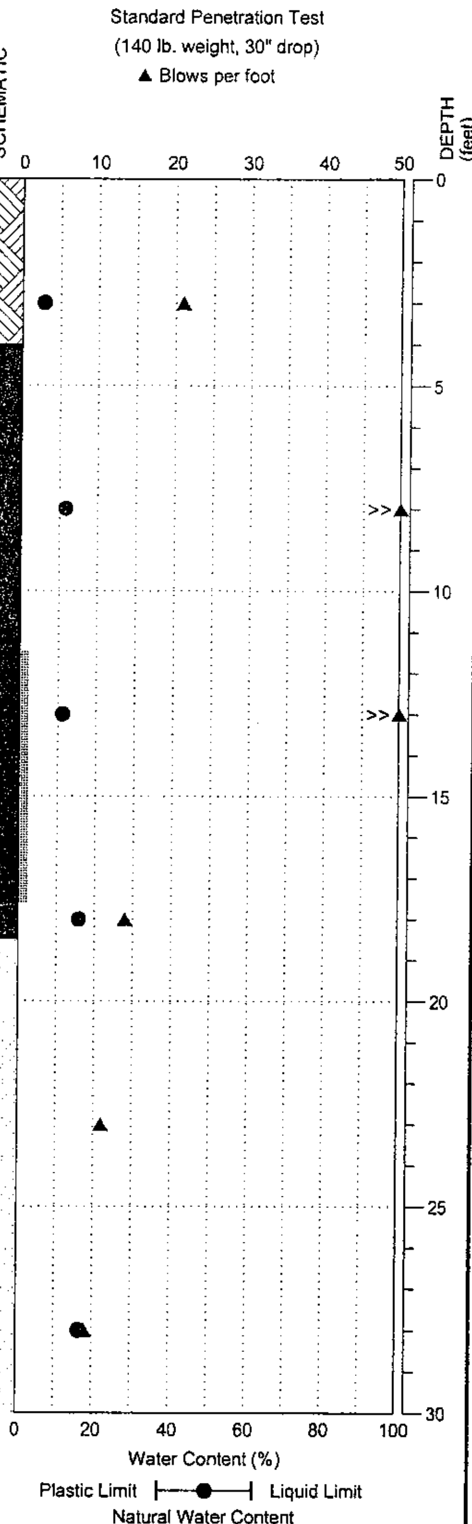
DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Hollow-Stem Auger, Simcoe 4000 tracked rig
SAMPLING METHOD: SPT w/ Cathead
LOCATION: See Site & Exploration Plan, Figure 2

SURFACE ELEVATION: 275.00 ± feet

DATE STARTED: 10/10/2003
DATE COMPLETED: 10/10/2003
LOGGED BY: B. Thurber

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
0	GM	Cuttings: Light gray, silty, sandy GRAVEL, dry.			
		Medium dense, light yellow-brown, silty, sandy, fine to coarse GRAVEL, dry. **(RECENT FILL)**	S-1	6-10-11	GS
5					
	GM	Glass in cuttings at 7 feet. Gravelly drill action, to 19.5 feet.			
		Very dense, stratified dark brown, yellow-brown, gray-brown, and reddish-brown (2-3 inch layers), silty, sandy, fine to coarse GRAVEL, moist. Glass fragments (clear) at 9 feet, in 2 inch layer of abundant glass; one piece of plastic sheeting. **(OLDER FILL with REFUSE)**	S-2	21-27-30	
10		6-inch long, 1/2-inch metal pipe in cuttings from approx. 10 feet.			
		Drove rock; blow counts overstated. Dense, dark brown, silty, sandy, fine to coarse GRAVEL, wet. Trace broken glass and partly decomposed, blackened wood. Peaty odor.	S-3	50/5"	
15					
		Medium dense, reddish-dark brown, silty, sandy, fine to coarse GRAVEL, wet. With a 1/2-inch black cinder.	S-4	8-7-7	
20		Fast, smooth drill action from 19.5 to 35 feet.			
		Medium dense, dark brown-gray, silty, sandy, fine GRAVEL, wet. One piece of packing tape.	S-5	4-5-6	pH/R
25	SM				
		Loose, dark brown, silty, fine gravelly, fine to coarse SAND, wet, over rust-mottled brown and gray grading to blue-gray, silty fine to medium SAND, wet. Trace broken glass, hard	S-6	2-4-5	GS pH/R OC
30					

DRAFT



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NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

BOW LAKE TRANSFER STATION
FMP UPDATE AND IMPLEMENTATION
TUKWILA, WASHINGTON

BORING:
BH-3

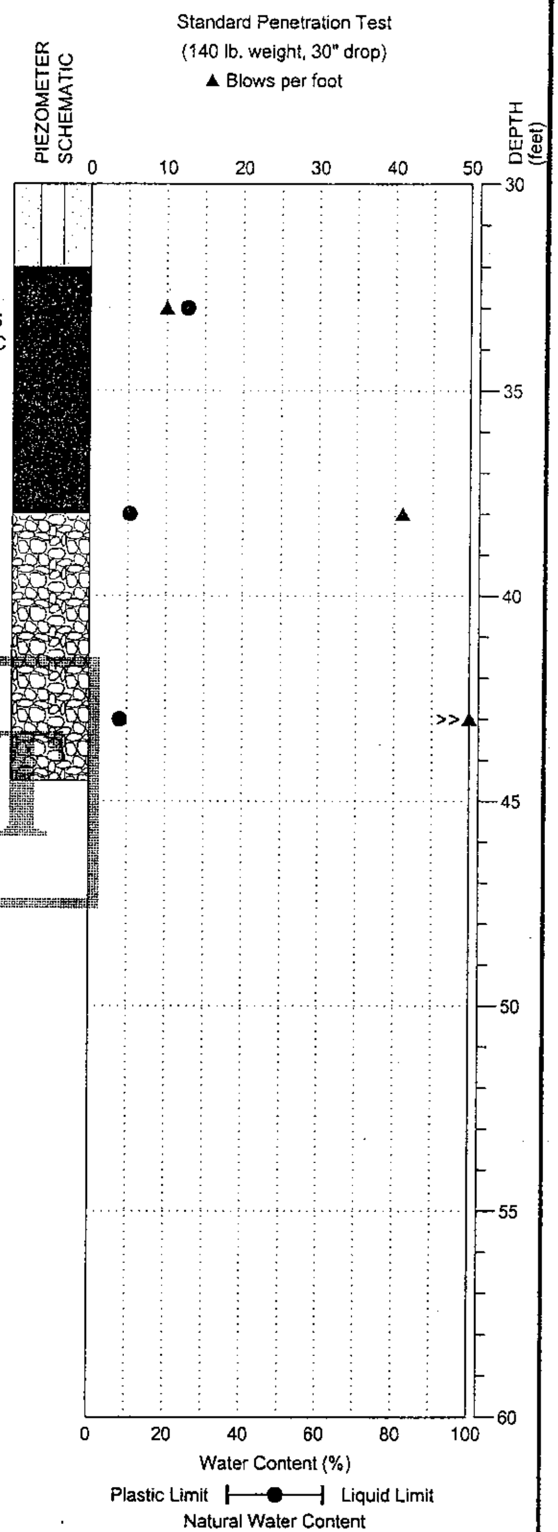
PROJECT NO.: 2003-008 FIGURE: A3

PZO 2003008.GPJ 1/9/04

DRILLING COMPANY: Holocene Drilling
SURFACE ELEVATION: 275.00 ± feet
DRILLING METHOD: Hollow-Stem Auger, Simcoe 4000 tracked rig
SAMPLING METHOD: SPT w/ Cathead
LOCATION: See Site & Exploration Plan, Figure 2

DATE STARTED: 10/10/2003
DATE COMPLETED: 10/10/2003
LOGGED BY: B. Thurber

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
30			plastic, partly decomposed wood.				
			Loose to medium dense, stratified gray, dark brown, and blackish-brown, silty, fine gravelly, fine to medium SAND, wet. Trace broken glass and mostly decomposed paper fiber in lower 6 inches, partly decomposed wood.		S-7	2-2-8	GS OC
35		SP	Minor gravelly drill action below 35 feet.				
			Slough on top of sample: Blackish-brown soil as above, with 1/2-inch thick layer of plastic bags. Dense, gray, clean, fine to medium SAND, moist.		S-8	7-20-21	
40			(ICE CONTACT STRATIFIED DRIFT)				
			Very dense, gray, clean, fine to medium SAND, moist. Massive.		S-9	20-31-46	
45			Borehole terminated at 44.5 feet. Minor ground water seepage encountered between 13 and 35 feet. Piezometer installed to 30 feet (1-inch schedule 40 PVC).				
50							
55							
60							



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HWA GEOSCIENCES INC.

BOW LAKE TRANSFER STATION
FMP UPDATE AND IMPLEMENTATION
TUKWILA, WASHINGTON

BORING:
BH-3

PROJECT NO.: 2003-008 FIGURE: A3

DRILLING COMPANY: Holocene Drilling SURFACE ELEVATION: 286.00 ± feet DATE STARTED: 10/9/2003
DRILLING METHOD: Hollow-Stem Auger, Simcoe 4000 tracked rig DATE COMPLETED: 10/10/2003
SAMPLING METHOD: SPT w/ Cathead LOGGED BY: B. Thurber
LOCATION: See Site & Exploration Plan, Figure 2

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
0	GM	Cuttings and drill action: Loose, brown, silty, sandy, fine to coarse GRAVEL, moist. Gravel subrounded. (RECENT FILL)			
2	GM	At 2 feet cuttings become dark brown. Drill action generally gravelly, from 2 to 33 feet. Dense, gray-brown, silty, fine to medium sandy, fine to coarse GRAVEL, moist. Trace wood fibers. (OLDER FILL with REFUSE) Hard, gravelly drill action at 3-3.5 and 6-6.5 feet.	S-1	24-17-19	
		Dense, dark brown to blackish-brown, with minor light brown, organic, silty, fine to medium sandy, fine to coarse GRAVEL, moist. Mild peaty odor. Hard, gravelly drill action at 10 to 11 feet.	S-2	11-17-19	
		Medium dense, stratified blackish-brown to dark gray, silty, sandy, fine to coarse GRAVEL, moist. Partly decomposed wood fragments in 1-inch layer at 14 feet.	S-3	11-8-6	
		Medium dense, reddish brown and gray-brown grading to blackish-brown, silty, sandy, fine to coarse GRAVEL, moist. Mild peaty odor. Trace broken glass, brick, and plastic sheeting in blackish-brown portion (18.5-19.5 feet). 2-inch thick partly decomposed wood.	S-4	4-8-10	
		Medium dense, stratified blackish-brown, blue-gray, and brown, organic, silty, sandy, fine to coarse GRAVEL, moist grading to wet. Trace broken glass and porcelain.	S-5	7-10-12	
30		Medium dense, dark brown with some blackish-brown, silty, sandy, fine GRAVEL, wet. Trace broken glass, brick fragments, partly decomposed wood fragments.	S-6	7-10-12	GS pH/R

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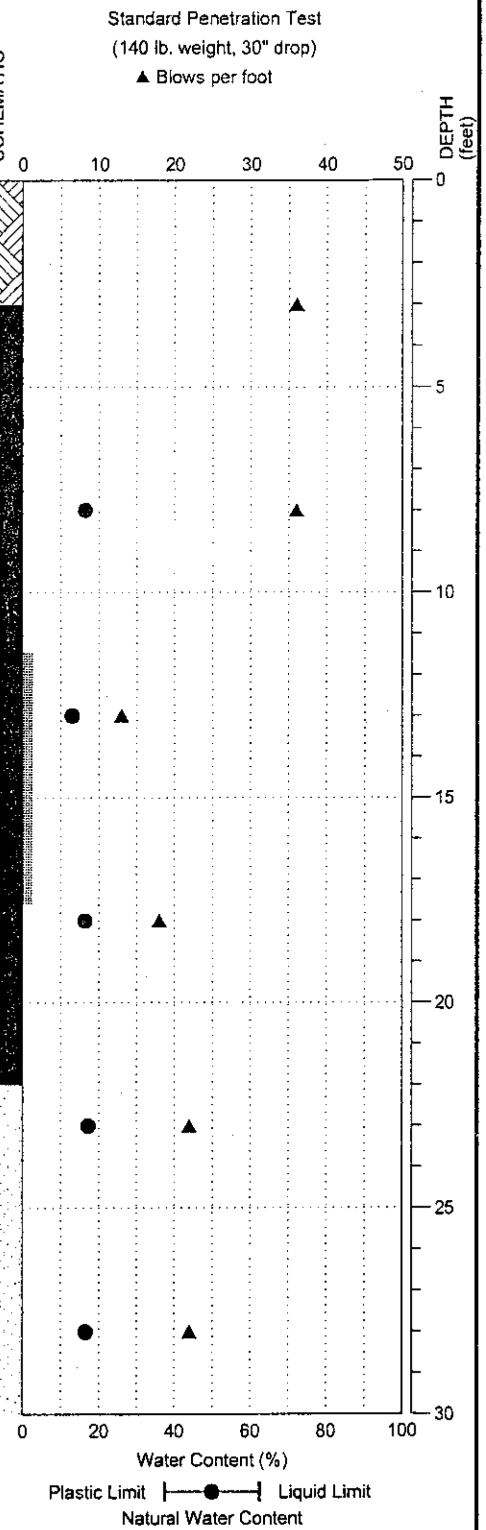

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HWA GEOSCIENCES INC.

BOW LAKE TRANSFER STATION
FMP UPDATE AND IMPLEMENTATION
TUKWILA, WASHINGTON

BORING: BH-4

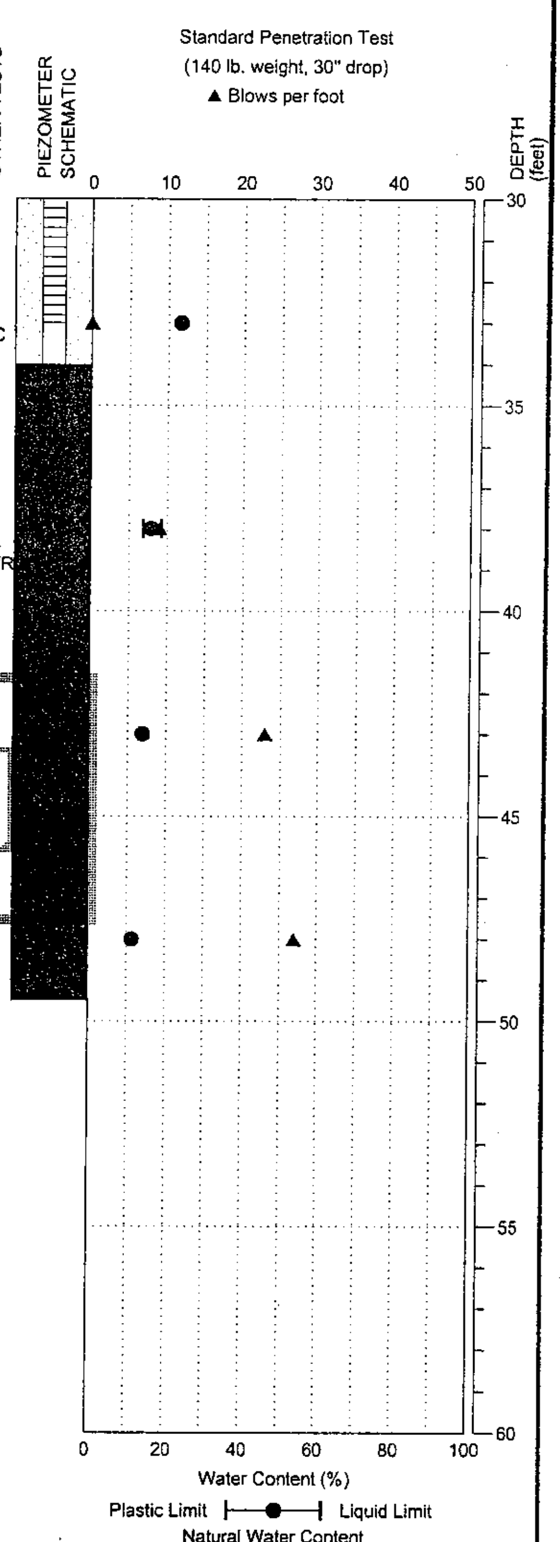
PROJECT NO.: 2003-008 FIGURE: A4

PZO 2003008.GPJ 1/9/04

DRILLING COMPANY: Holocene Drilling
SURFACE ELEVATION: 286.00 ± feet
DATE STARTED: 10/9/2003
DRILLING METHOD: Hollow-Stem Auger, Simcoe 4000 tracked rig
DATE COMPLETED: 10/10/2003
SAMPLING METHOD: SPT w/ Cathead
LOGGED BY: B. Thurber
LOCATION: See Site & Exploration Plan, Figure 2

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
30–35		Medium dense, dark brown, silty, gravelly SAND, wet. Trace glass fragments, including melted amber and green glass; trace partly decomposed wood fragments.	S-7		OC
35–41	SM	Loose, light gray and olive-brown, very silty, fine to medium SAND, moist. Rootlets at 39 feet. Massive appearance. **(WEATHERED DRIFT / COLLUVIUM)**	S-8	3-4-5	AL pH/R
41–49.5	SP	Medium dense, yellow-brown, clean, fine to medium SAND, damp to moist. 3 lenses of rust-brown, slightly silty sand. **(ICE CONTACT STRATIFIED DRIFT)**	S-9	7-12-11	
		Medium dense, light gray, clean, fine to medium SAND, moist. Stratified, with 3-5 mm lenses of slightly silty sand.	S-10	7-11-16	

Borehole terminated at 49.5 feet.
Minor ground water seepage encountered between 25 and 35 feet.
Piezometer installed to 33 feet (1-inch schedule 40 PVC).



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HWA GEOSCIENCES INC.

BOW LAKE TRANSFER STATION
FMP UPDATE AND IMPLEMENTATION
TUKWILA, WASHINGTON

BORING: BH-4

PROJECT NO.: 2003-008 FIGURE: A4

PZO 2003008.GPJ 1/9/04

DRILLING COMPANY: Holocene Drilling
SURFACE ELEVATION: 279.00 ± feet
DRILLING METHOD: Hollow-Stem Auger, Moblie B-61 truck rig
SAMPLING METHOD: SPT w/ Autohammer
LOCATION: See Site & Exploration Plan, Figure 2

DATE STARTED: 10/9/2003
DATE COMPLETED: 10/9/2003
LOGGED BY: B. Thurber

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
0		SM	6 inches sawdust (next to small pile) over quarry spall over loose to medium dense, olive-brown, silty SAND, damp. **(RECENT FILL)**					
		SP SM						
5			Medium dense, olive-brown, clean to slightly silty, fine to medium SAND, moist. (ICE CONTACT STRATIFIED DRIFT)		S-1	5-7-9		
10			Medium dense, light olive-brown, clean, fine to medium SAND, moist. Minor silty sand lenses, 1-10 mm thick.		S-2	8-13-14		
15			Medium dense, light olive-brown, slightly silty, fine to medium SAND, moist.		S-3	8-13-14	GS pH/R	
20			Very dense, light gray, clean, fine to medium SAND, damp.		S-4	14-26-35		
25			Dense, light gray, clean, fine to medium SAND, dry to damp. Stratified, 1- to 8-inch thick beds.		S-5	15-15-18	GS pH/R	
30								

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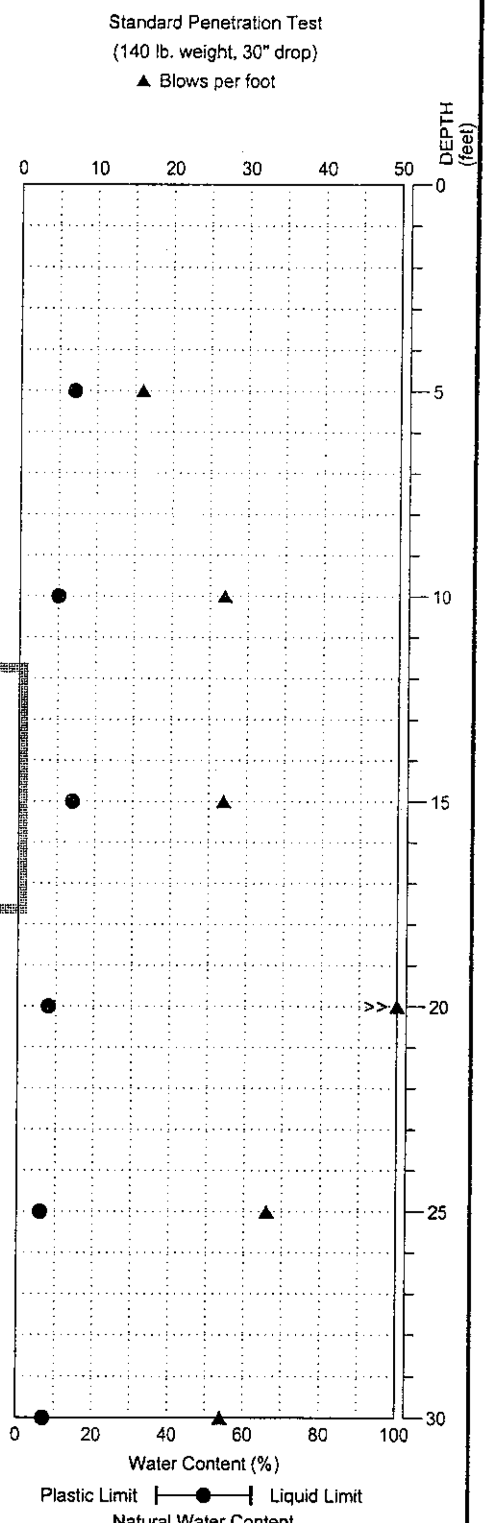

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HWA GEOSCIENCES INC.

BOW LAKE TRANSFER STATION
FMP UPDATE AND IMPLEMENTATION
TUKWILA, WASHINGTON

BORING:
BH-5

PROJECT NO.: 2003-008
FIGURE: A5

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Hollow-Stem Auger, Moblie B-61 truck rig
SAMPLING METHOD: SPT w/ Autohammer
LOCATION: See Site & Exploration Plan, Figure 2

SURFACE ELEVATION: 279.00 ± feet

DATE STARTED: 10/9/2003
DATE COMPLETED: 10/9/2003
LOGGED BY: B. Thurber

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
30			Medium dense, light gray, clean, fine to medium SAND, damp. Massive.		S-6	8-12-15		
35			Dense, light gray, clean, fine to medium SAND, damp. Stratified, beds 1- to 6-inches thick, with 4 mm silty sand lens.		S-7	9-16-20		
40			Very dense, light gray, clean, fine to medium SAND, dry to damp. Massive.		S-8	15-26-26		
			Borehole terminated at 41.5 feet. No ground water seepage encountered during drilling. Borehole abandoned with bentonite chips.					

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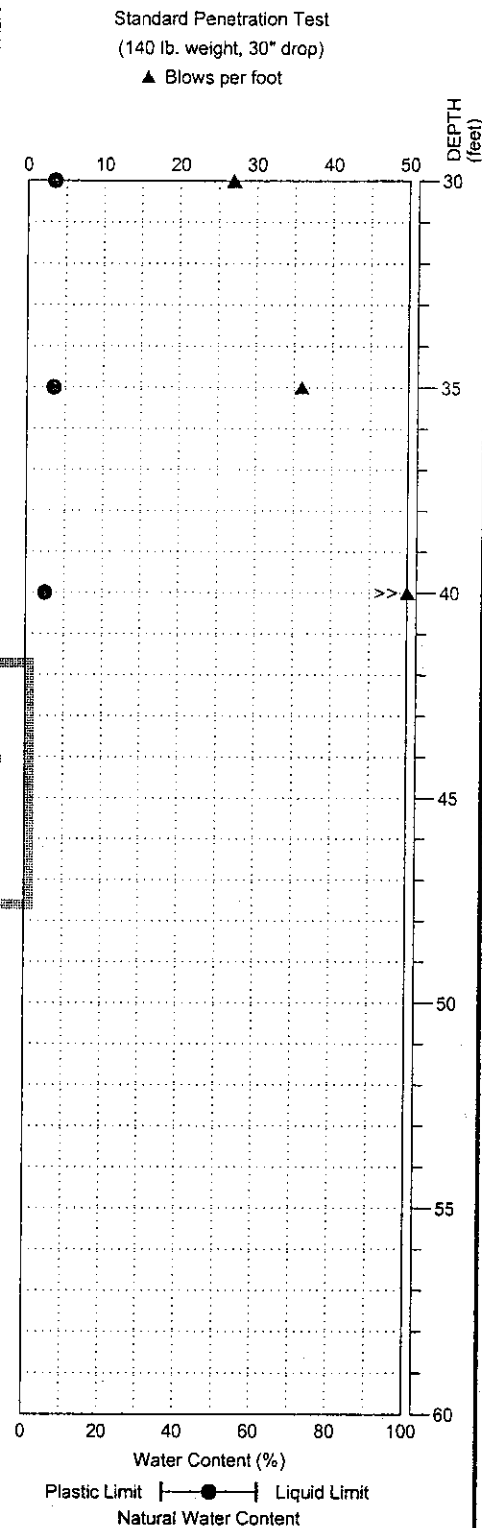

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HWA GEOSCIENCES INC.

BOW LAKE TRANSFER STATION
FMP UPDATE AND IMPLEMENTATION
TUKWILA, WASHINGTON

BORING:
BH-5

PAGE: 2 of 2

PROJECT NO.: 2003-008   FIGURE: A5

APPENDIX B

LABORATORY TEST RESULTS

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APPENDIX B

LABORATORY TESTING

Representative soil samples obtained from the borings were returned to the HWA laboratory for further examination and testing. Laboratory tests were conducted on selected soil samples to characterize relevant properties of the on-site soils. The laboratory testing program was performed in general accordance with appropriate standards as outlined below:

MOISTURE CONTENT OF SOIL: The moisture content of selected soil samples (percent by dry mass) was determined in general accordance with ASTM D 2216. The results are shown at the sampled intervals on the appropriate summary logs in Appendix A.

LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS (ATTERBERG LIMITS): Selected samples were tested using method ASTM D 4318, multi-point method. The results are reported on the Liquid Limit, Plastic Limit, and Plasticity Index chart, Figure B1.

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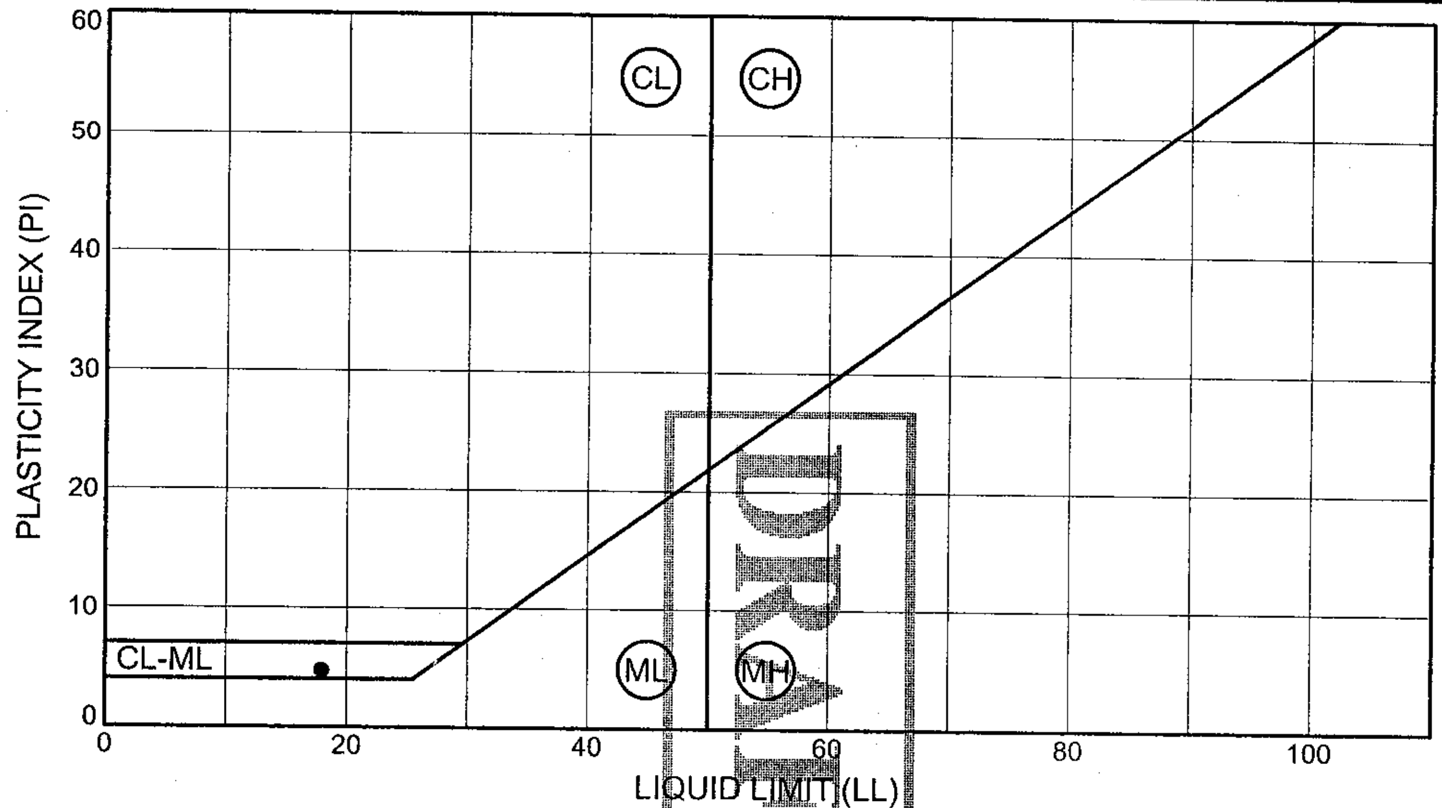
PARTICLE SIZE ANALYSIS OF SOILS: Selected samples were tested to determine the particle size distribution of material in general accordance with ASTM D422. The results are summarized on the Grain Size Distribution reports, Figures B2, B3 and B4, which also provide information regarding the classification of the sample.

RESISTIVITY TEST RESULTS: Testing was carried out on selected samples using WSDOT Test Method No. 417. The indicated minimum resistivity test results are tabulated as follows:

Sample	Sample Depth (feet)	Resistivity (ohm-cm)	Soil Type
BH-2, S-3	15 – 16.5	7,500	Native Glacial
BH-2, S-5	25 – 26.5	17,000	Native Glacial
BH-3, S-5	17 – 18.5	2,500	Older Fill
BH-3, S-6	28 – 29.5	1,500	Older Fill
BH-4, S-6	28 – 29.5	1,500	Older Fill
BH-5, S-8	38 – 39.5	2,400	Weathered Glacial/Colluvium
BH-5, S-3	15 – 16.5	17,000	Native Glacial
BH-5, S-5	25 – 26.5	26,000	Native Glacial

Moisture Content, Ash, and Organic Matter: Selected samples were tested in general accordance with method ASTM D 2974, using moisture content Method 'A' (oven dried at 105^{0} C) and ash content Method 'C' (burned at 440^{0} C). The test results are tabulated below, and represent percent by weight of dry soil.

Sample	Moisture Content (%)	Ash Content (%)	Organic Content (%)
BH-3, S-6	15.6	98.1	1.9
BH-3, S-7	24.3	96.5	3.5
BH-4, S-7	19.4	94.4	5.6



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	LL	PL	PI	% Fines
●	BH-4	S-8	38.0 - 39.5	(CL-ML) Olive brown, silty CLAY	16	18	13	5	

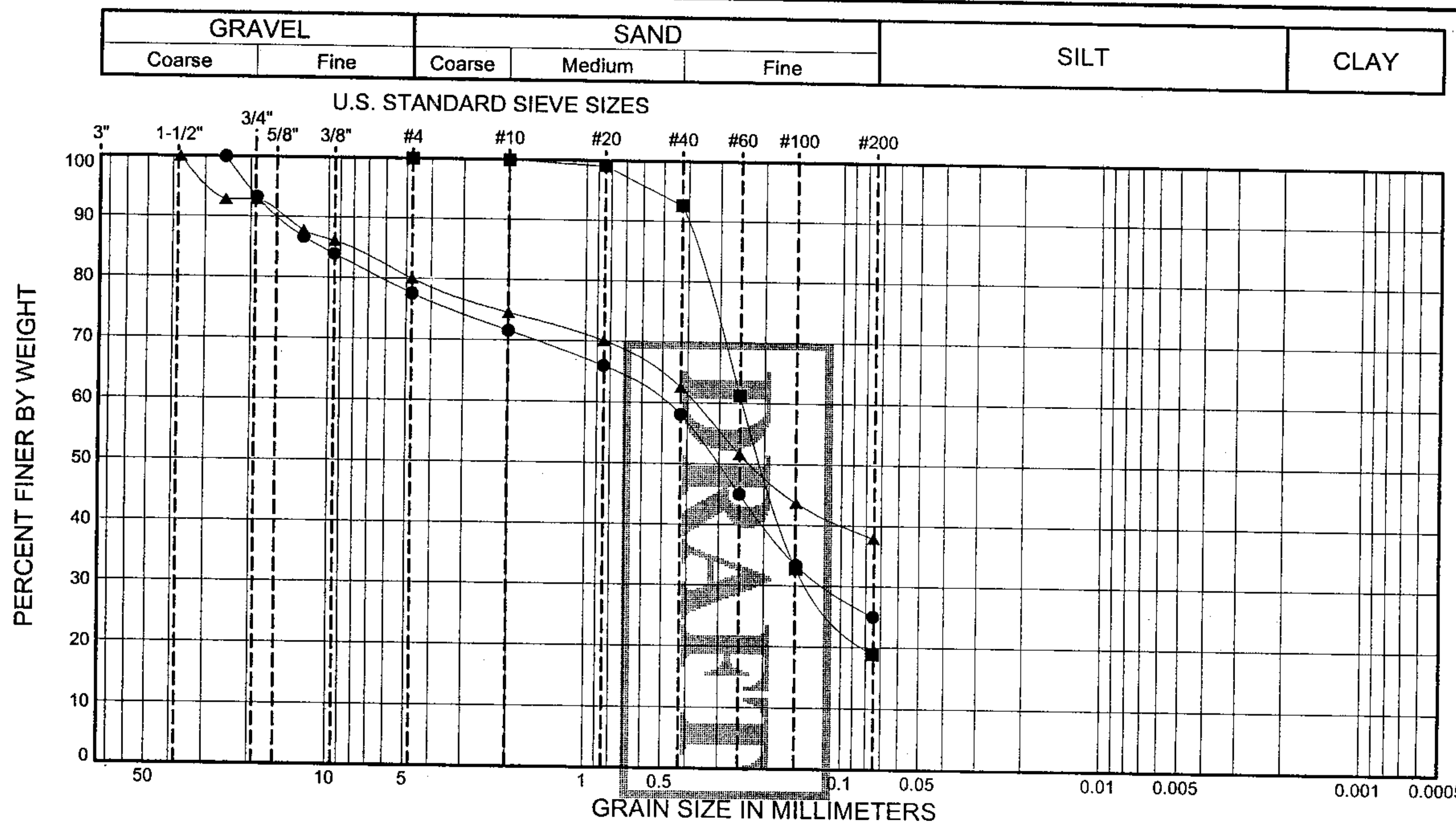
HWA GEOSCIENCES INC.

BOW LAKE TRANSFER STATION
FMP UPDATE AND IMPLEMENTATION
TUKWILA, WASHINGTON

LIQUID LIMIT, PLASTIC LIMIT AND
PLASTICITY INDEX OF SOILS
METHOD ASTM D4318

PROJECT NO.: 2003-008 FIGURE: B1

HWAATTB 2003008.GPJ 10/23/03



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION OF SOIL- ASTM D2487 Group Symbol and Name	% MC	LL	PL	PI	Gravel %	Sand %	Fines %
●	BH-2	S-2	10.0 - 10.7	(SM) Dark yellowish brown, silty SAND with gravel	9				22.4	52.5	25.0
■	BH-2	S-5	25.0 - 26.5	(SM) Olive brown, silty SAND	10					81.1	18.9
▲	BH-3	S-1	3.0 - 4.5	(SM) Light yellowish brown, silty SAND with gravel	6				20.0	42.0	37.9

HWA GEOSCIENCES INC.

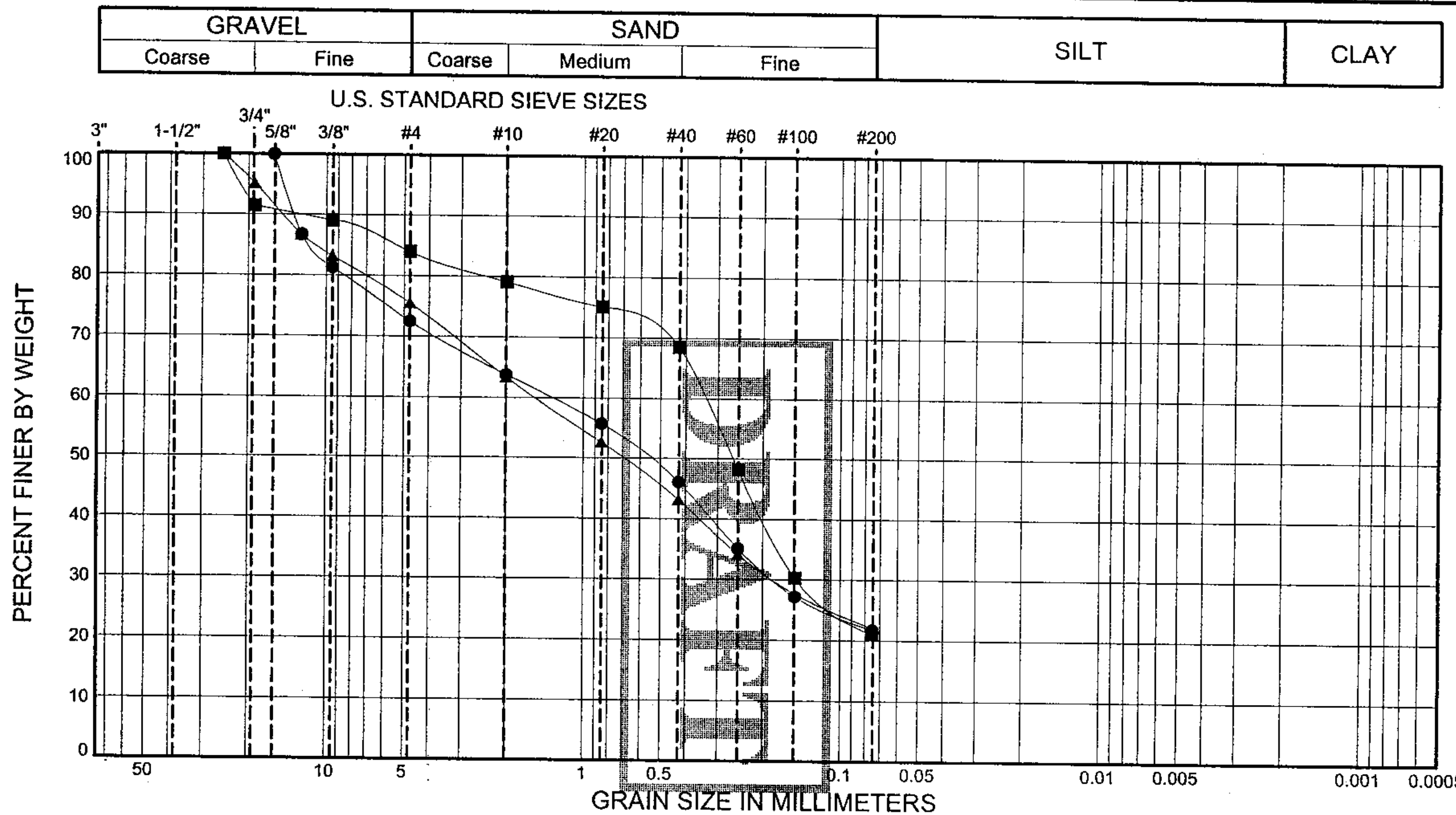
BOW LAKE TRANSFER STATION
FMP UPDATE AND IMPLEMENTATION
TUKWILA, WASHINGTON

PARTICLE-SIZE ANALYSIS OF SOILS METHOD ASTM D422

PROJECT NO.: 2003-008

FIGURE: B2

HWAGRSZ 2003008.GPJ 10/23/03



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION OF SOIL- ASTM D2487 Group Symbol and Name	% MC	LL	PL	PI	Gravel %	Sand %	Fines %
●	BH-3	S-6	28.0 - 29.5	(SM) Dark brown, silty SAND with gravel(1.9 % organics)	16				27.5	50.7	21.8
■	BH-3	S-7	33.0 - 34.5	(SM) Black, silty SAND with gravel(3.5% organics)	25				15.9	63.0	21.1
▲	BH-4	S-6	28.0 - 29.5	(SM) Dark brown, silty SAND with gravel	16				24.5	53.4	22.1

HWA GEOSCIENCES INC.

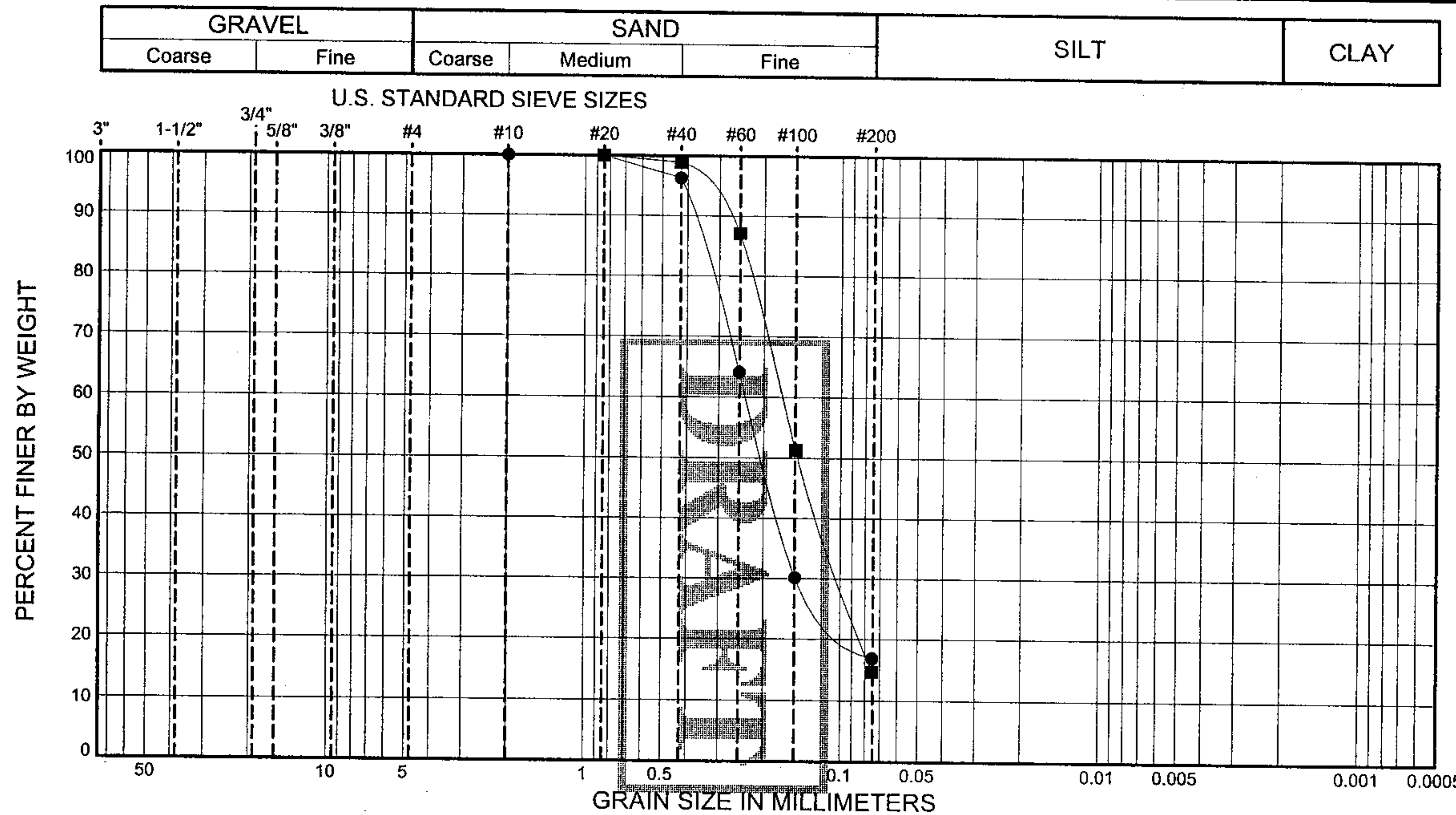
BOW LAKE TRANSFER STATION
FMP UPDATE AND IMPLEMENTATION
TUKWILA, WASHINGTON

PARTICLE-SIZE ANALYSIS
OF SOILS
METHOD ASTM D422

PROJECT NO.: 2003-008

FIGURE: B3

HWAGRSZ 2003008.GPJ 10/23/03



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION OF SOIL- ASTM D2487 Group Symbol and Name	% MC	LL	PL	PI	Gravel %	Sand %	Fines %
●	BH-5	S-3	15.0 - 16.5	(SM) Olive brown, silty SAND	14					83.1	16.9
■	BH-5	S-5	25.0 - 26.5	(SM) Grayish brown, silty SAND	6					85.3	14.7

HWA GEOSCIENCES INC.

BOW LAKE TRANSFER STATION
FMP UPDATE AND IMPLEMENTATION
TUKWILA, WASHINGTON

PARTICLE-SIZE ANALYSIS OF SOILS METHOD ASTM D422

PROJECT NO.: 2003-008 FIGURE: B4

HWAGRSZ 2003008.GPJ 10/23/03

CCI
Analytical
Laboratories, Inc.

CERTIFICATE OF ANALYSIS

CLIENT: HWA GEOSCIENCES INC.
19730 64TH AVE. W., SUITE 200
LYNNWOOD, WA 98036

DATE: 10/22/03
CCIL JOB #: 310052
CCIL SAMPLE #: 1
DATE RECEIVED: 10/13/03
WDOE ACCREDITATION #: C142

CLIENT CONTACT: BRIAN HALL

CLIENT PROJECT ID: BOW LAKE TRANSFER STA. #2003-008
CLIENT SAMPLE ID: BH-3, BH-4 CUTTINGS 10/10/03 16:00

DATA RESULTS

ANALYTE	METHOD	RESULTS*	UNITS**	ACTION LEVEL***	ANALYSIS DATE	ANALYSIS BY
TCLP ARSENIC	EPA-1311/6010	ND(<0.04)	MG/L	5.0 MG/L	10/20/03	RAB
TCLP BARIUM	EPA-1311/6010	1.6	MG/L	100.0 MG/L	10/20/03	RAB
TCLP CADMIUM	EPA-1311/6010	0.016	MG/L	1.0 MG/L	10/20/03	RAB
TCLP CHROMIUM	EPA-1311/6010	0.047	MG/L	5.0 MG/L	10/20/03	RAB
TCLP LEAD	EPA-1311/6010	1.2	MG/L	5.0 MG/L	10/20/03	RAB
TCLP MERCURY	EPA-1311/7470	ND(<.0002)	MG/L	0.2 MG/L	10/21/03	RAB
TCLP SELENIUM	EPA-1311/6010	ND(<0.04)	MG/L	1.0 MG/L	10/20/03	RAB
TCLP SILVER	EPA-1311/6010	ND(<0.03)	MG/L	5.0 MG/L	10/20/03	RAB

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* "ND" INDICATES ANALYTE ANALYZED FOR BUT NOT DETECTED AT LEVEL ABOVE REPORTING LIMIT. REPORTING LIMIT IS GIVEN IN PARENTHESES

** UNITS FOR ALL NON LIQUID SAMPLES ARE REPORTED ON A DRY WEIGHT BASIS

*** ACTIONS LEVELS ARE PROVIDED ONLY WHEN PARAMETER DATA IS USED FOR A GENERALLY CONSISTENT APPLICATION. WHEN PROVIDED, THEY SHOULD BE USED AS GUIDELINES ONLY. THE APPROPRIATE REGULATORY DOCUMENT SHOULD BE CONSULTED BEFORE MAKING ANY DECISIONS BASED ON ANALYTICAL DATA

APPROVED BY: [signature]

8620 Holly Drive • Everett, WA 98208 • 425 356-2600 • FAX 425 356-2626 • Seattle 206 292-9059

APPENDIX C

PREVIOUS INVESTIGATION RESULTS

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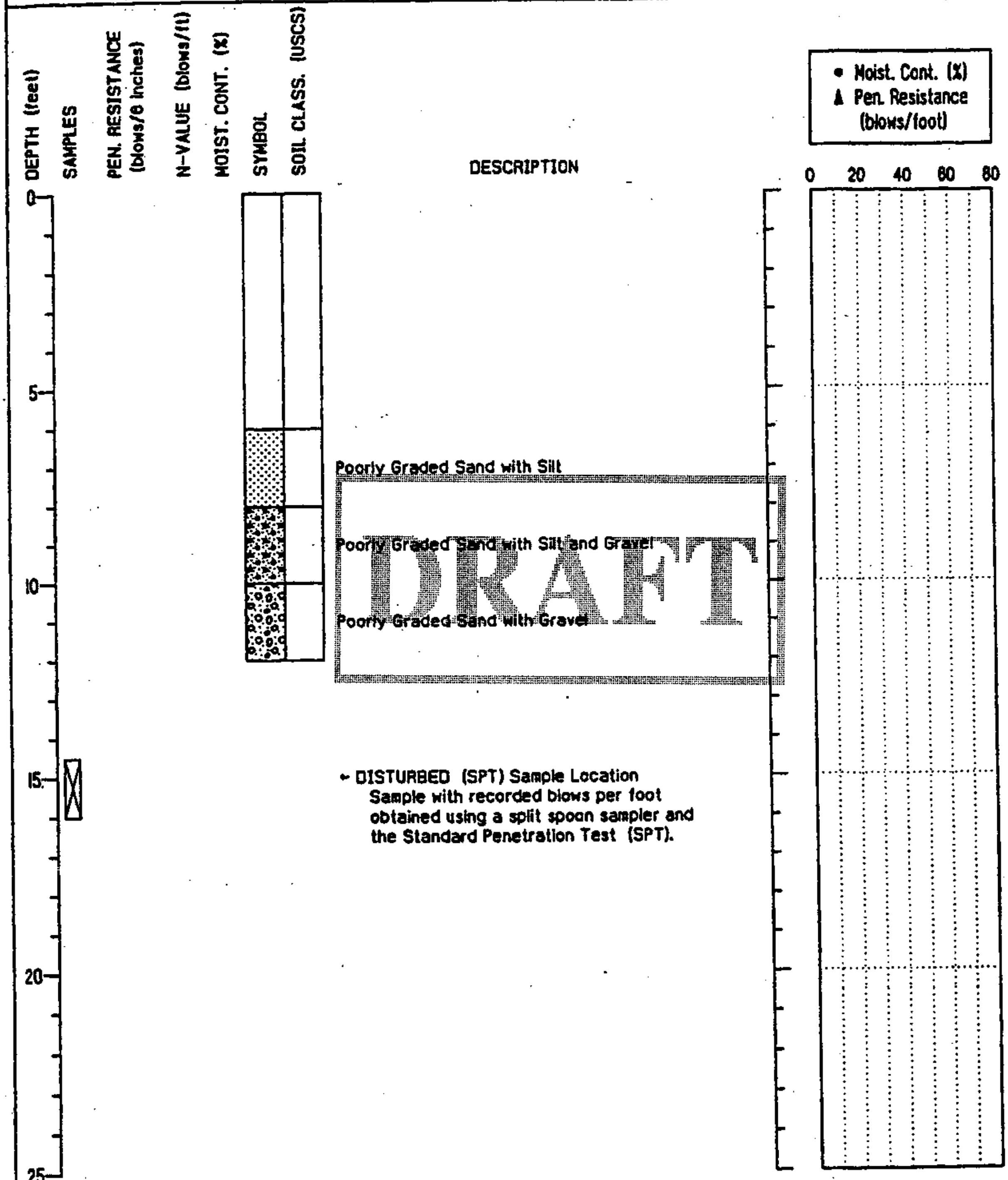
HONG WEST & ASSOCIATES, INC.

BORING LOG

DRILLING COMPANY:
DRILLING METHOD:
SAMPLING METHOD:

TOTAL DEPTH: Feet
SURFACE ELEVATION: Feet
MEASURING POINT EL.: Feet



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated.

PROJECT: BOW LAKE TRANSFER STA.

BORING: LEGEND

LOCATION: King County, Washington
DATE COMPLETED:
LOGGED BY:

PROJECT NUMBER: 93112-2

PAGE: 1 OF 1

Figure 2

HONG WEST & ASSOCIATES, INC. — BORING LOG

DRILLING COMPANY: Pacific Testing Laboratories
DRILLING METHOD: 4-inch I.D. CFHS Auger
SAMPLING METHOD: SPT

TOTAL DEPTH: 44.0 Feet
SURFACE ELEVATION: ±273 Feet
MEASURING POINT EL.: Feet

DEPTH (feet)	SAMPLES	PEN. RESISTANCE (blows/6 inches)	N-VALUE (blows/ft)	MOIST. CONT. (%)	SYMBOL	SOIL CLASS. (USCS)	DESCRIPTION
0							Loose to medium dense, brown, gravelly SAND: moist. Contains glass, slag, concrete and brick fragments.
							Obstruction at 2 feet. Pulled auger and moved 2 feet east.
	X	11-16-10	26				Obstructions between 3.5 and 8.5 feet. Refusal. Pulled auger and moved east 2 feet.
5							(FILL)
	X	11-12-50/2"					Artifically high blow count.
10							Soil is dark gray below ±10 feet with higher percent of fine gravel. Cobbles @ 10.5 feet.
							Medium dense, gray, poorly graded, fine to medium SAND with silt and gravel: moist. Numerous pieces of glass. Also wire, ashes, wood and a nail.
	X	8-9-12	21				(FILL)
15							
	X	17-10-16	26				Soil is similar but color varies from light to dark gray: slightly higher gravel content.
20							Obstruction @ 20 feet.
	X	20-25-26	51				Soil sample is brown. ▽ El 250' Possible Parking
25							Gravelly from 24.5 to 26.0 feet.

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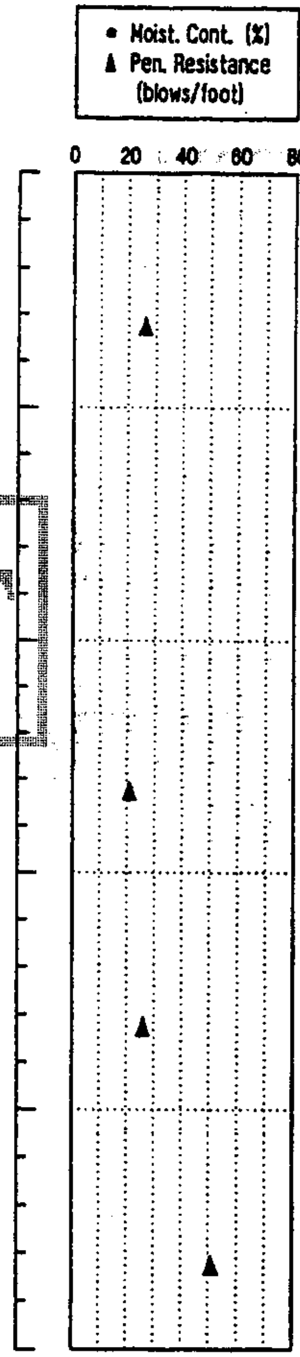


NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated.

PROJECT: BOW LAKE TRANSFER STA. BORING: BH-1

LOCATION: North WSDOT Fill Area
DATE COMPLETED: 02/21/94
LOGGED BY: Rod Faubion

PROJECT NUMBER: 93112-2

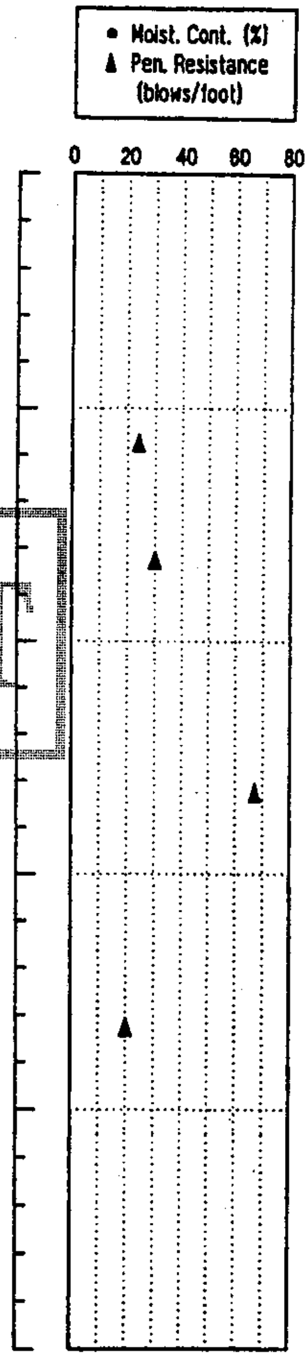
Figure 3

HONG WEST & ASSOCIATES, INC.
BORING LOG

DEPTH (feet)	SAMPLES	PEN. RESISTANCE (blows/6 inches)	N-VALUE (blows/ft)	MOIST. CONT. (%)	SYMBOL	SOIL CLASS. (USCS)	DESCRIPTION
25							
28	X	20-50/6"					At 28 feet, sampler bounces: wood in shoe of sampler. Artifically high blow count.
30	X	5-9-15	24				Medium dense, gray, poorly graded, fine to medium SAND with silt and gravel: moist. Few large (±¼-inch) roots in upper part of sample.
32.5	X	5-15-15	30				Sand with scattered 1" beds of silty sand.
35							
37.5	X	15-22-45	67				Becomes dense to very dense.
40							
42.5	X	10-10-10	20				Becomes medium dense.
45							
50							

DRAFT

Bottom of Boring at 44.0 feet.
SPT samples taken with free falling 140 lb. hammer dropping 30 inches.



PROJECT: BOW LAKE TRANSFER STA. **BORING: BH-1**

LOCATION: North WSDOT Fill Area
DATE COMPLETED: 02/21/94
LOGGED BY: Rod Faubion

PROJECT NUMBER: 93112-2

PAGE: 2 OF 2

Figure 4

TECHNICAL MEMORANDUM

TO: Karl Hufnagel, P.E. / R W Beck

PREPARED BY: Brian Hall, P.E. / HWA GeoSciences Inc.

SUBJECT: **FIELD VISIT - OBSERVATIONS AND CONCLUSIONS**
Bow Lake Transfer Station/Recycling Facility
King County, Washington

PROJECT NO.: 2003-008-21

DATE: March 24, 2005

This Technical Memorandum summarizes our observations and conclusions resulting from a field visit made to the Bow Lake Transfer Station/Recycling Facility on March 23, 2006. In addition to Brian Hall of HWA GeoSciences Inc. (HWA), the following were also present during the visit; Ian Sutton of RW Beck; Steve Bingham, Teresa Vanderburg and Deron Lozano of Adolfson; and Duane Hartman of Duane Hartman & Associates.

HWA previously prepared a draft geotechnical report of the WSDOT property, dated January 9, 2004. This previous report should be referred to for geological and geotechnical background information.

WSDOT PROPERTY

This current site visit was specifically undertaken to assess the steeply-sided ravine on the north side of the WSDOT property because the draft Facility Master Plan envisages a roadway with large retaining walls in close proximity to the ravine.

Observations

- The ravine is deep and has slopes averaging about 50% to 70%, except at the head where a near vertical face of about 15 to 20 feet high occurs. Below the near vertical face, extensive deposits of debris and colluvium are present.

- The near vertical face appears to have developed partly from erosion caused by discharge from a culvert located about 30 feet back from the face, and degradation caused by root wedging and freeze-thaw effects. We consider that degradation is currently occurring more aggressively than erosion because there is an absence of deep erosion scars and material falling away in slabs of 6 to 18 inches thick. In addition, the near vertical face extends over a width of 50 to 70 feet, which is a much greater width than would be expected from the size of the culvert.

- The material exposed in the near vertical face consists of dense, gray brown, uniform, massive, fine to medium sand, and is consistent with the soils encountered in our previous geotechnical borings. In these borings, we interpreted the soils as ice contact drift.

- No seepage was observed in the ravine slopes or in the base of the ravine. However, we understand that Adolfson have noticed seepage on the sidewall of the ravine, much lower down the ravine previously.

- A large, grated, catch basin just off the shoulder of I-5 opposite the culvert location appears to be inlet to the culvert. Water was draining into the catch basin from a pipe coming from under I-5 at an estimated flow rate of about 1 to 2 gallons/minute. Despite the flow occurring into the catch basin, no water was discharging from the culvert outlet and into the ravine. If this is the culvert inlet, then the culvert is leaking/broken and water is infiltrating into the permeable ice-contact deposits.

- Many of the trees surrounding the ravine have straight trunks indicating that slope creep is minimal. However, we observed evidence of at least two previous shallow slides in the ravine walls near the culvert outlet.

- Much of the high fill placed on the WSDOT property has been placed slightly back from the crest of the ravine slope leaving an irregular bench of approximately 20 feet wide.

Comments

We consider that the near vertical face will continue to degrade, and depending on the discharge volume through the culvert, will eventually degrade to a slope of around 1.5H:1V; the angle of repose of loose fine to medium sand. The foundations for any walls constructed near the ravine should be located below a line drawn at 1.5H:1V from the toe of the ravine. However, it is possible that buffer requirements for the creek within the ravine may result in the retaining walls being located outside this limit.

The major issue for slope instability is whether seepage may occur from less impermeable silt layers further down the slope. Such zones are particularly susceptible to sliding.

Recommendations

We recommend the following:

1. The land surveyors should develop cross sections at right angles to the slope at critical locations. These cross sections should extend from the base of the ravine to beyond the crest of the existing fill slopes. These sections would then be used as a basis for geological mapping and geotechnical slope stability assessments.

2. Undertake detailed geological mapping along the cross sections to prepare geological profiles for use in slope stability evaluations. The mapping should particularly aim to locate any silt layers and the location of seepage zones.

3. Drill at least 3 borings to prepare a geological profile through the slope to provide subsurface information for slopes stability calculations. Borings should extend to depths of around 50 feet. Alternatively, borings to depths of around 20 feet could be undertaken on the slope using hand portable drilling equipment.

LA PIANTA PROPERTY

This property is located on the south side of the facility, immediately east of the present entrance. The new entrance is planned to go through the property, and large retaining walls will be required to support the outside edge of the new road.

Observations

We could not enter on the property but observed the following from the existing roadway:

- The site is steeply sloping, and has a ravine near the south end (roughly opposite the existing entrance) that likely carries runoff draining from I-5.

- A flatter zone is present where the planned new roadway would enter into the existing transfer station site. Much of this area appears to be underlain by solid waste.

- The trees on the property generally have straight trunks and are not bent as is often associated with soil creep and sliding. The understory consists of brush and blackberry vines.

- The adjacent segment of entrance road is supported on a low fill, but is performing well despite the very heavy traffic.

Comments

For planning purposes at least 2 borings should be drilled on the property to allow slope stability and over stability of a retaining wall and provide information for foundation selection of the retaining wall.

TECHNICAL MEMORANDUM

TO: Karl Hufnagel, P.E. / R.W. Beck

PREPARED BY: Brad Thurber, L.E.G., Sa Hong, P.E. / HWA GeoSciences Inc.

SUBJECT: **SLOPE GEOTECHNICAL ISSUES**
Bow Lake Transfer Station/Recycling Facility
King County, Washington

PROJECT NO.: 2003-008-21

DATE: November 16, 2006

The purpose of this Technical Memorandum is to address qualitatively potential slope stability issues for design and reconstruction of the site facilities. During the master planning process, geotechnical conditions for the proposed reconstruction of the facility were evaluated by reviewing previous geotechnical reports for the existing facility, advancing several exploration borings and preparing a draft geotechnical report for the WSDOT parcel (HWA, 2004). Additionally, a geologic reconnaissance was performed in 2006 of slope conditions along several transects within the subject properties. This memorandum looks at two separate parcels that together make up the proposed project site: The WSDOT property and the King County parcel as shown in the attached Figure 2.

WSDOT Parcel

Observed Conditions

North Slope

Fill has been stockpiled to elevation 315 feet at the top of the north- to northeast-facing slope of the WSDOT parcel. This slope comprises part of a ravine bordering the north side of the site. The ravine bottom extends east-west and lies at elevation 170 feet, due north of the highest portion of the stockpile, and at 90 feet elevation to the northeast. The base of the stockpiled fill is approximately at elevation 250 feet (i.e. the fill is a maximum of 65 feet thick), based on previous borings and slope observations (HWA, 2004). The fill side slopes are inclined at about 20% in the upper half and about 30% in the lower half (see Slope Profile 1, Figure 3). The stockpiled fill consists of silty sand with gravel (till fill) in the upper 35 feet, and silty sand with cinder, ash, melted glass, and metal (derived from former refuse burning) in the lower 30 feet. Maple trees up to 18 inches in diameter were growing on the lower 30 feet or so of the stockpile, and partly vegetated erosional rills up to 2 feet deep and 5 feet wide were present. The fill is somewhat setback from the lower natural slope to the north, and the toe of the fill tapers into a gently sloping bench, from about 20 to 50 feet wide. Earthwork for the new facility will consist of the removal of a 40- to 60-foot thickness of old fill, and construction of a perimeter road that is partly on fill. The transfer station building will be founded partly on old fill and partly on native soils. Utilities will be trenched through the site as well.

The surficial soils along the bench are gravelly and very dense. The outer edge of the bench is the crest of the natural steep ravine slope, which descends at gradients of approximately 55 to 60% in the upper 35 feet, steepening to approximately 75 to 80% for the remainder of the slope down to a stream at the ravine bottom. The ravine slope soils consist of loose sand (colluvium) over very dense, clean, sand. The slope surface was probed with a 3-foot long, ½-inch diameter, steel rod. The thickness of loose sand was observed to be as little as ½ to 1 foot in an erosional side ravine, and 2½ feet to more than 3 feet along most of the slope. Many of the trees on the ravine slope have straight trunks indicating that slope creep is minimal. However, we observed evidence of at least two recent shallow slides in the 15- to 20-foot high walls near the ravine head (west end) by a culvert outlet.

Ground water seepage was observed at the western end of the ravine in very dense sand, at about 100 feet east of the head and a few feet above the stream, and at the heads of two side ravines approximately 100 feet higher than the stream.

East Slope

The eastern slope of the WSDOT parcel consists of the soil stockpile in the upper approximately 65 feet, inclined at up to 55%. The east property line fence is at or within 10 feet of the stockpile toe. Eastward from the toe is a gently-sloped bench, contiguous with the bench on the north side of the stockpile, inclined at approximately 20% and just over 100 feet in width. The outer edge of the bench forms the crest of the steep natural slope, inclined at approximately 85% in the upper 40 feet or so, then sloping more gently into a bowl-shaped area, with a second bowl down slope from it. Such bowls are a typical geomorphic expression of former land sliding. The slope beneath the second bowl down to the Green River Valley floor could not be observed from the bench (the natural slope was not reconnoitered below the bench, only observed from above). The bowls were vegetated with blackberry brambles, an indicator of ground disturbance (such as by land clearing or grading activities, or by sliding). Based on preliminary observations of the site and its surroundings, it appears that the east property line of the WSDOT parcel is a sufficient distance (approximately 100 feet) from the crest of the steep slope to provide an adequate buffer for on-site development up to the property line.

General Observations and Conclusions

The slopes on the north and east sides of the WSDOT parcel are judged to be Class 2, 3, and 4 per the City of Tukwila Municipal Code sensitive areas designation (TMC 18.45.120 A.). The ravine slope profile is convex, with the steepest portions at the bottom of the ravine. The slopes also are mapped by the City as an erosion hazard area.

No evidence of deep-seated sliding is evident on these slopes, only surficial soil creep and isolated shallow sliding. Such shallow slope movement will occur periodically over time as the underlying very dense sand mechanically weathers. Most of this movement will occur in the steep lower portion of the ravine slope, in relation to where ground water seepage occurs.

The natural processes of soil creep and skin sliding will continue whether or not the redevelopment takes place. However, present runoff to the slopes that could contribute to creep and shallow sliding will be reduced by the capture of stormwater and re-direction from proposed impervious surfaces. Such slope movement would not pose a threat to the proposed development. An appropriate buffer distance between the crest of steep natural slopes and proposed site development will be recommended by the geotechnical engineer during the design phase and approved or revised by the City, per Tukwila Municipal Code 18.45.120 C. Geotechnical borings will be conducted along the north-facing slope as needed to develop a geologic profile for slope and seismic stability calculations. Such calculations will determine current stability conditions and relative changes in stability as a consequence of construction. Geotechnical borings within the old fill will be advanced to determine appropriate foundation types for the building.

The outer edge of the perimeter access road, as envisioned in the preliminary design, will be supported on several feet of fill for a portion of the roadway. Some of the fill could be placed as a side-cast fill, where the existing cut bench is wide enough to accommodate the fill and steep slope buffer. The fill footprint could be reduced by construction of a retaining wall such as a mechanically-stabilized earth (MSE) wall, or a soldier pile wall. Geotechnical borings will be advanced and engineering analyses conducted in order to determine appropriate wall types and design details.

The proposed removal of approximately 40 to 60 feet of the existing fill stockpile will eliminate long-term issues of erosion and slope stability for that portion of the slopes. Stormwater runoff to the remaining fill and bench slopes will be significantly reduced by capture and tight-lining of stormwater from graded areas during construction, and over the long-term from proposed impervious surfaces (roofs and pavements). These remaining slopes will be graded and vegetated to reduce erosion potential.

Construction-period mitigations include maintaining temporary site grades to drain away from the steep slopes. Additionally, typical Temporary Erosion and Sediment Control Best Management Practices (TESC BMPs), such as silt fencing, mulch berms and placement of mulch and/or seeding in disturbed areas after grading, should be employed to limit potentially adverse effects of stormwater runoff during construction. Temporary soil stockpiles should be kept away from the crest of the steep slope a horizontal distance equal to twice the height of the stockpile, or other setback distance determined to be safe on the basis of detailed stability calculations.

The proposed facility reconstruction will result in a significant reduction of existing fill on the WSDOT parcel (e.g. a 40- to 60-foot thickness will be removed) such that construction of the new transfer station building and associated roadways will result in a net reduction of load at the top of the slopes. This fill removal will generally improve overall global (deep-seated) stability of the natural slopes below.

In summary, based on the preliminary investigations completed over the last several years, we do not see any slope conditions on the WSDOT parcel that present slope stability or slope erosion problems that cannot be dealt with through the application of conventional, routine, geotechnical design practices and TESC BMPs.

King County Parcel

Observed Conditions

The east slope of the existing transfer station is not as steep as the WSDOT parcel slopes, based on a topographic site plan developed from aerial photos and from our recent field reconnaissance work (see Slope Profiles 2 and 3, Figures 4 and 5). An extensive area of the slope is covered almost exclusively with invasive blackberry brambles, which could be indicative of past ground disturbance, but is more likely the result of landfill slope maintenance by the County. Previous geotechnical explorations (HWA, 1993) indicated fill and refuse depths up to 45 feet at the crest of the slope. The former landfill, with a soil fill cover, extends far down the slope to about elevation 160 feet (HWA, 1993).

Elevations of the eastern slope vary from approximately 245 feet, at the transfer station perimeter road, down to 80 feet at the southeastern property corner. Along most of the southern property line, the slope is traversed in an easterly direction by a 50-foot wide bench, which then traverses northeast and northward along contour and gently slopes to a

100-foot, or so, wide cut bench at the northeastern property corner. A narrow dirt access road (grown over) descends the slope from the north (crossing the LaPianta property from the WDOT parcel) to the wide bench area at the northeast corner.

The slopes above the bench and old road are covered with blackberries, and the surficial soil consists of brown silty sand, with some scattered refuse on the ground surface, consisting of old bottles, tin cans, and other metal and glass debris. Slope inclinations vary from 10 to 48 percent along short distances down the slope, indicative of modified land. The slope below the wide bench is more consistent, with gradients ranging from 40 to 45 percent. Scattered trees present in this lower slope consisted of 8- to 12-inch maples, 12- to 18-inch cottonwoods, and 6- to 8-inch alders, the latter of which had died. The lower trunks of the trees are bent or pistol-butt shaped, indicative of soil creep.

Based on visual observations, the toe of the slope east of the King County parcel, on the LaPianta property, consists of a 25-foot or so high cut slope at about a 1/2H:1V (Horizontal:Vertical) inclination. This steep cut is buttressed by an Ecology Block wall, four blocks high, backfilled with sand and gravel, at the edge of a paved lay-down yard on the Green River valley floor. The soils exposed at the top of the cut appear to be glacially over-consolidated silty sand with gravel.

Two 12-inch storm pipelines are visible on the slope: a corrugated plastic pipe along the upslope edge of the southern property line bench, and a 12-inch concrete-to-corrugated metal pipe located due east of the north end of the existing transfer building. In late October 2006, we observed no water flow at the culvert outlets on the slope, despite heavy rainfalls. However, according to site utility plans and visual observations, stormwater is discharged to the top of the fill slope, to the southeast of the transfer building. Surface expressions of the stormwater discharge consisted of fresh green grass, remains of wet-soil plants (Queen Anne's Lace) not seen elsewhere on the slope, and soft ground underfoot. No surface water was observed.

General Observations and Conclusions

The slopes of the east to south sides of the King County parcel are judged to be Class 2 and 3 per the City of Tukwila Environmentally Sensitive Areas designation (TMC 18.45.120 A). These slopes also are mapped by the City as an erosion hazard area.

No evidence of recent deep-seated sliding is evident on these slopes, and none has been documented in the past 20 years of our experience with the site. We did not observe any signs of erosion on the slopes. Some past soil creep is evident on a portion of the lower slopes within the property. The cut at the slope toe (on La Pianta property) does not show evidence of sliding, and the buttress wall appears to have supported the toe for many years. Any future sliding below the King County property should not affect the proposed development.

Redevelopment of the existing transfer station will result in little to no placement of additional fill, removal of refuse in a small area for new truck scales, and pavement of more area than the current extent of impervious surfaces.

Proper TESC practices will need to be implemented during construction in order to prevent concentration of stormwater onto the slopes.

Placement of any fill in relation to slope stability will be evaluated during the design phase. As appropriate, mitigation measures will be implemented to ensure fill placement is consistent with long-term stability requirements. It is noted that the conceptual design proposed in the Facility Master Plan does not contemplate adding any fill above the existing slope.

Captured stormwater will be either tight lined eastward to an appropriate discharge point on the Green River Valley floor, or dispersed and infiltrated along the slope. A geotechnical slope stability evaluation will be conducted during the design phase to determine an appropriate route and geotechnical parameters for construction of the stormwater pipeline, and/or slope stability for stormwater infiltration. A pipeline could be installed in a trench, or be laid on the ground surface and anchored to the slope with ground anchors such as helical piles.

In summary, based on the preliminary investigations completed over the last several years, we do not see any slope conditions on the King County parcel that present slope stability or slope erosion problems that cannot be dealt with through the application of conventional, routine geotechnical design practices and TESC BMPs.

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