SOUND TRANSIT

UNIVERSITY LINK LIGHT RAIL

PRELIMINARY ENGINEERING

GEOTECHNICAL CONSIDERATIONS REPORT

March 2006

Prepared For:
CENTRAL PUGET SOUND
REGIONAL TRANSIT AUTHORITY

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

PSTC

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LETTER OF TRANSMITTAL

Date: 31 March 2006

To: Mr. Robert Parsons
Sound Transit Link Light Rail
401 S. Jackson St., Union Station
Seattle, WA 98104-2826

Regarding: **SOUND TRANSIT LINK, CIVIL FACILITIES DESIGN
GEOTECHNICAL ENGINEERING**

FINAL PE GEOTECNICAL CONSIDERATIONS REPORT FOR UNIVERSITY LINK

We are sending the following attached items:

Number	Description
7 copies	University Link Preliminary Engineering, Geotechnical Considerations Report
1 copy	Unbound copy for reproduction
1 CD	PDF files of entire report

Bob,
Enclosed is the final version of our Preliminary Engineering, Geotechnical Considerations Report for University Link. Also enclosed is a CD, which includes PDF files of the report.

From: Ted Hopkins, L.E.G
Associate

C.: John Chirco, PSTC (2 copies)

21-1-08109-074

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ACRONYMS AND ABBREVIATIONS

AMS	accelerator mass spectrometer
Am Test	Am Test Laboratories
ANSI	American National Standards Institute
ASTM	American Society for Testing and Materials
BTEX	benzene, toluene, ethylbenzene, and xylenes
cc	cubic centimeters
CD	consolidated-drained
CE	Conceptual Engineering
Central Link	Central Puget Sound Link Rail Transit
CHK	Cherokee General Corporation
CH2M Hill	CH2M Hill, Inc.
CU	consolidated-undrained
CTC	CivilTech Corporation
DRO	diesel-range organics
DSTT	Downtown Seattle Transit Tunnel
Ecology	Washington State Department of Ecology
EPA	U.S. Environmental Protection Agency
EPS	Emerald Petroleum Services
ESA	Environmental Site Assessment
FHA	Fujitani Hilts & Associates, Inc.
Foss	Foss Environmental & Infrastructure
GDR	Geotechnical Data Report
gpm	gallons per minute
GRI	Geo-Recon International, Inc.
GRO	gasoline-range organics
HCID	Hydrocarbon Identification
HIE	Hughes InSitu Engineering, Inc.
HOL	Holocene Drilling Company
HSA	hollow-stem auger
I.D.	inside-diameter
I-5	Interstate 5
LL	liquid limit
LPA	Locally Preferred Alternative
MBI	Myers Biodynamics, Inc.
Metro	King County Metro
MLW	Mean Low Water
mm	millimeters
mm/min	millimeters per minute
NAD	North American Datum
NAVD	North American Vertical Datum

ACRONYMS AND ABBREVIATIONS (cont.)

NCDB	LB-235 North Corridor Tunnel Construction Design-Build
NWTPH	Northwest Total Petroleum Hydrocarbons
NWTPH-Dx	Northwest Total Petroleum Hydrocarbons as Diesel
NWTPH-G	Northwest Total Petroleum Hydrocarbons as Gasoline
NWTPH-HCID	Northwest Total Petroleum Hydrocarbons-Hydrocarbon Identification
O.D.	outside-diameter
ORO	oil-range organics
OW	observation well
PDC	Pitcher Drilling Company
PE	Preliminary Engineering
PGI	PacRim Geotechnical, Inc.
PI	plasticity index
PID	photoionization detector
PL	plastic limit
PPE	personal protective equipment
ppm	parts per million
PSTC	Puget Sound Transit Consultants
PVC	polyvinyl chloride
RQD	Rock Quality Designation
RSI	Resonant Sonic International, Ltd.
SAL	Salisbury & Associates, Inc.
SPT	Standard Penetration Test
SPU	Seattle Public Utilities
SR	State Route
TPD	Tacoma Pump & Drill
UFS	Universal Field Services
USCS	Unified Soil Classification System
UU	unconsolidated-undrained
VOC	volatile organic compound
VWP	vibrating-wire pressure transducer piezometer
WAC	Washington Administrative Code
WMI	Waste Management, Inc.
WSDOT	Washington State Department of Transportation
YGS	Yonemitsu Geological Services

SOUND TRANSIT LIGHT RAIL
UNIVERSITY LINK PRELIMINARY ENGINEERING
GEOTECHNICAL CONSIDERATIONS REPORT

1.0 INTRODUCTION

1.1 Site and Project Description

The proposed Central Puget Sound Light Rail Transit (Central Link) system would provide a mode of public transportation throughout the greater Seattle area and in Tacoma. In Seattle, Central Link includes corridors north and south of Downtown Seattle. The north and south corridors would be joined at the existing Downtown Seattle Transit Tunnel (DSTT). The north corridor has been the subject of extensive design studies to date, including a Locally Preferred Alternative (LPA) alignment, which the current north corridor alignment follows along much of its length. This report presents the results of field explorations and preliminary engineering (PE) recommendations of the Sound Transit University Link (University Link) route that runs between Downtown Seattle and University of Washington, as shown on the Vicinity Map, Figure 1.

The proposed University Link route begins from the C510 Pine Street Stub Tunnel (Stub Tunnel) currently being constructed under Pine Street at the north end of the DSTT, near the existing Convention Place Station, and crosses under Interstate 5 (I-5) to First Hill. The alignment continues under Capitol Hill. It then crosses beneath the Montlake Cut to the University of Washington, where it terminates just west of Husky Stadium (Figure 2). The light rail guideway and stations would all be underground.

1.2 Purpose and Scope

The purpose of our work was to provide geotechnical recommendations to the design team for their use in the PE phase of the project. Our work for this report consisted of characterizing subsurface conditions, preparing interpreted subsurface profiles, completing preliminary engineering analyses, and providing engineering conclusions and recommendations for PE design. This report should be reviewed in conjunction with its companion Geotechnical Data Report (GDR), dated March 2006, which contains the subsurface and laboratory data that were used as our basis for our characterization and recommendations. This report supersedes all

previous reports for portions of those alignments that match University Link. A list of the previous reports is presented below:

- Central Line Geotechnical Engineering Considerations Report for Staff-Recommended Locally Preferred Alternative Only, December 1998
- Preliminary Engineering (PE) Geotechnical Engineering Recommendations Report, December 31, 1999
- LB235 Geotechnical Characterization Report (GCR) (December 1999)
- CE Geotechnical Engineering Considerations Report, Convention Place Station to 45th Street Station, August 1, 2002
- Geotechnical Report, Portage Bay West Tunnel Routes, July 23, 2003
- CE Geotechnical Engineering Considerations Report, Modified Montlake Alignment, March 12, 2004

This report and its companion GDR have been commissioned by Sound Transit (ST) and their civil facilities design consultant, Puget Sound Transit Consultants (PSTC), for the specific purpose of assisting in the development of the PE design of the University Link. The information in both reports would be amended by additional geotechnical studies to be provided for the final design phase of the project.

1.3 Report Layout

This report contains four primary sections: (1) Alignment Description; (2) Regional Geology, Groundwater Conditions, and Tectonic Setting; (3) Subsurface Conditions and Characterization; and (4) Preliminary Geotechnical Recommendations. Preliminary geotechnical recommendations provided include earthquake engineering and geologic hazards, running tunnels, mined structures, shafts, cut-and-cover structures and portal excavations, retained cut-and-cover sections, retained fill sections, aerial structures, ground improvement, and instrumentation.

1.4 Acknowledgements

The following Shannon & Wilson staff contributed significantly to the effort of preparing this report. The project is under the overall coordination and review of Paul Godlewski, Project Manager. Ted Hopkins evaluated the subsurface conditions and characterized the geology along the project alignment. Richard Martin, Dan McHale, and Paul Van Horne evaluated the hydrogeological conditions. Bill Perkins developed the earthquake engineering considerations. Hisham Sarieddine, Monique Nykamp, and Laureen McKenna developed the geotechnical

recommendations. Mike Kucker and David Ward provided tunneling considerations and settlement calculations. Jim Wu, Bill Laprade, and Red Robinson reviewed the report.

1.5 Limitations

The conditions discussed and recommendations presented in this report are based on a limited number of subsurface explorations and laboratory testing that have been completed for the purposes of classifying and determining the general engineering properties of soils along the alignment. These discussions are suitable for preliminary engineering only and are not intended for final design or construction. The subsurface conditions presented are expected to be re-evaluated and the recommendations updated as additional geotechnical studies are conducted.

The locations of project features may change as project requirements are further developed. The information and discussions presented in this report were developed for the features presented on the attached figures, which were current at the time this report was prepared. Some project features have changed or have been modified since subsurface explorations were completed. Such changes have resulted in a few borings that terminate above the proposed invert elevation of below-grade features or borings that are not located as close to the alignment as desired. The geology and subsurface conditions in these areas are interpreted based on an extrapolation of conditions from available explorations.

The scope of the work for this report was for geotechnical engineering only; environmental studies were not conducted. A review of existing soil or groundwater contamination and the identification of potential sources of contamination were not part of the scope of services.

2.0 ALIGNMENT DESCRIPTION

The proposed University Link would consist of approximately 3.2 miles of running tunnels extending from the DSTT to just west of Husky Stadium at the University of Washington. The alignment includes two cut-and-cover stations and one crossover, and one ventilation shaft.

Beginning from the Stub Tunnel currently being constructed under Pine Street at the north end of the DSTT near the existing Convention Place Station, the alignment crosses under I-5 as a subway and advances eastward and uphill under Boren Avenue. The alignment continues under Bellevue and Summit Avenues, and turns to the north as it crosses under Boylston Avenue and Broadway E. to a proposed cut-and-cover Capitol Hill Station between Nagle Place and Broadway E. near E. Denny Way. The subway continues north between Broadway E. and 10th Avenue E. and then turns to the northeast near E. Harrison Street until it crosses under State

Route 520 (SR 520) and E. Montlake, near a proposed ventilation shaft to be located at Montlake Place E. and E. Roanoke Street, and proceeds to the north crossing under the Lake Washington Ship Canal and into the University District. On the north side of the ship canal, the alignment continues north along Montlake Boulevard N.E. and terminates at a proposed cut-and-cover crossover and University of Washington Station located adjacent to the Husky Stadium southeast of the intersection of Montlake Boulevard N.E. and N.E. Pacific Place.

The University Link alignment presented on the Site and Exploration Plan, Figure 2 (13 sheets) and discussed in this section is based on plans received from PSTC on March 7, 2006. Subsequent adjustments made to this alignment or grade and their implications to the data presented in this report should be considered during final design.

3.0 REGIONAL GEOLOGY, GROUNDWATER CONDITIONS, AND TECTONIC SETTING

3.1 General

The geology along the University Link consists of complex sequences of glacial and nonglacial deposits that include fine- and course-grained sediments that overlie bedrock at great depths. An understanding of the geologic history and the depositional processes that produced the soil stratigraphy in the project area is useful for understanding the engineering characteristics and predicted behavior of the soils that underlie the alignment. Such an understanding also provides a framework for anticipating subsurface conditions that may not have been disclosed directly by the exploration program, but may reasonably be expected based on past local project experience with similar geologic units.

3.2 Regional Geology

Seattle is located in the central portion of the Puget Lowland, an elongated topographic and structural depression bordered by the Cascade Mountains on the east and the Olympic Mountains on the west. This lowland is characterized by low, rolling relief with some deeply cut ravines and broad valleys. In general, the ground surface elevation is within 500 feet of sea level.

Six or more major glaciations originating in the coastal mountains of British Columbia have advanced southward across the Puget Lowland during the Pleistocene Epoch (2 million to about 10,000 years ago). The alternating glacial and interglacial episodes have filled the Puget Lowland with a complex sequence of glacial and nonglacial (deposited during interglacial times) sediments. During the most recent glaciation of the central Puget Lowland (Vashon Stade of

Fraser Glaciation), the thickness of ice was about 3,000 feet in the project area, resulting in overconsolidation of the underlying soils. The last ice covering the project area receded about 13,500 (radiometric) years ago. Since the last glaciation, complete or partial erosion of some deposits, as well as local deposition of other deposits, further complicates the geology.

Thick sequences of Pleistocene and Holocene sediments overlie bedrock along University Link. Based on deep drill holes and seismic profiling, the depth to bedrock north of Downtown Seattle is believed to be in the range of 2,400 to 2,700 feet. Bedrock exists at the ground surface at several locations in south Seattle, south of an east-west line extending from Bremerton to the south end of Lake Sammamish. This line has been identified as the Seattle Fault, which is now considered to be active. An active fault is one that has ruptured within the Holocene Epoch (the past 10,000 years).

3.3 Regional Groundwater Regime

In the Puget Sound area, the groundwater regime is highly variable. The complex glacial stratigraphy and groundwater recharge/discharge relationships in the Seattle area have a strong influence on the groundwater flow. Groundwater recharge typically occurs in the upland areas of Seattle. Groundwater movement is then, in principle, predominantly downward to the discharge areas, eventually draining to the major surface water bodies such as Lake Union, Portage Bay, Lake Washington, and Puget Sound.

The magnitude and direction of groundwater flow is controlled, in part, by the relative permeability (the ability of a soil to transmit a fluid; for water this is termed “hydraulic conductivity”) of the various glacial deposits. Groundwater in the stratigraphically higher, coarse grained, highly permeable deposits, such as glacial outwash, likely flows under unconfined conditions, i.e., the water table is present in the deposit. Groundwater in these units is commonly perched on top of till and lacustrine deposits, both typically having very low permeabilities. Much of the groundwater flows laterally and may discharge in springs or seeps along hillsides. However, a portion of this groundwater slowly percolates vertically downward through the less permeable till and lacustrine units to underlying, more permeable outwash or fluvial deposits where flow is predominantly horizontal. Groundwater in these stratigraphically lower, more permeable units may be unconfined or confined (i.e., excessive hydrostatic pressure causes the total hydraulic head to be higher than the top of the deposit).

Typical groundwater flow patterns consist of downward hydraulic gradients (the potential for groundwater to flow as measured by the difference in total hydraulic head between two points) in upland areas, and upward hydraulic gradients in water-bearing units close to major discharge

bodies. In these areas, groundwater heads may increase with depth in the stratigraphic section, and groundwater flow is upward. These typical groundwater flow patterns are not present in areas where very large cut-and-fill glacial deposits truncate the stratigraphic section and serve as major regional conduits of groundwater movement toward discharge areas.

The principal water-bearing units along the alignment are fluvial and outwash deposits of sand and gravel. These units are relatively permeable, thick, and laterally continuous over long distances. Other deposits, such as advance outwash, recessional outwash, and till-like deposits, may store and transmit significant amounts of groundwater; however, they are variably saturated and generally of limited areal extent. Though generally acting as weak aquitards, nonglacial lacustrine and glaciomarine deposits may also transmit significant quantities of groundwater locally. The rate of groundwater movement or discharge from these units is mostly dependent on their grain-size distribution, which may vary considerably. The principal aquitards along the alignment are glaciolacustrine deposits. These deposits typically have very low permeabilities and generally serve as a hydrologic barrier between overlying and underlying aquifers. Aquitards and aquifers are commonly laterally discontinuous along University Link. As a result, groundwater head and permeabilities can vary significantly over short distances.

3.4 Regional Tectonics and Seismicity

The tectonics and seismicity of the region are the result of ongoing, oblique, relative northeastward subduction of the Juan de Fuca Plate beneath the North American Plate between northern California and southern British Columbia and dextral strike-slip motion on the transform boundary between the North American and Pacific Plates farther south. The location of these plates and boundaries in the region and their relative movements are illustrated in Figure 3. The relative motion among these plates results not only in east-west compressive strain, but also in dextral shear, clockwise rotation, and north-south compression of accreted crustal blocks that form the leading edge of the North American Plate (Wells et al., 1998). As in most active convergence zones, the Cascadia Subduction Zone (CSZ), shown in Figure 3, contains a continental fore-arc consisting of accreted sedimentary and volcanic rocks in front of a landward mountainous, active volcanic arc. The project site is located within the continental fore-arc.

Within the present understanding of the regional tectonics, three broad seismogenic source zones have been identified, namely:

- The interplate portion of the CSZ, which may produce large mega-thrust events with earthquakes up to magnitude M_W 9.

- The deep intraslab portion of the CSZ, which has been the source of the largest historical earthquakes to affect the area.
- The relatively shallow, continental fore-arc.

3.4.1 Cascadia Subduction Mega-thrusts

The CSZ extends over a length of approximately 1,100 kilometers from northern California in the south to southern British Columbia in the north (see Figure 3). While there is a lack of historically observed interplate or mega-thrust earthquakes on the CSZ, significant paleo-seismological evidence has been found by several researchers to support the occurrence of mega-thrust earthquakes on the CSZ (e.g., Atwater, 1987 and 1992; Grant, 1989; Adams, 1990; Darienzo and Peterson, 1990; Clarke and Carver, 1992; Darienzo and Peterson, 1995; Adams 1996; Nelson et al., 1996; Meyers et al., 1996; Shennan et al., 1996). The paleo-seismological evidence suggests that large mega-thrust events on the interface between the subducting and overriding plates occur on average about every 400 to 600 years. Based on historical tsunami records in Japan (Satake et al., 1996) the most recent interplate event on the CSZ was a magnitude M_W 9 event on January 26, 1700. Because of the large uncertainty of the landward extent of a potential rupture surface, estimates of the distance between the project and a potential rupture surface range from about 50 to 150 km.

3.4.2 Cascadia Subduction Zone Intraslab

Earthquakes also originate within the subducting Juan de Fuca plate beneath the North American Plate. As the Juan de Fuca plate subducts beneath the North American plate, stress and physical changes in the subducting plate produce high-angle, normal faulting earthquakes. These types of earthquakes comprise the largest historic events to affect the project area including the magnitude (M_s) 7.1 Olympia earthquake of April 13, 1949; the magnitude (m_b) 6.5 Seattle-Tacoma earthquake of April 29, 1965; and the magnitude (M_W) 6.8 Nisqually earthquake of February 28, 2001. These events were located, respectively, at epicentral distances of approximately 65 km south-southwest (1949), 25 km south (1965), and 60 km south-southwest (2001) of Downtown Seattle. Ground shaking in the Seattle area was reported as intensity VIII (1949), VII (1965), and VI to VII (2001).

Only one strong ground motion recording instrument was operating in Seattle during the 1949 and 1965 earthquakes. In 1949, a peak horizontal ground acceleration of 0.07g was recorded at the U.S. Army Corps of Engineers building at 4735 E. Marginal Way (approximately 1 km south of the West Seattle Bridge on the east side of the Duwamish River). In 1965, a peak horizontal ground acceleration of 0.08 g was recorded at the Federal Office building at 909 First

Avenue (in the downtown area approximately one block east of Elliott Bay). A number of permanent and temporary recording instruments were located in Seattle during the 2001 Nisqually earthquake. Peak horizontal ground accelerations recorded in the Seattle area in the general vicinity of the project during the 2001 Nisqually earthquake ranged from 0.04 to 0.31g. While the project area is located above the seismogenic portion of the subducted plate, this source and associated earthquakes are relatively deep, between depths of approximately 30 and 65 km (the 1949, 1965, and 2001 earthquakes were at depths of 53, 63, and 52 km, respectively).

3.4.3 Continental Fore-arc

Shallow crustal sources in the continental fore-arc are postulated. Geophysical, geodetic, and geologic evidence support the hypothesis that the fore-arc (western leading edge of the North American Plate – see Figure 3) consists of two primary crustal blocks that are being dragged and pulled to the north parallel to the arc (Wells et al., 1998). These blocks include the coastal areas of Oregon and Washington and extend east to the Cascade Mountains. The southern block, consisting of the Coast Range and Willamette Lowland in Oregon and southern Washington, is moving northward and rotating clock-wise relative to a pole or pivot point located in eastern Washington. This motion translates into north-south compression and dextral shear in the northern block, consisting of the Olympic Mountains, Willapa Hills, and Central Puget Sound (in which the project is located), as it is compressed between the southern block and the relatively stationary Canadian Coastal Mountains to the north. It is estimated that the compression rate across the northern block is about 0.4 to 1.0 centimeters per year, and it is postulated that most of the compression and shearing takes place within the more fractured, Central Puget Sound region (Wells and Johnson, 2001, and Wells et al., 1998). This hypothesis is supported by the observation that the rate of historical shallow crustal seismicity is much greater in the Central Puget Sound region than elsewhere in the northern block, and the substantial evidence for Late Quaternary movement on structures within the Central Puget Sound region.

The underlying bedrock structure of the Central Puget Sound region is largely concealed by thick Quaternary deposits and repeated glaciation. Consequently, until the 1990s, crustal seismicity in the region generally had not been correlated with known or inferred structures within the fore-arc, and with the exception of two small minor scarps at the southeast corner of the Olympic Mountains, surface expression of Holocene fault ground surface rupture within western Washington had not been observed. Until the late 1980s, it had generally been accepted that shallow crustal events within the Lowland would have a maximum magnitude of about 6. However, geologic evidence developed since the early 1990s (e.g., Bucknam et al., 1992; Atwater and Moore, 1992; Karlin and Abella, 1992; Schuster et al., 1992; Jacoby et al., 1992;

Johnson et al., 1996; Pratt et al., 1997; Johnson et al., 1999; and Brocher et al., 2001) and structural and tectonic models (e.g., ten Brink et al., 2002; Wells et al., 1998) suggest that crustal block boundaries within the Central Puget Sound region are potentially seismogenic and capable of producing shallow crustal events of magnitudes up to 7.5.

The closest crustal block boundary to the project is the Seattle Fault Zone, which has been the subject of many of the recent seismologic studies. This zone is characterized as 60 to 65 kilometers long (east-west) south-dipping reverse or master fault at depth that produces a series of strands as it approaches the ground surface. Evidence of recent movement on the Seattle Fault includes raised bedrock terraces south of the inferred Seattle Fault, tsunami deposits north of the fault, and landslide deposits in Lake Washington, which have correlative dates of about 1,100 years before present (Bucknam et al., 1992; Atwater and Moore, 1992; Karlin and Abella, 1992; Schuster et al., 1992; and Jacoby et al., 1992). It has been postulated that these events were the result of reverse movement of the Seattle Fault, with the south side moving up approximately 7 meters relative to the north.

Analyses of seismic reflection data (Pratt et al., 1997, and Johnson et al., 1999) provide additional evidence of recent movement on the Seattle Fault. Johnson et al. (1999) analyzed high-resolution and conventional industry marine seismic reflection data and subsequently characterized the Seattle Fault as a 4 to 6 kilometer-wide (north-south) zone consisting of a series of east-west-trending fault strands as shown in Figure 4. Folds in the Quaternary section of the seismic reflection profile indicate that movement has occurred on at least some of the strands through the Holocene. Johnson et al. (1999) also identify a north trending strike-slip zone (Puget Sound Fault) in the center of Puget Sound that offsets the east-west trending strands of the Seattle Fault (Figure 4). While there is no paleoseismological evidence of rupture on this structure, Johnson et al. (1999) infer based on the observed offset of the Seattle Fault that the Puget Sound Fault is also likely to be active.

Brocher et al. (2004) postulate that the tip of the Seattle Fault (wedge tip) is located at a depth of about 4 kilometers beneath the Seattle Basin. The approximate location of the buried wedge tip is shown in Figure 4. The wedge tip is located north of the surface deformation zone and crosses the south end of the University Link alignment. However, because the fault tip is buried in this model, the zone of deformation at the ground surface is located farther south in the area of deformed Holocene sediments identified by Johnson et al. (1999) and Blakely et al. (2002).

Fault trenching studies by the U.S. Geological Survey (USGS) on the Toe Jam Hill (on Bainbridge Island) and Waterman Point (Kitsap Peninsula near Port Orchard) strands of the Seattle Fault Zone also indicate that movement in the zone has ruptured the ground surface during the Holocene. The trenching studies completed thus far suggest that at least four events ruptured the ground surface on this strand of the fault over the last 16,000 years (Nelson et al., 2003a and 2003b).

4.0 SUBSURFACE CONDITIONS AND CHARACTERIZATION

4.1 Exploration and Testing Program

We interpreted subsurface conditions along the alignment from 35 borings completed for this PE phase of University Link as well as subsurface explorations previously completed between Downtown Seattle and University of Washington for earlier phases of the Central Link project. We also utilized the logs of subsurface explorations completed previously by Shannon & Wilson and others for other projects in the vicinity of the alignment. The logs of these non-project borings were collected as part of a literature search that we conducted during earlier phases of the project. The collected data are summarized in the Previous Geotechnical Exploration Data Summary Report, Revision 2 (Shannon & Wilson, 2004a). The logs of the collected subsurface explorations from over 850 projects in the vicinity of University Link and previously studied alternatives have previously been provided to Sound Transit and are available for review.

The subsurface explorations, field tests, and laboratory tests completed for this study and for previous Central Link studies specifically referenced in the text of this report, or shown on accompanying figures, are included in the GDR. The GDR presents the data collected and describes the collection methods used. The GDR also contains the logs of subsurface explorations from projects other than Central Link that were utilized in our interpretation of subsurface conditions along University Link.

4.2 Geologic Units

To characterize subsurface conditions along the alignment from soils encountered in widely spaced borings, it is beneficial to classify these soils according to genesis (depositional process) so that stratigraphic correlations that more likely represent actual conditions between borings can be made. At the onset of the Central Link project, Shannon & Wilson developed a stratigraphic outline of geologic deposits likely to be encountered along the Central Link alignment. Each geologic unit comprises soils that are interpreted to have a common origin or process of

deposition and generally have similar engineering characteristics. However, more than one soil type and soils with different engineering characteristics may be found within each geologic unit.

We have modified the stratigraphic outline during the course of the subsurface exploration program as a result of soils encountered in project borings or as identified from exploration logs completed by others in the project vicinity. The resulting stratigraphic outline, from youngest to oldest, is presented below. The soil types and depositional environment for each of these geologic units are summarized in Figure 5. Note that not all of these geologic units were encountered or are expected to be present along University Link.

- Holocene Units (normally consolidated, nonglacial)
  - Fill (Hf)
  - Hydraulic Fill (Hh)
  - Colluvium (Hc)
  - Landslide Debris (Hls)
  - Alluvium (Ha)
  - Peat Deposits (Hp)
  - Estuarine Deposits (He)
  - Lacustrine Deposits (Lake) (Hl)
  - Beach Deposits (Hb)
  - Reworked Glacial Deposits (Hrw)

- Vashon Units (glacial)
  - Normally Consolidated Sediments
    - Recessional Outwash [Qvro]
    - Recessional Lacustrine Deposits [Qvrl]
    - Ice-Contact Deposits [Qvri]
    - Ablation Till [Qvat]
  - Glacially Overconsolidated Sediments
    - Lodgment Till [Qvt]
    - Glacial Till-Like Deposits [Qvd]
    - Advance Outwash [Qva]
    - Glaciolacustrine Deposits [Qvgl]
    - Glaciomarine Deposits [Qvgm]

- Pre-Vashon Units
  - Glacially Overconsolidated, Nonglacial Sediments (deposited during interglacial periods)
    - Fluvial Deposits [Qpnf]
    - Lacustrine Deposits [Qpnl]
    - Peat [Qpnp]

- Paleosol [Qpns]
- Landslide Deposits [Qpls]

— Glacially Overconsolidated, Glacial Sediments

- Outwash [Qpgo]
- Glaciolacustrine Deposits [Qpgl]
- Till [Qpgt]
- Till-Like Deposits [Qpgd]
- Glaciomarine Drift [Qpgm]

► Tertiary Bedrock Units

— Siltstone [Tsi]

— Sandstone [Tss]

— Claystone [Tcs]

— Volcaniclastic Rocks [Tvc]

The following discussion describes the geologic units and soil types along University Link, from youngest to oldest deposit.

4.2.1 Holocene (Normally Consolidated, Nonglacial) Units

Fill (Hf). Fill is soil that is placed by humans and can have widely variable properties, depending on the material used as fill and whether the fill was placed in an engineered or nonengineered fashion. In general, the fill encountered along the alignment consists of loose to dense granular material, such as silty sand. Cobbles and boulders are likely to be common in nonengineered fill. Fill soils were identified from the presence of irregular clasts of one soil type within soil of another type, from the presence of debris such as fragments of glass, wood, or coal, or from historical maps or plans showing previous ground surface contours different from present conditions. These soils also show irregular iron-oxide staining. Because drilling typically took place along streets or sidewalks, some of the fill encountered represents backfill material for utility trenches or fill placed during the original grading of the street. We identified fill locally along the upland portions of the alignment and more extensively in topographically low areas.

Colluvium (Hc). This unit consists of soils that have been transported downslope by the forces of gravity through processes such as creep and slope wash. These soils mantle most slopes and consist of widely variable material, depending on the parent material located upslope. This unit may contain scattered cobbles and boulders, depending on the parent material. These soils are usually loose or soft but may grade into the weathered portion of the underlying, in-

place material. These soils are not likely to be encountered along University Link except in a relatively thin layer at or near the ground surface on or at the base of slopes.

Landslide Debris (Hls). These deposits form from the downslope movement of soil by landslide action and are generally found on and at the toe of slopes. They are commonly highly disturbed, heterogeneous mixtures of several soil types, including organic debris, and are generally loose or soft with scattered dense or hard pockets. These soils are often similar to and gradational with deposits of Hc because both deposits form by the downslope movement of other soils and usually occur on or at the base of slopes. As such, Hls soils can have a wide variety of grain size, and may exhibit widely varying characteristics from place to place. Hls may also include soils that appear intact, particularly with cohesive material, which may represent displaced blocks of an otherwise undisturbed mass of soil. Hls may contain scattered cobbles and boulders, depending on the parent material. Although not encountered in the current phase of the University Link borings, Hls deposits exist on the steep slope at the north end of Capitol Hill and in the filled swale west of I-5.

Alluvium (Ha). This river or creek deposit is generally associated with historical streams and includes finer-grained overbank deposits. The relative density of this unit ranges from very loose to very dense and is comprised of sand, silty sand, and gravelly sand. Ha deposits are present in the filled swale west of I-5.

Peat Deposits (Hp). This unit consists of very soft to medium stiff peat, peaty silt, and organic silt deposited in local depressions. These deposits and are present in the filled swale located just west of I-5.

Lacustrine Deposits (Hl). Lacustrine (lake) deposits formed from the deposition of fine-grained soil local depressions. The unit was found to be very soft to stiff or very loose to medium dense and composed of silt, clayey silt, silty clay, and clay with scattered sandy seams and laminations. It commonly contains scattered organic debris and was commonly gradational at the top with Hp and at the bottom with Qvri and Qvrl. Cobbles and boulders are rare in this unit, but are likely to exist at or near the contact with the underlying unit.

4.2.2 Vashon Units

4.2.2.1 Normally Consolidated Sediments

Recessional Outwash [Qvro]. This glaciofluvial sediment was deposited as glacial ice retreated from the Puget Lowland. Where Qvro overlies glacially overridden granular deposits, such as Vashon advance outwash [Qva] and Pre-Vashon outwash [Qpgo],

differentiating the contact between the two deposits was commonly difficult. Qvro sediments typically consisted of loose to very dense, clean to silty sand, gravelly sand, or sandy gravel. Cobbles and boulders are common in this unit. The unit was identified along most of University Link at the ground surface between First Hill and the University of Washington Station.

Recessional Lacustrine Deposits [Qvrl]. These glaciolacustrine sediments were deposited in depressions in quiet water as the glacial ice retreated from the Puget Lowland. This unit comprises loose to dense, silty, fine sand and soft to hard silty clay to clayey silt. The clayey sediments are generally of low plasticity. Cobbles and boulders are rare in this unit, but are most likely to exist at the contact with underlying deposits.

Ice-Contact Deposits [Qvri]. This unit is a heterogeneous soil mixture deposited against or adjacent to ice during the wasting of glacial ice. These sediments range from loose to dense, gravelly, silty sand to soft to very stiff, silty clay or clayey silt with some sand. This unit contains scattered cobbles and boulders.

Ablation Till [Qvat]. These heterogeneous soils were deposited during the wasting of glacial ice and were generally not reworked after initial deposition. These soils are generally poorly sorted and consist of a wide range of grain sizes (a diamict), similar to Qvt, but are not glacially consolidated. These soils are typically dense, gravelly, silty sand to silty, gravelly sand with some clay and commonly contain cobbles and boulders. This unit was identified along University Link in the topographic low between First Hill and Capitol Hill (in the vicinity of E. Pine Street, Station NB 1065+00) and in the Pine Street Stub Tunnel. Qvat may grade laterally or vertically into lodgment till [Qvt].

4.2.2.2 Glacially Overconsolidated Sediments

Lodgment Till [Qvt]. Lodgment till was deposited along the base of the advancing glacial ice sheet and subsequently was overridden by the ice. This unit generally consists of gravelly, silty sand to silty, gravelly sand with nonplastic to low plasticity fines (a nonclayey diamict). Qvt is glacially consolidated and is generally very dense. Qvt can also contain cleaner seams that may transmit water. Cobbles and boulders are common in this unit. This unit was identified at or near the ground surface in the upland areas along most of University Link.

Till-Like Deposits [Qvd] are also diamicts and, as defined for the project, are glacially derived and glacially consolidated sediments that have properties intermediate between Qvt and Qva. Qvd soils are generally very dense and have similarities to Qvt. Qvd is more

variable, commonly has fewer fines, and is more likely to contain water-bearing seams than Qvt. Cobbles and boulders are common in this unit. The unit is gradational with Qvt and exists near the ground surface of most upland areas along the project alignment.

Advance Outwash [Qva]. These glaciofluvial sediments, deposited as the glacial ice advanced through the Puget Lowland, typically consist of very dense, clean to silty, fine or fine to medium sand but also includes silty, gravelly sand to sandy gravel with various amounts of silt. This unit contains scattered cobbles and boulders. The unit was overridden by the advancing glacial ice and is dense to very dense. This unit underlies Qvt along several portions of University Link near the ground surface.

Glaciolacustrine Deposits [Qvgl]. These soils are the result of deposition of suspended sediments in quiet water in proglacial lakes in the Puget Lowland. This unit consists of very stiff to hard, silty clay and to a lesser extent clayey silt. Qvgl contains scattered beds of silt and fine sand, but typically consists almost exclusively of fines (all passing the No. 200 sieve). Cobbles and boulders are uncommon in this unit. Qvgl includes both low- and high-plasticity clay but, as a whole, are slightly less plastic than the pre-Vashon glaciolacustrine [Qpgl] soils. Qvgl sediments are commonly laminated to bedded with some fracturing present. These soils are not commonly sheared or slickensided like older Qpgl deposits encountered along the alignment. Qvgl commonly grades into sediments from the previous interglacial period [Qpnl], and so may contain seams and layers of cohesionless silt with scattered to abundant fine, organic fragments. The unit was identified over short distances at scattered locations along University Link.

Glaciomarine Drift [Qvgm]. Soils of this unit were deposited in lakes or marine water by icebergs, floating ice, and gravity currents. These soils generally consist of poorly graded granular material with a clayey matrix (a clayey diamict). Qvgm soils vary considerably from very dense, gravelly, silty sand with a trace of clay to silty, clayey sand to hard, silty clay with varying percentages of sand and gravel. Cobbles and boulders are common in this unit. Qvgm may grade into and contain layers of Qvgl.

4.2.3 Pre-Vashon Units

4.2.3.1 Overconsolidated, Nonglacial Sediments, Deposited During Interglacial Periods

Fluvial Deposits [Qpnf]. These are glacially overridden deposits of rivers and creeks. These sediments typically consist of clean to silty, fine or fine to medium sand with

lesser amounts of slightly silty, gravelly sand and sandy gravel with a trace of silt. These soils are glacially consolidated and are generally very dense. Soils of this unit are very similar to other outwash and fluvial deposits [Qva and Qpgo] but are differentiated by the presence of organic material or by stratigraphic position. The organic material encountered within Qpnf was typically small pieces of wood or clasts of fine organic material. This unit contains scattered cobbles and boulders. This unit is present along most of University Link.

Lacustrine Deposits [Qpnl]. These fine-grained sediments were deposited in quiet water in large and small depressions. The soils of this unit varied between very dense, silty, fine sand to hard, slightly clayey silt but are most typified as cohesionless silt with small amounts of fine sand. This unit also includes layers of clayey silt to silty clay. Sediments that were classified as Qpnl commonly exist as layers above, below, or within glaciolacustrine [Qpgl] soils. Qpnl soils, however, are generally much less clayey than Qpgl and are commonly wet or contain wet seams, and have scattered to abundant fine organic fragments. Cobbles and boulders are rare in this unit, but are most likely to exist at the bottom of the unit. This unit was found along University Link at scattered locations along much of Capitol Hill.

Peat Deposits [Qpnp]. This unit consists of peat, peaty silt, and organic silt. These soils were deposited in local depressions and are relatively thin and of limited lateral extent. These soils have been glacially overridden and, unlike Holocene peat deposits, are hard. The non-organic portion of Qpnp soils is silt and clayey silt of low plasticity. Cobbles and boulders are rare in this unit, but are most likely to exist at the bottom of the unit. Qpnp was encountered in scattered locations along Capitol Hill.

Paleosols [Qpns] are soils formed from the subareal weathering of other deposits during previous interglacial times and have variable grain size distributions, depending on the nature of the underlying unit from which they were formed. These soils have been overridden by subsequent glaciations and are generally very dense or hard. Qpns soils usually have a greenish cast from excavation and reburial and are clayey from weathering. These soils commonly contain organic material, such as wood fragments and roots. This unit contains scattered cobbles and boulders. Qpns deposits are associated with Qpnp deposits in several places along the alignment.

4.2.3.2 Glacial Units

Outwash [Qpgo]. This unit is glaciofluvial sediment deposited as the glacial ice advanced or retreated through the Puget Lowland. It typically consists of very dense, clean to silty, fine or fine to medium sand with a trace of coarse sand and fine gravel. Less commonly,

Qpgo sediments are coarser with less silt and more gravel. This unit contains scattered cobbles and boulders. Qpgo soils are very similar to Qva and Qpnf and were differentiated from them by their lack of organics or from stratigraphy. These soils were found along much of University Link. A significantly thick, natural, cut-and-fill deposit of this unit is found beneath Capitol Hill, north of Volunteer Park.

Glaciolacustrine Deposits [Qpgl]. These deposits formed from the deposition of suspended sediments in proglacial lakes in the Puget Lowland. These soils consist of very stiff to hard, silty clay, and to a lesser extent, clayey silt, with scattered beds of silt and fine sand. Qpgl includes both low- and high-plasticity clay but is generally of higher plasticity than Qvgl soil. Qpgl soils are often laminated to bedded but may also lack any bedding features (massive). The Qpgl soils commonly exhibit sheared and slickensided zones, particularly along Capitol Hill, south of Volunteer Park. The sheared and slickensided clays are generally more highly plastic than the adjacent glaciolacustrine soils that are not sheared or slickensided. Some Qpgl soils also contain high-angle seams of sand that do not appear to be depositional and may have been emplaced through shearing or as sand dikes during major earthquakes. The unit was found at depth along much of University Link.

Because Qpgl soils are much older than Qvgl soils, they have been subjected to greater amounts of glacial loading and unloading and may be expected to be more deformed than Qvgl, which has been subjected to only one glacial episode. These deformational features may also be the result of tectonic deformation. Broad zones of weak and highly sheared clay soils exist along the alignment, particularly south of Volunteer Park. Movement along fractures and shear zones may occur during excavation and support.

Qpgl commonly contains thin, clastic-rich layers and is gradational with Qpgm. As such, this unit may contain scattered cobbles and boulders. A few gravel-sized concretions were also found in Qpgl soils. Qpgl soils also contain seams or layers of cohesionless silt with scattered to abundant fine, organic fragments.

Till [Qpgt]. This unit was deposited as lodgment till at the base of an advancing glacial ice sheet and was overridden by the ice. Qpgt soils are similar to Qvt soils and generally consist of very dense, gravelly, silty sand to silty, gravelly sand with nonplastic to low plasticity fines. Cobbles and boulders are common in this unit. The unit was identified at scattered locations along the alignment.

Till-like Deposits [Qpgd]. This sub-glacially reworked deposit is intermediate between till and outwash. The unit is composed of very dense, silty, gravelly sand; silty sand;

and sandy gravel. Cobbles and boulders are common in this unit. The unit was identified at scattered locations along the alignment.

Glaciomarine Drift [Qpgm]. Soils of this unit were deposited in lakes or marine water by icebergs, floating ice, and gravity currents. These soils generally consist of poorly graded granular material with a clayey matrix (a clayey diamict). Qpgm soils vary considerably from very dense, gravelly, silty sand with a trace of clay, to silty, clayey sand, to hard, silty clay with varying percentages of sand and gravel. Cobbles and boulders are common in this unit. Qpgm commonly grades into and contains layers of Qpgl. This unit was found along most of the alignment.

4.3 Generalized Subsurface Conditions

Our interpretation of the distribution of geologic units known or expected to be present along University Link is illustrated on the geologic profile shown in Figure 7 (13 sheets).

In developing the geologic profiles, we have interpreted soil stratigraphy from information obtained in the widely spaced borings and their estimated soil behavior based on laboratory and field observations and testing, as well as previous construction experience in the Seattle area. As a result, the actual contacts between strata and the characteristics and behavior of the soil may vary significantly from that presented on the subsurface profiles.

On the geologic profiles, the soil strata have been delineated based on geologic units. Different soil types as well as different engineering characteristics may be found within each geologic unit. It is expected that refinements of the distribution and composition of the geologic units, as well as a better understanding of their engineering behavior, would occur as the geotechnical evaluation progresses and more information becomes available.

The University Link alignment is underlain by a thick sequence of glacially overridden (over consolidated) soils comprising both glacial and nonglacial deposits. Normally consolidated or slightly overconsolidated sediments, consisting of Holocene and Vashon recessional-type deposits, occur as a relatively thin veneer over the upland areas of First Hill and Capitol Hill. This veneer of less dense or less stiff sediments, in general, is on the order of 10 to 30 feet thick. In topographically low areas of the alignment, however, these Holocene or Vashon recessional deposits are commonly thicker.

Normally consolidated sediments are as thick as about 60 feet in a relatively narrow trough that lies parallel to the west face of First Hill, between I-5 and the Convention Place Station (Station

NB 1045+00). This swale may have formed from subareal or subglacial streams trapped against the flank of First Hill and Capitol Hill during retreat of the Vashon glaciation. Where the alignment crosses the swale, the swale appears to be filled with normally consolidated sediments comprising landslide deposits [Hls], peat [Hp], alluvium [Ha], and fill [Hf].

Normally consolidated sediments are present to an approximate depth of 45 feet in a shallow topographic swale that crosses the alignment at E. Pike Street (Station 1065+00). The topographic swale and the normally consolidated deposits of ablation till [Qvat] and recessional outwash [Qvro] may be the result of subglacial reworking or ice-marginal processes. As a result, these normally consolidated deposits may have an irregular contact with, and grade into, adjacent glacially overconsolidated deposits of till [Qvt] and [Qva].

Beneath the veneer of normally consolidated sediments are glacial and nonglacial sediments that have been overridden by one or more glacial ice sheets. These deposits consist of very dense, granular soils and hard, cohesive soils. The glacially consolidated sediments encountered in the explorations and shown on the geologic profiles and cross sections may represent up to three of the six or more glacial and intervening interglacial episodes thought to have occurred in the Puget Lowland. Except for the last glacial episode (Vashon) and where we have obtained age dates on organic specimens in soils (see LB235 GDR, October 1999), specific deposits could not be correlated with specific glacial or interglacial episodes.

In general, Capitol Hill (and its southward extension First Hill) has a core of pre-Vashon glacial and nonglacial sediments that form an elongated ridge that mirrors the present ground surface; Vashon-age sediments lie on top of, and against the flanks and ends of the older ridge. The glacially overridden deposits of the Vashon glaciation are typically 30 to 80 feet thick. The core of pre-Vashon sediments consists largely of an assemblage of cohesive, glaciolacustrine clays and silts [Qpgl] with intervening layers of cohesionless silt and fine sand [Qpnl]. Beneath First Hill and the south half of Capitol Hill (approximately south of E. Prospect Street), layers of glaciomarine drift [Qpgm] and outwash and fluvial sands [Qpgo and Qpnf] are present both above and below the Qpgl/Qpnl assemblage. It is largely within the Qpgl and Qpnl sediments that the tunnel would be constructed.

As previously discussed in the LPA GCR, several deep, cut-and-fill features, referred to as the St. Marks swale, are present between E. Prospect Street, in the vicinity of Volunteer Park, and the southern edge of the University of Washington. These large swales are cut into fine-grained soils [Qpgl] and filled with fluvial and outwash sands [Qpnf and Qpgo] and to a lesser extent, silt and fine sand [Qpnl]. A similar channel filled with Vashon outwash sand [Qva] lies north of the

Montlake Cut. These cut-and-fill features likely represent former deep meltwater channels that cut across the ridge that forms Capitol Hill. The localization of these channels at the north end of Capitol Hill appears to have persisted through several glacial events. High groundwater heads are present in the sand and silt deposits that fill these swales.

North of the swale at Boyer Avenue E., the ground surface is underlain by a 30- to 50-foot-thick layer of Vashon deposits. The Vashon deposits consist of till [Qvt], till-like [Qvd], and outwash sand [Qva] deposits. These granular deposits do not occur in separate distinct layers, but commonly as one layer that varies laterally and vertically with the different deposits grading into each other. The layers gradational nature may represent sediment reworking by subglacial streams.

The layer of Vashon deposits are underlain by a thin layer of nonglacial deposits [Qpnl and Qpnf], which in turn, is underlain by a 50- to-70 feet-thick layer comprising pre-Vashon till [Qpgt], till-like [Qpgd], and glaciomarine [Qpgm] deposits, with relatively thin layers of outwash sand and gravel [Qpgo] and clay [Qpgl]. Most of the soils within this layer have similar soil characteristics and are gradational with each other. This layer of till, till-like, and glaciomarine deposits is underlain by glaciolacustrine clay and silt [Qpgl].

4.4 Generalized Hydrogeologic Conditions

4.4.1 General

The hydrogeologic conditions along University Link are complex as a result of numerous aquifer/aquitard sequences, which control the presence and movement of groundwater. The following subsections describe the general groundwater conditions in the vicinity of the tunnel and stations based on groundwater levels measured in observation wells and the vibrating wire piezometers (VWPs). Refer to the geologic profile (Figure 7) when reviewing the discussion of hydrostratigraphy and groundwater conditions that follows.

The piezometric surfaces shown on the geologic profiles and discussed in the report should be considered preliminary because of the wide spacing of the borings and the limited number of measuring devices installed in each boring. The piezometric surface represents the level to which groundwater would rise in a well installed within a soil unit and is equivalent to atmospheric pressure. Piezometric surface lines presented in Figure 7 are based on current and previous groundwater level measurements and are approximate. Groundwater levels likely fluctuate seasonally and could differ from those shown on the profiles at any given time. Piezometric surface lines are inferred between observation wells and/or VWPs. Absence of a

piezometric surface line for portions of the geologic profiles does not necessarily indicate the absence of groundwater in that area.

We have estimated average values of hydraulic conductivity (K) for the predominant geologic units within the tunnel horizon. These values are summarized in Table 13 and were estimated from site-specific gradation curves, slug tests, and aquifer tests.

4.4.2 C510 Pine Street Stub Tunnel to Capitol Hill Station

The University Link segment between the Stub Tunnel and Capitol Hill station is characterized by: (1) interlayered, mostly fine-grained, saturated soils under confined pressure and (2) near-surface, moist to wet, granular soils with unconfined groundwater. Saturated Qpnl and Qpgl are likely to be encountered at tunnel depth along most of the alignment within this segment. Saturated Qpgm and Qvgm may also be encountered locally at the tunnel depth. The tunnel invert is just above unconfined Qpgo and Qpnf along the western portion of this segment. Perched groundwater above the tunnel is present within Qpnl, Qva, Qvgm and Qvro soils.

The groundwater hydraulic heads in Qpgm, Qpnl, and Qpgl just east of I-5 are at an elevation of about 140 feet. Hydraulic head may rise to about elevation 240 feet in the middle of the segment, based on a single monitoring point; however, hydraulic heads in these units measured previously to the south of the current alignment (in the First Hill vicinity) were as high as elevation 280 feet. The wells completed in the deep Qpgm, Qpnl, and Qpgl at about tunnel depth indicate that the hydraulic head in these units rise to about elevation 300 feet in the vicinity of the Capitol Hill Station, or about 45 to 50 feet above the tunnel crown in the vicinity of the station.

Perched groundwater with heads rising to the east and north appear to be present in the granular Qvro, Qva, Qvgm, and Qpnf soils that overlie the finer-grained Qpgl, Qpnl, and Qpgm soils discussed above. Observation wells located south of the present University Link alignment indicate the presence of perched groundwater at an elevation of about 210 feet within Qpnf and Qva soils located above the tunnel just east of I-5. Perched groundwater appears to be present in Qva, Qvro and Qvgm between about elevations 250 and 290 feet, rising northward, in the middle of this segment. A Qva layer located in the vicinity of Howell Street exhibits confined groundwater conditions with hydraulic heads at an elevation of about 315 feet. The proposed tunnel intercepts Qvgm in the vicinity of Howell Street and may encounter limited groundwater within this unit.

4.4.3 Capitol Hill Station to E. Prospect Street

The segment between the Capitol Hill Station and E. Prospect Street is characterized by interlayered soils that are predominantly fine-grained, saturated, and under confined pressure. Less is known about the groundwater conditions north of Capitol Hill Station because borings are more widely spaced. Saturated soils at tunnel depth between Capitol Hill and E. Prospect are predominantly Qpnl and Qpgl. There are no monitoring points within Qpnf, Qva, or Ha soils located above the tunnel elevation; however, based on observations made elsewhere along the alignment, these soil units may have perched groundwater.

The high groundwater heads (approximate elevation 310 to 320 feet) observed in the Qpnf and Qpnl in the vicinity of Capitol Hill Station likely persist through this segment. Although no groundwater measuring devices are located in the overlying Qva, the hydraulic heads in the Qpnf suggest that Qva may contain groundwater. Because of the presumed downward hydraulic gradient in this upland area south of Capitol Hill Station, hydraulic heads in the Qva may be higher than those measured in Qpnf.

Groundwater heads within the Qpnl and Qpgl appear to decrease north of about E. Mercer Street. Somewhere between 12th Avenue E. and E. Prospect, finer grained Qpgl and Qpnl soils transition to Qpgo and Qva soils. Based on observations made elsewhere along the alignment, these Qpgo and Qva soils may be water-bearing (areas north of E. Prospect Street indicate a groundwater elevation of about 220 feet in Qpgo soils).

4.4.4 E. Prospect Street to Montlake Cut

In general, the segment between E. Prospect Street and the Montlake Cut can be characterized by: (1) a thick sequence of outwash and fluvial soils at higher elevations that are water-bearing and likely unconfined; (2) interlayered, fine-grained soils that are saturated and under confined pressure; and (3) occasional layers of outwash (Qpgo) that may be saturated and may be under confined pressure.

The saturated soils located at the tunnel depth along this segment primarily include Qpgl and Qpnl. The proposed tunnel also intercepts Qpnl/Qpnf soils in the vicinity of E. Garfield Street and Qpgm between E. Howe and E. Miller Streets. The proposed tunnel intercepts a second Qpnf unit in the vicinity of Boyer Avenue E. All of these soil units may contain saturated zones. Perched groundwater may be present above the tunnel in Qpgo, Qva, Qpnf, and various Holocene-aged soils. The Qpgo appears to have perched water in the vicinity of 15th Avenue E. at about elevation 220 feet.

There are a limited number of monitoring points within the deep Qpgl and Qpnl soils at the tunnel depth. A VWP located at E. Garfield Street (NB-250) indicates that the Qpnl soils at tunnel depth are saturated and under confined pressure, with water levels between about elevation 190 and 200 feet (about 130 feet above the tunnel crown in this area). The Qpnl water level elevation likely decreases to the north of E. Garfield Street, mimicking the decreasing ground surface elevation.

The proposed tunnel intercepts a Qpnf unit in the vicinity of Boyer Avenue E. This unit may be under confined conditions in places. The groundwater head measured at NB-387 is at about elevation 35 feet and is about 35 feet above the tunnel crown. The Qpnf pinches out to the north. North of about E. Lynn Street, the proposed tunnel intercepts Qpgm soils. The Qpgm soils are under confined conditions, with groundwater heads between elevation 55 and 60 feet, corresponding to about 70 to 80 feet of head above the crown of the tunnel.

Groundwater heads in the Qpgl and Qpnl soils generally decline with topography as the alignment approaches the Montlake Cut because of groundwater discharge to the water bodies adjacent to the northern end of Capitol Hill. Because of this discharge relationship, groundwater heads in the soils near and directly beneath the Montlake Cut are expected to be above the water level of Portage Bay.

4.4.5 Montlake Cut to University of Washington Station

The segment between the Montlake Cut and the University of Washington Station can generally be characterized by a thick, unconfined sequence of saturated advance outwash (Qva) and saturated, interlayered, fine-grained soils under confined pressure.

The proposed tunnel intercepts saturated soils including Qpgl, Qva, Qpnf, Qvd, and Qpgo. Groundwater may be perched above units such as Qvt, Qvrl, and Qpgl within Qva, Qvro, and Holocene-aged soils along the alignment.

The groundwater heads in Qpgl increase northward of the Montlake Cut. This trend suggests that groundwater in the unit discharging from the Qpgl that lies north of the Montlake Cut to the Lake Washington Ship Canal. Heads are between elevation 20 and 40 feet in the unit, which is about 60 feet above tunnel invert near the Montlake Cut. A thick sequence of water-bearing Qva exists between the University of Washington Station and Stevens Way. The groundwater head elevations appear to be similar to those observed in the Qpgl in this area.

4.5 Geotechnical Characterization of Geologic Units

4.5.1 Properties of Individual Geologic Units

Data from current and previous field and laboratory testing were evaluated to develop a better understanding of the nature of the geologic units along University Link. Test results for samples with the same interpreted geologic unit were grouped, and non-parametric statistical analyses were performed and are presented in a series of tables and plots. Summaries are presented for (1) field penetration resistance; (2) water content, Atterberg limits, sticky limits, and activity; (3) unit weight and specific gravity; (4) grain size distributions; (5) consolidation characteristics; and (6) strength properties of soils.

Quantitative summaries of geotechnical parameters and index tests for each geologic unit are shown in Tables 1 through 4. Engineering properties for each geologic unit are summarized in Tables 5 through 7. Each table presents a statistical evaluation (generally including count, minimum, maximum, average, and standard deviation) of the test results for each unit. To give an indication of the skew of the data, the median (50^{th} percentile) is presented for those data sets that do not appear to be normally distributed.

Penetration resistance data, summarized in Table 1, contained a significant number of outliers (defined as data that is more than 1.5 times the interquartile range, either above the 75th percentile or below the 25th percentile). To provide a better indication of penetration resistance values that are characteristic of a geologic unit, a modified range excluding those outliers is presented in the table. Because several types of hammers and samplers were used during previous Central Link subsurface explorations, analyses were performed during an earlier phase of the Central Link project to correlate blow counts generated by different hammer/sampler systems. The results of this evaluation are included in Appendix B of the PE Geotechnical Engineering Recommendations Report, dated December 1999.

Only a limited number of laboratory tests were conducted for many of the geologic units. Therefore, the results may not be representative of the geologic unit and the use of these values should be evaluated. Data with greater statistical significance would be available as more data is collected during subsequent phases of the project. Not all units discussed in Section 4.4, Geologic Units Description, are included in these tables. Tables summarizing subsurface conditions along the alignment, tunneling and excavation characteristics, and preliminary recommended engineering properties are presented as Tables 8 through 17. These tables are discussed in subsequent sections of this report.

In addition to the statistical evaluations performed on the data, grain size distribution analyses performed on similar geologic units were combined and plotted to evaluate the variation of grain size distribution within each unit. These results are presented in Figures 8 to 24. Similar composite plots were developed for Atterberg limits testing and are presented in Figures 25 through 35.

4.5.2 Grouping of Similar Geologic Units

The geologic units along the project alignment, although differing in genesis, can be grouped into five major soil groups with similar engineering properties and behaviors to simplify engineering considerations. These five groups are:

- **Group 1 – Fill and other Non-overridden Soils** – Fill [Hf], all Holocene Deposits [Hc, Hls, Ha, Hp, He, Hl, Hb, and Hrw], and all Vashon recessional soils [Qvro, Qvrl, Qvri, and Qvat] with widely varying consistencies or densities but have not been glacially overconsolidated.
- **Group 2 – Cohesive Silt and Clay** – Glaciolacustrine [Qvgl and Qpgl] deposits consisting of very stiff to hard clayey Silt and silty Clay.
- **Group 3 – Cohesionless Silt and Fine Sand** – Nonglacial lacustrine [Qpnl] deposits consisting of very dense Silt, fine sandy Silt, and silty fine Sand.
- **Group 4 – Till and Till-like Deposits** – Glacial deposits [Qvt, Qvd, Qpgt, Qpgd, and Qpgm] consisting of a dense to very dense, heterogeneous mixture of Silt, Sand, and Gravel, and varying amounts of clay. Glacial Till [Qvt and Qpgt] has nonplastic to low plasticity fines, while Till-like Diamict [Qvd and Qpgd] has fewer fines and is more cohesionless. Glaciomarine drift [Qvgm and Qpgm] generally has a clayey matrix but may be quite variable. Paleosols [Qpns], though of limited extent, can also be included in this group.
- **Group 5 – Cohesionless Sand and Gravel** – Glacial outwash [Qva and Qpgo] and nonglacial fluvial [Qpnf] deposits consisting of very dense Sand, gravelly Sand, and sandy Gravel.

The general soil characteristics of these five soil groups are discussed below. Anticipated ground behavior and engineering characteristics of these soil groups are discussed in Section 5.0.

The Fill and other Non-overridden Soils (Group 1) includes all soils that have not been glacially overconsolidated, i.e. stratigraphically younger than Vashon till. This group comprises all Vashon recessional soils, all Holocene soils, and fill. These soils have properties that vary widely but are all normally consolidated. These soils occur at or near the ground surface and are not likely to be present along the tunnel horizon.

The Cohesive Silt and Clay soils (Group 2) consists of glaciolacustrine [Qvgl and Qpgl] deposits that are composed primarily of very stiff to hard, interbedded silts and clays with lenses and layers of cohesionless fine sand and silt. These soils may be laminated or without apparent bedding. Gravel, cobbles, and boulders, though typically not common, are likely to exist in these deposits and may be encountered as isolated clasts or in seams or layers. Atterberg limits for these soils are shown in Figures 29 and 34 (Qvgl and Qpgl, respectively). A composite plot of grain size distributions for these soils is shown in Figure 36. While both Qpgl and Qvgl are generally classified as either silty clay or clayey silt, Qvgl is typically less plastic and less fractured or slickensided than Qpgl.

The Cohesionless Silt and Fine Sand deposits (Group 3) comprise Qpnl and Qpnp, which consist primarily of very dense silt and fine sand with small amounts of clay and scattered to abundant organic fragments. Although most typically cohesionless to slightly cohesive, these soils are moderately cohesive in places. The cohesive portions of this unit tend to occur in scattered to abundant lenses or layers. The Atterberg limits for these units are included in Figures 31 and 32, respectively. The Atterberg limits data presented for is soil group is biased as Atterberg limits testing were generally conducted on only samples that exhibited plasticity, which represents only a small portion of Qpnl. Atterberg limits testing was generally not performed on the non-plastic samples of these soils. A summary plot of measured grain size distributions for Group 3 soils is shown in Figure 37.

The Till and Till-like Deposits (Group 4) comprise Qvd, Qvt, Qpgm, Qpgd, Qpns, and Qpgt, which are similar in that they generally are heterogeneous mixtures of gravel, sand, and silt or clay. The Qpns and Qpgm soils generally have cohesive fines, however, while Qvd, Qvt, Qpgd, and Qpgt typically have non-plastic or low plasticity fines. These soils commonly have a consistency similar to very soft rock or lean concrete. Cobbles and boulders are common in these deposits. A composite plot of grain size distributions for these soils is shown in Figure 38.

The Cohesionless Sand and Gravel deposits (Group 5) comprise Qva, Qpnf, and Qpgo, which consist of dense to very dense, silty sand to sandy gravel, with some sandy silt. A composite plot of grain-size distributions for these soils is shown in Figure 39. These soils are generally poorly graded sand and gravelly sand, but include sandy gravel. Based on the soil samples tested, these soils have a bi-modal distribution, with a subgroup consisting of slightly silty to silty, fine to medium sand (Group 5a) and a group of clean to slightly silty, gravelly sand (Group 5b).

4.5.3 Boulders

Boulders would likely be encountered during construction of University Link and may be a critical factor in tunneling with a closed-face tunnel boring machine (TBM). Boulders that are too large to pass through the rotating cutterhead and too strong to break up would impact tunneling. A boulder is defined as a rock that is too large to pass through a 12-inch-square opening. Boulders in the Puget Lowland generally consist of igneous or metamorphic rock with an unconfined compressive strength of 15,000 to 60,000 pounds per square inch (psi).

Based on our experience on other tunneling projects in the Seattle area, boulders would routinely be encountered, in our opinion, in the following geologic units: Hf, Qvro, Qvat, Qvt, Qvd, Qpgt, Qpgd, and Qpgm. Cobbles and boulders are also likely to exist in all other geologic units in variable amounts, particularly concentrated in zones. Essentially all of the glacial units along the alignment could contain some cobbles and boulders. Boulders could occur anywhere within a geologic unit; however, a greater percentage of boulders may be presented along contacts between geologic units.

It is difficult to quantify the likelihood of encountering boulders to when tunneling through the various geologic units from relatively widely spaced, small-diameter borings. A statistical analysis was performed for previous project studies to estimate the number and size of boulders likely to be encountered during tunneling through Qpgm and Qpgl. A discussion of the statistical approach and presentation of results are presented in Appendix C of the PE Geotechnical Engineering Recommendations Report, dated December 31, 1999.

For our 1999 boulder study, Shannon & Wilson developed a model for estimating the number and size of boulders from suspected encounters of boulders in project borings based on boulder shape, size distribution, and boulder volume fraction. The aspect ratio of boulders was determined from measurements of the three principal axes of boulders encountered in the DSTT, and from similar measurements by another investigator of boulders encountered in the expansion of the Toronto subway. The modeled size distribution of boulders was based on boulders encountered during soldier pile installation for the Toronto subway. The boulder volume fraction was determined by assigning a probability to inferred boulder encounters in the drilling of the 100- and 200- series North Corridor borings completed at the time of the study. The bolder volume fraction calculated was compared to the boulder volume fraction estimated from a deep excavation in Qpgm and Qpgl soils on the city block on the northwest corner of Pine Street and 6th Avenue.

The validity of the modeling approach was tested using DSTT data by comparing the actual number of boulders encountered during construction of the DSTT to the number of boulders inferred to have been encountered in the DSTT borings. The results of this study were then modified and extrapolated to other geologic units, not part of the boulder study, based on our experience. The size and average number of boulders per 1,000 cubic yard (cy) of in situ material that should be anticipated are defined in Table 18. This table was previously presented in the Tender Geotechnical Baseline Report for the LB235 contract (Shannon & Wilson, 2000).

5.0 GEOTECHNICAL ENGINEERING CONSIDERATIONS

5.1 General

The following sections present geotechnical considerations for preliminary design of University Link. The University Link alignment consists of approximately 3.2 miles of twin, running tunnels, each approximately 21 feet in outside diameter, extending from the Stub Tunnel north to the vicinity of Husky Stadium at University of Washington. The running tunnels are spaced approximately 40 feet apart, measured between centerlines, except adjacent to the Montlake Ventilation Shaft and at the I-5 undercrossing where centerline-to-centerline spacing approaches 80 feet. The running tunnels would be excavated using TBMs, except for a short length of tunnel beneath Pine Street that would connect the existing Pine Street Stub tunnel with the proposed retrieval shafts west of I-5 (see Figure 2, Sheet 1). These portions of the running tunnel would be mined using the sequential excavation method (SEM). SEM would also be used for tunnel cross-passages and adits at the Montlake Ventilation Shaft. Underground stations, crossovers, and portals, would be constructed using cut-and-cover excavations

Two underground stations (Capitol Hill and University of Washington stations) would be constructed along University Link for pedestrian access. These stations would be constructed by conventional cut-and-cover, top-down construction methods and would extend to as much as about 100 feet below the ground surface. A crossover is to be located south of University of Washington Station. Numerous cross-passage tunnels would be constructed between the running tunnels and would be located at intervals of approximately 800 feet.

5.2 Earthquake Engineering and Earthquake-induced Geologic Hazards

Seismic design criteria for the project are provided in Section 8A – Supplemental Criteria for Seismic Design of the Light Rail Transit System Design Criteria Manual, Revision 1, December 2001 (Section 8A). Procedures for design of buried structures (e.g., tunnels, cut-and-cover structures, shafts) are outlined in this section along with required design ground motion

parameters for stiff/dense soils. The parameters provided in Section 8A are generally consistent with the soils along University Link and are suitable for preliminary design.

Earthquake-induced geologic hazards include landsliding, fault rupture, settlement, soft soil ground motion amplification, and liquefaction and associated effects (such as loss of shear strength, bearing capacity failures, loss of lateral support, ground oscillation, and lateral spreading). Figure 4 shows the location of liquefaction-susceptible soils and potential landslide areas in the vicinity of University Link (Grant et al., 1992).

The risks posed by these geologic hazards are relatively low along the project alignment. As shown in Figure 4, University Link does not cross liquefaction-susceptible soils. The relatively dense/hard nature of the glacially-overridden soils in which the structures are located have a very low susceptibility to liquefaction or settlement and preclude the type of ground motion amplification observed at sites underlain by soft soils. Figure 4 also shows splays of the Seattle Fault Zone at or near the ground surface. While geologic evidence suggests that this fault zone is active (Holocene rupture of the ground surface), the northernmost fault splay is thought to be located more than one mile south of the southern end of the project alignment.

Generally, areas identified as susceptible to landslides under static conditions are considered to have an increased likelihood of slope movement during earthquakes. Figure 4 shows that University Link does not cross potential landslide areas where the likelihood of slope movement is considered to be relatively high.

5.3 Running Tunnels

5.3.1 General

The tunnels are expected to be located almost entirely in glacially overconsolidated soils. A geologic profile constructed along the northbound centerline, is shown in Figure 7 (13 sheets). The subsurface conditions shown on the profiles were interpreted from explorations ranging from less than 100 feet apart to as much as ½ mile apart. As a result, the configuration, location, and continuity of geologic contacts should be considered preliminary and approximate. From our experience on numerous tunnel projects in the Seattle area, more closely spaced explorations typically reveal a more complex and varied stratigraphy than conditions interpreted from widely spaced borings. Additional subsurface explorations to reduce spacing between borings are needed for final design.

A summary of the general subsurface conditions for the running tunnels is presented in Table 8. Subsurface conditions are presented by alignment segments based on similarity of

geology. Table 8 summarizes the borings located near the alignment and the geologic units encountered between two tunnel diameters above the crown and one diameter below the invert of the present design tunnel elevation.

5.3.2 Ground Behavior During Running Tunnel Construction

This section discusses tunnel construction considerations related to the anticipated behavior of the soil along the proposed tunnel alignment. These considerations are based on our tunneling experience in similar soils in the Seattle area. Ground behavior in a tunnel excavation is a function of soil and groundwater conditions, depth of soil cover, tunnel size and configuration, and thickness of the soil pillar between the tunnels. Ground behavior is also a function of construction means and methods, such as ground improvement measures, excavation and initial support methods, timing and sequence of excavation and support, and workmanship. Excavation methods and support requirements should be developed taking into account these factors.

The Tunnelman's Ground Classification System (Heuer, 1974) for soil behavior is presented in Table 9 to aid in discussion of the expected ground behavior of the soil. The expected ground behavior described in the table applies to an unsupported tunnel face. Ground behavior may be significantly improved if ground modification measures are employed or a closed-face TBM is used.

As discussed in Section 4.5.2, the soils along the alignment can be divided into five soil groups. A summary of tunneling and excavation characteristics of these five soil groups is presented in Table 10. Boring logs, geologic unit descriptions and properties, and geologic profiles should still be utilized when more detailed soil information is needed. The table includes Group 1 soils, Fill and Non-overridden Soils, which is not likely to be encountered during tunnel construction, but has been included for completeness.

Based on available subsurface data, Tables 11, and 12 provide a breakdown by stationing and percentage of tunnel length, respectively, of the soil groups likely to be encountered during tunnel excavation for full and mixed face conditions. Where mixed-face conditions are present, the soil group with the worst ground behavior characteristics would likely control the selection and effectiveness of excavation methods and groundwater control measures.

Group 1 – Fill and other Non-overridden Soils. This soil group is not discussed because its soils are not likely to be encountered within the tunnel horizon.

Group 2 – Cohesive Silt and Clay. As indicated in Table 11, a significant portion of the University Link tunnel alignment would be excavated through glaciolacustrine silts and clays. Approximately 40 percent of the tunnel alignment would encounter this type of ground full-face and an additional approximate 43 percent in a mixed-face condition (Table 12).

The cohesive silts and clays are typically fractured or slickensided, but may be massive in places. These soils commonly contain slickensided or sheared zones. When not fractured, slickensided, or sheared, the cohesive silt and clay deposits are considered stable for tunneling and have stood unsupported in tunnel faces below the groundwater table for some time in previous but smaller and much shallower tunnel excavations throughout the Seattle area [DSTT, Mt. Baker Ridge Tunnel (MBRT) test adits, Columbia Center pedestrian tunnels]. However, where present, fractures, slickensides, and shear zones can substantially reduce the soil mass strength, in spite of the high intact strength of the glacially overridden silts and clays, and could cause raveling or block failure into an unsupported tunnel heading. Such failures in fractured clays have occurred in road cuts and foundation excavations in the Seattle area.

This unit of Group 2 soils has a low hydraulic conductivity and, in a mixed face condition, perched water may be encountered in granular soils above the cohesive silts and clays. Water perched above this unit would be difficult to dewater, particularly near the contact between the units. Consequently, cohesionless soils encountered above the cohesive silt and clay contact may become unstable and flow into the excavation unless the ground is improved by grouting, pre-drainage, or freezing. Scattered to abundant lenses or seams of saturated cohesionless sand or silt may also be present within this unit. These lenses or seams are not likely to be hydraulically connected to the regional groundwater regime, but may run or flow into the heading of an open-face shield if not drained or otherwise controlled or modified. Face breasting alone may not be sufficient to control and stabilize these cohesionless soils and more positive water control methods may be necessary. The fine grain size and low permeability of Group 2 soils would make grouting and dewatering difficult.

In mixed face conditions, trapped methane may be present in granular soils below the cohesive silts and clays. If encountered, methane concentrations are expected to be low and should dissipate quickly. However, appropriate methane detection and ventilation systems would be essential during tunneling. Several tunnels in Seattle (West Seattle Tunnel, Matthews Beach Sewer Tunnel, and Fort Lawton Sewer Tunnel) were temporarily shutdown from a few hours to a complete shift because of high methane concentrations.

The cohesive silt and clay soils can be "sticky" at certain moisture contents. Soils that stick to equipment could increase excavation time by requiring the cleaning of excavation conveyors, buckets, and muck cars and could clog tunnel machine cutterheads unless conditioners or special muck chute and paddle designs are used.

Group 3 – Cohesionless Silt and Fine Sand. As indicated in Tables 11 and 12, only about 2 percent of the tunnel alignment along University Link is expected to be excavated full-face through this type of ground. However, approximately 43 percent of the alignment would encounter Group 3 soils in a mixed-face condition.

The soils that have some cohesion (slightly clayey to clayey) would behave similar to, but somewhat more poorly than the cohesive silt and clay deposits (Group 2) described above. The silt and fine sand deposits with no cohesion, however, are expected to have little to no standup times, particularly when wet. Under hydrostatic pressure, these soils would become unstable at the heading and would flow or run into the excavation unless the ground is improved by grouting, dewatering, or freezing. Eductor well points, spaced 10 to 20 feet apart, have been effective in dewatering these fine-grained cohesionless soils in shallow tunnels (DSTT). Once these soils were drained, they stood well in a moderate-sized, shield-supported opening (20-foot diameter) and were considered to be good tunneling ground with minor face support. Jet grouting could also be utilized locally to stabilize these soils. However, considering the difficulty and cost for successfully implementing these methods, the use of a TBM with positive face control (Earth Pressure Balance [EPB] or Slurry) may be the most effective and least risky method for tunneling through these soils.

Group 4 – Till and Till-like Deposits. As indicated in Tables 11 and 12, only about 3 percent of the tunnel alignment along University Link is expected to be excavated full-face in Till and Till-like Deposits, and approximately 18 percent of the alignment would encounter these deposits in a mixed-face condition.

Soils of this group include both clayey and sandy deposits. The clayey Till and Till-like Deposits are primarily glaciomarine drift [Qvgm and Qpgm], which typically have a more clayey matrix and are somewhat more variable in texture and strength than the sandy deposits [Qvt, Qvd, Qpgt, and Qpgd]. In general, the clayey Till and Till-like Deposits have lower strengths and lower permeabilities than the sandy Till and Till-like Deposits. The permeability of the sandy Till and Till-like Deposits varies with the composition. In places, the clayey Till and Till-like Deposits are similar to and gradational with the Very Stiff to Hard Clay soils, but have a higher content of granular material.

Till and Till-like Deposits are expected to stand vertically in a tunnel heading or shaft walls with very little support and for prolonged periods of time, except in water-bearing zones that have a matrix with less silt and clay. Group 4 soils, as a whole, are lowly permeable, but perched water is often present in more pervious seams, and in overlying permeable soils. Water-bearing silt and sand lenses within these deposits are typically not hydraulically connected to the regional groundwater regime. Depressions and irregularities in the top of these deposits would make complete dewatering of overlying granular deposits difficult. Significant silt content and varying quantities of clay content make it nearly impossible to dewater or grout this unit.

Because of their consistency, these soils may be difficult to excavate with standard soil excavation equipment. These soil units have been excavated in past tunnels using roadheaders (University of Washington Southwest Campus Utilidor Tunnel), digger shields with ripper teeth (DSTT), hoe-rams (east shaft of the Mercer Street Tunnel) and TBMs equipped with picks and disc cutters (West Seattle Tunnel and Mercer Street Tunnel). The constituent granular portion of these units are considered to be very abrasive, causing excessive wear to excavation equipment, as experienced on the West Seattle Tunnel and the Mercer Street Tunnel.

Group 5 – Cohesionless Sand and Gravel. As indicated in Tables 11 and 12, about 3 percent of the tunnel alignment along University Link is expected to be excavated full-face in cohesionless sand and gravel [Qpgo and Qpnf], and approximately 16 percent of the alignment would encounter these deposits in a mixed-face condition.

Because these deposits are cohesionless, they are expected to be unstable in an open-face tunnel excavation without dewatering prior to excavation, or controlling the flowing material by using a TBM with positive face control such as EPB or slurry methods. Selection of the most appropriate ground modification approach would depend upon a reach-by-reach evaluation of the soil grain-size distribution along the tunnel.

As evident on the grain size distribution plot (Figure 39), the grain size distribution of Group 5 soils varies considerably. Based on the soil samples tested, these soils tend toward having a bi-modal distribution, with a subgroup consisting of slightly silty to silty, fine to medium sand (Group 5a) and a group of clean to slightly silty, gravelly sand (Group 5b). This real or apparent bimodal distribution is an important consideration, in that Group 5b soils (the coarser range of Group 5) may be near the limits of soils suitable for EPB tunneling, as discussed further in Section 5.3.4.

5.3.3 Stability Factor

In addition to the information available from previous projects in the Seattle area, the ground behavior of the Very Stiff to Hard Clay and the clayey portions of the Till and Till-Like Deposits can be estimated using empirical methods. This is useful, given that the planned facilities include several unique features.

A criterion commonly used to provide an initial assessment of the stability of underground openings in cohesive soils is the stability factor, which was developed by Broms and Bennermark (1967) and later adapted to running tunnels by Peck (1969). The stability factor uses the ratio of total overburden pressure to the soil's undrained shear strength to estimate potential difficulties in tunneling and is defined as:

$$N_T = \frac{P_z - P_a}{S_u}$$

where P_z is the overburden pressure at the depth of the centerline of the tunnel, P_a is the air, earth or slurry pressure above atmospheric (if any) exerted against the tunnel face and S_u is the undrained shear strength of the clay.

For $N_T < 3$, the ground is anticipated to behave elastically. For $N_T < 5$, tunneling can be accomplished without significant difficulties. For $N_T > 5$, cohesive soils would tend to squeeze into the tailskin void before grouting or liner expansion can be accomplished and would likely result in difficulties in mining, such as working of the face and deformation of the tunnel perimeter before the lining is installed. Excessive squeezing or working at the tunnel heading and perimeter may occur when $N_T > 6$. When $N_T > 7$, the tunnel machine would be difficult to steer and rapid deformation of the tunnel face and perimeter would likely occur in an open-face tunnel shield (Peck, 1969 and Heuer, 1974).

The stability factor does not explicitly take into account fracturing in clayey soils and, consequently, these correlations can be expected to overestimate the stability of the hard, fractured clays found in Seattle. Because this empirical formula was not developed for openings greater than 20 to 25 feet in diameter or for sequentially excavated openings, the stability factor should be considered as a rough approximation. With adjacent excavations separated by intervening pillars, pillar stability must be evaluated separately. The width of the soil pillar between the University Link running tunnels would range from about one to three tunnel diameters (20 to 60 feet).

5.3.4 Excavation Methods

Successful tunneling through the variable soil and groundwater conditions that would potentially be encountered along University Link requires that the TBM be selected to fit the ground conditions or that the ground be modified (dewatered, conditioned, or grouted) to fit the machine selected. Because of the wide range of material properties and their abrupt transition from one to another, the presence of groundwater, and the considerable depth of the tunnels, no single TBM or methodology would be best suited for tunneling the entire alignment. Whichever TBM and tunneling methodology is selected, some performance compromises would be necessary. A flexible yet well-proven excavation and support system should be selected to respond to the varied ground conditions of the project.

In general, an open-face TBM would not be suitable where a tunnel is to be driven through cohesionless silts, sands, and gravels below the water table without extensive dewatering, ground modification, or use of compressed air. A closed-face tunneling machine (EPB or slurry) capable of accommodating the changing ground conditions and groundwater heads may be best suited to these areas. However, extensive lengths of tunnel alignment in cohesive soils may not be suitable for a slurry machine, where the clay slurry used to transport the excavated soils could become overloaded and difficult to separate. Similarly, the gradation may become too coarse for an EPB machine without the use of excessive quantities of soil conditioning additives. The presence of cobbles and boulders also needs to be considered when selecting TBM equipment, as crusher or disc cutters may be needed to break up boulders.

Tunnels have been excavated, although at shallower depths, for over 100 years in Seattle using a wide range of techniques. Within the last 35 years, the primary equipment and methods used for these highly variable conditions have included:

- Open-face digger shield with full-face breasting, combined with dewatering and/or compressed air and compaction and/or permeation grouting at selected critical locations.
- Convertible EPB tunneling machine, with and without disc cutters for boulders, with and without a screw conveyor, with soil conditioning additives, and with and without dewatering.
- Slurry tunneling machine with disc cutters for boulders, primarily for river crossings and in flowing sands.

The use of an open-face digger shield is the least expensive and most versatile tunneling excavation method. Normally, open-face shields have hydraulically-activated breasting doors to provide support to much of the face while a backhoe-type digger is used to excavate the soils in

the lower half of the shield. The hydraulic doors are usually sufficient for maintaining stable heading conditions in cohesive soils as well as in cohesionless soils that are either above the groundwater table, have been dewatered, or otherwise stabilized with compressed air or ground improvement techniques. Open-face shields could be used effectively in tunneling through the glaciolacustrine [Qvgl and Qpgl] and glaciomarine drift [Qpgm] units; provided that any layers of saturated silts and sands are dewatered or grouted ahead of the face. The presence of fractures, slickensides, or sheared zones within Qvgl or Qpgl would not present difficulties for a closed-face TBM, whether EPB or slurry. However, ravelling or block failure of such ground could pose problems for an open-face machine.

The use of an open-face shield in cohesionless, granular soils including glacial outwash [Qpgo and Qva] and nonglacial fluvial [Qpnf] deposits is possible but risky, and would require extensive pre-drainage or some form of ground improvement prior to tunneling. The nonglacial lacustrine [Qpnl] deposits are difficult to drain and are practically impossible to grout. When saturated, these soils become unstable and would flow into the heading of an open-face shield and around the tail skin. Consequently, open-face shields may not be appropriate in these soils. The open-face shield allows access to the entire tunnel heading which makes the removal of boulders and other obstructions, such as piles, tiebacks, and abandoned utilities, much easier. Therefore, open-face shields might be necessary and applicable to sections of the tunnel where extensive obstructions are thought to exist and where ground improvement is possible to stabilize the soils.

With properly designed positive support, a closed-face tunneling machine should be capable of excavating a wide range of soil materials from hard, sticky clays to very dense glaciomarine drift to flowing sand and silt. The tunneling machine must be able to control water inflows with high piezometric heads. Much of the sandy and gravelly soils are abrasive and have resulted in heavy wear on rotating cutterheads, seals, and bearings.

Experience suggests that regular, daily to weekly maintenance, including checking and replacing selected cutter teeth, would be required. Major maintenance and/or replacements of seals and repair of muck chutes, muck augers, cutter saddles, and partial refacing of the cutterhead may be required every one to two miles, as experienced at the West Seattle Tunnel (Oatman et al., 1997). Routine maintenance has been accomplished using airlocks with water pressures up to 100 feet. However, greater water pressure heads in granular soils may require partial dewatering or ground improvement with grouting or freezing. In the EPB machine a variety of additives (such as polymers, surfactant foams, bentonite, vermiculite, and water) have

been used to condition the granular soils to a "paste" consistency. Soil additives have also been used extensively to reduce abrasion and resultant wear to the cutters, cutterhead, and seals.

Slurry tunneling machines have been used for several tunneling projects in saturated granular soils in Europe and Asia. The use of a bentonite slurry as a carrier medium for the spoils reduces the need for other soil conditioning additives but requires a large, complex, slurry separation plant at ground surface. The slurry separation process is much less effective in silt and clay soils. Slurry machines up to 45 feet in diameter have been used in Europe. Most slurry machines used in the U.S. have been less than 10 feet in diameter and have been used for microtunnels.

Boulders and cobbles may be encountered throughout the glacial units and may be concentrated along erosional contacts. Direct access to the face may be required for boulder removal. For the 32,600 feet of 9-foot-diameter drifts excavated for the MBRT, the digger shield was stopped about a dozen times to remove and break up boulders larger than about 2 feet in diameter. At least one boulder, 7 feet in diameter, was encountered in the main excavation. For the DSTT, tunnel advance was stopped several times to remove and break up boulders larger than about 3 feet in diameter. In deep, potentially flowing soils, access to the face for boulder removal may require compressed air, coupled with through-the-face grouting, steel or timber support, and/or dewatering. Disc cutters mounted on the cutterhead to augment the soil picks or spades may be able to break up some of the boulders in the glaciomarine drift [Qpgm] and possibly in the glaciolacustrine and lacustrine clays [Qpgl and Qpnl]. However, the discs would likely pluck out the boulders or pieces of boulders in the sand and silt units, and the freed fragments would then tend to roll around in the face, resulting in damage to the cutterhead, overexcavation, and/or increased ground losses that could lead to unacceptable ground surface settlements. Nests of boulders encountered along the Chambers Creek Sewer Tunnel (Douglas et. al., 1985) resulted in excessive ground losses and chimney-like settlements ahead of a wheel excavator, which prompted the substitution of a fully-breasted, open-face digger shield.

5.3.5 TBM Selection

Published criteria offer guidance in the selection of EPB or slurry TBMs. Suitability of both methods is primarily dependent on permeability and consistency of the ground, which can be characterized by grain size distributions. Herrenknecht has published a series of guidelines (Herrenknecht, 1994; Maidl, Herrenknecht, and Anheuser, 1996) that indicate the range of grain size distributions over which slurry and EPB methods are appropriate. The criteria include both the range for optimal operation, as well as extended limits over which operation may be practical with the appropriate use of soil conditioning additives (polymers and foams). Langmaack has

published similar relationships (Jancsecz, Krause, and Langmaack, 1999; Langmaack, 2002; Langmaack, 2003). These guidelines were developed from soil conditions encountered on tunneling projects familiar to the authors at the time of their publication. The range of soil gradations suitable for tunneling has broadened from advancements in conditioner and tunneling technology and from successful completion of tunnels in soil conditions not previously encountered using a particularly method. Herrenknecht's and Langmaack's TBM guidelines are discussed below. Groundwater heads and permeability control the upper limits shown on their guidelines. Generally, the lower the groundwater head the coarser the material that can be handled by the TBM.

The average gradations for the four soil groups, Group 2 through Group 5, are compared with Herrenknecht's guidelines (Maidl, Herrenknecht, and Anheuser, 1996, Figure 10-5) for the suitability of the slurry TBM in Figure 40. Soil Group 1 is not included in the figure because these soils are rarely encountered within the tunnel horizon. Similarly, Figure 41 shows a comparison of Langmaack's guidelines (Langmaack, 2002, Figure 1) for slurry TBMs with the gradations of the four soil groups. Both Figures 40 and 41 show that Group 5 soils tend to fall within the desired grain size distribution range both guidelines. Although portions of the coarser-grained fraction of the Group 5 soils (GP5b) slightly exceed Herrenknecht's limit, we believe that the limit can likely be extended to even coarser soils with recent advances in the use of conditioners. However, the finer-grained fraction of the Group 4 soils and the entire gradation of the Group 3 and 2 soils fall outside the finer limit of both guidelines. These silty and clayey soils would tend choke the slurry and likely make removal of fines from the slurry difficult and expensive (unless the fine-grained soils have sufficient consistency to act primarily as a coarser-grained soil).

The guidelines from Herrenknecht and Langmaack for the suitability of EPB TBMs are compared with the four soil gradations in Figures 42 and 43, respectively. Soil Groups 2, 3, and 4 as well as the finer-fraction of the Group 5 soils (GP5a) fall within Herrenknecht's guidelines. The coarser fraction of the Group 5 soils (GP5b) essentially falls within the range that Herrenknecht considers "where EPB TBMs should not be applied under water pressure (Maidl, Herrenknecht, and Anheuser, 1996, page 281)." For soils coarser than this limit, Herrenknecht's states that the "permeability is too high" to build up support pressure even with the use of conditioners. Again, we believe that recent advances in soil conditioners have likely extended this limit into coarser-grained soils. All four soil groups fall within Langmaack's guidelines for EPB TBMs with soil conditioners (Langmaack, 2002, Figure 1; Langmaack, 2003, Figure 12).

The depth of the tunnels, coupled with the likely presence of potentially flowing silts [Qpnl] or sands [Qpnf or Qpgo] would likely dictate the use of a closed-face tunneling machine for more than half of the alignment. For granular soils a slurry TBM may be more suitable. Conversely, for finer-grained soils an EPB TBM may be slightly more suitable. However, either a slurry or EPB TBM would likely be satisfactory provided that appropriate conditioning agents are used.

The need for large volumes of conditioning agents would affect the tunneling costs and should be considered. The need for conditioning is dependent to a large extent on the permeabilities of the soils and groundwater heads and is discussed for slurry and EPB in subsequent sections of this report. Table 13 gives a rough estimate of soil permeabilities for various tunnel segments.

Note that there are no soils identified along the tunnel horizon with permeability estimates in the 10^{-1} cm/sec range. However, a nominal percentage of tunnel length in each segment is shown for conservatism.

5.3.5.1 Bentonite Consumption for Slurry TBM

During operation of a slurry TBM, loss of support fluid into the ground at the face would occur as a filter cake is established. Consumption of this support fluid, typically a bentonite slurry, is primarily dependent on the permeability of the ground. Little published information is available on the relationship between bentonite consumption and permeability. Richards (2005) reported that a tunnel in France followed the general relationship shown below:

Permeability (cm/sec)	Consumption (% by weight)
10^{-4} or less	0 to 0.5*
10^{-3}	1
10^{-2}	2
10^{-1}	3

* Estimated by Shannon & Wilson based on trend in French data.

5.3.5.2 Soil Conditioner Usage for EPB TBM

Efficient operation of an EBP TBM typically requires use of soil conditioning polymers and foams. Polymer is often used to reduce permeability and extend the range of EPB

operation into coarser-grained soils. Based on several published case histories (Milligan, 2000), the following table presents ranges of typical polymer usage based on permeability:

Permeability (cm/sec)	Polymer Usage (% by weight)
10^{-4} or less	0
10^{-3}	0.1
10^{-2}	0.5
10^{-1}	1 to 2*

* The lower and higher values are based on lower and higher groundwater heads, respectively

Foams are commonly used in conjunction with polymers to reduce permeabilities, decrease abrasion, and reduce torque requirements. Foams are typically 90 to 95 % air. They are created by using a surfactant to reduce the surface tension at the air-water interface. Usage rates vary greatly, depending on soil pore volume (gradation), water content, permeability, and consistency, as well as the support pressure in the excavation chamber and the amount of polymer in use. Based on a typical Foam Expansion Ratio (air volume per foaming solution volume) of about 10 (typically varies from 8 to 12), and a Foam Injection Rate (foam volume per soil volume) of 30 to 60%, a typical agent usage rate of foaming solution would be about 3 to 6 % (EFNARC, 2001; Degussa, 2005). A foaming solution may vary from 1 to 10% surfactant agent, the rest being water.

5.3.6 Support Requirements

Support systems for running tunnels can be categorized as either “two-pass” or “one-pass” linings. Two-pass systems consist of an initial and a final lining. The initial lining is installed immediately behind the TBM and is designed to support the soil and groundwater loads until the final lining is placed, to prevent inflow of soil, and to safely accommodate all construction loads such as liner handling and assembly forces as well as TBM jacking pressures. The final lining is installed later and is used to support all long-term loads, to provide the desired grade and interior surface, and to provide waterproofing. The one-pass lining is installed immediately behind the TBM and is designed to act as both the initial and final tunnel support. Tunnel support systems used in Seattle over the last 25 years include:

- Two-pass steel rib and timber lagging followed by cast-in-place concrete.
- Two-pass expanded concrete segments followed by cast-in-place concrete.
- Two-pass bolted steel liner plate followed by cast-in-place concrete.
- One-pass gasketed, bolted, pre-cast concrete segments.
- One-pass steel ribs and shotcrete.

The selection of support systems would depend on many factors including soil type, tunnel depth, and groundwater levels. As indicated on Table 12, for almost 60 percent of University Link, the tunnels would be driven through low permeability soils (Groups 2 and 4) including glaciomarine drift [Qpgm] and glaciolacustrine deposits [Qpgl]. In these soils, any of the above support systems could be used. However, where the tunnels intersect water bearing, granular soils [Qpgo, Qva, and Qpnf] or cohesionless silt [Qpnl], the choice of initial support becomes limited without dewatering or ground modification (grouting or freezing). If dewatering or ground modification is used effectively, then any of the above support systems could be used. In areas where dewatering would be difficult, such as along contacts between highly permeable and lowly permeable soils, deep tunnel sections with high hydrostatic heads, and where fine-grained cohesionless soils are encountered, a gasketed one-pass concrete segmental lining may be more practical and cost-effective.

Difficulties with gasket and segment alignment for the West Seattle Tunnel (Alki CSO Project) installation resulted in over 100 gallons per minute (gpm) leakage for about two miles of tunnel under water heads of only 10 to 30 feet. However, during the same year, similar gasketed segments were installed in a two-mile-long outfall tunnel 250 feet below the ocean surface with less than 20 gpm of inflow (San Diego Outfall Tunnel). These two experiences illustrate the need for high quality workmanship and construction monitoring in single-pass linings. More recently (2001 and 2002) a single-pass, bolted/doweled, gasketed concrete segmental liner was used on the 6,200-foot-long, 17-foot O.D. Denny Way CSO/Mercer Street Tunnel. Minor cracking of segments and offsets of gaskets resulted in leakages of less than 20 gpm for the 6,200 feet of tunnel. These leaks were nearly eliminated with follow-up patching and grouting at completion of the tunnel excavation. Excellent performance of one-pass linings has come to be expected in other parts of the world. With good design, specifications, and very careful construction, a one-pass, gasketed, bolted, segmental lining should be capable of accommodating the water heads, soil conditions, and depths anticipated along the alignment of the running tunnels.

5.3.7 Support Loads

The loading on the tunnels would depend on many factors including soil and groundwater conditions, tunnel depth and size, and type of support. For a two-pass system of supports, a distinction is usually made between anticipated ground loads for initial support and those for final linings. The tunnels along University Link are located at approximate depths of 75 to 305 feet and would be excavated through a variety of glacially consolidated soils. Approximate clear distance between the running tunnels ranges from 20 to 100 feet.

Preliminary recommended properties for each of the major geologic units anticipated to be encountered in the running tunnels ranges of shear modulus, shear strength, K_o, unit weight, and permeability are presented in Table 14. These values are based on soil borings, laboratory testing, and in situ pressuremeter tests completed during this and previous design phases of the project, as well as previous experience with Seattle ground conditions.

For preliminary design, soil pressures on the initial support systems for the running tunnels can be assumed to be equivalent to approximately two tunnel diameters of total soil load, plus the maximum hydrostatic pressure if the liners (such as gasketed, bolted, or pre-cast concrete segments) do not allow free drainage of the soils. For preliminary design, final support for the running tunnels should be designed for the full overburden soil pressure and the maximum full hydrostatic pressure. Current groundwater measurements indicate that up to 120 feet of hydrostatic pressure would act on the tunnel liners.

Very little data is available from either local or worldwide projects regarding long-term loads on tunnels in hard, glacially overconsolidated clays. The most pertinent data is from deep tunnels in London clay which suggest that long-term soil creep would eventually result in full overburden loads on the tunnel lining (Peck et al., 1969). In addition, intersections between openings (tunnels and shafts) tend to concentrate and multiply soil loads. Therefore, at tunnel and/or shaft intersections, the preliminary design pressures may be assumed to double at the point of intersection and decrease within one diameter of the larger opening (Peck et al., 1969).

5.3.8 Groundwater Flows and Control

Most of the tunneling would be completed in or below water-bearing soils. The extent to which groundwater inflows would affect tunneling operations and tunneling machine selection depends on groundwater heads, the soils present at the tunnel horizon, and the method of tunnel excavation and support. Tunneling using an open-face shield has been successfully completed in the Seattle area in glaciomarine drift [Qpgm] and glaciolacustrine [Qvgl and Qpgl] silt and clay with up to 40 feet of groundwater head without the need for dewatering. These soil units generally have very low hydraulic conductivities and have sufficient short-term stability immediately after excavation. However, the DSTT project showed that where zones of cohesionless silt or sand are encountered, flowing ground could occur with less than 10 to 20 feet of groundwater head.

Glacial outwash [Qpgo and Qva] and nonglacial fluvial [Qpnf] deposits of sand and gravel are present at several locations along the tunnel horizon. In a saturated, unsupported tunnel excavation, glacial outwash [Qpgo] and fluvial [Qpnf] sands would likely flow or run into

the heading. Pumping tests for the Mercer Street Tunnel performed in Qpgo soils indicate that they could be dewatered successfully at depths of 100 to 200 feet. Tunneling using an open-face TBM was, therefore, feasible; however, pumping rates of 200 to over 1,000 gpm would have been required for long lengths of tunnel. Obtaining permits to dispose of such large quantities of water into the city's combined sewers would have been difficult, especially during winter months. The University Link tunnel depth is great with high hydrostatic heads present. To determine the feasibility of dewatering in these units, studies should be performed during final design to evaluate whether open-face tunneling would be feasible.

Dewatering of cohesive clays and silts [Qpgl and Qpnl] and glaciomarine drift [Qpgm] is generally not necessary. However, for silts with low clay content, flowing conditions may develop during excavation. Since the hydraulic conductivity of the silt is generally low, dewatering these soils would be very difficult, time consuming, and costly, even if all of the seams and lenses could be located and intercepted. On the DSTT, a layer of cohesionless silt about 20 to 40 feet thick was successfully dewatered using vacuum eductor/ejector wells spaced 10 feet apart over a length of about 300 feet of twin tunnels. The tunnel and silt layer was at a depth of about 40 to 50 feet. Such a dewatering system, while technically feasible, would be very expensive for dewatering deep deposits of silts and sands over long lengths of tunnel. Even for a closed-face EPB machine capable of balancing the measured groundwater heads, the location and extent of water-bearing layers as predicted from additional explorations and from probes ahead of the face, would be useful information for assessing abrupt changes in groundwater pressures, earth pressures, and the need for soil conditioning additives. Adequate subsurface data would also be needed for closed-face TBMs to locate "safe havens" of suitable soil where back-loading cutters can be changed or the cutterhead can be serviced, preferably without compressed air or dewatering.

As discussed in Section 5.3.5, the final liner should be designed to resist the maximum full hydrostatic pressure. Temporary initial liners may permit some groundwater inflows, as long as these inflows do not carry or "pipe in" soil. Such erosion could result in adverse ground losses that could affect facilities at the surface. Groundwater inflow quantities within the outwash [Qpgo] soils could become significant if a permeable initial liner, such as steel ribs and timber lagging, is used. Depending on the grade of the tunnel, operations may require that the groundwater inflow be pumped which may result in significant operational costs during tunneling. For these reasons, it may be desirable to require the tunnel to have a watertight or water-resistant initial liner. Initial support system should also be designed for the maximum hydrostatic pressure if the liners (such as gasketed, bolted, or pre-cast concrete segments) do not allow free drainage of the soils.

A summary of the tunneling characteristics (stability, excavatability, standup time, ground modification, and groundwater flows) for the soil groups anticipated to be encountered along the alignment is presented in Table 10. This table summarizes the discussion of these characteristics in the previous sections.

5.4 Mined Structures

5.4.1 General

Mined structures proposed along University Link include short lengths of tunnel that would connect the proposed TBM retrieval shafts west of I-5 to the C510 Pine Street Stub Tunnel. Other mined structures include cross-passages that would connect the two parallel running tunnels and adits for the proposed Montlake Ventilation Shaft.

The following sections address our preliminary engineering for the mined structures. Our analyses assume that the mined structures would be constructed by SEM, sometimes called the New Austrian Tunneling Method (NATM). General subsurface conditions at the proposed mined structure locations are summarized in Table 8 and are shown on the subsurface profile presented in Figure 7. Table 8 includes the anticipated soil and groundwater conditions based on a relatively small number of borings completed near currently proposed levels of mined structures. Soil conditions are generally summarized for two diameters above the station crown to one diameter below.

5.4.2 Ground Behavior at Mined Structures

This section describes the anticipated ground behavior at the Pine Street Connector Tunnels, Montlake Ventilation Shaft adits, and cross-passages along the alignment. Our description of anticipated ground behavior is based on available exploration data and our experience in the Seattle area. Ground behavior is a function of soil and groundwater conditions, depth of soil cover above the mined structure and its size and configuration, effectiveness of any implemented ground improvements, excavation and initial support methods, timing and sequence of excavation and support, and workmanship. Additional considerations that can affect ground behavior include location and configuration of the crossovers, and location and excavation method employed in any pedestrian cross-passages between the running tunnels. As such, these factors should be considered when excavation and support requirements are developed.

Excavation equipment, methods, and support requirements may vary at each mined location based on soil and groundwater conditions. It is essential to sequence and construct the staged excavations to maintain the strength and integrity of the soil over and adjacent to the

openings. Soil reinforcement using chemical or cement grouting, barrel vaults, soil bolts, spiling, and/or face wedges may be required to maintain stability, especially in sandy soils or clayey soils that are highly slickensided or sheared. Ground stabilization and dewatering would be most effective if performed from the running tunnels or shafts prior to excavating enlarged cross-passages. Grouting and dewatering would likely be effective in only relatively clean sand deposits. Further discussion on considerations related to behavior of soil and groundwater are presented in Section 5.3.2

5.4.2.1 Pine Street Connector Tunnels

The Pine Street Connector Tunnels would connect the proposed TBM retrieval shafts west of I-5 to the C510 Pine Street Stub Tunnel as shown on Figures 2 (Sheet 1 of 13). The north-bound and south-bound connector tunnels would be about 85 and 115 feet long, respectively. Subsurface conditions are presented in Figure 7, Sheet 1 of 13), which are based on soils and groundwater encountered in borings NB-398, NB-399, and NB-400.

Construction of the connector tunnels using SEM is anticipated to be difficult because of poor and variable ground conditions and potential obstructions including timber piling and wood from the Pine Street trestle and existing tiebacks from the C510 Pine Street Stub Tunnel construction. Most of the lower half of the two tunnels would be excavated in hard or very dense, glaciolacustrine (Qpgl) and glaciomarine (Qpgm) soils; however, much of the upper half of the tunnel excavations will likely encounter less dense or stiff landslide debris (Hls) and possibly fill (Hf), alluvium (Ha), and peat (Hp) deposits.

Qpgm and Qpgl soils are generally considered to be stable ground for mined excavation. The Qpgm soils are very dense with a hard clayey matrix that provides a relatively long standup time between excavation and the installation of initial ground support. These Qpgm soils could have the consistency of soft rock and similar soils have been excavated in the past using road headers, diggers with ripping teeth, and TBMs equipped with picks and disc cutters. Cobbles and boulders may be encountered in these soils. These soils have scattered sheared seams and slickensided surfaces that may form wedges or blocks of soil, which could cause instability in the tunnel crown and heading face unless some form of pre-support such as spiling, face wedges, and/or face bolts are provided during excavation.

Qpgl soils are also favorable ground for tunneling; however, these glaciolacustrine clays have scattered sheared seams and are likely to contain fractures or slickensided surfaces. The Qpgl soils may also contain seams and partings of silt and fine sand. Cobbles and boulders may also be encountered in these soils. The Qpgl soils are expected to

have a moderate standup time where the deposit is massive and very short to no standup time in areas that are highly fissured or jointed and where layers of saturated, cohesionless silt and sand are encountered. The fissured or jointed nature of these soils combined with bedding surfaces are commonly the cause of instability and may require the use of spiling, face wedges, and or face bolts to maintain stability during excavation. A layer of low plasticity silt and wet, cohesionless silt and sand (Qpnl) is likely present in portions of the tunnel excavations. These soils would have little standup time and would likely run or flow into the excavation unless the ground is successfully improved by grouting and/or dewatering prior to excavation. Jet eductor or vacuum lances may be the most effective dewatering method in the fine cohesionless soil layers.

The upper half of the connector tunnels intercept a topographic swale eroded into the Qpgl and Qpgm soils discussed above. The swale has been filled with recent deposits of Hf, Ha, Hp, and Hls, and boulders could be concentrated at the base of the swale. These soils have not been glacially overridden and were typically wet when encountered in borings. These soils are likely to have little or no standup time and would be classified as fast ravelling to flowing. The crown and face of an SEM-excavated tunnel located in these soils would need to be supported by a combination of grouting, spiling or barrel vaults, face wedges or bolts, lattice girders, shotcrete, and soils nails.

In addition to poor ground conditions, a timber trestle once crossed the swale before being filled. Timber piles, as well as boulders and wood or other debris could be encountered. Tieback tendons and anchors from adjacent structures could also be encountered. Tiebacks that extend into public right-of-ways should have been de-stressed following construction completion but might not have been. Tiebacks from the Pine Street Stub Tunnel, under construction, extend into the location of the proposed connector tunnel. SEM excavation and support around these tiebacks will be difficult, particularly if they have not been de-stressed.

5.4.2.2 Montlake Ventilation Shaft Cross Adit and Launch Tunnels

The Montlake ventilation shaft is located approximately 300 feet south of SR-520 on E Roanoke St (NB Sta.1182+80 to 1183+50). A ventilation cross adit would extend outward approximately 38 feet to either side (east and west) of the vent shaft to intercept the running tunnels, which would be excavated after shaft and adit construction. The cross adit would have a diameter up to approximately 37 feet. Short launch tunnels, approximately 15 feet long, would be excavated parallel to the running tunnels, both north and south from either end of the cross adit. The launch tunnels would have an approximate diameter of 26 feet. The excavation for the cross adit and launch tunnels would be located approximately 84 to 122 feet below the existing

ground surface. Soil and groundwater conditions at the ventilation shaft and adit are characterized by boring NB-251 (see Figure 7, Sheet 11 of 13).

The adits are anticipated to be excavated in glaciolacustrine soils (Qpgl). These soils are anticipated to be favorable ground for tunneling and mined station construction. However, the glaciolacustrine clays have scattered sheared seams and are likely to contain fractures or slickensided surfaces. The Qpgl soils may also contain seams and partings of silt and fine sand. Cobbles and boulders may also be encountered in these soils. These soils are expected to have a moderate standup time where the deposit is massive and very short to no standup time in areas that are highly fissured or jointed and where layers of saturated, cohesionless silt and sand are encountered. The fissured or jointed nature of these soils combined with bedding surfaces are commonly the cause of instability in slopes and tunnel arches, walls, and headings. At the Beacon Hill Tunnel, currently under construction, similar soils with shears and fractures were prone to raveling conditions in the crown and face of the heading during incremental excavation and required the use of spiling and face wedges or bolts to stabilize the excavation face and perimeter until the lattice girders and shotcrete were installed.

Depending on the geometry of the contact between the Qpgl and the overlying soils, the excavation at the top of the adits could encounter till (Qpgt). This unit is generally considered to be stable ground for mined excavation. The soil is very dense and could provide a relatively long standup time between excavation and the installation of initial ground support, depending on the silt and clay content. Qpgt commonly has the consistency of soft rock and has been excavated in the past using road headers or diggers with ripping teeth. This stratum, however, could contain lenses or layers with less silt and clay binder that could significantly reduce standup time, particularly if groundwater is present.

5.4.2.3 Cross-Passage Tunnels

Emergency cross-passages would be constructed approximately every 800 feet along the running tunnels, except when near a station or tunnel portal where distance from cross-passage to the station or portal would be as much as approximately 1,200 feet. Cross-passages would connect the northbound (NB) and southbound (SB) running tunnels and would be primarily for emergency egress of passengers. It is recommended that final locations of cross-passages would only be established after additional borings are made in the final design program.

The cross-passages would be 10 to 12 feet high and wide (inside dimensions) and would extend for lengths of approximately 20 feet, depending on the distance between the

running tunnels. Excavation of the cross-passages would be conducted at depths ranging from 50 to 300 feet below the existing ground surface. These tunnels would likely be excavated by using SEM using either full face or top heading and bench methods.

In situ soil stresses would be high (up to double the overburden pressure) within about 20 feet of the running tunnels because of stress redistribution around the tunnels (Peck 1969). In addition, soil within 5 to 10 feet of the running tunnels would likely be disturbed as the result of tunnel construction. These two factors would likely affect the overall behavior of the soils in the vicinity of the running tunnels.

Soil and groundwater conditions at the cross-passages would vary from glaciolacustrine [Qpgl] silts and clays to saturated outwash sands and gravels [Qpgo]. Tables 11 and 12 summarize soil groups likely to be encountered during tunneling. Where possible, cross-passages should be located in tunneling-favorable units, such as glaciolacustrine [Qpgl] and glaciomarine [Qpgm] deposits, both of which are cohesive and have low hydraulic conductivities.

The Qpgl unit can be locally massive or fractured and would likely contain lenses or layers of cohesionless silt and fine sand. Assuming an average undrained shear strength of 5,500 pounds per square foot (psf), stability factors for the deepest cross-passages are between 6 and 7. Although the stability factors are only a rough estimation, they indicate that the clays would be overstressed and would tend to squeeze and work at the heading and in the walls and crown of the excavation for cross-passages in the deepest portions of the tunnel alignments. If fractured or overstressed, the soils may ravel and would require pre-support ahead of the excavation. Because of the high stress levels, new fractures are expected to develop around the excavations.

Groundwater levels may be up to 130 feet above the cross-passage excavations in the deepest portions of the tunnel alignments. However, the glaciolacustrine [Qpgl] soils have low hydraulic conductivities, and water flows are not expected from these units except when lenses or layers of wet, cohesionless silts or sands are encountered. If lenses or layers of saturated cohesionless silt and sand within Qpgl are encountered near the crown of the tunnel, water may flow into the excavation and ground improvement may be necessary to limit ground loss and maintain stability.

Methane was measured during laboratory testing on several samples of clay [Qpgl], as discussed further in Section 5.13.2. Methane may exist in lenses of granular soils within Qpgl, but the presence of methane within these soils is anticipated to be limited. These

clays are not likely to yield large quantities of methane in an adequately ventilated tunnel heading.

A number of cross-passages would likely be excavated through nonglacial lacustrine [Qpnl] deposits. These soils consist primarily of very dense, cohesionless silt and fine sand with zones or lenses containing small amounts of clay resulting in scattered seams or layers with small to moderate cohesion. The soils that have some cohesion (slightly clayey to clayey) would behave similarly to, but significantly poorer than, the glaciolacustrine [Qpgl] deposits described above. The cohesionless silts and sands would maintain reasonable stability when dry or damp, but when saturated would become unstable at the heading and flow into the excavation unless the ground is improved. Similar soils under approximately 30 feet of hydrostatic pressure were encountered during construction of the DSTT. At the DSTT, these soils became unstable at the heading of the open-face digger shield, and tunneling was delayed until dewatering could be accomplished. Chemical grouting of these fine-grained soils was attempted, but was not effective. Dewatering was accomplished using closely spaced eductor or ejector well points positioned on either side of the tunnels. Once the soils were drained, they stood well in a 21-foot-diameter heading with minor breasting for face support.

Cross-passages may be constructed in glaciomarine drift [Qpgm]. These soils consist of a very dense or hard mixture of sand, gravel, silt, and clay and typically stand well in a tunnel heading. The unit is relatively impermeable but may contain saturated lenses of cohesionless silts and sands that may require additional stabilization techniques at exposed faces. The soils may be difficult to excavate due to their high strength and the potential presence of cobbles and boulders.

Cross-passages may also be constructed through nonglacial fluvial deposits [Qpnf] and glacial outwash [Qpgo]. These deposits consist of sand and gravel with scattered cobbles and boulders. Groundwater heads in excess of 90 feet are expected in the deepest portions of the tunnel alignments. The soils are very dense but cohesionless, and are expected to run or flow into an excavation when wet without ground modifications. Even dewatered, these soils may tend to run into an unsupported excavation. Complete dewatering may not be possible if these granular soils are encountered in a mixed-face condition above soils with a low hydraulic conductivity [e.g. Qpgm, Qpgl, Qpns, and Qpnl]. Methane may also be encountered in these granular soils, particularly in the deep Qpgo deposits beneath First Hill. Ground improvement methods would likely need to be conducted from the running tunnels and should be accomplished well before cross-passage construction begins.

5.4.3 Excavation Sequence and Methods

For this report, we have assumed that SEM would be used to construct the mined structures. The SEM method consists of a staged sequence of excavation and support, together with the application of the observational method for monitoring ground behavior and modifying construction procedures accordingly. Quality control and attention to detail at all stages of excavation and support is critical. Large underground openings are created by sequentially excavating and supporting a series of smaller drifts, which are inherently more stable than the larger or full-face openings. These smaller drifts are typically supported with mesh reinforced or steel fiber-reinforced shotcrete (MRS or SFRS) and soil bolts, often supplemented by spiling, forepoling, and lattice girders. In cohesionless soils, drainage, horizontal soil mixing, jet grouting, or freezing may also be required. Further details on this type of construction are premature for this phase of the project. However, it should be noted that the size and sequence of drifts and pre-drainage or pre-support, as well as the nature and timing of initial support, would be different for hard clay versus sand or silt.

In general, the soils encountered in the borings in the vicinity of the mined structures are either dense to very dense or very stiff to hard. Excavation of these glacially consolidated soils is expected to require heavy excavating equipment designed to dig in dense or hard soils. Special hydraulic splitters may also be required to remove cobbles and boulders, especially in the glaciolacustrine soils [Qpgl], glaciomarine drift [Qpgm], and near the contacts between soil units where concentrations of cobbles and boulders may be higher. Boulders with diameters of about 7 feet have been reported during construction of the MBRT, and boulders with unconfined compressive strengths of up to 30,000 pounds per square inch (psi) were measured during construction of the West Seattle Tunnel (Alki Transfer/Combined Sewer Outfall [CSO] project).

The glaciolacustrine [Qpgl] and lacustrine [Qpnl] units may be relatively easy to excavate using standard hydraulic excavation equipment, such as large-toothed backhoes, as has been accomplished for the DSTT and the MBRT. However, these soils can become "sticky" when slightly wet. A soil that sticks to equipment could increase excavation time because of additional time necessary to clean excavation conveyors, buckets, and muck cars. Depending on its size and depth, the excavation may be completed in sections without pre-drainage in these soils. The exception may be where a significant thickness of silt or sand exists within the unit. In these areas, pre-drainage and pre-excavation support using spiling may be required.

Large portions of the glaciomarine drift [Qpgm] are likely to have the excavation characteristics of lean concrete. Qpgm deposits have been successfully excavated with difficulty using heavy-duty toothed backhoes or hydraulic hoe-rams. Roadheaders, normally used to

excavate rock or concrete, have been used to excavate glacial till on recent utility tunnels at the University of Washington. Where glaciomarine drift was encountered in the DSTT, excavation rates increased because of the increased stability and standup time of these soils. However, these very dense soils were abrasive, causing the rate and amount of wear on the excavating equipment to increase. Depending on the size and depth of excavation and groundwater conditions, we anticipate that these soils could be excavated in small drifts with little or no pre-support or pre-drainage. However, sand lenses and layers of cohesionless silt are likely present within this unit, which could require pre-drainage or pre-support with spiling and possibly face-support with breasting and/or shotcrete.

5.4.4 Standup Times

Standup time is generally defined as the time between excavation and the installation of initial ground support. Standup time depends not only on the soil and groundwater conditions, but also on the size, depth, configuration, and sometimes the orientation of the unsupported section because of the presence of soil structure, such as slickensides. A soil may have a standup time of several hours when the heading is unsupported for a distance of 3 feet, but only several minutes when the heading is unsupported for a distance of 20 feet before initial support is installed.

In general, the glaciomarine drift [Qpgm] is expected to have a relatively long standup time; however, it may have a very short to no standup time in areas where significant sand layers and groundwater are present. Glaciolacustrine silt and clay [Qvgl and Qpgl] is expected to have a moderate standup time where the deposit is massive, and very short to no standup time in areas that are highly fissured and blocky. The glacial outwash [Qpgo] is expected to have very short to no standup time with a tendency to flow or ravel unless this behavior is improved by pre-support and pre-drainage. The presence of tunnel intersections, narrow pillars (less than 15 feet thick), wide and/or flat spans, and groundwater would significantly reduce standup time in all soil units. Table 10 presents a summary of standup times for the different soil groups anticipated along the project alignment.

5.4.5 Support Requirements

For most of the mined structures, initial support would probably consist of steel ribs and timber lagging or shotcrete. Some mined structures may require special measures such as grouting, dewatering, or freezing. Soil anchors may also be used to help provide support, reinforce soil pillars, and/or reduce the number of lattice girders in selected areas where glaciomarine drift [Qpgm] or glaciolacustrine [Qpgl] deposits are encountered. In areas that

require spiling over the crown, such as cohesionless silt layers and fractured zones in both the glaciolacustrine [Qpgl] and lacustrine [Qpnl] units, the spiling could be incorporated into the lattice girder and shotcrete system. Where relatively clean sand [Qpgo, Qva, and portions of Qpnf] is encountered, soil stability and stand-up time may be improved with chemical and/or cement grouting, and/or dewatering. For the DSTT, silicate grouts were used very effectively in the clean sands over short lengths of the alignment. Specialized dewatering systems such as vacuum eductor/ejector wells, with dewatering points spaced as close as 20 feet were used to effectively dewater and stabilize silty fine sand on that project as well.

Once the initial support is installed, a waterproof polyvinyl chloride (PVC) membrane is typically installed against the initial support and a finished liner system is constructed. The finished liner might be cast-in-place concrete or shotcrete.

5.4.6 Support Loads

The loading on the mined structures would depend on many factors including soil and groundwater conditions, depth, size and configuration of openings, sequence of excavation, and type of support. The mined structures would be located at approximate depths of 50 to 215 feet and would be excavated through a variety of glacially consolidated soils.

Based on our previous experience with Seattle soil conditions, theoretical horizontal loads may vary from 50 to 200 percent of the vertical loads (K_o=0.5 to 2.0). Horizontal stresses with K_o values of 2 or more have been measured in Seattle by pressuremeter tests, but actual excavations apparently have not exhibited abnormal deformations or loads during construction. However, the conditions of restraint and the effects of free-field stresses may be significantly different in a large deep mined opening than any other construction to date in Seattle. The MBRT was designed for a K_o of both 0.5 and 1.5. Pressuremeter tests conducted for this project indicate a similar range of K_o values. The results of the In Situ Pressuremeter Testing are presented in Appendix C of the GDR. Preliminary engineering recommendations for ranges of shear modulus, K_o, shear strength, unit weight, and permeability coefficients for each of the major geologic units anticipated to be encountered in the construction of the mined structures are presented in Table 13. These values are based on the Conceptual and Preliminary Engineering explorations, laboratory tests, in situ pressuremeter tests, and previous experience with similar subsurface conditions in the Seattle area. We recommend that excavation and support analyses be performed that accommodate the full range of soil properties including the worst-case scenarios. Future explorations and tests may allow further refinement of these values.

Glaciolacustrine clays [Qpgl] have exhibited swelling characteristics on a few past tunnel projects. During construction of the test shaft and adits for the MBRT, long-term swell deformations of up to 2 inches were observed where water was dripping or condensing on exposed clay surfaces. Swelling of the soils exerted sufficient pressure to fail 6-inch-square invert strut timbers spaced approximately 4 feet apart. This behavior was observed only where the clays were in direct contact with standing water and where they were left unsupported or unconfined for prolonged periods of time. Swelling behavior has not been observed in laboratory tests conducted for this project on the glaciolacustrine clays [Qpgl], but further testing should be accomplished during final design.

If the mined structures are required to be watertight, the final liner systems should be designed to accommodate the maximum groundwater pressures measured at each structure. Available groundwater measurements indicate that up to 150 feet of hydraulic pressure head at boring NB-250 (Station 1146+60 near the north end of Capitol Hill). Additional discussions about groundwater issues are presented in Section 5.4.7.

The general configurations of the mined structures are relatively complicated and would likely result in complex stress-strain conditions, particularly where openings intersect. Due to the complex configuration of these unusually large and deep openings, we recommend that sophisticated computer analyses, such as Fast Lagrangian Analysis of Continua (FLAC) or ABACUS be used to model the sequence of excavation and support, and the accompanying soil and groundwater loads. During final design, both two and three-dimensional analyses should be performed to evaluate the effects of various construction sequences on liner and soil pillar loads. Methods and requirements for pre-reinforcement of soil columns and pillars may also be needed, particularly where pillar widths are less than 30 feet. The support loading discussions presented above are preliminary and should be used as a general guideline until more detailed evaluations are completed during the final design stage.

5.4.7 Groundwater Flows and Control

A regional groundwater level, as well as numerous perched water levels above lowly permeable layers, has been encountered in borings along most of the project alignment. The deeper the excavation below these groundwater surfaces, the greater the long-term hydraulic pressure head would act on the excavation, regardless of the soil hydraulic conductivity. Excavations that penetrate predominately cohesionless soils, such as outwash, fluvial, or lacustrine deposits [Qvro, Qva, Qpnf, Qpgo, and Qpnl], may encounter inflow rates proportional to the hydraulic head. Groundwater inflows from lowly permeable units, such as Qpgl, Qpgm and others, are generally expected to be relatively low. Seams or lenses of sands within these

units, however, could result in adverse inflow, causing flowing sand and/or unstable subgrade conditions in tunnels or deep excavations. In some areas, till-like soils [Qvd and Qpgd] are extremely heterogeneous and groundwater inflow rates may be difficult to predict.

Significant natural cut-and-fill channels filled with coarse-grained deposits are present beneath Capitol Hill. These channels are cut into soils with a lower hydraulic conductivity. Other filled channels are likely to be detected during future explorations or during construction. When such channels are encountered unexpectedly in a tunnel heading, significant groundwater inflows into an excavation can result. Design and construction of shafts and stations should be accomplished so that they do not provide conduits from wet units to dry units, thus altering piezometric levels, and increasing or decreasing water pressures on support systems.

As excavations for mined structures may require a generally open mined excavation that is supported as the excavation proceeds sequentially, any water bearing cohesionless soil would need to be fully dewatered or otherwise stabilized in advance of tunneling. Since the soil types at the mined structure locations vary from sand to clay, dewatering requirements and methods would vary as well. The methods of controlling groundwater would depend on the soil type within the excavation and the amount of groundwater pressure head at the elevation of the excavation.

Methods of controlling groundwater inflow into excavations include pre-drainage from the ground surface using deep wells or eductor/ejector well points or pre-drainage from inside of the excavation using horizontal wells or eductor/ejector well points placed from the running tunnels, or using ground improvement methods to reduce the amount and impact of groundwater flow toward the excavation. Methods to improve the ground and to reduce groundwater flow toward the excavation include cement or chemical grouting from the ground surface or from within previously excavated openings such as the running tunnels or the mined excavation itself, jet grouting, and ground freezing.

Many of the mined structures may be constructed within units that would likely require only minor measures to control groundwater inflows. These units generally have a low hydraulic conductivity but may contain interbeds and discontinuous lenses of saturated, cohesionless, silts and sands that would likely flow if encountered without dewatering. Therefore, it may be necessary to explore ahead of the primary excavation to identify and control such lenses before they are encountered to prevent uncontrolled flow into the excavation.

These lenses could be drained using horizontal drains, vertical wells, or other groundwater control methods suitable for the characteristics of the granular layer. Thick

sequences of primarily silt could be encountered in the pre-Vashon glaciolacustrine [Qpgl] and nonlacustrine [Qpnl] units. The silt is commonly too fine to pre-drain with typical dewatering methods, except closely spaced eductor wells or vacuum well points, and too fine to be chemically grouted. Where glacial outwash or fluvial deposits are encountered in cross-passages, drainage can be accomplished using horizontal drains, vertical wells, or other suitable groundwater control methods.

Groundwater with high hydrostatic heads may be present at proposed station locations and other cut-and-cover excavations for University Link. Depending on the construction methods used, dewatering or aquifer depressurization may be necessary during excavation and construction of cut-and-cover structures. We recommend aquifer testing be performed for design purposes as part of the next phase of work.

Aquifer testing should include, but not be limited to, pumping tests and slug tests. We recommend that aquifer testing be performed at the Capitol Hill Station, and University of Washington Station and Cross-over. In addition, aquifer testing should be performed at any proposed cut-and-cover locations along the alignment where the water level is within 5 feet of the proposed excavation base.

5.5 Montlake Ventilation Shaft

5.5.1 General

A deep ventilation shaft is proposed between Capitol Hill and University of Washington Stations, just south of SR 520. This shaft would be approximately 110 feet deep and 22 feet in diameter and would be located on the Hop-In Market property, north of E. Roanoke Street, between Montlake Boulevard N.E. and the SR 520 Off-ramp. Based on subsurface conditions encountered in boring NB-251, the shaft would extend through approximately 65 feet of hard silt [Qpnl] and very dense sand and silt [Qvd/Qpnf/Qpgm/Qpgd/Qpgo], 15 feet of very dense sand and gravel [Qpgt], and then into hard, silty clay [Qpgl]. Measured water levels in boring NB-251 indicate that head associated with the Qpgm/Qpgd units is around elevation 45 feet, corresponding to about 95 feet of head above the bottom of the shaft excavation.

5.5.2 Design Considerations

As discussed above, the ventilation shaft would extend through water-bearing, granular soils and saturated cohesive soils. Therefore, shaft construction would require dewatering of the granular soils or ground treatment to limit the influence of groundwater, or watertight shoring to allow construction of the shafts below the groundwater table.

Since the ventilation shaft excavation would be relatively small in areal extent, if required, dewatering the outwash deposits could likely be accomplished by using a few deep wells, provided that the soil conditions are suitable for deep well dewatering. For the Qpnl deposits, jet eductor or vacuum well points may be the most effective method for dewatering these fine cohesionless soils. Even with wells around the excavations, some groundwater inflow would likely occur at contacts between coarse- and fine-grained deposits. In addition, dewatering in an area where there is soft or loose and compressible soil at the ground surface can cause ground settlement that would affect structures and utilities in the area.

In areas where dewatering is deemed uneconomical or technically infeasible given the soil conditions, soil improvement or watertight shoring methods should be used. Soil improvement could consist of grouting or freezing the more granular soil zones to limit groundwater inflows and allow excavation of the shaft. For grouting, permeation chemical or synthetic grout would be the likely choice. Ground freezing has been completed successfully to several hundred feet in Europe and other places in the United States, but not in the Seattle area. Ground freezing was used to depths of 180 feet on the First Avenue utilidor tunnel with mixed results. One shaft was frozen very capably while the other shaft was incompletely frozen and required considerable remedial measures to provide a watertight structure. Watertight shoring methods to the depths required might include slurry or secant pile walls. These shoring systems can be installed for the full depth of the shaft or just through the water-bearing units.

The shaft excavation would likely be circular with a final cast-in-place concrete or shotcrete lining installed after the temporary shoring is installed. If the excavation is dewatered, the temporary shoring could consist of soldier piles and lagging (timber); ring beams and lagging (timber, steel liner plate, or shotcrete); or soil nails and shotcrete. Similar shoring systems have been used to retain excavations to depths of up to 140 feet in soils in Seattle, but not to depths of about 230 feet. Waterproofing can be provided by a membrane installed between the temporary shoring wall and the final cast-in-place concrete liner. As an alternative, watertight shoring, such as slurry or secant pile walls, could be used for both temporary and final support.

Design lateral pressures for shafts would depend on many factors including soil properties, groundwater levels, in situ stress conditions, the sequence of excavation, the type of initial and final support, and drainage provisions. Earth pressures on the completed permanent shaft liners are expected to be equivalent to those on the temporary initial support systems, provided that the surrounding soils do not experience long-term creep. Currently, we do not anticipate that the glacially consolidated sands or clays would creep over time.

The stiffness of the support system and the proximity of the bracing to the advancing excavation bottom of the shaft would have a significant impact on lateral earth pressures. Actual lateral earth pressures would vary depending on whether a flexible or rigid support system is used, how the system is installed, when it is installed relative to the excavation sequence, and the type and degree of internal or external bracing. A rigid support system may not allow sufficient yielding of the soil to mobilize active earth pressure conditions or soil arching around the shafts. For this condition, lateral earth pressures close to at-rest earth pressures with no soil arching would apply. Conversely, a flexible support system would allow greater movement, resulting in decreased earth pressures to an active condition, due to soil arching and the ability of the surrounding soil to partially support itself. Examples of rigid and flexible support systems commonly used in Seattle include:

- **Flexible Support Systems**: In Seattle, soldier piles and lagging (timber); ring beams and fagging (timber, steel liner plate, or-shotcrete); and soil nails and shotcrete have been used to retain excavations to depths of about 140 feet in soils without significant groundwater pressure. Soldier pile alignment and installation could be difficult due to obstructions, such as boulders, as well as variations in soil properties and groundwater. If dewatered, the soils that are anticipated in the shafts generally have sufficient standup time and strength to permit the placement of ring beams and lagging or soil nails and shotcrete in depth intervals of 3 to 5 feet.

  The primary advantage of these support systems is that they allow sufficient movement for soil arching and active earth pressure conditions around the shaft. The disadvantages include dewatering outside the shaft, increased deformations and settlements around the shafts, and the construction of a permanent shaft liner.

- **Rigid Support Systems**: Secant or tangent concrete pile walls and slurry walls have been used in Seattle for support of cuts and shafts. The drilled concrete piles (shafts) normally have good vertical alignment to depths of 120 feet or less, due to the limitations associated with the heavier and larger installation equipment and casings that are required at greater foundation depths, and the general expertise of the local drilled shaft specialty contractors. The stability of open holes may also be a problem in flowing soils and would likely require drilling mud, or casing during installation.

  Slurry walls can be used in most soil and groundwater conditions, and may be installed through obstructions or small boulders. Slurry walls are commonly constructed to depths of 100 feet or less, although, there have been slurry walls constructed to depths of 200 feet or more. The advantages of the slurry or secant pile walls is that no regional dewatering is required outside the excavation, the walls can be used as the permanent shaft liner, and radial deformations and vertical settlements are typically very small.

  As mentioned above, the primary disadvantage of these rigid walls is that the permanent earth pressures are higher.

A large component of the lateral pressure for the design of a support system arises from the surrounding groundwater. The installation of a long-term reliable drainage system would reduce groundwater pressures on the shaft wall. However, there may be concerns about permanently lowering the groundwater levels in the area, the conveyance of groundwater and possible contaminants from one aquifer level to another, and the need to discharge the drainage water. It is our understanding from discussions with PSTC that the shaft liners should be designed for full hydrostatic pressures.

Because the shaft is designed for full hydrostatic pressures, the bottom of the shaft may experience full uplift force. Resistance to uplift force would be provided by the weight of the shaft lining and base and the shear resistance between the lining and the surrounding soils. The magnitude of this shear resistance would depend on construction methods and the continuity between the support system and surrounding soils. Assuming some form of continuity between the liner and soil can be achieved, it is anticipated that the shaft walls would provide between 1,000 and 3,000 psf of adhesion in the glacially consolidated, cohesive soils. Frictional resistance in the granular soils above the clays may be estimated using frictional coefficients ranging from 0.35 to 0.5. These values represent our estimate of ultimate soil strengths; an appropriate factor of safety should be applied. Additional uplift resistance could be obtained with tiedown anchors through the invert slab and/or a bottom seal. It is recommended that the resistance to uplift force exceed the calculated uplift force by 10 percent.

The shear strengths of the glacial units beneath the base of the Montlake Ventilation Shaft sites are, in our opinion, high enough to resist bottom heave or blowout, provided that, before excavation, the saturated granular soils have been dewatered, grouted, or otherwise disconnected from the granular soils and water outside the excavation perimeter. Alternatively, the potential for blowouts may be decreased by installing an impervious shaft wall embedded sufficiently below the bottom of the excavation, similar to a cofferdam, prior to shaft excavation.

Numerical analyses of tunnel/shaft intersections by others have indicated increased lateral pressures near the intersection. Traditionally, earth pressures have been considered to double at an intersection (Peck 1969). This load increase can be assumed to decrease linearly to zero at a distance of one tunnel or shaft diameter from the intersection.

Lateral pressures due to surcharge loads from adjacent buildings, structures, and utilities should also be included in the shaft design. Recommendations for surcharge loads are presented in Figure 44.

In addition to the sources of pressure discussed above (earth, hydrostatic, additional pressures due to intersections, and surcharge loads), lateral pressure increments due to seismic loading should also be applied to the shafts. Lateral earth pressure increments due to seismic loading are not included in the provided earth pressure diagrams.

5.5.3 Methods for Estimating Lateral Earth Pressures

Unfortunately, shafts of the large diameters and depths proposed for this project have never been constructed in Seattle and case histories for large diameter, deep shafts constructed in soils could not be found. Commonly used methods for estimating lateral earth pressures are appropriate for braced excavations with linear walls that are less than 100 feet in depth. However, these methods do not account for soil arching around the excavation that may reduce the lateral earth pressures. Theoretical methods for estimating earth pressures on circular shafts have been developed by several authors including Terzaghi (1943), Berezantzev (1958), and Prater (1976). These methods are not supported by empirical data and are typically limited to cohesionless soils. A brief discussion of the methods, limitations, and conclusions regarding their use in the evaluation of earth pressures acting on large diameters and deep shafts is presented below:

- **Apparent Earth Pressure (AEP):** The AEP or apparent pressure envelope is an empirical method for estimating strut loads in braced temporary excavations. It was never intended to represent the pressure distribution against shoring or permanent walls. However, it is commonly used for this purpose. The method was developed from actual measurements made in excavations using linear walls and internal bracing. The excavations were typically less than 100 feet in depth. This method does not account for soil arching around the excavation and is not appropriate for circular excavations.
- **Rankine (K_a and K_o):** The Rankine theory is commonly used for estimating lateral earth pressure against retaining and shoring walls as well as permanent structural walls. The method estimates lateral pressures as a function of vertical stress multiplied by a lateral coefficient of earth pressure (K). As a result, the pressure increases in direct proportion to depth. The K coefficients range from a maximum at-rest state (K_o) to a minimum active state (Ka). The at-rest state is equivalent to the in situ condition and assumes no movement of the soils. Conversely, the active condition assumes that the soils move sufficiently to fully mobilize their shear strength and reduce lateral pressures. Since movement would occur during excavation, the lateral pressures using at-rest conditions are probably overly conservative. This method, regardless of whether K_o or K_a is used, does not consider the affects of soil arching around the shaft and also is not appropriate for circular excavations.
- **Prater (1976):** The paper by Prater provides a theoretical method for estimating lateral earth pressure on circular shaft liners. This method is similar to those presented by others

on circular excavations (Terzaghi, Berezantzev, etc.). It uses a Coulomb-type analysis with conical sliding surfaces that account for reduced pressures due to soil arching around the shaft. This method results in very low pressures, less than half of those calculated by the AEP or Rankine methods. The method assumes the soils are in an active state for the entire shaft depth and, unfortunately, there are no case histories with measurements to document such low pressures in such a deep shaft in clay. As a result, we are concerned about the validity of this method for design of a deep shaft.

- **FLAC (Fast Lagrangian Analysis of Continua):** FLAC is a finite difference soil-structure interaction program developed by the Itasca Consulting Group. The program was used to model and evaluate soil loading on the shafts, considering several construction and support methods. For this study, two-dimensional FLAC models were developed for a rigid shoring system (secant pile or slurry wall) for the entire shaft. The magnitude of the earth pressure depends on the stiffness of the support system (deformation allowed) and the construction method used. The use of FLAC requires many assumptions and is mathematical with no verification from case histories or field measurements. Although limited to some extent, it appears that this method is currently the best available alternative for reasonably estimating the lateral earth pressures on the shafts.

5.5.4 Preliminary Lateral Earth Pressure Estimates

Similar to the cut-and-cover stations, a rigid shoring system, such as secant or tangent pile walls, or concrete slurry walls, is proposed to be used for the construction of the Montlake Ventilation Shaft. The rigid walls would be constructed from the ground surface and would be incorporated into the permanent structure. A top-down construction sequence is proposed for the construction of the shafts. Since the proposed rigid shoring system is a watertight system, dewatering during construction of the shafts could be accomplished inside the excavation only, after the installation of the shoring system is complete.

The following paragraph presents the sequence of steps performed in our FLAC analyses, regarding assumed soil conditions and applied construction methods.

- Initial, effective stress conditions are assigned to the soil layers, based on the results of our field explorations, pressuremeter test results, and theoretical and empirical data. Table 15 presents the generalized subsurface conditions and FLAC input parameters assumed at the Montlake shaft location.
- A circular, slurry wall or secant pile wall is installed to the final shaft depth. Analyses are performed for different wall thickness. The groundwater level remains static at the currently known depth during construction of the shaft.
- Soil is then excavated in 5-foot lifts. Hydrostatic pressure is incrementally applied to the outside of the shaft, below the known groundwater level, as excavation proceeds.

The recommended lateral earth pressures for design of permanent Montlake Ventilation Shaft are presented in Figure 45. The recommended lateral pressures in this figure include both earth pressures and hydrostatic pressures. Additional pressures due to the intersections with tunnel staging and patron access chambers (as discussed in Section 5.5.2) are not considered in our recommendations. Also, recommended lateral earth pressures presented in Figure 45 do not include surcharge and seismic loading. Such pressures should be applied in accordance with the established design criteria.

5.6 Cut-and-cover Station Excavations

5.6.1 General

We understand that the Capitol Hill Station and University of Washington Station would be constructed using cut-and-cover techniques. General subsurface conditions of proposed cut-and-cover stations and crossover locations are summarized in Table 16. Table 16 also indicates groundwater conditions based on the results of available borings completed near proposed facilities. This section presents preliminary geotechnical engineering considerations for design of cut-and-cover structures.

5.6.2 Excavation Support Methods

The feasibility of excavation support methods for cut-and-cover structures depends on such factors as the nature of the soil and groundwater conditions of the site, the landslide history of the area, the depth and width of the excavation, the proximity and sensitivity of adjacent existing structures and utilities, the compatibility of the support system with the proposed construction, and the general expertise available in the local construction industry. Depending on these considerations, flexible or rigid excavation support systems may be considered. As discussed in previous sections of this report, flexible wall systems result in greater ground movements adjacent to the excavation, resulting in decreased earth pressures, thus they are typically more economical than more rigid walls.

The suitability of the temporary excavation support system should be evaluated with respect to the allowable ground movements considering the existing adjacent structures and utilities. It is recommended that structural condition surveys be performed for all existing structures located within a distance of the excavation equal to about 1.5 to 2 times its depth. Based on the results of these surveys and the foundation support conditions of each structure, movement limits should be developed for each building. These movements should be compared with the predicted excavation-induced ground movements. If smaller allowable ground

movements are required to limit potential distress to the adjacent existing facilities, rigid wall systems could be considered to support the excavation, or it may be necessary to underpin the adjacent existing facilities. In addition, if right-of-way restrictions exist, it may be appropriate to incorporate the temporary excavation support system into the walls of the permanent stations structure to reduce the required clearance envelope.

In the Puget Sound area, flexible support systems typically consist of either continuous, interlocking steel sheet piles; soldier piles with wood or concrete lagging or shotcrete, possibly supported by tieback anchors; or soil nail wall systems. The appropriate flexible wall system generally depends on the nature of the soil conditions and whether or not groundwater is present. Sheet piles provide a relatively watertight wall and are commonly used for loose and/or soft soil conditions. They cannot be driven significant distances into dense, granular materials or hard, overconsolidated, cohesive deposits. Soldier piles and lagging systems are generally used for major excavations above groundwater level and underlain by more competent soil conditions. In Seattle, soldier piles and lagging are typically used for intact, hard clay materials and/or dense, granular soils without adverse groundwater conditions. If groundwater is encountered, however, the soils could be dewatered prior to excavation and placement of the lagging. Soldier pile and lagging systems have also been used as trench support for shallow utility installations in loose sand and soft clay. For these applications, removable steel plates are commonly used as lagging. Soil nail systems are generally used for sites where the subsurface conditions consist of hard, massive cohesive soils or dense granular soils without the presence of adverse groundwater conditions. Such soils also need to be able to stand vertically in limited exposures to enable soil nail installation reinforcement, drainage, and shotcrete placement along the excavated soil face.

All three flexible wall systems have been successfully used in the Puget Sound area. Most of the major excavations completed in Downtown Seattle have been supported by tieback soldier pile and lagging walls. More recently, soil nails and shotcrete have been installed successfully to support a number of excavations in the downtown area, but typically in dense granular soils and hard massive cohesive soils.

Rigid wall systems are typically used for excavations where groundwater drawdown needs to be limited outside the excavation, and in areas where excavation-related ground movements must be minimized. Such conditions could result in lateral design loads that are higher than for a flexible support system. Rigid wall systems could consist of secant or tangent pile walls, or concrete slurry walls. Secant or tangent pile walls using conventional drilled shafts are typically used as rigid walls in the Puget Sound region. A secant pile wall consists of a series of intersecting concrete-filled boreholes. Typically, the concrete of the intersected pile, located

between two reinforced, high-strength concrete piles, is composed of low strength "lean mix" material. A tangent pile wall consists of closely spaced reinforced concrete-filled boreholes that do not intersect.

Secant- or tangent pile walls were used for the Pioneer Square, Westlake, and University Street Stations of the DSTT located beneath Third Avenue and Pine Street. At these station excavations, it was important to control and reduce ground movements. In addition, station right-of-way was limited such that the excavation support walls were incorporated into the permanent structures. Large-diameter cylinder pile walls have also been used for many deep excavations in Seattle; particularly those located in landslide-prone areas, including walls constructed for the I-5 freeway through Downtown Seattle, portions of I-90/Corwin Place, the Mount Baker Ridge Tunnel portals, and the Convention Place Bus Station walls.

Few excavations in the Seattle area have been supported by concrete slurry walls. One example would be Interstate 90 through Mercer Island. It should be noted, however, that conventional slurry walls, when properly installed, perform better than secant or tangent pile walls in soft or loose soils and where groundwater cutoffs are required. In addition, slurry wall construction would allow the release of some of the locked-in pressure in glacially overridden soils. This locked-in pressure resulted from the weight of about 3,000 feet of ice that once covered these soils. Thus, a slurry wall would result in a more significant decrease of lateral earth pressures than a secant or tangent pile wall.

On a unit cost basis, rigid wall systems are more costly than the flexible wall systems. However, the overall excavation support system costs could be more comparable, if adverse groundwater conditions including hazardous or contaminated materials are present such that pumping, treatment, and disposal of groundwater is required for the flexible wall, but not the rigid wall. The rigid concrete walls of the temporary excavation support system are often incorporated into the permanent structure, if possible, so that the soil and groundwater loads are shared by the temporary and permanent walls. This reduces the required quantities and therefore the cost of the conventional cast-in-place concrete walls for the permanent structure.

5.6.3 Bracing of Walls

Bracing for excavation support systems could include internal and/or external members. Internal bracing is typically provided by struts (horizontal braces) or rakers (inclined braces). External bracing is usually provided by tiebacks or soil nails. Depending on the proposed depth of the excavations, the temporary excavation support system would generally require multiple levels of bracing. Typical practice is to use a continuous or discontinuous horizontal waler

generally spaced 10 to 15 feet apart vertically to transfer loads from the ground support wall to the bracing. In cut-and-cover excavations, braces typically extend across the excavation with or without intermediate vertical support depending on the width of the excavation. Because of the relatively deep excavations required for this project and their limited width, the required excavations generally would not be conducive to berm and raker type construction. The use of external members such as tieback anchors or soil nails for support of temporary excavation support walls is normally desirable to allow construction (both excavation and permanent structure construction) to proceed without interference from the internal bracing members. However, right-of-way issues may prevent the use of external bracing.

Most excavations in Downtown Seattle have been supported by tiebacks. Selected DSTT stations, however, were internally braced with horizontal pipe struts. Tieback soil anchor capacities vary depending on soil conditions. Contractors in the Puget Sound region, however, use drill tools and casings that can more readily accommodate tiebacks that consist of 6 to 7 steel strands, which correspond to maximum anchor capacities equal to about 200 kips. If required for the design of the excavation support system, tiebacks installed in very dense granular soils could achieve higher anchor capacities. The length and capacity of the tieback anchors or soil nails may be limited by the available right-of-way required from the adjacent private property owners.

5.6.4 Design Considerations

The walls and bracing for excavation support systems for cut-and-cover structures should be designed for loads due to lateral pressures from soil, groundwater, surcharges, axial loads from wall ends and tiebacks, and construction surcharge loads, as applicable. The lateral earth pressures should be developed based on ground conditions encountered and allowable ground movements adjacent to the excavation.

Based on discussions with PSTC, we understand that a rigid shoring system, such as secant or tangent pile walls or concrete slurry walls, would be used for construction of the cut-and-cover stations. The rigid walls would be incorporated into the permanent structure. A top-down construction sequence is proposed for construction of the cut-and-cover structures. For each of the stations, a temporary ground support system would be installed to excavate approximately 15 feet below the ground surface. From this level, the rigid concrete walls for the station would be installed, and then the station's concrete roof would be constructed. The roof would serve as the first internal brace for the cut-and-cover structure. Excavation would proceed beneath the station roof until the next level of internal bracing is reached. After that brace is in place, the excavation and bracing sequence would continue until the bottom of the excavation is reached and the base of the station is constructed.

We understand that design of the cut-and-cover structures would generally not include the installation of permanent drainage systems. For these conditions, hydrostatic pressures should be included in the lateral design pressures. The hydrostatic pressures should consider both perched and permanent groundwater levels.

Groundwater pressures should be considered for design of the permanent base slabs of the cut-and-cover structures. Resistance to uplift force would be provided by the weight of the cut-and-cover structure and the shear resistance between its walls and the surrounding soils. The magnitude of this shear resistance would depend on the construction methods and continuity between the support system and surrounding soils.

Lateral pressures due to surcharge loads from adjacent buildings, structures, and utilities should be added to the earth and water pressures described above. Recommendations for surcharge loads are presented in Figure 44.

The permanent structures should also be designed to accommodate dynamic lateral earth pressures during a seismic event. For preliminary analyses of the permanent structure walls, we recommend that earth pressures plus a dynamic pressure increment be applied to the structure. The seismic earth pressures would be determined from the deflection of the structure walls during an earthquake. The wall deflection can be estimated from site response analyses. Using a deflection approach should be considered for seismic design in the future phases.

The excavation support system should extend below the bottom of the excavation to provide an adequate factor-of-safety (FS) against toe instability due to lateral pressures below the lowest bracing level. In addition, the wall embedment should also be designed to support the vertical loads acting on the wall, including the vertical components of construction surcharges, and permanent structure dead and live loads (if required).

Preliminary recommendations for design of the excavation support systems required for the project are presented in Section 5.6.6.

5.6.5 Groundwater Control

Based on the groundwater level readings measured in the borings and our experience in the Seattle area, it is our opinion that groundwater would be encountered during the proposed excavations and excavation support installation. Groundwater control requirements should be developed for the Contractor so excavation work can be completed in the dry, and basal stability is maintained during excavation and placement of station structures. Where groundwater is

encountered, and if required, it could be lowered by vertical sand drains, well points, wells, or other appropriate means to the base of all excavations. Dewatering should be accomplished as necessary so that all work can be accomplished in the dry. As an alternative to dewatering, watertight shoring methods, such as slurry or secant pile walls, could be used. These shoring systems can be installed for the full depth of the shaft or just through the water-bearing units. Aquifer depressurization (lowering piezometric head) may be necessary in addition to watertight shoring methods to achieve basal stability.

To satisfy the basal stability of all excavations that are underlain by an aquitard followed by an aquifer, it is recommended that if a site is to be dewatered, the piezometric head in the aquifer be lowered so that the total weight of the more impervious aquitard located above the water-bearing unit (aquifer) exceeds the calculated uplift force due to the water pressure in the underlying aquifer by a minimum of 10 percent. If no dewatering is accomplished, resistance to the uplift force could be provided by the weight of the structure, the shear resistance between its walls and the surrounding soils, and if required, additional uplift resistance could be obtained with tiedown anchors through the invert slab and/or a bottom seal. It is recommended that the resistance to uplift force exceed the calculated uplift force by 10 percent.

The Contractor should be responsible for the control of surface water and groundwater wherever encountered. The Contractor should also be responsible for controlling the effects from construction dewatering and for impacts on adjacent structures due to lowering the groundwater level. Prior to initiation, the method of dewatering selected by the Contractor should be evaluated by a geotechnical engineer or hydrogeologist experienced in groundwater control, as improper dewatering methods could cause difficulties in excavations and disturb the integrity of foundation soils. Additionally, achieving dewatering goals may be difficult when layered soils are encountered. General groundwater control recommendations are presented in the following sections for the proposed station and portal excavations required for the project.

5.6.6 Preliminary Excavation Support Recommendations

This section presents preliminary recommendations for design of the permanent below grade cut-and-cover stations. These preliminary recommendations should be re-evaluated based on the results of future explorations, as well as field and laboratory testing programs.

Table 17 presents the geologic units anticipated at each cut-and-cover site, and our preliminary recommendation of at-rest and active lateral earth pressure coefficients for estimating soil loads, groundwater levels, dynamic pressure increments, and subgrade reaction values for both walls and base slabs. Earth pressures provided for permanent walls assume that

the structure walls are internally braced, thus have uniform pressure distributions. Preliminary recommendations of lateral pressures for design of the permanent cut-and-cover structures are presented in the following sections.

5.6.6.1 Capitol Hill Station

The proposed Capitol Hill Station would be located between Nagle Place and Broadway E. near E. Denny Way (between approximate Stations 1083+71 to 1087+51), as shown in Figure 2 (Sheet 4 of 13). It would be constructed using cut-and-cover methods. The proposed excavations would be approximately 540 feet long by 75 feet wide and would extend to depths varying between 90 and 100 feet, depending on the vertical profile selected for the tunnel. Based on available subsurface information, contamination may potentially be encountered during excavation or dewatering.

Subsurface Conditions. The upper 5 to 10 feet of material in the vicinity of the station has not been glacially overridden but generally consists of dense to very dense granular soils. The soil deposits include Fill (Hf), recessional outwash [Qvro], and till-like deposits [Qvat, Qvri, and Qvd]. These normally consolidated soils are underlain by till and till-like soils that extend to depths of 40 to 60 feet below the ground surface. These soils consist of very dense, silty, gravelly sand to silty, clayey, gravelly, sand [Qvt, Qvd, and Qvgm]. Within this layer of till and till-like soils is a layer of very dense, clean to silty sand that may be as thick as 10 feet.

The till and till-like layer is underlain by a layer of fluvial sand and gravel deposits [Qpnf], consisting of very dense, slightly silty to silty, sandy gravel to slightly silty to silty sand. The layer of Qpnf is underlain by interbedded, finer-grained layers of glaciolacustrine [Qpgl] and nonglacial lacustrine [Qpnl] deposits that vary in thickness between 5 and 40 feet. The glaciolacustrine materials [Qpgl] consist of very stiff to hard, silty clay and clayey silt with a trace of fine sand, while the Qpnl soils generally consist of very dense, silty, fine sand to fine sandy silt and hard, slightly clayey silt with organics.

Based on the results of the explorations and water level measurements collected from the observation wells and piezometers installed in nearby borings, the advance outwash [Qva], fluvial [Qpnf], and lacustrine [Qpnl] deposits appear to be water-bearing and under relatively high, confined pressure. Water level measurements indicate that the piezometric surface of these units may extend upward to within 10 feet of the ground surface and 80 to 90 feet above the station bottom (Figure 7, Sheet 4 of 13). Dewatering and/or aquifer

depressurization would likely be necessary during excavation within water-bearing units due to the presence of high hydrostatic pressure and coarse-grained soils.

Excavation Support Considerations. As discussed earlier, rigid concrete walls (secant or tangent pile walls, or slurry walls) that could be incorporated into the permanent station structure are being considered for use as a temporary excavation support system. Because the station would be designed for full hydrostatic pressures, its bottom may experience full uplift forces due to the piezometric head of the fluvial [Qpnf] material and the lower nonglacial lacustrine [Qpnl] layers. As discussed earlier, resistance to uplift force would be provided by the weight of the station and the shear resistance between its walls and the surrounding soils. For the proposed rigid concrete support system, it is anticipated that the station walls would provide between 1,000 and 3,000 psf of adhesion in the clay. Frictional resistance in the granular soils above the clays may be estimated using frictional coefficients ranging from 0.5 to 0.6. These values are based on our estimate of ultimate soil strengths; an appropriate FS should be applied. Additional uplift resistance could be obtained with installation of tiedown anchors through the invert slab and/or a bottom seal. It is recommended that the resistance to uplift force exceed the calculated uplift force by 10 percent.

Based on the results of subsurface explorations completed in the vicinity of the station site and our experience in similar soils, we developed preliminary lateral earth pressures for the temporary ground support proposed to excavate to the station roof level, and for the rigid concrete walls for the station structure. The recommended preliminary lateral earth pressures for the temporary ground support, which are presented in Figure 46, assume that a flexible shoring system, such as soldier pile and lagging wall supported by internal bracing and/or tiebacks, would be used, and that adequate groundwater control would be provided to lower the groundwater level below the excavation subgrade until the station is constructed and the excavation is backfilled.

Our preliminary recommendations of lateral pressures for design of rigid concrete walls are presented in Figure 47. Recommended values for secant or tangent pile walls and for slurry walls are provided. As discussed in Section 5.6.2, slurry wall construction would allow greater release of some of the locked-in stresses in glacially overridden soils. As a result, lateral earth pressures for a slurry wall system are lower than lateral earth pressures for a secant or tangent pile wall system.

Figures 46 and 47 also present preliminary recommendations for allowable skin friction and end-bearing parameters for design of axial capacities at the Capitol Hill Station; and

Figure 47 presents preliminary recommendations for estimating uplift pressure acting at the base of the station and for design of tiedown anchors. It should be emphasized that the lateral pressure and axial capacity recommendations presented in Figures 46 and 47 are preliminary and based on limited available subsurface information. They should be reviewed upon completion of additional explorations.

5.6.6.2 University of Washington Station and Crossover

The University of Washington Station is located just north of the Montlake Cut between Montlake Boulevard N.E. and the University of Washington football stadium as shown in Figure 2 (Sheets 12 and 13 of 13). A cross-over is located just south of the station. The station and cross-over would be constructed using cut-and-cover methods. The proposed excavations would be approximately 840 feet long and 85 feet wide, and would extend to an approximate depth of 110 feet.

Subsurface Conditions. The soils at the station and cross-over are glacially overridden below a depth of about 5 to 20 feet below the ground surface. The non-overridden soils consist of fill (Hf) and recessional outwash [Qvro] and are generally loose to very dense, silty sand and gravel with some sandy silt. Hf soils may be clayey. A layer of till and till-like deposits [Qvt and Qvd] that is about 10 to 30 feet thick underlies the layer of non-overridden soils. The Qvt and Qvd soils are overconsolidated and consist of very dense, silty, gravelly sand to slightly gravelly, silty sand.

Below the layer of Qvt and Qvd, soil conditions vary considerably north to south. At the location of the cross-over, the soils underlying Qvd and Qvt soils generally consist of hard, silty clay to clayey silt [Qpgl] at depth with several layers of coarse- to fine-grained deposits in between. These intervening layers are very dense and range from fine sandy silt to silt [Qpnl] with some gravel and clay [Qpgm or Qpnl] to slightly silty to silty sand and gravelly sand. To the north, the depth to the hard cohesive soils [Qpgl] increases and the other coarse- and fine-grained soil layers are absent. Between the Qvt/Qvd layer and the deep Qpgl soils is a thick section of slightly silty to silty sand with gravelly layers [Qva]. This soil geometry may be the result of subglacial erosion, similar to the deep swales cut into Qpgl at the north end of Capitol Hill. Although not encountered in our borings, cobbles and boulders could be present within the Qva soils as well as the Qpgt and Qpgm layers.

Much of the stadium excavation is likely to be in Qva soils. The change from predominantly Qpgl soils at depth to Qva soils at depth is interpolated between borings and is not known. Groundwater levels are approximately 35 to 40 feet below the ground surface, at or

slightly above the elevation of the water in the Montlake Cut to the south and Union Bay to the east. Water-bearing Qva, Qpgo, and Qpgl may have high hydrostatic pressure and may therefore need to be depressurized to achieve basal stability during excavation and construction. Should construction dewatering of Qva soils be necessary, large flows may be expected because of high permeabilities and the proximate bodies of water. The layer of Qpgo beneath the cross-over may also yield considerable water. The presence of interlayered soil units, such as those below the cross-over, may decrease the difficulty associated with meeting dewatering goals.

Excavation Design Considerations. As discussed earlier, rigid concrete walls (secant or tangent pile walls, or slurry walls) that could be incorporated into the permanent structure of the proposed facilities are being considered for use as a temporary excavation support system. Because these structures would be designed for full hydrostatic pressures, its bottom may experience full uplift forces due to the piezometric head of the outwash [Qva] material and the lower glaciolacustrine [Qpgl] layers. As discussed earlier, resistance to uplift force would be provided by the weight of the station and the shear resistance between its walls and the surrounding soils. For the proposed rigid concrete support system, it is anticipated that the station walls would provide between 1,000 and 3,000 psf of adhesion in the clay. Frictional resistance in the granular soils above the clays may be estimated using frictional coefficients ranging from 0.5 to 0.6. These values are based on our estimate of ultimate soil strengths; an appropriate FS should be applied. Additional uplift resistance could be obtained with installation of tiedown anchors through the invert slab and/or a bottom seal. It is recommended that the resistance to uplift force exceed the calculated uplift force by 10 percent.

Based on the results of subsurface explorations completed in the vicinity of the station site and our experience in similar soils, we developed preliminary lateral earth pressures for the temporary ground support proposed to excavate to the station roof level, and for the rigid concrete walls for the station structure. The recommended preliminary lateral earth pressures for the temporary ground support, which are presented in Figure 48, assume that a flexible shoring system, such as soldier pile and lagging wall supported by internal bracing and/or tiebacks, would be used, and that adequate groundwater control would be provided to lower the groundwater level below the excavation subgrade until the station is constructed and the excavation is backfilled.

Preliminary recommendations of lateral pressures for design of rigid concrete walls for the station are presented in Figure 49. Figure 49 presents recommendations for a secant or tangent pile wall, and for slurry walls. As discussed in Section 5.6.2, the slurry wall construction would allow greater release of some of the locked-in stresses in glacially overridden

soils. As a result, lateral earth pressures for a slurry wall system are lower than lateral earth pressures for a secant or tangent pile wall system.

Figures 48 and 49 also present preliminary recommendations for allowable skin friction and end-bearing parameters for design of axial capacities at the University of Washington Station, and Figure 49 presents preliminary recommendations for estimating uplift pressure acting at the base of the station and for design of tiedown anchors. It should be emphasized that the lateral pressure and axial capacity recommendations presented in Figures 48 and 49 are preliminary and based on limited available subsurface information. They should be reviewed upon completion of additional explorations.

5.7 Significant Features

5.7.1 I-5 Undercrossing

The proposed vertical tunnel alignment would require tunneling under the I-5 roadway, east of the Stub Tunnel, which would require cutting through the existing heavily reinforced I-5 cylinder pile walls shown in Figure 2 (Sheet 1 of 13) and Figure 7 (Sheet 1 of 13). Further, the horizontal alignment of this section of the tunnel is constrained by critical structures such as the deep foundations for the I-5 NB roadway structures, the Pine Street/Boren Avenue Bridge and the Paramount Theater. As a result of Washington State Department of Transportation (WSDOT) liaison committee meetings, PSTC and Shannon & Wilson performed a feasibility study to assess how the cylinder pile walls would be impacted by tunnel construction. The results of this study were published in a technical memorandum titled “I-5 Undercrossing Alternatives at Convention Place Station,” dated July 29, 1998. This preliminary assessment of the subsurface conditions at the I-5 Undercrossing location was conducted based on a review of published articles and reports, geotechnical information from existing projects, and WSDOT files and construction records.

As shown in Figure 7 (Sheet 1 of 13), the geology in the vicinity of the I-5 Undercrossing consists primarily of recent fill (Hf), landslide (Hs), and peat (Hp) deposits over pre-Vashon glacial deposits. The soil conditions in this area are complicated by past landslides, which may have included debris avalanches and rotational slumps. Prior to the construction of I-5, the western toe of Capitol Hill ran along the eastern edge of the existing Convention Place Station. In the early 1900s, a trough at the base of this hillside that ran northward to Lake Union was filled with random fill during the development of this area. In the 1960s, serious landslides occurred during I-5 construction, especially north of the University Link alignment, and consequently, the heavily reinforced cylinder pile walls were constructed along the upslope side

of the interstate to retain the excavations, and to reduce the potential for reactivation of old landslides and the initiation of new ones. Possibly because of the previous landslide processes, it is difficult to trace individual geologic units from boring to boring because their presence is quite variable over short distances.

The surficial soils west of I-5 are generally random fill materials that are highly variable in depth, composition, and consistency or relative density, and that are underlain by normally consolidated, post-glacial soils and some organic material. Tunneling would primarily be through glacial soils below the fill and the normally consolidated soils. However, low standard penetration resistances and the fractured nature of soil samples indicate that some of these natural soils have been disturbed and modified by landsliding. Based on the construction records of I-5, excavations for the cylinder pile walls in the vicinity (walls W-30 and W-38) encountered primarily glaciolacustrine deposits; however, it is unclear if these fine-grained soils are pre-Vashon or Vashon in age. Because of limited existing subsurface information, the stratigraphy is unclear. Granular glaciomarine drift, outwash, and/or fluvial deposits were encountered at the bases of a few of the cylinder piles for Wall W-38 and at similar elevations in a number of borings completed for adjacent projects. Construction experience at the Washington State Convention and Trade Center suggests that a concentration of boulders may be encountered at the top of the glaciomarine drift unit.

Two construction methods are being considered for the I-5 crossing by the transit tunnels. The first method involves excavation of cut-and-cover pits between the two pairs of cylinder pile walls (Walls W-30 and W-38 to the west, and Walls D and W-36 to the east). A rigid box built between the pairs of walls would strengthen the individual wall structure and increase the overall stability of these cylinder pile walls. Openings through the cylinder pile walls would be framed to receive the tunnels, and knock-out panels would be constructed to allow the passage of the tunneling machine. Note that this would require two cut-and-cover pits for each tunnel.

The second construction method involves doing all of the strengthening and framing underground while tunneling through the cylinder pile walls to minimize the impact on I-5 vehicle traffic. However, during development of this construction method, it was found that it would be necessary to install permanent tiebacks through each cylinder pile wall prior to cutting through the walls to maintain their stability. Although working access from the ground surface would not be required for excavation purposes, it would be required for installing the permanent tiebacks. An open-face shield would be required for this method to allow working access at the face. Also, depending on soil conditions, ground treatment and/or dewatering may be required.

Based on discussions with WSDOT, preliminary engineering has advanced using the first method. The first method appears to involve less construction risk, has less potential impact on the overall construction schedule, and has a lower estimated direct construction cost.

Preliminary stability analyses were performed for the most critical case using the first method, with the cut-and-cover box structure between Wall W-38 (10-foot-diameter piles) and Wall W-30 (5-foot, 6-inch-diameter piles). A two-dimensional computer soil-structure interaction analysis was conducted using FLAC to evaluate approximate displacements of the walls and the surrounding ground. The results indicated a maximum horizontal movement of ⅛ inch at the top of Wall 38 and a maximum settlement of ¼ inch behind the wall. It is believed that the calculated displacements would be less if a more sophisticated three-dimensional analysis, which could take into account soil arching, was used.

For comparison, a FLAC analysis was also conducted for the most critical case using the second construction method, tunneling through Wall 38 after permanent tiebacks are installed from I-5. Estimated maximum horizontal displacements at the top of the wall and maximum settlements behind the wall were greater than for the first construction method. Therefore, the preliminary analyses indicate that the first method provides greater rigidity and less movement than the second method.

5.7.2 Montlake Cut Undercrossing

University Link crosses, as a running tunnel, beneath the Montlake Cut approximately 200 feet east of the Montlake drawbridge; see Figure 2 (Sheet 12 of 13). As indicated in Figure 7 (Sheet 12 of 13), the crown of the tunnel would be at an approximate elevation ranging from –45 feet at the south end of the cut to –35 feet at the north end (NAVD 88). Based on available information, the bottom of the Montlake Cut is at approximate elevation of –15 feet. Therefore, beneath the cut, the soil cover above the tunnel crown would range from 20 to 30 feet. Also, based on available information, the current alignment would run beneath the sheetpiles located along the north side of the Montlake Cut. We understand that these sheetpiles were installed for the construction of the cut. Considering the impact of insufficient soil cover on tunnel construction verifying the elevation of the bottom of the Montlake Cut is critical, in our opinion.

In the Montlake cut along University Link, glacially overconsolidated deposits were encountered at or close to the ground surface. These deposits consist of a sequence of till and till-like deposits [Qvt, Qvd, Qpgm, and Qpgt] with some outwash sand and gravel layers [Qpgo]. These deposits are underlain by glaciolacustrine silt and clay deposits [Qpgl] that exist at an elevation of about –15 feet.

As discussed in Section 4.4.2, the heads in the Qpgl and Qpnl units increase with distance south and north of Portage Bay. This trend also suggests that groundwater in the unit is discharging from these units to Portage Bay. Heads are between elevation 20 and 40 feet in these units and are about 60 to 80 feet above tunnel invert near the Montlake Cut. Therefore, heads in the soils near and directly beneath the cut should be expected to be above the water level elevation of the cut. The magnitude of head above the cut cannot be estimated.

It should be noted that utility tunnels cross the Montlake Cut in the vicinity of the project alignment, as described in the following section, Section 5.10.3. Impacts of tunneling on these utility structures should be evaluated.

5.7.3 Existing Utility Tunnels

A 54-inch-diameter water main crosses the Montlake Cut approximately 130 feet east of the Montlake Bridge, and then runs along the east side of Montlake Boulevard N.E. Beneath the cut, the water main run through a 152-inch outside diameter utility tunnel with an invert elevation of approximately –56 feet. The 48-inch-diameter North Trunk Sewer crosses the cut along the west side of the bridge, approximately 225 feet west of the centerline of NB track.

A 138-inch-diameter sewer tunnel lies beneath the western portion of Montlake Boulevard N.E., south of the IMA building and beneath the Burke Gilman trial north of the IMA building. The tunnel was constructed in 1908 and 1909 using both cut-and-cover and hand-mined tunneling methods. Numerous voids, 2 to 18 feet high, were encountered above the tunnel in borings advanced in 1972. The top of the tunnel lies at a depth of 25 to 30 feet below the existing ground surface. In addition, there are other utility tunnels, as indicated on the profile provide by PSTC, which cross the alignment on the UW campus. The impact of the Link tunnels on settlement of the existing utilities should be evaluated during subsequent stages of design.

5.8 Ground Movements

5.8.1 General

The ground movements (surface settlements) presented in this report are the result of a two-dimensional analysis utilizing largely empirical methods. The ground movements represent a starting point for discussions within the design team and with the owner to better quantify the risks and uncertainties associated with tunneling and underground construction. The values provided are not estimations and are not intended to be predictive. The intended purpose is to identify areas where ground improvement, prescriptive construction methods, instrumentation, and/or minor adjustments to the alignment or layout of structures may be required. The values

provided are intended to be conservative without being pessimistic. They are intended to represent a preliminary assessment of credible ground movements without the benefit of having defined the following:

- Possible limitations on means and methods in the plans and specifications
- Possible prescriptive requirements in the plans and specifications
- Final layouts and excavation limits
- Layout and types of instrumentation
- The experience of the contractor and the experience of the crew

Areas of potential concern identified during the preliminary design phase should be reevaluated as the design of the structures progresses and as the plans and specifications are developed because of the interdependence between means, methods, workmanship, plans, specifications, and ground movements.

In general the analysis was performed perpendicular to the long axis of the structure analyzed. For example, the contours provided for the running tunnel are based on the results of two-dimensional analysis performed at 100-foot intervals along the alignment. At the cut-and-cover stations and the ventilation shaft where three-dimensional geometrical considerations were expected to have an impact on ground movements, additional work was performed and the ground movements were assumed to be additive.

The results of our analysis are summarized as contours in Figure 63. The contours represent possible surface expressions of ground movements and are not necessarily representative of movements at the elevation of foundation elements for structures or utilities adjacent to the alignment. It is also important to note that the ground movements from the analysis may be slightly larger, by up to almost 0.2 inches, than indicated on Figure 63 because of the selected contour interval.

5.8.2 Methodology

The ground movements provided in this report are for discussion purposes only. They do not expressly take into account the following factor that may influence ground movements:

- Vertical and horizontal curves
- Ground disturbance at the entry and exit to cut and cover structures
- 3D end effects for mined cross passages
- Geologic constraints such as a glacial till cap
- Abrupt geologic contacts
- Existing structures, which may limit the zone of influence

- Ground improvement such as compaction grouting
- Excavations for cutting the I-5 cylinder piles

Some of the above items are partially taken into account by the empirical manner in which the formulas were derived. Empirical relationships are based on historic records for projects where these factors may have impacted the observed ground movements.

5.8.2.1 Running Tunnels

Ground movements result from the inevitable elastic response as well as over-excavation or loosening of the soils, insufficient support of soils, and groundwater drawdown. Some loosening and movement of the soil around an excavated tunnel would occur even if the contractor performs all needed ground improvements and all of the tunneling equipment and support systems are performing correctly. Ground movement is always an issue in an urban environment such as along University Link. For much of the alignment, settlement trough widths are expected to be about one and one-half to two times the tunnel depths, resulting in relatively wide, shallow settlements with very small differential movements. The method used to generate the ground movements are based on Wang et al., 2000. Where a 10- to 50-foot thickness of very hard or very dense glacially consolidated soil is located above the tunnel crown, and/or tunnel depths are in excess 3 to 5 tunnel diameters (60 to 100 feet), surface settlements are likely to be less than shown in Figure 63 and may even be negligible with EPB or SLURRY TBMs and a well-grouted, single-pass lining.

The following assumptions were made for the running tunnels:

- Two-dimensional analysis is appropriate.
- The selection of appropriate tunneling means and methods and reasonable standard of care during construction.
- End effects are negligible or accounted for in analysis of other structures.
- Ground losses equal to 1.0 percent of the excavated volume for most of the alignment.
- Ground loss equal to 1.5 percent of the excavated volume for the portion of the alignment under I-5.
- Draw angle of 40 degrees for both clay and sand.
- Sand or clay soil model selected based on predominant soil type at each analysis point (every 100 feet).
- No additional ground losses due to abrupt changes in geology.
- No additional ground losses due to vertical or horizontal curves.

Typically settlements for most of the running tunnels are less than 1 inch. Magnitudes up to 1 inch were computed north and south of the Capitol Hill Station and next to St. Demetrios Church near the Boyer Ave. E. ground depression, also because of reduced tunnel depths in these areas.

Slightly larger than average surface settlements of 1.2 inches were computed for the I-5 undercrossing, because of the very shallow depth of cover, the relatively tight curve in the tunnel, and previous ground disturbance and relaxation of the soil during construction of I-5.

5.8.2.2 Cut-and-cover Structures

Figures 64A and 64B present summaries of the maximum lateral wall movements and settlements that occurred adjacent to several excavations accomplished in dense soils that were supported by both flexible and rigid wall systems. These figures are based on the work of Clough and O'Rourke's (1990) along with data from six excavation projects completed in Seattle to provide insight into the performance of excavations in glacial soils. Not all of these projects are represented in Figure 64B because it could not be confirmed that the published settlements were actually the maximum settlements that occurred adjacent to the excavations.

In general, with the exception of the excavation for the Seattle First National Bank project, the observed ground movements for these excavations are less than the average movements noted on the figures. It was reported that the greater movements observed at the Seattle First National Bank excavation occurred partially because of inadequate vertical bearing capacity of the soldier piles (Shannon and Strazer, 1970). It should be noted that this project was one of the first projects in the Seattle area where tiebacks were used to support a deep excavation. The references for these six projects are provided in Figure 64.

Based on these case histories and our experience, it is anticipated that for planning purposes, the maximum lateral wall movement would be about 0.15 percent of the excavation depth for relatively stiff walls in very dense granular material. For estimating purposes, it can also be assumed that the ground settlement immediately behind the wall would be approximately equal to the lateral wall movements. For preliminary design, the settlement distribution away from the wall was estimated using the empirical relationship presented in Figure 64C for cut-and-cover structures. The empirical relationship is based on data from three Downtown Seattle projects. In developing the relationship, less weight was given to the points associated with the Seattle First National Bank where inadequate vertical bearing capacity of the soldier piles was suspected to have caused additional ground movements.

A maximum of 2.2 inches of settlement was calculated at Capitol Hill Station. The other 3 stations show up to 2 inches of settlement.

5.8.2.3 Montlake Ventilation Shaft

The same basic method of calculating ground movements for the cut-and-cover excavations was used for the ventilation shaft. The only significant difference is calculating the settlement distribution away from the wall of the shafts. The empirical relationship used for the shafts is presented in Figure 64C and is based on settlement measurements at the exploratory test shaft recently completed for the Central Link Beacon Hill Station. The data from the test shaft indicates that the distribution of settlement contours is much tighter behind the wall for circular shafts than for long walls. It is important to note that data from the test shaft is preliminary, and the maximum settlement may not have been recorded. The ventilation shaft at E. Roanoke St. shows over 3.6 inches of calculated settlement. This settlement accounts for the relatively large diameter adits that will be constructed to connect the ventilation shaft to the running tunnels. The analysis of the adits associated with the ventilation shaft does account for the 3D end effects.

5.8.3 Sensitive Structures and Facilities

The following presents a preliminary list, in alphabetical order, of sensitive structures and facilities along the alignment that will require additional analysis during subsequent design phases to assess the need for additional geotechnical evaluation, potential structure support, or the potential need for mitigation of damage caused by ground movements. Depending on the final alignment and specified construction means and methods, it is possible that no additional analysis or design will be required for these structures and facilities.

- Lincoln Reservoir
- Montlake Boulevard Bridge
- SCL Transmission Lines Under Denny Way
- SPU Water Storage Tower at Volunteer Park
- SPU Water Tunnel at Lake Washington Ship Canal
- University of Washington (UW) Husky Stadium

The magnitude of settlement for each of these structures or utilities was not specifically computed here, but the general magnitude in their vicinity can be estimated from the surface settlement contours, shown in Figure 63.

5.9 Ground Modification

An array of ground improvement techniques have been developed and used in the tunneling industry over the last 30 years to:

- Improve the soil strength
- Reduce permeability
- Compensate for tunnel ground loss to reduce settlement

To accomplish these goals a variety of pro-active and reactive ground improvement techniques have been developed that include various forms of dewatering, ground freezing and grouting. All of these ground improvement techniques have been used successfully on tunnel projects within the last 20 years in the Seattle area.

Dewatering – Dewatering may be either proactive or reactive. However, most deep well systems are used as proactive systems because of the time needed to install the wells and operate the pumping system to adequately improve the ground prior to tunneling or excavation. Predominant dewatering systems include:

- Deep dewatering wells spaced 10 to over 100 feet apart in clean sands and gravels
- Eductor-ejector wells spaced 5 to 20 feet apart in silty sands to clean sands
- Vacuum wells spaced 2 to 20 feet apart in a wide range of granular soils

The deep wells and eductor-ejector wells are typically installed from ground surface, several weeks to a month or more ahead of construction to allow the pumping to be effective in lowering the groundwater table. The vacuum wells are often installed as the excavation advances and, for design purposes, are generally limited to an assumed vertical lift of about 15 feet between the vacuum pump and the well tip.

Multiple perched water levels, silty soils, and complex geology would complicate the dewatering process. It is also generally impracticable to completely dewater a granular layer. Even with closely spaced wells, some groundwater would typically remain mounded on top of an impermeable layer between wells. However, dewatering methods are often successful in reducing hydrostatic pressures in permeable soils. Groundwater control and soil loss may be more manageable when combined with other ground modification methods, as experienced at the Beacon Hill Station.

Freezing – Ground freezing is generally considered to be a pro-active ground improvement technique due to month or more that it takes to install the closely spaced pipes and thoroughly freeze and adequate thickness of ground. Freeze pipe have been installed at spacing of 2 to 5

feet around shafts, open cuts, cross passages and other structures to strengthen the ground and decrease permeability and groundwater flows from granular soils. Freezing can be adversely affected by the presence of salt water or brackish water and significant groundwater gradients or flow velocity. At depths of greater than about 80 to 100 feet, maintaining hole alignment and spacing can also be difficult and holes spaced too widely would not adequately freeze the ground.

Grouting – A wide variety of grouting systems have been developed and utilized on Seattle area projects over the last 30 years on projects such as the Seattle Bus Tunnels, Henderson CSO, Denny CSO and current Beacon Hill Project. Grouting techniques include:

- Proactive
  - Jet Grouting
  - Soil Mixing
  - Permeation Grouting
  - Fracture Grouting
- Reactive
  - Compaction Grouting

Both jet grouting and soil mixing involve the replacement of soils with a cementitious material to form a mass nearly concrete-like material. In jet grouting the grout is sprayed under high pressure through jet nozzles attached to a drill string that is lowered and raised through the ground. The diameter to the jet grouted soil would depend on soil conditions, jet pressure and rate of withdrawal of the pipe string. Jet grout pressures are not high enough to replace glacial till, medium stiff to hard clay and silts, and peats. Jet grouting does work well and would form 3- to 7-foot-diameter columns in granular soils such as cohesionless silts, fine to coarse sand, and gravel. A normal maximum depth for jet grouting is about 100 feet; however, the Beacon Hill Project has installed a jet grouted mat at a depth of 150 feet, along one of the proposed station tubes.

Soil mixing involves the augering in of cementitious material and mixing it with native soils. Specialized auger rigs with one, two, or three adjacent augers have been used to depths of up to 100 feet for improve a wide range of soils to a near concrete-like consistency.

Permeation and fracture grouting are techniques that involve the injection of chemical mixtures or finely-ground cementitious material. These techniques either fill existing voids or new fractures created by the introduction of the grout under high pressure. Injection pressures must overcome the hydrostatic head, but are otherwise limited only by the pump capacity.

Compaction grouting involves the injection of very stiff mortar-like grout under relatively high pressures into previously drilled holes in the soil. The objective of compaction grouting is to displace and compact soils in situ, rather than mix or fill interparticle voids. As the volume of the grout bulb increases, the soil is densified through compaction. Compaction grouting is typically used as a remedial or mitigation measure for specific, more local, settlement-related or densification projects. Vibro-compaction and vibro-replacement techniques are generally utilized for mitigation of more global or widespread areas.

5.10 Soil Chemistry and Gas Considerations

This section discusses tunneling and other construction considerations related to soil chemistry and gases. Considerations include soil pH, soil corrosion potential, and the presence of gases that are potentially hazardous for tunneling

5.10.1 Soil pH and Muck Disposal

The acidity or alkalinity of soils, noted as pH, could impact the setup of cementitious materials and could also impact disposal costs of excavated spoils. From Shannon & Wilson's current Beacon Hill Station work, we understand that the Washington State Department of Ecology currently restricts landfill disposal of uncontaminated soil and rock spoils to materials with a pH less than 8.5. Consequently, soils with a pH greater than 8.5 can only be disposed at landfills that have been licensed to accept contaminated materials or at landfills that do not allow surface runoff or infiltration of water with a pH greater than 8.5.

No pH testing was performed during the current University Link work Soil. During previous studies for the LB235 alignment, corrosion parameter testing included tests of pH, electrical resistivity, chlorides, sulfates, and sulfides. Testing of pH and other corrosion parameters were performed on nearly 350 samples along the LPA. The values of pH ranged from 6.1 to 9.7 with a mean value of approximately 8.2. Refer to the LB235 GCR (Shannon & Wilson, 1999a), Section 7.6, for a discussion of the results of pH and other corrosion parameter testing.

5.10.2 Methane and Hydrogen Sulfide

A gas-monitoring program was conducted during previous design phases of the project to evaluate whether methane or hydrogen sulfide gases, which are potential hazards for tunneling, were present in soils along the LPA. No gas monitoring was performed during the current University Link work other than PID measurements of soil samples during drilling.

5.10.2.1 Methane

Methane is generated underground by bacterial decomposition of organic matter in anoxic environments, and by thermal decomposition of organic matter during coalification and petroleum generation. Methane gas is recognized as a potential hazard for tunneling in the Seattle area. Nearly 40 years ago, significant quantities of methane were encountered during construction of the Lake City sewer tunnel from Portage Bay to Matthews Beach on Lake Washington (Metropolitan Engineers, 1963). It was first detected in borings in the University District where it was "touched off by an arc welder" (methane is combustible in air at concentrations of between 5 and 15 percent). In the early 1990s, tunneling for the West Point Combined Sewer Overflow (CSO) project at Fort Lawton was interrupted periodically because of excessive accumulations of methane (Fulcher, 1993). During the pre-construction exploration program at Fort Lawton, a cutting torch accidentally ignited gas issuing from a piezometer. The flame continued until it was extinguished. The gas contained 70 percent methane by volume. Minor amounts of methane were also reported in borings completed along the alignment of West Seattle Tunnel in 1996, and tunneling was interrupted once during its construction (Oatman et al., 1997). There is no documentation that methane was encountered during the completion of either the Mt. Baker Ridge or Downtown Seattle Transit tunnels. However, since both of these projects involved open-face tunnel boring machines and had robust ventilation systems, any small quantities of methane that may have been encountered may have passed without notice.

Methane is expected to be present intermittently along University Link and may occur in significant concentrations at some locations. It has been detected in headspace samples collected from borings and observation wells completed for previous design phases of the project. Methane was detected in 25 of 254 soil samples that were tested from borings drilled during previous design in the entire north corridor (Table F-2 of the GDR), and in 20 of 233 samples in the vicinity of University Link. The concentrations of methane in the soil headspace samples ranged from below detection to 16.2 percent of the gas present. With one exception, the soil samples where methane was detected were collected from organic glaciolacustrine, lacustrine, or fluvial geologic units that are known to contain organic material and are likely sources for the methane. Samples from these units are also generally fine-grained and, thus, are more likely to have retained methane during sample collection and handling. Methane was not detected in the more granular units; however, it is expected that highly porous soils would present the greatest risk to tunneling. The porous units are more likely to accumulate significant amounts of gas when overlain by fine-grained soil units and, because of their higher

permeability, more likely to release larger volumes of gas when encountered in the tunnel excavation.

Methane was detected in 15 of the 112 soil samples collected within Pre-Vashon glaciolacustrine deposits [Qpgl] in the north corridor. Lenses and layers of hard organics, which may be a source of methane, have been encountered in the Qpgl. Methane was also detected in 7 of 97 soil samples obtained in a non-glacial lacustrine deposit [Qpnl], and 2 of 3 soil samples collected in a non-glacial paleosol deposit [Qpns]. Both the Qpnl and Qpns typically contain organics and are expected to also generate methane. One sample in which methane was detected was collected from the Pre-Vashon glaciomarine drift deposit [Qpgm]. The source of methane in the Qpgm is unknown, and the unit is generally free of organics.

Methane was detected in the headspace of 14 of the 49 observation wells where measurements were taken in the north corridor and in 14 of the observation wells in the vicinity of University Link (Table F-3 of the GDR). The concentrations of methane in the wellhead samples ranged from 0.1 to 27.3 percent of the gas present. The results of borehole monitoring are presented in Table F-1 of the GDR.

Given the available data, it is expected that methane may be encountered during tunneling activities, especially from the beginning of the alignment to the proposed Capitol Hill Station. Tunneling equipment should be equipped with methane sensors, alarms, and automatic shutoffs, and construction electrical systems should be designed to operate in potentially gassy environments.

5.10.2.2 Hydrogen Sulfide

A limited gas-monitoring program was previously conducted to evaluate whether hydrogen sulfide, a potential hazard for tunneling, is present along the proposed alignment. Hydrogen sulfide is generated underground by bacterial decomposition of organic matter in anoxic environments and by thermal decomposition of organic matter during petroleum generation. Hydrogen sulfide gas has not previously been identified as a significant potential hazard for tunneling in the Seattle area. Hydrogen sulfide was detected in 14 of the 45 observation wells in previous project borings in the north corridor and in 12 of the observation wells in proximity to the University Link alignment. The concentrations of hydrogen sulfide measured in headspace samples collected at the wellhead ranged from 1 to 6 parts per million (ppm). Monitoring for this hazardous gas should be made during any subsequent borings.

Hydrogen sulfide was not detected in headspace samples obtained from boreholes during drilling (Table F-1 of the GDR); however, it is expected that drilling mud may have inhibited the gas release in the boreholes.

5.10.2.3 Montlake Landfill

The low, flat-lying area east of Montlake Boulevard is the original location of Union Bay. Much of Union Bay was a marsh and has been filled for expansion of the University of Washington athletic facilities. Much of the fill consists of garbage, as portions of the area served as a municipal landfill beginning in 1926. Thick deposits of soft peat and clay and loose sand underlie the garbage and fill. These deposits thicken to the east as the underlying glacially overridden deposits drop in elevation to the east. The former marsh and landfill are closest to the alignment just north of the University of Washington Station, as indicated in Figure 2 (Sheet 13 of 13).

Decomposition of garbage in the landfill is known to generate a significant amount of methane. Peat may also be generating methane. Methane may exist at low levels in the soils along Montlake Boulevard. No methane was detected in most of the shallow monitoring wells installed in 2003 along the west side of Montlake Boulevard; however, low levels of methane were detected, and landfill debris was encountered in one boring west of Montlake Boulevard N.E., directly east of the cyclotron building.

Testing for lateral migration of methane generated from this landfill has not been completed as part of this study. In our opinion, the probability of lateral migration toward the proposed alignment is low because the relative density or consistency of the soil is very dense or hard, thus creating a barrier to gas migration.

5.11 Instrumentation

5.11.1 General

A geotechnical instrumentation program is strongly recommended to assist in the monitoring, documentation, and control the quality of construction of the University Link system. The primary objectives of the instrumentation program are to:

- Indicate whether or not the tunneling procedures used are maintaining surface and subsurface settlements within acceptable limits.
- Provide early warning of adverse trends.

- Provide the Engineer and Contractor with sufficient data to determine the source of excessive lost ground and to plan remedial measures.
- Determine when ground modifications (underpinning, grouting, etc.) need to be implemented to protect structures.
- Monitor the degree to which these protective or remedial measures are limiting damage to structures and to provide early warning when alternative means of protection are necessary.
- Provide data for settling legal disputes either between the Contractor and the Owner or with owners of adjacent structures.
- Evaluate the performance and structural integrity of the tunnel and station lining systems.
- Monitor the performance of temporary construction structures.
- Confirm design assumptions and provide data that could improve future designs.

Selected instrumentation should be installed prior to construction and used to measure groundwater levels, deformations, and loads. Groundwater measurements should include piezometric elevations, pressures, and flows around and within the tunnels and station excavations. Various forms of deformations should be monitored including horizontal and vertical movements of support systems (tunnels, mined stations, cut-and-cover stations, etc.), soils adjacent to the excavations, and adjacent structures and utilities. The measurement of loading may include lateral loads in deep excavation support systems (bracing, tiebacks, etc.), hoop stresses in temporary and permanent liners (shaft, tunnel, and mined station), and soil contact pressures between the soils and structures.

Due to the significant depths of some portions of the alignment, very little data is available from other projects with comparable depths. Since the design process is typically based on some empirical data, this lack of comparable case histories hinders the design process. Carefully instrumented shaft, station, and tunnel supports could therefore provide a wealth of deformation and load data that can be applied to more economic support systems on future portions of the project.

5.11.2 Preconstruction Survey

Prior to the beginning of instrumentation or construction, an extensive inspection survey of all buildings, structures, and utilities along the entire project alignment should be undertaken. The survey should document the existing condition of each facility with diagrams, sketches, photographs, and video recordings. These records should include, but not be limited to, length and width of existing cracks, number of cracks, locations of water marks, condition of door and

window jams, condition of paint, etc. The surveys should be conducted in the presence of representatives of the building owner, Contractor, Engineer, and Owner. A formal report of every facility should be developed and signed by each member of the group.

5.11.3 Instrumentation Systems

Instrumentation systems should be developed to monitor the response of the ground and adjacent structures and utilities to the construction of the University Link facilities. Construction of these facilities would require deep excavations and support systems, tunneling, SEM, and deep foundations. Further details of instrumentation systems are presented in the LPA report.

5.11.4 Monitoring Frequency

Monitoring frequency would vary widely for each of the instrument systems and for each category of construction. Gages should be installed and a minimum of four readings, at least one week apart, should be obtained prior to the start of construction to provide a stable baseline. Monitoring of some instruments, such as deep settlement gages, may be required every few hours as tunnels are excavated past the instrument sections. Instruments installed around shafts or cut-and-cover structures may require monitoring on a daily to weekly basis depending on the rates of advance. Generally, a reading should be taken for each 5 to 10 feet of depth increase.

5.11.5 Data Reduction and Reporting

All data should be collected and expeditiously reduced and presented in useful, legible and well-labeled plots. In general, the plots should include construction information on depths or stationing of the advancing excavation. Plots might also include geotechnical data, including soil layers and groundwater levels, or other features which may impact the interpretation of the data.

Since the collected and reduced data may be critical to assessing the successful undertaking of the project, the data must be made available within a few hours to the Contractor and owner’s representative for their use. Due to the large quantities of data that would be collected and reduced on a daily basis, it should be summarized in a brief memorandum. This

memorandum should document the readings taken and any salient and important readings that may lead to significant adjustments in construction procedures.

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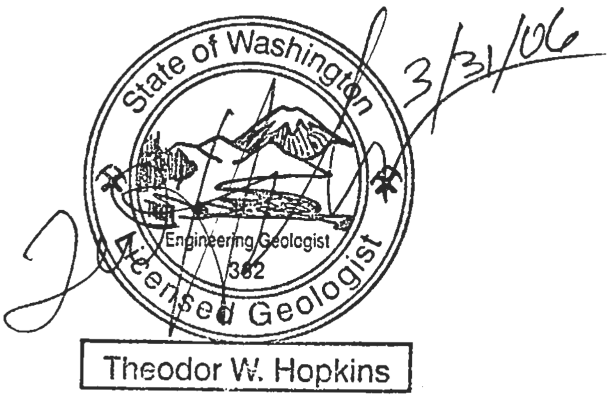

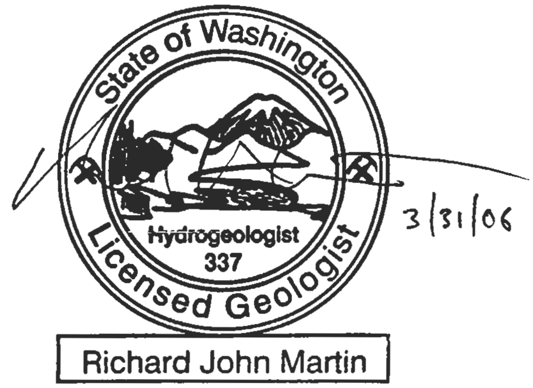

Theodor W. Hopkins, L.E.G.
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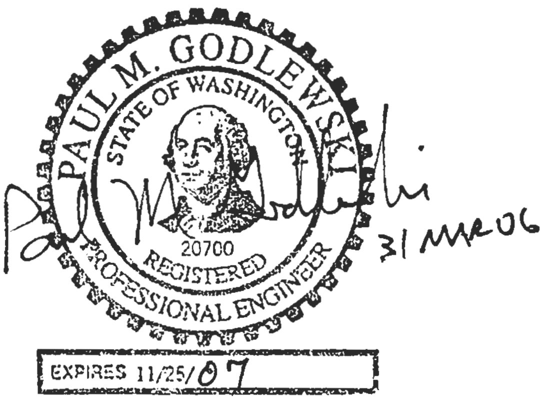

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Hydrogeologic items related to hydrogeology, hydrogeologic setting, hydrostratigraphy and groundwater control were prepared by or prepared under the direct supervision of Richard J. Martin, L.H.G.,

Geological items related to geology, geologic unit designations and descriptions, and soil descriptions and grouping were prepared by or prepared under the direct supervision of Theodor W. Hopkins, L.E.G.

Geotechnical items related to soil engineering properties, ground characterization, ground conditions, tunneling considerations, and shoring were prepared by or prepared under the direct supervision of Paul M. Godlewski, P.E.

Geotechnical items related to soil engineering properties, ground characterization, ground conditions, tunneling considerations, and shoring were prepared by or prepared under the direct supervision of Paul M. Godlewski, P.E.

REFERENCES

Adams, J., 1990, Paleoseismicity of the Cascadia Subduction Zone: evidence from turbidities off the Oregon-Washington Adams margin: Tectonics, v. 9, no. 4, p. 569-583.

Adams, J., 1996, Great earthquakes recorded by turbidities off the Oregon-Washington coast, *in* Rogers, A.M., Walsh, T.J., Kikkoman, W.J., and Priest, G.R., eds., Assessing Earthquake Hazards and Reducing Risk in the Pacific Northwest, U.S. Geological Survey Professional Paper 1560, p. 147-158.

Algermissen, S.T., Perkins, D.M., Thenhaus, P.C., and others, 1982, Probabilistic estimates of maximum acceleration and velocity in rock in the contiguous United States: U.S. Geological Survey Open-File Report 82-1033.

Algermissen, S.T., Perkins, D.M., Thenhaus, P.C., and others, 1990, Probabilistic earthquake acceleration and velocity maps for the United States and Puerto Rico: U.S. Geological Survey Miscellaneous Field Studies Map MF-2120.

American Association of State Highway and Transportation Officials (AASHTO), 2002, Standard specifications for highway bridges (16^{th} ed.): Washington, D.C., American Association of State Highway and Transportation Officials, 677 p.

American Society for Testing and Materials (ASTM), 2000, Annual book of ASTM standards: Soil and Rock, Building Stone, Geosynthetics: Philadelphia, Penn., V.04.08.

Atwater, B.F., 1987, Evidence for great Holocene earthquakes along the outer coast of Washington State: Science, v. 236, p. 942-944.

Atwater, B.F., and Moore, A.L., 1992, A tsunami about 1,000 years ago in Puget Sound, Washington: Science, v. 258, p. 1614-1617.

Bartlett, S.F., and Youd, T.L., 1995, Empirical prediction of liquefaction-induced lateral spread: Journal of Geotechnical Engineering, v. 121, no. 4, p. 316-329.

Berezantzev, V.G., 1958, Earth pressure on cylindrical retaining walls, *Proceedings, Brussels Conf. On Earth Pressure Problems*, II, p.21-27.

Blakely, R.J.; Wells, R.E.; Weaver, C.S.; and Johnson, S.Y., January 2002. “Location, structure, and seismicity of the Seattle fault zone, Washington: Evidence from aeromagnetic anomalies, geologic mapping, and seismic-reflection data,” *GSA Bulletin*, Volume 114, No. 1, pp. 169-177.

Borst, A.J., Conley, T.L., Russell, D.P., and others, 1990, Subway design and construction for the Downtown Seattle Transit Project, *in* Lambe, P.C., and Hansen, L.A., eds., Design and

performance of earth retaining structures, Ithaca, N.Y., 1990, Proceedings: New York, American Society of Civil Engineers, Geotechnical Special Publication no. 25, p. 510-524.

Brocher, T.M., Blakely, R.J., and Wells, R.E., 2004, Interpretation of the Seattle uplift, Washington, as a passive-roof duplex: Bulletin of the Seismological Society of America, v. 94, no. 4, p. 1379-1401.

Brocher, T.M.; Parsons, T.; Blakely, R.J.; Christensen, N.I.; Fisher, M.A.; Wells, R.E.; and the SHIPS Working Group, 2001. "Upper Crustal structure in Puget Lowland, Washington: Results from the 1998 Seismic Hazards Investigation in Puget Sound," *Journal of Geophysical Research*, vol. 106, no. B7, pp. 13,541-13,564, July 10,2001.

Broms, B.B., and Bennermark, H., 1967, Stability of clay at vertical openings: Journal ASCE, v. 3 p. 71-94.

Brown, D.A., and Shie, C.F., 1991, Modification of P-Y curves to account for group effects on laterally loaded piles, *in* McLean, F.G., Campbell, D.A., and Harris, D.W., eds., Geotechnical Engineering Congress 1991, Boulder, Colo., Proceedings: New York, American Society of Civil Engineers, Special Publication No. 27, v. 1, p. 479-490.

Bucknam, R.C., Hemphill-Haley, E., and Leopold, E.B., 1992, Abrupt uplift within the past 1,700 years of southern Puget Sound, Washington: Science, v. 258, p. 1611-1613.

Central Artery/Tunnel, Design Policy Memorandum (DPM) No. 60, Foundation Type Selection Evaluation, Rev. 1, June, 1995, Bechtel/Parsons Brinckerhoff, Boston, MA.

Clarke, S.H., Jr., and Carver, G.A., 1992, Late Holocene tectonics and paleoseismicity, southern Cascadia Subduction Zone: Science, v. 255, p. 188-192.

Clough, G.W., and O'Rourke, T.D., 1990, Construction induced movements of in situ walls, *in* Lambe, P.C., and Hansen, L.A., eds., Design and performance of earth retaining structures, Ithaca, N.Y., 1990, Proceedings: New York, American Society of Civil Engineers, Special Publication No. 25, p. 439-470.

Clough, G. W., Weber, P. R., and Lamont, J., 1972, Design and observation of a tied-back wall, *in* Performance of earth and earth-supported structures, Lafayette, Ind., 1972, Proceedings: New York, American Society of Civil Engineers, v. 1, pt. 2, p. 1367-1389.

Coffman, J.L., and von Hake, C.A., 1973, Earthquake history of the United States (revised ed.): U.S. National Oceanic and Atmospheric Administrative Publication 41-1.

Darienzo, M., and Peterson, C., 1990, Investigation of coastal neotectonics and paleoseismicity of the southern Cascadia margin as recorded in coastal marsh systems," *in* Jacobson, M.L.

ed., National Earthquake Hazards Reduction Program, Summaries of Technical Reports, Volume XXXI, U.S. Geological Survey Open File Report 90-680, p. 131-139.

Darienzo, M.E., and Peterson, C.D., 1995, Magnitude and frequency of subduction-zone earthquakes at four estuaries in Northern Oregon: Journal of Coastal Research, v. 10, p. 850-876.

Degussa, 2005, literature on web site @ www.degussa-ugc.com

Douglas, P.M. et al., 1985, Chambers Creek Sewer Tunnel, *Rapid Excavation and Tunneling Conference Proceedings*, Vol. 2, p. 582-610.

EFNARC, 2001, Specification and Guidelines for the use of Specialist Products for Soft Ground Tunneling, Association House, November.

Ensoft, Inc., 1998, Design of deep foundations: piles and drilled shafts under lateral and axial loadings, a seminar/workshop featuring computer programs from Ensoft, Inc., Austin, Texas, April 23-24, 1998: Austin, Tex.

Frankel, Arthur, and others, 1996, National seismic-hazard maps, June 1996: documentation: U.S. Geological Survey Open File Report 96-532.

Fulcher, Brian, 1993, The Fort Lawton parallel tunnel, Seattle, Washington, in Bowerman, L.D., and Monsees, J.E., eds., Rapid Excavation and Tunneling Conference, Boston, Mass., 1993, Proceedings: Littleton, Colo., Society for Mining, Metallurgy, and Exploration, Inc., p. 183-201.

Gower, H.D., Yount, J.C., and Crossos, R.S., 1985, Seismotectonic map of the Puget Sound region, Washington: U.S. Geological Survey Miscellaneous Investigations Series Map I-1613, scale 1:250,000.

Grant, W.C., 1989, More evidence from tidal-marsh stratigraphy for multiple late Holocene subduction earthquakes along the northern Oregon Coast: Geological Society of America Abstracts with Programs, v. 21, no. 4, p. 86.

Grant, W.P., Perkins, W.J., and Youd, T.L., 1992, Evaluation of liquefaction potential, Seattle, Washington: U.S. Geological Survey, Open File Report 91-441-T.

Grant, W. P., Yamane, G., and Miller, R. P., 1984, Design and performance of Columbia Center shoring wall, Seattle, Washington, *in* Ramaswamy, S. D., and Tam, C. T., eds., International Conference on Tall Buildings, Singapore, 1984, Proceedings: Singapore, The Institution of Engineers, p. 651-661.

Herrenknecht, M., 1994, EPB or slurry machine: The choice: Tunnels and Tunnelling, pp. 35-36, June.

Heuer, R.E., 1974, Important ground parameters in soft ground tunneling: Proceedings of Specialty Conference on Subsurface Exploration for Underground Excavation and Heavy Construction.

Holtz, R.D., Christopher, B.R., and Berg, R.R., 1995, Geosynthetic design and construction guidelines, participant notebook, NHI course no. 13213: McLean, Va., Federal Highway Administration Report no. FHWA HI-95-038, 412 p. [Available from National Technical Information Services, Springfield, VA 22161 as NTIS Report PB95-270500.]

Hyndman, R.D., and Wang, K., 1993, Thermal constraints on the zone of major thrust earthquake failure: the Cascadia Subduction Zone: Journal of Geophysical Research, v. 98, p. 2039-2060.

International Code Council, 1998, International building code, 2000, final draft: Birmingham, Ala., International Code Council.

International Conference of Building Officials, 1997, 1997 uniform building code: Whittier, Calif., International Conference of Building Officials, 3 v.

Jancsecz, S., Krause, R., and Langmaack, L., 1999, Advantages of soil conditioning in shield tunneling: Experiences of LRTS Ismir *in* Proceedings, ITA World Tunnel Congress, Challenges for the 21st Century, Oslo, vol. 2, pp. 865-875, A.A. Balkema, Rotterdam.

Jacoby, G.C., Williams, P.L., and Buckley, B.M., 1992, Tree ring correlation between prehistoric landslides and abrupt tectonic events in Seattle, Washington: Science, v. 258, p. 1621-1623.

Johnson, S.Y., Dadisman, S.V., Childs, J.R., and others, 1999, Active tectonics of the Seattle fault and central Puget Sound, Washington - implications for earthquake hazards: Geological Society of America Bulletin, v. 111, no. 7, p. 1042-1053.

Johnson, S.Y., Potter, C.J., and Armentrout, J.M., 1994, Origin and evolution of the Seattle basin and Seattle fault: Geology, v. 22, p. 71-74.

Johnson, S.Y., Potter, C.J., Armentrout, J.M., and others, 1996, The southern Whidbey Island fault, an active structure in the Puget Lowland, Washington: Geological Society of America Bulletin, v. 108, p. 334-354.

Karlin, R.E., and Abella, S.E.B., 1992, Paleoearthquakes in the Puget Sound region recorded in sediments from Lake Washington, USA: Science, v. 258, p. 1617-1620.

Kirmani, M. and Highfill, S.C., (1996), Design & Construction of the Circular Cofferdam for Ventilation Building No. 6 at the Ted Williams Tunnel, Civil *Engineering Practice, 11, pp.* 31-50.

Langmaack, L., 2003, Europe and Asia: Application of new TBM conditioning additives: http://www.degussa-ugc.com/NR/rdonlyres/1DE9EA0E-B0D1-45D5-8F2F-DAC6898CFB9D/17167/TechPaperBauma_engl.PDF

Langmaack, L., 2002, Soil conditioning for TBM – Chances and limits *in* Proceedings, Conference on Underground Works: Living Structures, Toulouse, AFTES, Paris.

Lisowski, M., 1993, Geodetic measurements of strain accumulation in the Puget Sound Basin, *in* Large earthquakes and active faults in the Puget Sound Region, a conference sponsored by the Quaternary Research Center and U.S. Geological Survey, Seattle, Wash., 1993, Program Notes: Seattle, Wash., 8 p.

McVay, Michael, Casper, Robert, and Shang, T.I., 1995, Lateral response of three-row groups in loose to dense sands at 3D and 5D spacing: Journal of geotechnical engineering, v. 121, no. 5, 436-441.

Ma, L., Crosson, R.S., and Ludwin, R.S., 1996, Western Washington focal mechanisms and their relationship to regional tectonic stress: U.S. Geological Professional Paper 1560, p. 257-284.

Mabey, M. A., and Youd, T. L., 1991, Liquefaction hazard mapping for the Seattle, Washington urban region using LSI: Denver, Colo., U.S. Geological Survey Technical Report CEG-91-01.

Maidl, B., Herrenknecht, M., and Anheuser, L., 1996, Mechanized shield tunneling: Berlin, Ernst and Sohn.

Makdisi, F.I., and Seed, H.B., 1978, Simplified procedure for estimating dam and embankment earthquake-induced deformation, Journal of the Geotechnical Engineering Division, v. 104, no. GT7, July.

Metropolitan Engineers, 1963, Final report, subsurface investigation, proposed Lake City Tunnel, Seattle, Washington: Report by Metropolitan Engineers, Seattle, Wash., Frank J. Kerschner, Chief Engineer, Seattle, Wash., Oct. 11, 19 p.

Meyers, R.A., Smith, D.G., Jol, H.M., and Peterson, C.D., 1996, Evidence for eight great earthquake-subsidence events detected with ground penetrating radar, Willapa Barrier, Washington: Geology, v. 24, p. 99-102.

Mitchell, C. L. B., and Nykamp, M. A., 1998, Shoring analysis, design and construction at the Seattle Symphony's Benaroya Hall, *in* Prakesh, S., ed., Fourth International Conference on Case Histories in Geotechnical Engineering, St. Louis, Mo., 1998, Proceedings: Rolla, Mo., University of Missouri-Rolla, Paper 5.08.

Nelson, A.R., Shennan, I., and Long, A.J., 1996, Identifying coseismic subsidence in tidal-wetland stratigraphic sequences at the Cascadia Subduction Zone of western North America: Journal of Geophysical Research, v. 101, p. 6115-6135.

Nelson, A.R, Johnson, S.Y., Kelsey, H.M., Sherrod, B.L., Wells, R.E., Bradley, L.-A., Okumura, K., and Bogar, R., 2003a, Late Holocene earthquakes on the Waterman Point reverse fault, another ALSM-discovered fault scarp in the Seattle fault zone, Puget Lowland, Washington: Geological Society of America Abstracts with Program, (Abstract for talk in topical session on "Earthquake geology in reverse faulting terrains" at the Nov 03 Annual Meeting of The Geological Society of America in Seattle Washington).

Nelson, A.R., Johnson, S.Y., Kelsey, H.M., Wells, R.E., Sherrod, B.L., Pezzopane, S.K., Bradley, L.-A., and Koehler, R.D., III, 2003b, Late Holocene earthquakes on the Toe Jam Hill fault, Seattle fault zone, Bainbridge Island, Washington: Geological Society of America Bulletin, v. 115, 1388-1403.

Oatman, Martin, and Lenahan, Lawrence, 1997, West Seattle tunnel, *in* Rapid Excavation and Tunneling Conference, Las Vegas, Nev., 1997, Proceedings: Littleton, Colo., Society for Mining, Metallurgy, and Exploration, Inc., p. 749-763.

Peck, R.B., 1969, Deep excavation and tunneling in soft ground, state-of-the-art volume: 7th International Conference on Soil Mechanics & Foundations, Mexico City, p. 225-290.

Peck, R.B., Deere, D.U., Monsees, J.E., Parker, H.W., and Schmidt, B, 1969, Some design considerations in the selection of underground support systems: Report prepared by University of Illinois Department of Civil Engineering, Urbana, Ill., Contract no. 3-0152, for Office of High Speed Ground Transportation and Urban Mass Transportation Administration, Washington, D.C., November.

Prater, E.G., 1976, An examination of some theories on earth pressure shaft linings, Canadian Geotechnical Journal, 14, 91, p. 91-106.

Pratt, T.L., Johnson, S.Y., Potter, C.J., and others, 1997, Seismic-reflection images beneath Puget Sound, western Washington state: Journal of Geophysical Research , v. 102, p. 469-490.

Reese, L.C. and O'Neill, M.W., 1988, Drilled shafts: construction procedures and design methods: McLean, Va., Federal Highway Administration, Publication no. FHWA-HI-88-042, Publication no. ADSC-TL-4, 564 p.

Reese, L.C., and Wang, S.T., 2002, Technical manual of documentation of computer program LPILE^{PLUS} 4.0 for Windows, a program for the analysis of piles and drilled shafts under lateral loads: Austin, Texas, ENSOFT, Inc.

Richards, D., 2005, Memorandum, North Link tunnels – estimated bentonite consumption rate for slurry shield TBMs: By Don Richards for Puget Sound Transit Consultants, Seattle, WA, March 10.

Riddihough, R.P., 1984, Recent movements of the Juan de Fuca Plate System: Journal of Geophysical Research, v. 89, no. B8, p. 6980-6994.

Robertson, P., 1997, Direct use of cone penetration testing: the primary investigation tool, *in* Seismic Short Course on Evaluation and Mitigation of Earthquake Induced Liquefaction Hazards, 3rd: San Francisco, Calif.

Robinson, R.A., Kucker, M.S., Feldman, A.I., and Parker, H.W., 1987, Ground and liner behavior during construction of the Mt. Baker Ridge Tunnel, in Jacobs, J.M., and Hendricks, R.S., eds., Rapid Excavation and Tunneling Conference, New Orleans, La., 1987, Proceedings: Littleton, Colo., Society for Mining, Metallurgy, and Exploration, Inc., v. 1, p. 309-327.

Robinson, R.A., Kucker, M.S., and Parker, H.W., 1991, Ground behavior in glacial soil for the Seattle transit tunnels, *in* Wightman, W. D., and McCarry, D. C., eds., Rapid Excavation and Tunneling Conference, Seattle, Wash., 1991, Proceedings: Littleton, Colo., Society for Mining, Metallurgy, and Exploration, Inc. (SME), p. 93-117.

Satake, K., Shimazaki, K., Tsuji, Y., and others, 1996, Time and size of a giant earthquake in Cascadia inferred from Japanese tsunami records of January 1700: Nature, v. 379, no. 6562, p. 246-249.

Schuster, R.L., Laiger, R.L., and Pringle, P.T., 1992, Prehistoric rock avalanches in the Olympic Mountains, Washington: Science, v. 258, p. 1620-1621.

Seed, R.B., and Harder, L.F., 1990, SPT-based analysis of cyclic pore pressure generation and undrained residual strength, *in* Duncan, J.M., ed., H. Bolton Seed Memorial Symposium, Proceedings: Vancouver, British Columbia, BiTech Publishers, Ltd., v. 2, p. 351-376.

Shannon & Wilson, Inc., 1998, Geotechnical memorandum, 1st Avenue South water main seismic upgrade, earthquake-induced ground displacements: Report: By Shannon & Wilson, Inc., Seattle, WA, Job Number W-8311-01, for Summit Technology, Seattle, WA, 1 v., August.

Shannon & Wilson, Inc., 1998, Draft Geotechnical Engineering Considerations Report for Staff-Recommended Locally Preferred Alternative Only: Report: By Shannon & Wilson, Inc., Seattle, WA, Job Number W-8101-74, for Central Puget Sound Regional Authority, Seattle, WA, December.

Shannon & Wilson, Inc., 1999a, Geotechnical Characterization Report, Sound Transit Light Link Rail Central Line, Contract LB 235: By Shannon & Wilson, Inc., Seattle, WA, Job Number W-8110-70, for Central Puget Sound Regional Authority, Seattle, WA, December.

Shannon & Wilson, Inc., 1999b, Geotechnical Engineering Recommendations Report, Sound Transit, Light Link Rail Central Line: By Shannon & Wilson, Inc., Seattle, WA, Job Number W-8101-70, for Puget Sound Transit Consultants, Seattle, WA, December.

Shannon & Wilson, Inc., 2000, Tender Geotechnical Baseline Report, Sound Transit Link Light Rail, Central Line, Contract LB-235: By Shannon & Wilson, Inc., Seattle, WA, Job Number W-8110-90, for Puget Sound Transit Consultants, Seattle, WA, February.

Shannon & Wilson, Inc., 2001, Memorandum, Montlake Alignment, Sound Transit Link Light Rail, Central Line: By Shannon & Wilson, Inc., Seattle, WA, Job Number 21-1-08110-650, for Puget Sound Transit Consultants, Seattle, WA, December.

Shannon & Wilson, Inc., 2002a, Conceptual Engineering: Geotechnical Engineering Considerations Report, North Alternative Alignments, Convention Place Station to N.E. 45th St. Station, Sound Transit Link Light Rail, Central Line: By Shannon & Wilson, Inc., Seattle, WA, Job Number 21-1-08108-070, for Puget Sound Transit Consultants, Seattle, WA, July.

Shannon & Wilson, Inc., 2002b, Geotechnical Engineering Considerations Report, North Alternative Alignments, Sound Transit Link Light Rail, Central Line: By Shannon & Wilson, Inc., Seattle, WA, Job Number 21-1-08108-170, for Puget Sound Transit Consultants, Seattle, WA, August.

Shannon & Wilson, Inc., 2003, Geotechnical Report, C520 Pine Street Stub Tunnel, Sound Transit Link Light Rail, Central Line: By Shannon & Wilson, Inc., Seattle, WA, Job Number 21-1-08110-870, for Puget Sound Transit Consultants, Seattle, WA, July.

Shannon & Wilson, Inc., 2004a, Geotechnical Engineering Considerations Report, Sound Transit Link Light Rail, Conceptual Engineering, Modified Montlake Alignment: By Shannon & Wilson, Inc., Seattle, WA, Job Number 21-1-08108-175, for Puget Sound Transit Consultants, Seattle, WA, March.

Shannon & Wilson, Inc., 2004b, Preliminary Engineering: Previous Geotechnical Exploration Data Summary Report, Revision 2, Sound Transit Link Light Rail, Central Line: By Shannon & Wilson, Inc., Seattle, WA, Job Number W-8101-60, for Puget Sound Transit Consultants, Seattle, WA, April.

Shannon & Wilson, Inc., 2006, Geotechnical Data Report, Sound Transit North Link Light Rail, Preliminary Engineering: By Shannon & Wilson, Inc., Seattle, WA, Job Number 21-1-08109-070, for Puget Sound Transit Consultants, Seattle, WA, February.

Shannon, W.L., and Strazer, R.J., 1970, Design and performance of tie back system for Seattle-First National Bank Building: Civil Engineering, v. 40, no. 3, p. 62-64.

Shennan, I., Long, A.J., Rutherford, M.M., Green, F.M., Innes, J.B., Lloyd, J.M., Zong, Y., and Walker, K.J., 1996, Tidal marsh stratigraphy, sea-level change and large earthquakes, I: a 5000 year record in Washington, USA: Quaternary Science Reviews, v. 15, p. 1023-1059.

Sound Transit, Regional Transit Authority, 2001, Link Design Criteria Manual, Supplemental Criteria for Seismic Design of the Link Light Rail Transit System, Seattle, WA, Revision 1, December.

ten Brink U.S., P.C. Molzer, M.A. Fisher, R.J. Blakely, R.C. Bucknam, T. Parsons, R.S. Crosson and K.C. Creager, 2002, Subsurface geometry and evolution of the Seattle fault zone and the Seattle basin, Washington: Bulletin of the Seismological Society of America, v. 92, p. 1737-1753.

Terzaghi, K., 1943, Theoretical Soil Mechanics, J. Wiley & Sons, New York, p. 202-215.

Wang, Z.W., Sampaco, K.L, Fischer, G.R., Kucker, M.S., Godlewski, P.M., and Robinson, R.A., 2000, Modeling for predicting surface settlements due to soft ground tunneling, North American Tunneling '00, Ozdemir, L. (ed.), Balkema, Rotterdam, p. 645-652.

Washington State Department of Transportation (WSDOT), 1998, Bridge design manual: Olympia, Washington, Washington State Department of Transportation report M23-50, 2 v.

Washington State Department of Transportation (WSDOT) and American Public Works Association, 2002, Standard specifications for road, bridge, and municipal construction: English Units, M 41-10: Olympia, Wash.

Weichert, D.H., and Hyndman, R.D., 1983, A comparison of the rate of seismic activity and several estimates of deformation in the Puget Sound area, *in* Proceedings of Workshop XIV, Earthquake Hazards of the Puget Sound Region: U.S. Geological Survey Open-File Report 83-19, p. 105-130.

Wells, R.E., Weaver, C.S., and Blakely, R.J., 1998, Fore-arc migration in Cascadia and its neotectonic significance: Geology, vol. 26, p. 759-762.

Wells, R.E., and Johnson, S.Y., 2001, Deformations of Western Washington resulting from northward migration of the Cascadia Forearc: Seismological Research Letters, v. 72, no. 2, March/April.

Winter, D. G., 1990, Pacific First Center, performance of the tieback shoring wall, *in* Lambe, P. C., and Hansen, L. A., eds., Design and performance of earth retaining structures, Ithaca,

N.Y., 1990, Proceedings: New York, American Society of Civil Engineers, Geotechnical Special Publication no. 25, p. 764-777.

Xanthakos, P.P. (1994), Slurry *Walls* as *Structural Systems,* McGraw-Hill, p. 45.

Yount, C. J., Dembroff, G. R., and Barats, G. M., 1985, Map showing depth to bedrock in the Seattle 30' x 60' quadrangle, Washington: U.S. Geological Survey Miscellaneous Field Studies Map MF-1692, scale 1:100,000.

Yount, J. C., and Gower, H. D., 1991, Bedrock geologic map of the Seattle 30' x 60' quadrangle, Washington: U.S. Geological Survey Open-File Report 91-147

TABLE 1
SUMMARY OF PENETRATION RESISTANCE BY GEOLOGIC UNIT[1]

Interpreted Geologic Unit[2]	BLOW COUNT (blows per foot)									
	Standard Penetration Test (N)[3]						3" O.D Split Spoon Sampler/ Dam			
	Count	Minimum	Maximum[5]	Range (excluding outliers)[6]	Average[7]	Standard Deviation[8]	Count	Minimum	Maximum[5]	Ran (exclu outlie
Hf	59	2	200	2 - 86	21	18	2	16	22	16 -
Hls	10	13	94	13 - 28	19	3	-	-	-	-
Hl	2	8	12	8 - 12	-	-	-	-	-	-
Ha	7	5	31	5 - 31	16	7	-	-	-	-
Hp	2	21	24	21 - 24	-	-	-	-	-	-
Qvro	101	6	136	6 - 119	48	23	2	41	82	41 -
Qvrl	12	6	61	6 - 61	32	14	-	-	-	-
Qvri	9	4	100	18 - 100	65	22	-	-	-	-
Qvat	43	8	300	8 - 184	62	39	1	31	31	-
Qvt	191	37	1200+	37 - 378	161	72	24	77	450	77 -
Qvd	128	38	600	38 - 358	143	64	6	200	600	310 -
Qva	230	7	660	7 - 242	113	45	29	9	300	9 -
Qvgl	4	32	190	32 - 139	38	-	1	47	47	-
Qvgm	176	50	1200+	50 - 418	182	75	21	100	1200+	100 -
Qpnf	470	31	600	31 - 308	141	64	112	32	768	32 -
Qpnl	455	29	707	29 - 225	119	42	43	14	1200+	14 -
Qpnp	8	49	300	49 - 211	113	33	2	108	114	108 -
Qpns	30	30	480	30 - 197	97	34	2	36	43	36 -
Qpgo	273	23	1200+	23 - 436	181	93	67	27	1200+	65 -
Qpgl	607	15	600	15 - 151	64	29	26	14	300	14 -
Qpgt	20	100	800	100 - 570	211	83	-	-	-	-
Qpgd	25	52	900	52 - 465	224	109	16	150	1200+	-
Qpgm	142	31	1200+	31 - 384	116	65	14	120	600	120 -

NOTES:

(1) This table summarizes the blow count results from the applicable borings, drilled as part of the CE, PE and current phases.

(2) Descriptions of the geologic units are presented in Section 4.2.

(3) For the Standard Penetration Test (SPT) a 2" outside-diameter (O.D.) split spoon sampler is driven using a 140 lb. hammer freely falling 30 inches.

(4) A 300 lb. hammer was used to drive the sampler.

(5) Partial blow counts (less than 12 inches of penetration achieved) were extrapolated to obtain a penetration value in blows per foot (e.g., 50/6" = 100 or 50/1" = 600 bpf) for the purpose of statistical evaluation only. Extrapolated values were limited to 1,200 bpf.

(6) An outlier is an observation which appears to differ in characteristics from the bulk of the data set. The interquartile range is the difference between Outliers are defined (for our purposes) as being more than 1.5 times the interquartile range either above the 75th percentile or below the 25th percen distribution, 1 out of 143 observations would be outliers.

(7) The mean (arithmatic average) was calculated using the extrapolated penetration values (in blows per foot) as described in Note 5. Outlier values w

(8) Standard deviation not calculated for units with counts less than 5. Outlier values were not included to determine this value.

TABLE 2

SUMMARY OF WATER CONTENT, ATTERBERG LIMIT, STICKY LIMIT, AND ACTIVITY BY GEOLOGIC UNIT(1)

Interpreted Geologic Unit(2)	Water Content (%)					Atterberg Limits(3,4) Liquid Limit, LL (%)					Plastic Limit, PL (%)					Plasticity Index, PI (%)					Sticky Limit (			
	Count	Minimum	Maximum	Average	Standard Deviation	Count	Minimum	Maximum	Average	Standard Deviation	Count	Minimum	Maximum	Average	Standard Deviation	Count	Minimum	Maximum	Average	Standard Deviation	Count	Minimum	Maximum	Average
Hf	69	0	41	15	7	2	23	27	25	-	2	14	15	15	-	2	7	13	10	-	-	-	-	-
Hls	25	11	43	22	7	4	23	32	27	-	4	16	19	17	-	4	6	13	10	-	-	-	-	-
Hl	2	22	33	27	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Ha	7	17	71	32	20	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Hp	8	16	199	61	62	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qvro	109	4	25	14	5	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qvrl	12	18	41	30	7	4	30	54	44	-	4	21	27	24	-	4	9	27	20	-	-	-	-	-
Qvri	11	9	24	15	4	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qvat	45	6	28	13	4	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qvt	199	1	46	11	4	2	15	17	16	-	2	12	13	13	-	2	3	4	4	-	-	-	-	-
Qvd	126	3	23	12	3	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qva	251	3	25	13	5	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qvgl	5	23	35	29	5	1	50	50	50	-	1	29	29	29	-	1	22	22	22	-	-	-	-	-
Qvgm	218	5	28	13	4	14	15	32	22	4	14	13	19	15	2	14	1	15	7	4	-	-	-	-
Qpnf	616	3	47	17	6	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qpnl	818	8	68	26	5	48	24	86	37	12	48	20	32	25	3	48	1	59	11	11	-	-	-	-
Qpnp	9	15	126	53	38	2	34	55	45	-	2	28	43	36	-	2	6	12	9	-	-	-	-	-
Qpns	52	10	38	22	6	7	29	66	42	14	7	14	33	23	6	7	7	43	19	13	-	-	-	-
Qpgo	382	2	53	15	5	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qpgl	1050	8	62	32	7	176	30	110	63	18	173	19	48	27	5	176	5	77	36	15	3	43	72	5
Qpgt	20	6	21	11	4	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qpgd	36	6	23	13	4	2	20	22	21	-	2	14	18	16	-	2	3	5	4	-	-	-	-	-
Qpgm	200	5	46	18	8	19	26	83	43	19	19	12	35	20	6	19	10	48	23	14	-	-	-	-

NOTES:

(1) This table summarizes laboratory test results from the applicable borings, drilled as part of the CE, PE and current phases, along or in the vicinity of

(2) Descriptions of geologic units are presented in Section 4.2.

(3) Atterberg Limit test results are also presented graphically on Figures 25 through 35 by geologic unit.

(4) Atterberg Limit tests performed on non-plastic samples were not included in this summary.

(5) Activity is defined as the percent passing the number 200 seive divided by plasticity index.

TABLE 3
SUMMARY OF UNIT WEIGHT AND SPECIFIC GRAVITY BY GEOLOGIC UNIT(1)

Interpreted Geologic Unit(2)	Dry Density (pcf)					Wet Density (pcf)					Specific Gravity				
	Count	Minimum	Maximum	Average	Standard Deviation(3)	Count	Minimum	Maximum	Average	Standard Deviation	Count	Minimum	Maximum	Average	Standard Deviation
Hf	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Hls	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Hl	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Ha	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Hp	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qvro	1	114	114	114	-	1	134	134	134	-	1	2.7	2.7	2.7	-
Qvrl	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qvri	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qvat	-	-	-	-	-	-	-	-	-	-	2	2.7	2.7	2.7	-
Qvt	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qvd	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qva	1	83	83	83	-	1	92	92	92	-	-	-	-	-	-
Qvgl	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qvgm	5	114	142	125	14	5	134	157	142	10	1	2.7	2.7	2.7	-
Qpnf	16	94	115	104	7	16	119	137	128	5	2	2.8	2.8	2.8	-
Qpnl	130	88	125	100	6	130	114	157	127	6	4	2.7	2.7	2.7	-
Qpnp	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qpns	5	101	114	106	6	5	126	133	129	3	-	-	-	-	-
Qpgo	19	97	119	107	7	19	111	144	126	8	1	2.7	2.7	2.7	-
Qpgl	205	63	118	91	9	205	91	139	120	6	2	2.8	2.8	2.8	-
Qpgt	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qpgd	1	125	125	125	-	1	142	142	142	-	-	-	-	-	-
Qpgm	6	88	124	101	14	6	117	147	129	12	2	2.7	2.8	2.7	-

NOTES:

(1) This table summarizes laboratory test results from the applicable borings, drilled as part of the CE, PE and current phases, along or in the vicinity of the University Link route.

(2) Descriptions of geologic units are presented in Section 4.2

(3) Standard Deviation not calculated for units with counts less than 5.

TABLE 4
SUMMARY OF GRAIN SIZE DISTRIBUTION BY GEOLOGIC UNIT[1]

Interpreted Geologic Unit[2]	GRAIN SIZE DISTRIBUTION[3] % Gravel[4]					% Sand[5]					% Fines[6]					Percent		
	Count[8]	Minimum	Maximum	Average	Standard Deviation	Count	Minimum	Maximum	Average	Standard Deviation	Count	Minimum	Maximum	Average	Standard Deviation	Count[9]	Minimum	Maximum
Hf	6	3	27	14	10	6	40	63	50	9	6	28	53	36	10	1	8	8
Hls	5	0	5	3	2	5	21	39	30	7	5	59	76	67	6	5	14	2
Hl	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Ha	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Hp	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qvro	15	0	38	5	10	15	54	96	82	12	15	4	22	12	5	1	6	6
Qvrl	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qvri	2	8	9	8	-	2	48	63	56	-	2	29	43	36	-	-	-	-
Qvat	3	15	22	19	-	3	51	60	55	-	3	17	34	26	-	-	-	-
Qvt	22	5	22	11	5	22	46	71	58	7	22	23	47	31	6	3	3	4
Qvd	20	1	45	16	11	20	34	93	60	13	20	4	56	24	11	1	5	5
Qva	43	0	77	18	21	43	20	94	72	18	43	2	29	10	6	-	-	-
Qvgl	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qvgm	36	0	31	10	8	36	19	94	48	13	36	6	78	42	15	10	3	1
Qpnf	124	0	67	11	18	124	28	98	80	17	124	2	42	9	6	2	1	1
Qpnl	85	0	6	0	1	85	0	92	24	26	85	8	100	75	26	47	1	4
Qpnp	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qpns	9	0	25	14	9	9	0	52	30	18	9	24	100	57	22	6	5	2
Qpgo	56	0	61	5	12	56	30	98	81	16	56	2	64	14	12	3	2	4
Qpgl	9	0	0	0	0	9	0	26	3	8	19	74	100	97	6	14	15	7
Qpgt	4	4	14	7	-	4	40	58	50	-	4	38	56	43	-	1	4	4
Qpgd	6	0	9	4	4	6	31	76	59	17	6	15	67	37	18	1	6	6
Qpgm	25	2	31	11	6	25	12	72	42	16	29	20	83	49	18	19	4	2

NOTES:

(1) This table summarizes laboratory test results from the applicable borings, drilled as part of the CE, PE and current phases, along and in the vicinity of the Ur

(2) Descriptions of geologic units are presented in Section 4.2

(3) Grain Size distribution results are also shown graphically on Figures 8 through 24 by geologic unit.

(4) Percent Gravel is equal to the percent of material (by weight) with grain sizes between 51mm and 4.7mm.

(5) Percent Sand is equal to the percent of material (by weight) with grain sizes between 4.7mm and 0.08mm.

(6) Percent Fines is equal to the percent of material (by weight) finer than 0.08mm.

(7) Percent finer by weight (also known as the clay-size fraction).

(8) Count equals total number of sieve analyses performed (smallest sieve size is #200 sieve [0.07mm]).

(9) Count equals the total number of hydrometer analyses performed.

TABLE 5
SUMMARY OF ONE-DIMENSIONAL CONSOLIDATION PROPERTIES BY GEOLOGIC UNIT(1)

Interpreted Geologic Unit(2)	Count	Depth			Preconsolidation Pressure (tons per square foot)			Estimated Overconsolidation Ratio			Modified Compression Index $C_{c\varepsilon}$(3)			Modified Recompression Index $C_{r\varepsilon}$(3)		
		Minimum	Maximum	Average	Minimum	Maximum	Average	Minimum	Maximum	Average	Minimum	Maximum	Average	Minimum	Maximum	Average
Hf	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Hls	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Hl	2	13	40	27	0.3	1.0	0.7	1.4	1.5	1.5	0.2	0.4	0.3	0.02	0.03	0.02
Ha	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Hp	1	-	-	9	-	-	0.1	-	-	0.8	-	-	0.4	-	-	0.07
Qvro	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qvrl	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qvri	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qvat	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qvt	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qvd	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qva	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qvgl	2	34	66	50	22	25	24	8	10	9	0.08	0.10	0.09	0.01	0.02	0.02
Qvgm	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qpnf	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qpnl	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qpnp	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qpns	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qpgo	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qpgl	5	101	151	121	9	32	21	2.2	6	4	0.1	0.2	0.2	0.01	0.05	0.03
Qpgl	4	161	191	179	15	38	28	1.9	4	3	0.07	0.2	0.1	0.009	0.03	0.02
Qpgl	4	207	263	232	21	30	26	1.7	3	2	0.2	0.3	0.2	0.030	0.04	0.04
Qpgt	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qpgd	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Qpgm	1	-	-	146	-	-	22	-	-	2	-	-	0.3	-	-	0.07

NOTES:

(1) This table presents the results of one-dimensional consolidation tests performed on soil samples from geologic units encountered during boring drilling along the current northbound alignment. Only 4 consolidation tests were performed on samples from borings presented in the February 2006 Geotechnical Data Report (2006 GDR). All 4 samples were located within the interpreted Qpgl geologic unit. Data presented for geologic unit Qpgm is based on a test performed on a SB series boring sample. The remaining data is based on tests performed on samples from the NB series borings that are not included in the 2006 GDR.

(2) Descriptions of the geologic units are presented in Section 4.2.

(3) $C_{c\varepsilon} = \frac{C_c}{1 + e_o} = \frac{\Delta\varepsilon}{\Delta \log p'}$ and $C_{r\varepsilon} = \frac{C_r}{1 + e_o} = \frac{\Delta\varepsilon}{\Delta \log p'}$

TABLE 6

SUMMARY OF TOTAL STRESS SHEAR STRENGTH BY GEOLOGIC UNIT(1) FROM UNCONSOLIDATED UNDRAINED TRIAXIAL TESTS

Interpreted Geologic Unit(2)	Count	Depth			Undrained Shear Strength (tons per square foot)			Axial Strain a (%)	
		Minimum	Maximum	Average	Minimum	Maximum	Average	Minimum	Maxim
Hf	-	-	-	-	-	-	-	-	-
Hls	-	-	-	-	-	-	-	-	-
Hl	4	40	51	44	0.1	2.5	0.8	1.1	15
Ha	-	-	-	-	-	-	-	-	-
Hp	-	-	-	-	-	-	-	-	-
Qvro	-	-	-	-	-	-	-	-	-
Qvrl	-	-	-	-	-	-	-	-	-
Qvri	-	-	-	-	-	-	-	-	-
Qvat	-	-	-	-	-	-	-	-	-
Qvt	1	-	-	17	-	-	2.1	-	-
Qvd	2	40	47	44	3.9	14	8.9	2.1	15
Qva	-	-	-	-	-	-	-	-	-
Qvgl	2	40	74	57	1.5	1.7	1.6	15	15
Qvgm	1	-	-	76	-	-	1.6	-	-
Qpnf	-	-	-	-	-	-	-	-	-
Qpnl	22	57	280	170	0.7	12	6.9	5.2	15
Qpnp	-	-	-	-	-	-	-	-	-
Qpns	1	-	-	52	-	-	3.0	-	-
Qpgo	-	-	-	-	-	-	-	-	-
Qpgl	57	64	274	180	1.0	11	3.5	1.1	15
Qpgt	-	-	-	-	-	-	-	-	-
Qpgd	-	-	-	-	-	-	-	-	-
Qpgm	2	121	313	217	2.8	11	6.7	15	15

NOTES:

(1) This table presents the results of unconsolidated undrained triaxial (UU TXC) tests performed on soil samples recovered from borings drilled during the CE and PE de TXC tests included in this table, which represent the data for the geologic unit Qpgm, were performed on SB series boring samples. Twenty-one of the tests were perf located adjacent to the current University Link route. The remaining tests were performed on recovered samples from NB series borings, which were drilled adjacent t

(2) Descriptions of the geologic units are presented in Section 4.2.

(3) Failure of a sample is not achieved before reaching an axial strain of 15%. Undrained Shear Strength at 15 percent axial strain is considered as "failure".

TABLE 7
SUMMARY OF EFFECTIVE STRESS SHEAR STRENGTH BY GEOLOGIC UNIT[1]

Interpreted Geologic Unit [2]	Effective Strength Parameters								
	φ' [3]			ψ' [4]			c' [5] (tsf)		
	Minimum	Maximum	Average	Minimum	Maximum	Average	Minimum	Maximum	Average
Hf	-	-	-	-	-	-	-	-	-
Hls	-	-	-	-	-	-	-	-	-
Hl	-	-	31	-	-	-	-	-	0
Ha	-	-	-	-	-	-	-	-	-
Hp	-	-	-	-	-	-	-	-	-
Qvro	-	-	-	-	-	-	-	-	-
Qvrl	-	-	-	-	-	-	-	-	-
Qvri	-	-	-	-	-	-	-	-	-
Qvat	-	-	-	-	-	-	-	-	-
Qvt	-	-	-	-	-	-	-	-	-
Qvd	-	-	-	-	-	-	-	-	-
Qva	38	41	40	5	9	13	0	0	0
Qvgl	-	-	32	-	-	-	-	-	0.3
Qvgm	-	-	41	-	-	-	-	-	0
Qpnf	38	42	40	-	-	-	0	0	0
Qpnl	35	40	37	-	-	-	0	0	0
Qpnp	-	-	-	-	-	-	-	-	-
Qpns	-	-	-	-	-	-	-	-	-
Qpgo	39	42	40	6	9	11	0	0.3	0
Qpgl (Relatively Intact)	27	32	29	-	-	-	0.3	0.8	0.6
Qpgl (Discontinuous Slickensided)	16	27	22	-	-	-	0	0.3	0
Qpgl (Residual)	12	18	15	-	-	-	0	0	0
Qpgt	-	-	-	-	-	-	-	-	-
Qpgd	-	-	-	-	-	-	-	-	-
Qpgm/ Qpns	32	39	35	-	-	-	0.3	0.75	0.5

NOTES:

(1) Effective strength parameters given above are based on results from the Consolidated Drained and Consolidated Undrained Triaxial Compression tests performed for CE and PE phases.

(2) Descriptions of geologic units are presented in Section 4.2.

(3) ϕ'=effective stress friction angle

(4) ψ'=efffective stress dilation angle

(5) c'=effective stress cohesion

TABLE 8
SUMMARY OF SUBSURFACE CONDITIONS
ALONG BORED TUNNEL SECTIONS

Segment	Approx. Length (feet)	Approx. Elevation of Rail (feet)	Approx. Depth of Cover above Crown (feet)	Average Depth to Rail (feet)	Closest[1] Boring(s)	Geologic Units near Tunnel Level[2]	Measured Groundwater Elevations[3] (feet)
~NB 1046+00 to NB 1048+00 (I-5 Undercrossing [UC])	200	102 to 104	17 to 40	35 to 57	NB-301 NB-201	**Qpgl, Qpnl, Qpnf,** Qpgo	NA
~NB-1048+00 to NB-1072+00 (I-5 UC to Capitol Hill Station)	2,400	105 to 180	85 to 130	102 to 147	NB-202 NB-302 NB-282 NB-283	**Qpgl, Qpnl, Qpgm,** Qpnf, Qpgo	135 (Qpgl) 136 (Qpgl) 123 (Qpgo) 242 (Qpnl)
~NB 1072+00 to NB 1083+00 (I-5 UC to Capitol Hill Station)	1,100	180 to 238	71 to 92	90 to 110	NB-314 NB-315 NB-383 NB-384 NB-385	**Qpgl, Qpnl, Qvgm,** Qpgm, Qpnf	274 (Qvt) 289 (Qvt) 307 (Qvt) 314 (Qvt) 317 (Qvt)
~NB 1083+00 to NB 1112+00 (Capitol Hill Station to Montlake Vent Shaft)	2,900	240 to 180	70 to 205	90 to 220	NB-386 NB-392 NB-393 NB-249	Qpnf, **Qpnl, Qpgl**	317 (Qpnl) 320 (Qpnl) 317 (Qpnl) 280 (Qpgl)
~NB 1112+00 to NB 1132+00 (Capitol Hill Station to Montlake Vent Shaft)	2,000	180 to 100	205 to 310	220 to 330	NA	**Qpgl, Qpnl**	NA [5]
~NB 1132+00 to NB 1183+00 (Capitol Hill Station to Montlake Vent Shaft)	4,800	100 to -50	65 to 300	80 to 320	NB-280 NB-250 NB-115 NB-387 NB-116	Qpgo, **Qpnl, Qpgl, Qpnf, Qpgm**	230 (Qpgl) 190 (Qpnl) 33 (Qpnf) 56 (Qpgm)

TABLE 8
SUMMARY OF SUBSURFACE CONDITIONS
ALONG BORED TUNNEL SECTIONS

Segment	Approx. Length (feet)	Approx. Elevation of Rail (feet)	Approx. Depth of Cover above Crown (feet)	Average Depth to Rail (feet)	Closest[(1)] Boring(s)	Geologic Units near Tunnel Level[(2)]	Measured Groundwater Elevations[(3)] (feet)
~NB 1184+00 to NB 1201+75 (Montlake Vent Shaft to University of Washington [UW] Station)	1,720	-45 to -80	20 to 125 [(4)]	40 to 145 [(4)]	NB-388	**Qpgl**	NA

NOTES:

(1) Borings within approximately 100 feet of the alignment were considered.

(2) Based on soils encountered in closest borings for 2 diameters above and 1 diameter below tunnel level.(soils encountered at tunnel level are in **bold**; geologic units are based on closest recent boring).

(3) Measured groundwater levels reported in this table do not necessarily account for all potential perched water conditions. The geologic unit presented above reflects the unit encountered at the installed well screen/piezometer depth. Groundwater well screen depth/ piezometer locations were determined based on knowledge of conceptual design information at the time of installation. Where mined station construction was previously proposed, well screens/ piezometers were not located at shallow elevations.

(4) Shallower depths are based on assumed bottom of Mountlake Cut shown on Figure 7 (Sheet 12 of 27).

(5) NA = None available

TABLE 9
TUNNELMAN'S GROUND CLASSIFICATION

Classification		Behavior	Typical Soil Type
Firm		Heading can be advanced without initial support, and final lining can be constructed before ground starts to move.	Loess above the water table; hard clay, marl, cemented sand and gravel when not highly overstressed.
Raveling	**Slow Raveling**	Chunks or flakes of material begin to drop out of the arch or walls sometime after the ground has been exposed, due to loosening or to overstress and "brittle" fracture (ground separates or breaks along distinct surfaces, opposed to squeezing ground). In fast raveling ground, the process starts within a few minutes, otherwise the ground is slow raveling.	Residual soils or sand with small amounts of binder may be fast raveling below the water table, slow raveling above. Stiff fissured clays may be slow or fast raveling depending upon degree of overstress.
	Fast Raveling		
Squeezing		Ground squeezes or extrudes plastically into tunnel, without visible fracturing or loss of continuity, and without perceptible increase in water content. Ductile, plastic yield and flow due to overstress.	Ground with low frictional strength. Rate of squeeze depends on degree of overstress. Occurs at shallow to medium depth in clay of very soft to medium consistency. Stiff to hard clay under high cover may move in combination of raveling at execution surface and squeezing at depth behind surface.
Running	**Cohesive Running**	Granular materials without cohesion are unstable at a slope greater than their angle of repose (±30° to 35°). When exposed at steeper slopes they run like granulated sugar or dune sand until the slope flattens to the angle of repose.	Clean, dry granular materials. Apparent cohesion in moist sand, or weak cementation in any granular soil may allow the material to stand for a brief period of raveling before it breaks down and runs. Such behavior is cohesive-running.
	Running		
Flowing		A mixture of soil and water flows into the tunnel like a viscous fluid. The material can enter the tunnel from the invert as well as from the face, crown, and walls, and can flow for great distances, completely filling the tunnel in some cases.	Below the water table in silt, sand or gravel without enough clay content to give significant cohesion and plasticity. May also occur in highly sensitive clay when such material is disturbed.
Swelling		Ground absorbs water, increases in volume, and expands slowly into the tunnel.	Highly pre-consolidated clay with plasticity index in excess of about 30, generally containing significant percentages of montmorillonite.

(Heuer, 1974)

TABLE 10
TUNNELING AND EXCAVATION CHARACTERISTICS BY SOIL GROUP

Soil Group[1]	Soil Type	Geologic Units[2]	Stability	Excavatability	Standup Time	Ground Modification for tunneling	Groundwater Flows	Comments
1	Fill and Other Non-Overridden Soils	Hf, Hls, Ha, Hp, Qvri, Qvro, Qvat, Qvrl	Unstable without pre-drainage and support.	Loose to very dense, relatively easy to excavate, cobbles and boulders.	Very short to none.	NA	High in saturated zones of sand and silt.	These units can be highly variable and should be reviewed as site-specific.
2	Cohesive Silt and Clay	**Qpgl,** Qvgl	Favorable except where slickensided, fractured, and blocky, and where lenses or layers of cohesionless silt and sand may run or flow.	Very stiff to hard, relatively easy to excavate, may become sticky when wet, cobbles and boulders.	Moderate where massive, short to none where slickensided, fractured, and blocky.	Pre-drain cohesionless silt and sand layers. Spiling in highly slickensided, fractured, and blocky zones.	Minor, except in layers of saturated cohesionless silt and sand.	Qpgl and Qvgl can have highly variable characteristics; refer to text for discussion.
3	Cohesionless Silt and Fine Sand	**Qpnl,** Qpnp	Unstable without pre-drainage and support.	Very dense/ Hard, relatively easy to excavate.	Very short to none.	Pre-drain with closely spaced vacuum/eductor well points.	Moderate where saturated.	Qpnl and Qpnp can have highly variable characteristics; refer to text for discussion.
4	Till and Till-like Deposits	**Qpgm, Qpgd,** Qvd, Qvt, Qvgm, Qpns, Qpgt	Favorable, lenses or layers of cohesionless silt and sand may run or flow.	Dense to very dense, concrete-like and difficult to excavate, cobbles and boulders.	Relatively long; however, very short to none in zones of cohesionless silt and sand.	Pre-drain cohesionless silt and sand layers.	Minor, except in zones of saturated sand and silt.	None
5	Cohesionless Sand and Gravel	**Qpgo, Qpnf,** Qva	Unstable without pre-drainage and support.	Dense to very dense, relatively easy to excavate, cobbles and boulders.	Very short to none.	Pre-drain with deep wells.	High where saturated.	None

NOTES:
(1) See text for explanation of grouping geologic units into designated soil groups.
(2) Geologic units listed in **bold** are the primary units encountered along the University Link route.
(3) NA = Not Applicable

TABLE 11
SUMMARY OF TUNNEL EXCAVATION CONDITIONS BY STATIONING

Anticipated Excavation Condition[(1)] (Mixed or full face)	Anticipated Soil Type (by group)[(2)]	NB Station location		Approximate Length (ft)
		Approximate Start	Approximate End	
Full	Group 4	1044+00	1045+00	100
Mixed	Groups 2 over 3	1045+00	1052+00	700
Full	Group 2	1052+00	1058+00	600
Mixed	Group 4 over 2	1058+00	1061+00	300
Mixed	Group 3 over 4 over 2	1061+00	1067+00	600
Mixed	Group 3 over 2	1067+00	1078+00	1,100
Mixed	Group 4 over 3 over 2	1078+00	1082+00	400
Mixed	Group 3 over 2	1082+00	1084+00	200
Capitol Hill Station (Cut and Cover)		1084+00	1087+00	-
Mixed	Groups 2 over 3	1087+00	1108+00	2,100
Full	Group 2	1108+00	1140+00	3,200
Mixed	Groups 3 over 2	1140+00	1150+00	1,000
Full	Group 2	1150+00	1161+00	1,100
Mixed	Groups 5 over 2	1161+00	1167+00	600
Mixed	Group 4 over 2	1167+00	1179+00	1,200
Full	Group 2	1179+00	1183+00	400
Vent Shaft		1183+00	1184+00	-
Full	Group 2	1184+00	1202+00	1,800
U of W Station (Cut and Cover)		1202+00	1210+00	-
Mixed	Groups 5 over 4 over 5	1210+00	1218+00	800
Full	Group 2	1218+00	1222+00	400
Mixed	Groups 2 over 3	1222+00	1225+00	300
Full	Group 2	1225+00	1231+00	600
Mixed	Groups 3 over 2	1231+00	1233+00	200
Full	Group 3	1233+00	1238+00	500
Mixed	Groups 5 over 3	1238+00	1243+00	500
Full	Group 5	1243+00	1250+00	700
Mixed	Groups 4 over 5	1250+00	1251+00	100
Full	Group 4	1251+00	1256+00	500
Mixed	Groups 4 over 5	1256+00	1259+00	300

NOTES:

(1) Anticipated excavation conditions are based on the interpreted subsurface conditions indicated on the geologic profile for University Link (Figure 7). Only subsurface conditions between the crown and the invert of the proposed tunnel were considered.

(2) The following group designations were used in this table:
(Refer to the main text for full descriptions of the soil groups.)

Group 1 - Fill and other non-overridden soils

Group 2 - Cohesive silt and clay

Group 3 - Cohesionless silt and fine sand

Group 4 - Till and Till-like deposits

Group 5 - Cohesionless sand and gravel

TABLE 12
SUMMARY OF ANTICIPATED SOIL TYPE BY PERCENTAGE OF TUNNEL LENGTH

Anticipated Soil Type	Percentage
Full face through Group 2	40%
Full face through Group 3	2%
Full face through Group 4	3%
Full face through Group 5	3%
Mixed face - Groups 2 over 3	15%
Mixed face - Groups 3 over 2	12%
Mixed face - Groups 4 over 2	7%
Mixed face - Groups 2 over 4	0%
Mixed face - Groups 2 over 5	0%
Mixed face - Groups 5 over 2	3%
Mixed face - Groups 3 over 5	0%
Mixed face - Groups 5 over 3	2%
Mixed face - Groups 4 over 5	2%
Mixed face - Groups 5 over 4	4%
Mixed face - Three or more Soil Groups	5%

Distribution of Soil Type for Mixed-Face Conditions:

Summary by Soil Group	Percentage
Group 2	43%
Group 3	35%
Group 4	18%
Group 5	16%

NOTES:

(1) Soil types are based on the interpreted subsurface conditions indicated on the geologic profile for University Link (Figure 7). Only subsurface conditions between the crown and the invert of the proposed tunnel were considered.

(2) The following group designations were used in this table:
(Refer to the main text for full descriptions of the soil groups).

Group 1 - Fill and other non-overridden soils
Group 2 - Cohesive silt and clay
Group 3 - Cohesionless silt and fine sand
Group 4 - Till and Till-like deposits
Group 5 - Cohesionless sand and gravel

TABLE 13
ESTIMATED SOIL PERMEABILITY BY TUNNEL SEGMENT

Tunnel Segment (length, ft)	Permeability (cm/sec)	% of length
Stub Tunnel To Capitol Hill Sta. (3,850)	10^{-4} or less	80
“	10^{-3}	10
“	10^{-2}	7
“	10^{-1}	3
Capitol Hill Sta. To UW Sta. (11,330)	10^{-4} or less	80
“	10^{-3}	15
“	10^{-2}	4
“	10^{-1}	1

TABLE 14
RECOMMENDED ENGINEERING PROPERTIES
FOR TUNNELING AND CROSS-PASSAGE TUNNELS

Geologic Unit[1]		Shear Modulus, G x 1000 (psi)[2]		Poisson's Ratio	At-Rest Earth Pressure Coefficient, Ko	Drained Shear Strength Parameters[3]		Undrained Shear Strength (tsf)	Total Unit Weight (pcf)	Hydraulic Conductivity Coefficients[10] (cm/sec)		Specific Storage (ft^{-1})
		Initial (max)[4]	Secant (failure)[5]			c' (tsf)	ϕ' (degrees)			Horizontal	Vertical	
Qpgl/Qvgl[6]	***Relatively Intact***	40 - 70	12 - 30	0.3 - 0.4 (drained) 0.5 (undrained)	0.6 - 1.5	0.3 - 0.8	27 - 32	0.3-10.8	118 - 130	5×10^{-8} to 1×10^{-6} (8)	1×10^{-8} to 5×10^{-7} (8)	1×10^{-6} to 1×10^{-3} (8)
	Discontinuous Slickensides					0.0 - 0.3	16 - 27					
	Slickensided (Residual) [7]	8 - 14	2.4 - 6			0	12 - 18					
Qpnl/Qpnp		45 - 75	20 - 40[9]	0.3 - 0.4 (drained and undrained)	0.5 - 1.1	0	35 - 40	3.7-18	120 - 135	1×10^{-5} to 1×10^{-3}	1×10^{-6} to 1×10^{-4}	1×10^{-6} to 1×10^{-3}
Qpgm/Qpns		40 - 120	30 - 60		0.7 - 1.5	0.3 - 0.75	32 - 39	3.6-7.1	130 - 150	5×10^{-7} to 5×10^{-4}	5×10^{-8} to 5×10^{-5}	1×10^{-6} to 1×10^{-3}
Qvt/Qpgt					-	0	40 - 45	-		5×10^{-7} to 5×10^{-4}	1×10^{-7} to 1×10^{-4}	
Qvd/Qpgd					0.6 - 1.2					1×10^{-5} to 1×10^{-3}	1×10^{-6} to 1×10^{-4}	
Qpgo/Qpnf/Qva		45 - 90	30 - 70		0.5 - 1.2	0-0.3	38-42	-	125 - 140	5×10^{-4} to 1×10^{-1}	1×10^{-4} to 5×10^{-1}	1×10^{-6} to 1×10^{-2}

Notes:

(1) Descriptions of geologic units are included in Section 4.2

(2) Elastic (Young's) Modulus = 2 x G x (1 + poisson's ratio)

(3) c' = effective stress cohesion
ϕ' = effective stress friction angle
ψ' = effective stress dilation angle

(4) Interpreted from seismic downhole test results at strains of approx. 10^{-4} percent.

(5) Interpreted from pressure meter test results at strains of approx. 0.5 to 1.5 percent.

(6) Relatively intact = no observed slickensides or other discontinuities;
Discontinuous slickensides = observed with partial slickensides;
Slickensided = observed through-going slickensides.

(7) Based on 20 percent of intact soil range.

(8) Hydraulic conductivity coefficients and specific storage values assume relatively intact clays.

(9) The upper limit was estimated based upon experience with Seattle soils.

(10) Values of hydraulic conductivity coefficients are average values for the particular geologic unit and are based on site-specific gradation curves and slug tests.

TABLE 15
GENERALIZED FLAC[1] INPUT FOR MONTLAKE VENT SHAFT

Soil Layer[2]	Upper Boundary Depth (feet)	Lower Boundary Depth (feet)	Effective Unit Weight, γ' (pcf)[3]	Effective Friction Angle (degrees)	Cohesion, c' (psf)	Poisson's Ratio[4]	Young's Modulus (ksi)[4],[5]		Initial At-Rest Earth Pressure Coefficient, K_o[6]
							Slurry Wall	Secant Pile Wall	
Very dense SAND and sandy SILT	0	10	130	40	0	0.35	135	162	1.3 to 1.2
Very dense SAND and sandy SILT	10	80	67.5	40	0	0.35	135	162	
Hard silty CLAY	80	-	62.5	26	600	0.48	45	74.5	1.5

Notes:

(1) Recommended lateral earth pressures were determined by performing numerical analyses using the two-dimensional finite difference soil-structure interaction program (FLAC) developed by Itasca Consulting Group (1998). The soil properties presented above were used in the analyses.

(2) Soil layer propertiess are based on the subsurface conditions encountered in existing available borings completed in vicinity.

(3) Effective unit weights were used in the analyses. As excavation was performed in the analyses, an additional pressure was applied to account for hydrostatic pressures acting against the shaft walls.

(4) The cohesive layers shown above were assumed to be in an undrained state.

(5) Shear and bulk moduli used in the analyses can be derived using the Young's moduli and poisson's ratios given above and the following relationships based on the Theory of Elasticity:
Shear Modulus, $G = E/[2*(1+\nu)]$
Bulk Modulus, $K = E/[3*(1-2*\nu)]$
Modulus of Elasticity, $E = 9*K*G/(3*K+G)$

(6) Initial at-rest earth pressure coefficients are based on the emperical relationships provided by Terzaghi (1943), Berezantzev (1958), and Prater (1972).

(7) Recommended earth pressures are provide on Figure 45 for the Montlake Vent Shaft at Roanoke St, which has a shaft diameter of 22 feet and a shaft depth of 110 feet.

TABLE 16
SUMMARY OF SUBSURFACE CONDITIONS AT
CUT-AND-COVER STRUCTURES

Section	Type	Approx. Length (feet)	Approx. Elevation of Rail (feet)	Average Depth to Rail (feet)	Closest[1] Boring(s)	Geologic Units Anticipated in Excavation[2]	Measured Groundwater Elevation[3] (feet)
Capitol Hill Station NB 1083+75 to NB 1087+56	Cut-and-cover	400	240	90	NB-385 NB-390	**Hf, Qvat, Qvgm, Qpnf, Qpnl, Qpgl**	318 (Qpnf) 318 (Qpnf)
Montlake Vent Shaft ~NB 1183+20	Shaft	100	-54	110 to 140	NB-251	**Hf, Qvd, Qpnl, Qpnf, Qpgm, Qpgo, Qpgt, Qpgl**	46 (Qpnl/Qpgm)
Double Cross-over NB 1201+75 to NB 1205+50	Cut-and-cover	370	-44 to -42	110	NB-389 NB-252	**Hf, Qvt, Qpnl, Qva, Qpgm, Qpgo, Qpgl**	27 (Qpgl)
University of Washington Station NB 1205+50 to NB 1211+15	Cut-and-cover	570	-42 to -40	90	NB-253	**Hf, Qvro, Qvd, Qvt, Qva,** Qpgl	31 (Qva)

NOTES:

(1) Borings within approximately 100 feet of the alignment were considered.

(2) Based on our interpretation of subsurface conditions as shown on Figure 7.

(3) Measured groundwater levels reported in this table do not necessarily account for all potential perched water conditions. The geologic unit presented above reflects the unit encountered at the installed well screen/piezometer depth. Groundwater well screen depth/ piezometer locations were determined based on knowledge of preliminary design information at the time of installation. Where mined station construction was previously proposed, well screens/ piezometers were not located at shallow elevations.

(4) NA = None available

TABLE 17
PRELIMINARY RECOMMENDED PARAMETERS
FOR DESIGN OF RETAINING WALLS FOR CUT-AND-COVER STATIONS

Section	Geologic Units[1]	Active Earth Pressure Coefficient, Ka	At-rest Earth Pressure Coefficient, Ko			Total Unit Weight (pcf)	Depth of Gorundwater Level[2] (feet)	Dynamic Pressure Increment[3] ODE/ MDE	Modulus of Subgrade Reaction		
			Typical Retaining Walls	Secant Pile Wall	Slurry Wall				Horizontal Constant, n_h (pci)[4]	Horizontal Coefficient, k_1 (psi)[4]	Vertical Coefficient, k (psi)[5]
Capitol Hill Station	Hf/ Qvat	0.31	0.47	0.47	-	120			30 to 50	-	100 to 125
NB 1083+75 to NB 1087+56	Qvgm/Qpnf/Qpnl	0.22	0.36	0.7	0.5	130	10	5H/ 24H	-	300 to 500	100 to 125
	Qpnl/ Qpgl	NA[6]	NA	0.9	0.5	125			-	200 to 300	50 to 80
University of Washington Station	Hf	0.31	0.47	0.47	-	120			30 to 50	-	100 to 125
and Crossover	Qvro/Qvd/Qvt	0.22	0.36	0.8	0.5	130	30	5H/ 24H	-	300 to 500	100 to 125
NB 1201+75 to NB 1211+15	Qva/ Qpgl	NA	NA	1.0	0.6	130			-	200 to 300	50 to 80

NOTES:

(1) Descriptions of geologic units are included in Section 4.2.

(2) Depth of assumed groundwater level recommended for calculating hydrostatic pressures acting on below-grade structures. For temporary shoring design, it was assumed that dewatering would be performed, and the groundwater level would be approximately two feet below the bottom of the excavation.

(3) It is recommended that the dynamic earth pressure increments be considered together with both hydrostatic and active/ at-rest earth pressures for design of below-grade structures. These pressures are based on the Operational Design Earthquake (ODE) and Maximum Design Earthquake (MDE) with peak ground accelerations (PGAs) of 0.18 and 0.77, respectively.

(4) Recommended for design of side walls for permanent structures.
$k_1=n_h*z$ for normally consolidated soils, where z is depth below ground surface; k_1 is constant for overconsolidated soils.

(5) Recommended for design of base slabs, assuming that the length of the slab is at least twice greater than its width.

(6) NA = Not Applicable

(7) NR = Not Recommended

(8) NE = Not Encountered

TABLE 18
ESTIMATED NUMBER OF BOULDERS

Boulder Size[1]	Average Number of Boulders per 1,000 cy of In-Situ Material[2],[3]		
	Qpgm, Qpgd, Qpgt, Qvd, Qvt, Qvat, Qvro, Hf	Qpgl, Qpgl/Qpnl, Qpgo, Qpns, Qpnf, Qva, Qvri, Ha, Hls, Hc	Qpnp, Qpnl, Qvgl, Qvrl, Hl, Hp
1 to 3 feet	40	5	1
3 to 5 feet	4	0.5	0.1
> 5 feet	0.4	0.05	0.01

NOTES:

(1) Defined as the size of boulder that will not pass through a square opening of that size, no matter how it is oriented in the square opening. For example, a boulder with dimensions of 2.5 feet by 2.5 feet by 4.5 feet would not pass a 1-foot-square opening, but would pass through a 3-foot-square opening. Hence, it is considered to be in the "1 to 3 feet" size.

(2) The number of boulders is averaged over the length of the alignment.

(3) Nesting and concentrations of boulders could occur over 1,000 cy sections at frequencies of up to five times the values provided in the table.

(4) This table was previously presented in the Tender Geotechnical Baseline Report for the LBC35 Contract (Shannon & Wilson, 2004).

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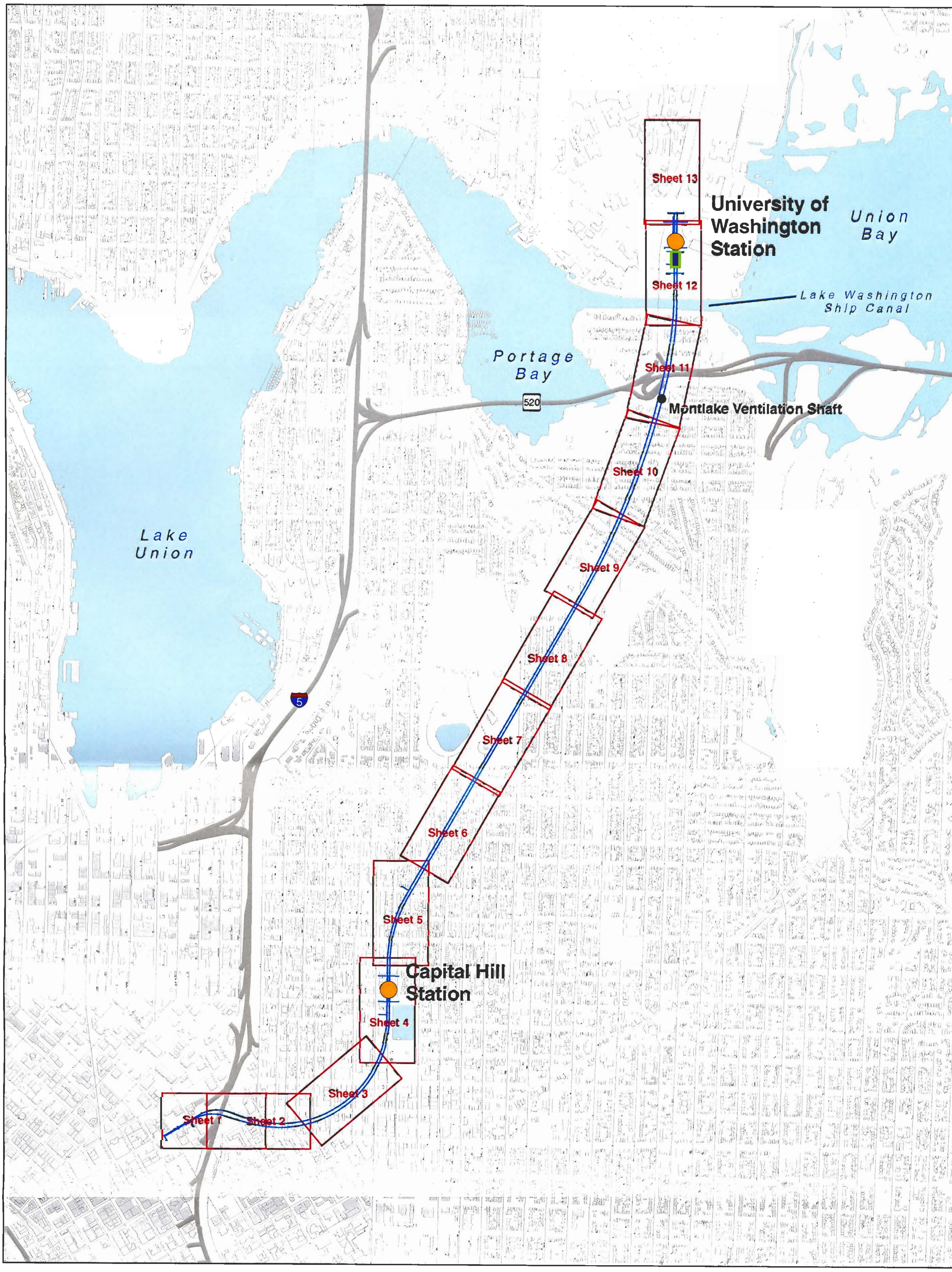

Notes:

1. Street and water base data provided by the City of Seattle, 1999.
2. Alignment "N35_L00_KA(3-7-06).dwg" provided by PSTC, 03-7-06.

N

LEGEND

University Link

Station

Vent

Cross-over

0 1500 3000 4500

Feet

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VICINITY MAP OF PROPOSED ALIGNMENT AND FACILITY LOCATIONS

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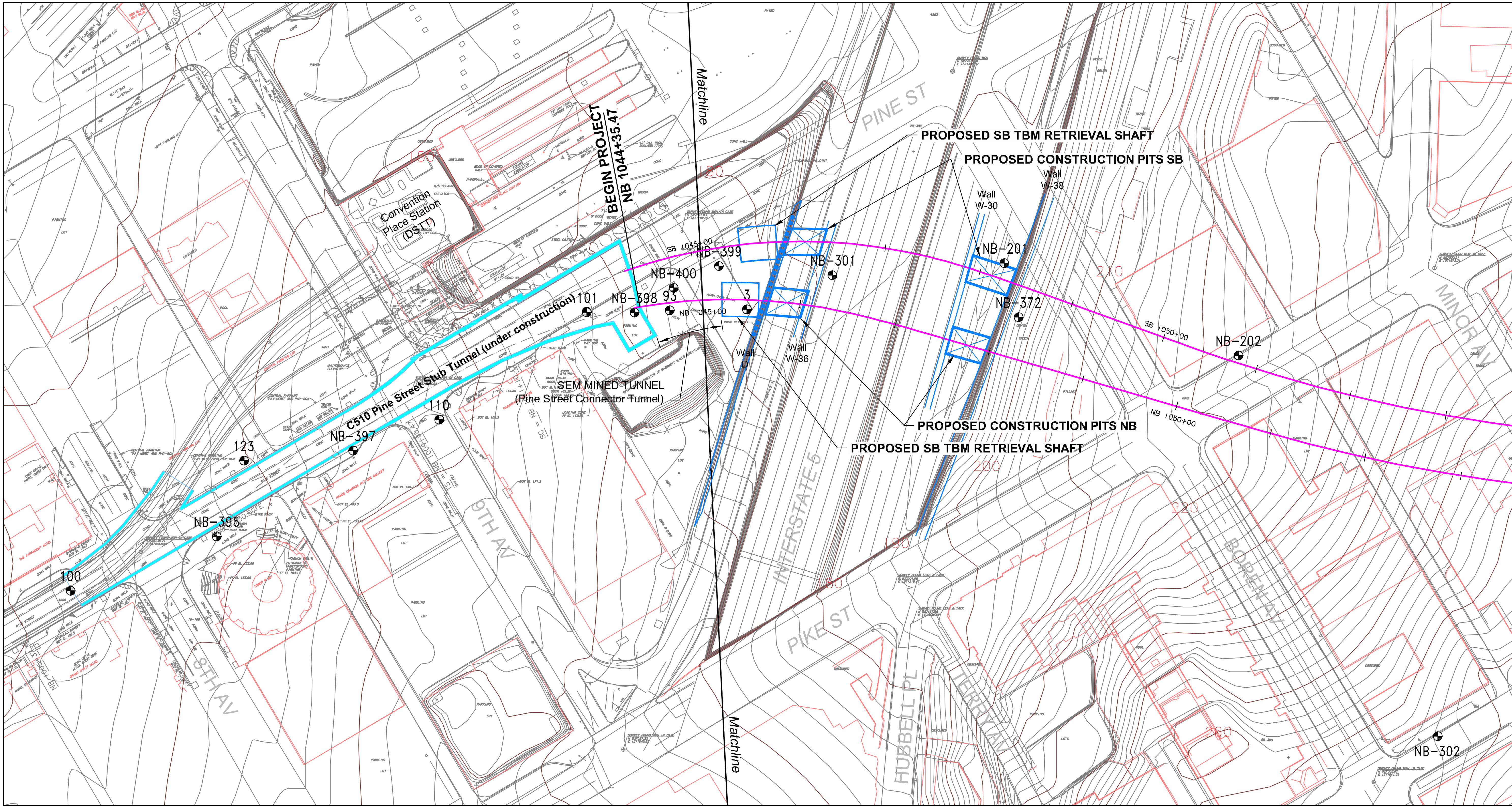
FIG. 1

FIG. 1



File: J:\211\08109-074\21-1-08109-074 Site Plan.dwg   Date: 03-28-2006   Author: SAC

STATIONING BETWEEN MATCHLINES

NB 1210+00 to 1225+00

0 100 200

Scale in Feet

SHEET LOCATION KEY

NOTES

1. Figure based on electronic files provided by PSTC. Alignment "N35_L00_KA.dwg" received 3-7-06.

2. Vertical datum: NAVD88.

LEGEND

NB-282 Current Project Boring Designation

NB-389 Previous Project Boring Designation

3692 Previous Non-Project Boring Designation and Approximate Location

N

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SITE AND EXPLORATION PLAN
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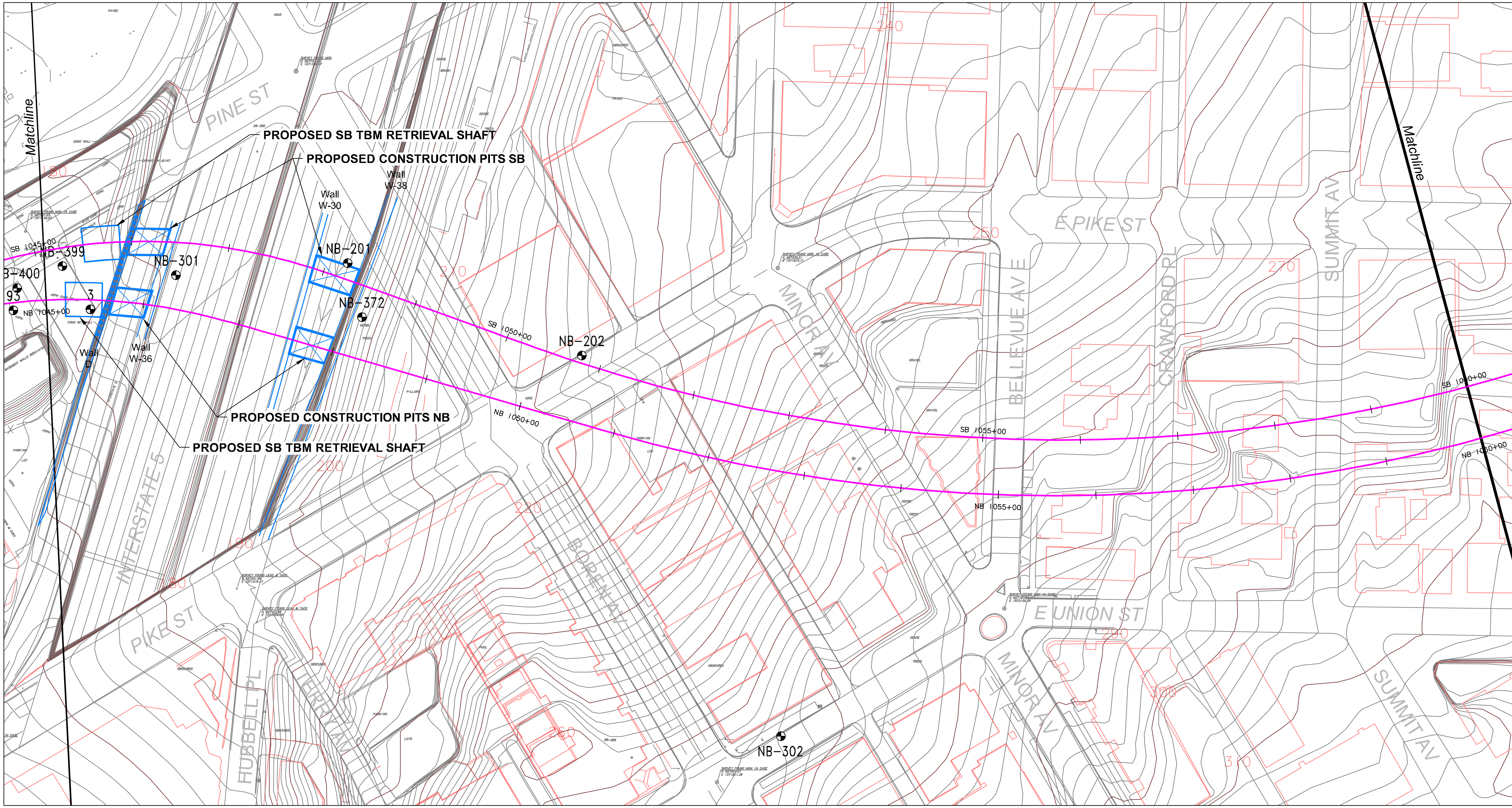
FIG. 2
Sheet 1 of 13



STATIONING BETWEEN MATCHLINES

NB 1210+00 to 1225+00

SHEET LOCATION KEY

0 100 200

Scale in Feet

NOTES

1. Figure based on electronic files provided by PSTC. Alignment "N35_L00_KA.dwg" received 3-7-06.
2. Vertical datum: NAVD88.

LEGEND

NB-282 Current Project Boring Designation

NB-389 Previous Project Boring Designation

3692 Previous Non-Project Boring Designation and Approximate Location

N

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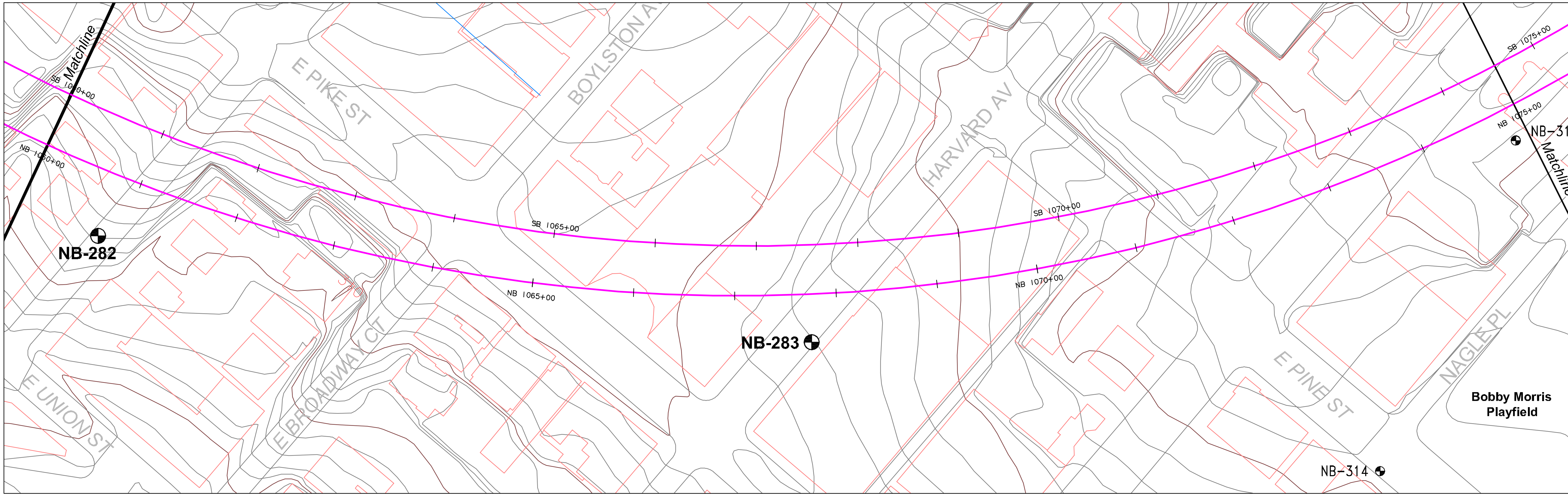
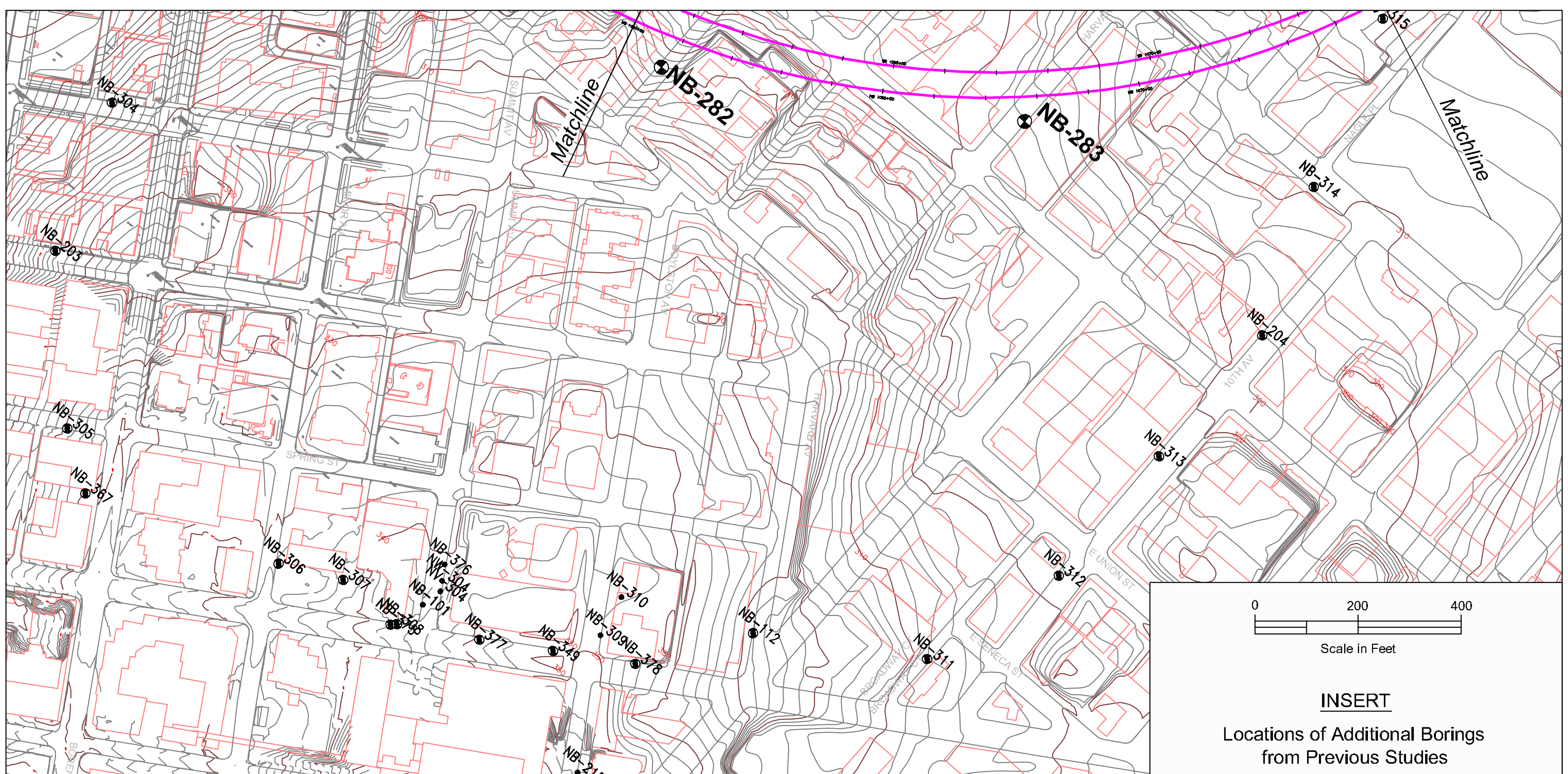
FIG. 2
Sheet 2 of 13

File: J:\211\08109-074\21-1-08109-074 Site Plan.dwg Date: 03-28-2006 Author: SAC



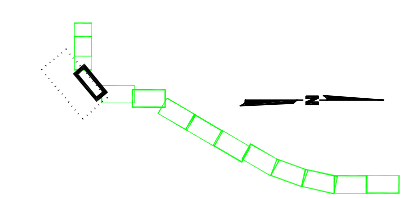


INSERT
Locations of Additional Borings from Previous Studies

STATIONING BETWEEN MATCHLINES

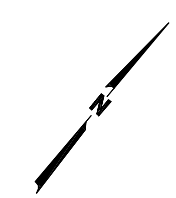
NB 1060+00 to 1075+00

SHEET LOCATION KEY



LEGEND

NB-282	Current Project Boring Designation
NB-389	Previous Project Boring Designation
3692	Previous Non-Project Boring Designation and Approximate Location



0 100 200

Scale in Feet

NOTES

1. Figure based on electronic files provided by PSTC. Alignment "N35_L00_KA.dwg" received 3-7-06.
2. Vertical datum: NAVD88.

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FIG. 2
Sheet 3 of 13

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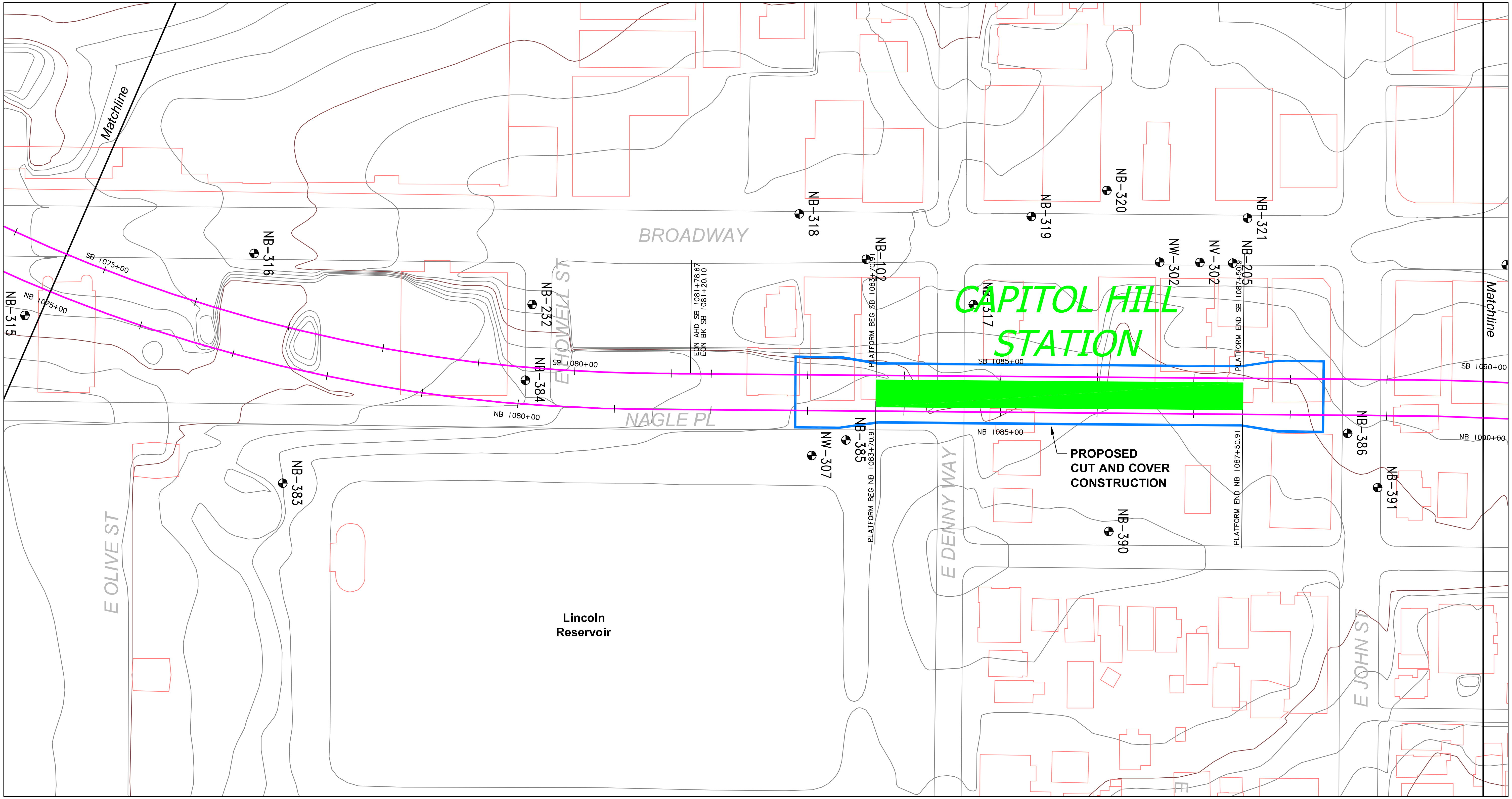

STATIONING BETWEEN MATCHLINES

NB 1210+00 to 1225+00

SHEET LOCATION KEY

0 100 200

Scale in Feet

NOTES

1. Figure based on electronic files provided by PSTC. Alignment "N35_L00_KA.dwg" received 3-7-06.
2. Vertical datum: NAVD88.

LEGEND

NB-282 Current Project Boring Designation

NB-389 Previous Project Boring Designation

3692 Previous Non-Project Boring Designation and Approximate Location

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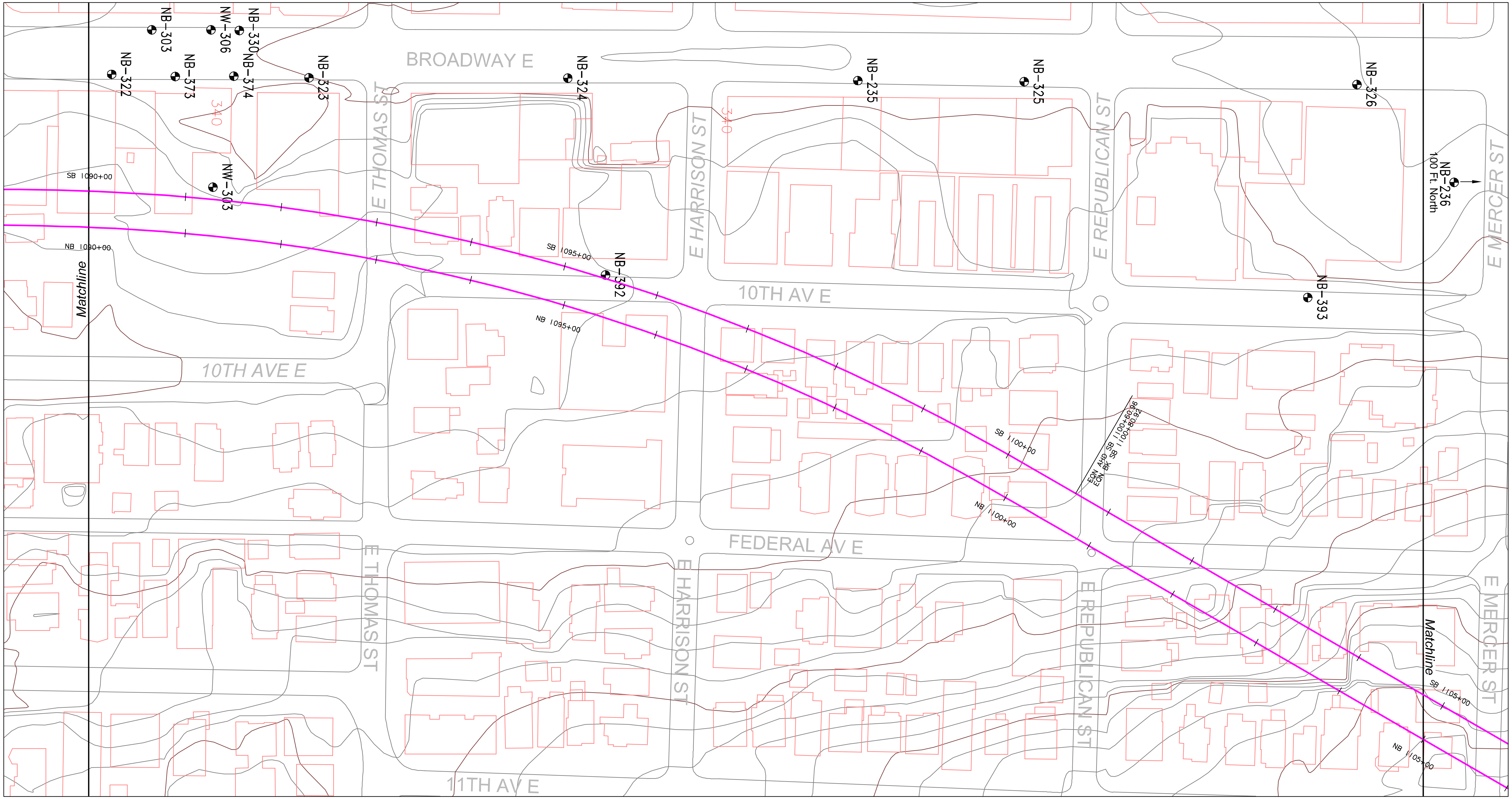
FIG. 2
Sheet 4 of 13

File: J:\211\08109-074\21-1-08109-074 Site Plan.dwg Date: 03-28-2006 Author: SAC



STATIONING BETWEEN MATCHLINES

NB 1210+00 to 1225+00

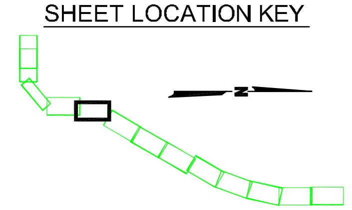
0 100 200

Scale in Feet

NOTES

1. Figure based on electronic files provided by PSTC. Alignment "N35_L00_KA.dwg" received 3-7-06.
2. Vertical datum: NAVD88.

SHEET LOCATION KEY



LEGEND

NB-282	Current Project Boring Designation
NB-389	Previous Project Boring Designation
3692	Previous Non-Project Boring Designation and Approximate Location

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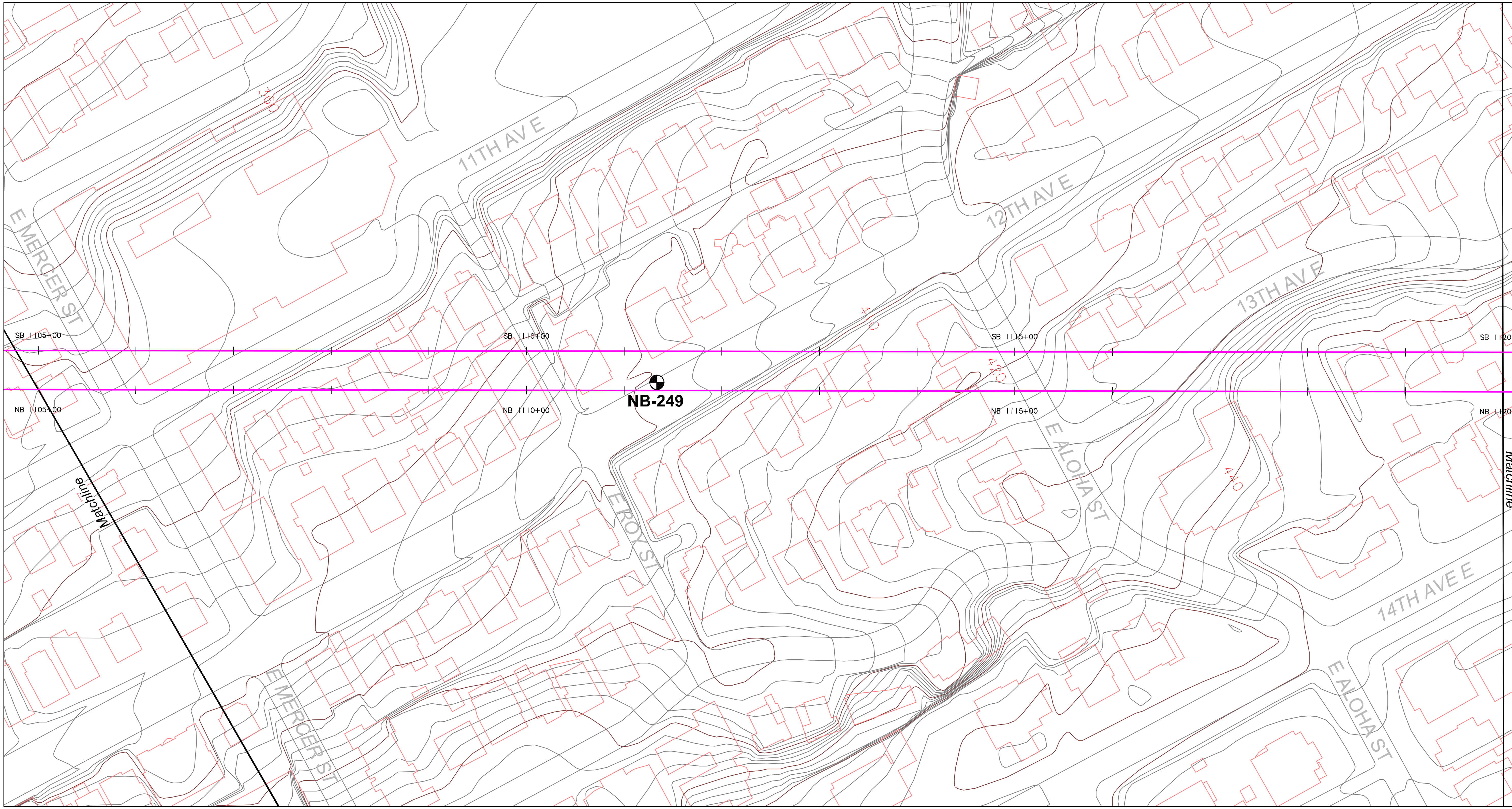
FIG. 2
Sheet 5 of 13

File: J:\211\08109-074\21-1-08109-074 Site Plan.dwg Date: 03-28-2006 Author: SAC



STATIONING BETWEEN MATCHLINES

NB 1210+00 to 1225+00

SHEET LOCATION KEY

0 100 200

Scale in Feet

NOTES

1. Figure based on electronic files provided by PSTC. Alignment "N35_L00_KA.dwg" received 3-7-06.
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LEGEND

NB-282	Current Project Boring Designation
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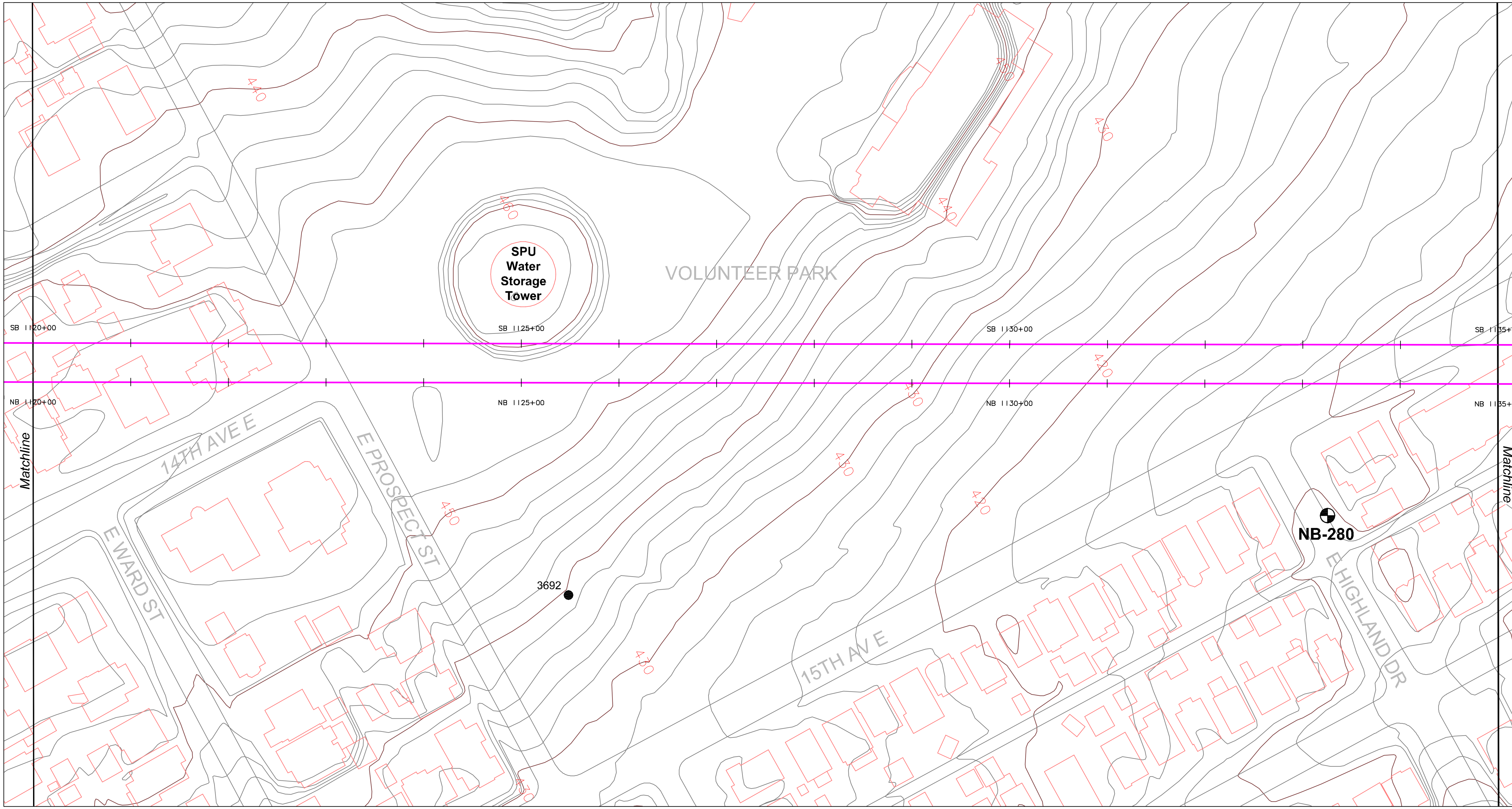
FIG. 2
Sheet 6 of 13

File: J:\211\08109-074\21-1-08109-074 Site Plan.dwg Date: 03-28-2006 Author: SAC

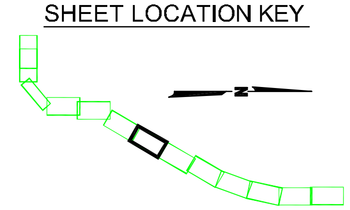


STATIONING BETWEEN MATCHLINES

STATIONING BETWEEN MATCHLINES

NB 1210+00 to 1225+00

SHEET LOCATION KEY



0 100 200

Scale in Feet

NOTES

1. Figure based on electronic files provided by PSTC. Alignment "N35_L00_KA.dwg" received 3-7-06.
2. Vertical datum: NAVD88.

LEGEND

NB-282 Current Project Boring Designation

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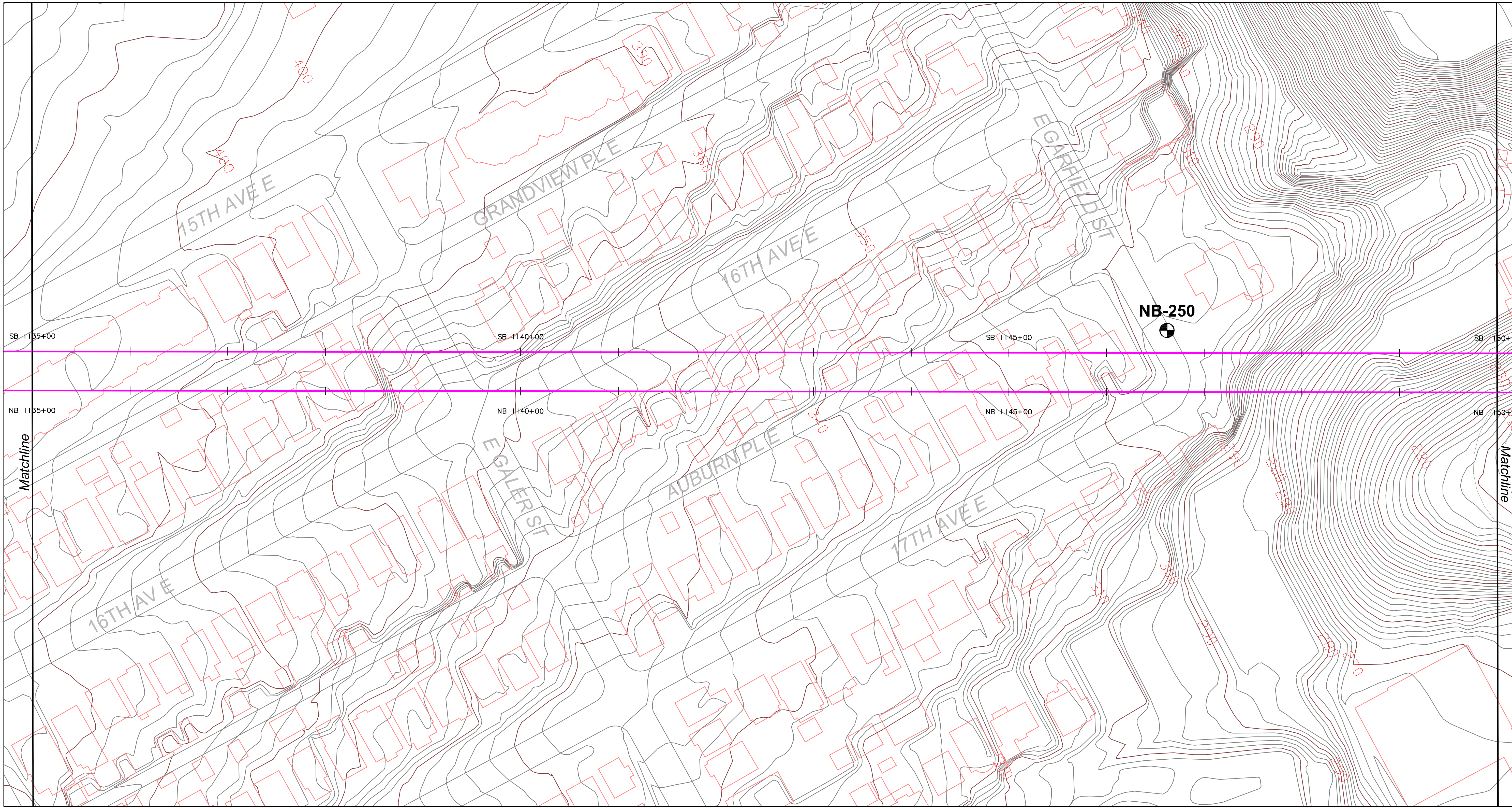
FIG. 2
Sheet 7 of 13

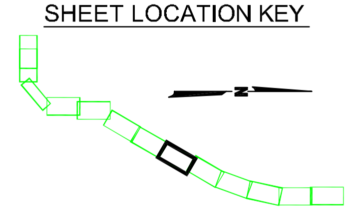
File: J:\211\08109-074\21-1-08109-074 Site Plan.dwg Date: 03-28-2006 Author: SAC



STATIONING BETWEEN MATCHLINES

NB 1210+00 to 1225+00

SHEET LOCATION KEY



0 100 200

Scale in Feet

NOTES

1. Figure based on electronic files provided by PSTC. Alignment "N35_L00_KA.dwg" received 3-7-06.
2. Vertical datum: NAVD88.

LEGEND

NB-282	Current Project Boring Designation
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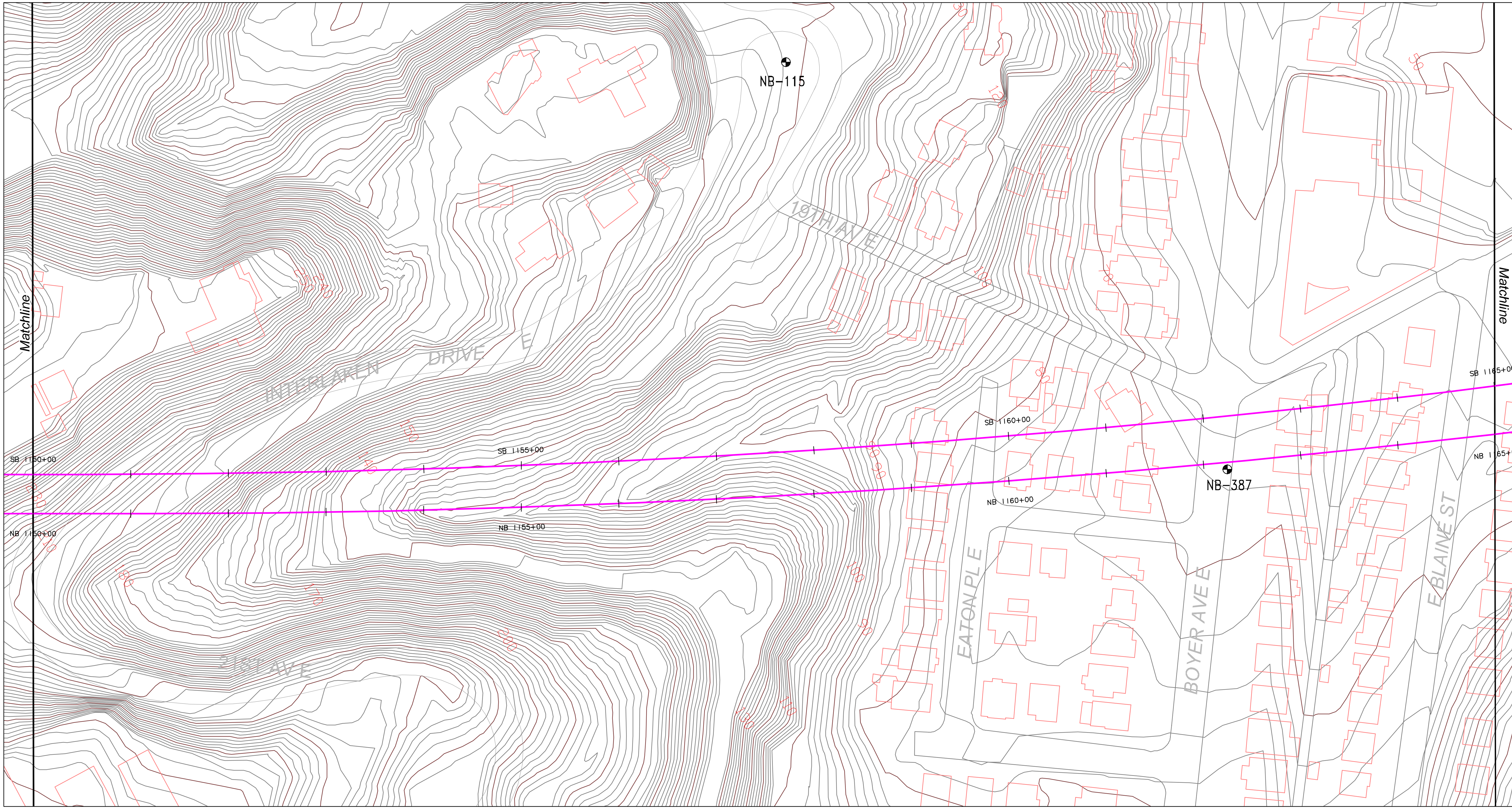
FIG. 2
Sheet 8 of 13

File: J:\211\08109-074\21-1-08109-074 Site Plan.dwg Date: 03-28-2006 Author: SAC



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NB 1210+00 to 1225+00

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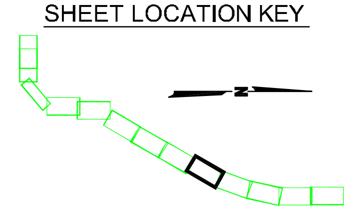

0 100 200

Scale in Feet

NOTES

1. Figure based on electronic files provided by PSTC. Alignment "N35_L00_KA.dwg" received 3-7-06.
2. Vertical datum: NAVD88.

LEGEND

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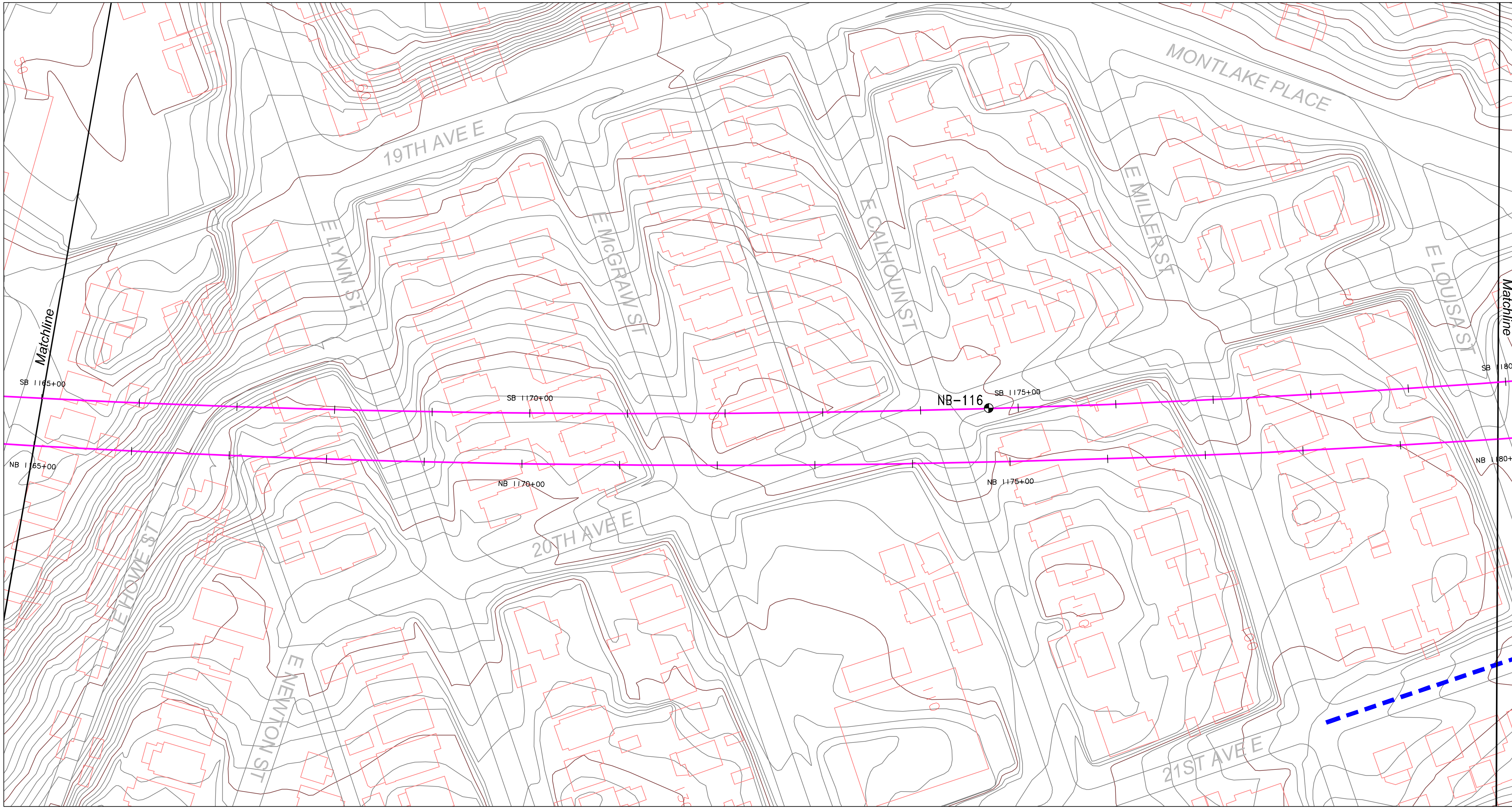
FIG. 2
Sheet 9 of 13

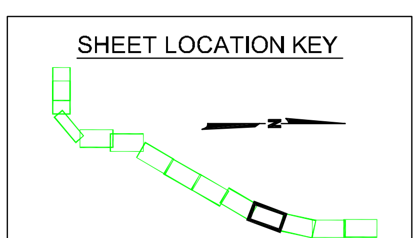
File: J:\211\08109-074\21-1-08109-074 Site Plan.dwg Date: 03-28-2006 Author: SAC

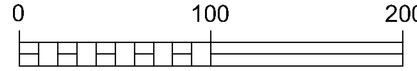


STATIONING BETWEEN MATCHLINES

NB 1210+00 to 1225+00

SHEET LOCATION KEY





Scale in Feet

NOTES

1. Figure based on electronic files provided by PSTC. Alignment "N35_L00_KA.dwg" received 3-7-06.
2. Vertical datum: NAVD88.

LEGEND

NB-282	Current Project Boring Designation
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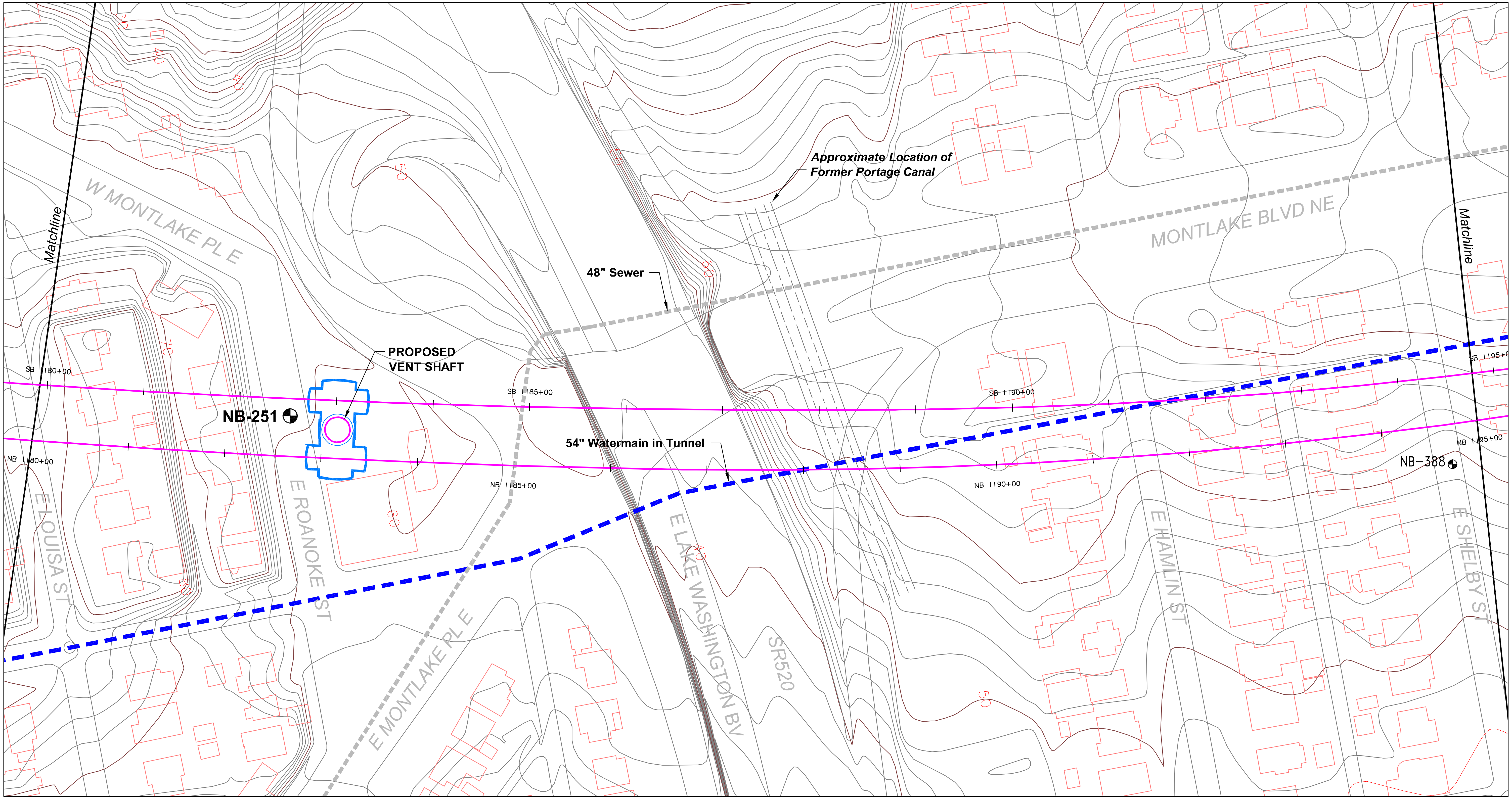
FIG. 2
Sheet 10 of 13

File: J:\211\08109-074\21-1-08109-074 Site Plan.dwg Date: 03-28-2006 Author: SAC



STATIONING BETWEEN MATCHLINES

NB 1210+00 to 1225+00

SHEET LOCATION KEY

0 100 200

Scale in Feet

NOTES

1. Figure based on electronic files provided by PSTC. Alignment "N35_L00_KA.dwg" received 3-7-06.
2. Vertical datum: NAVD88.

LEGEND

NB-282	Current Project Boring Designation
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FIG. 2
Sheet 11 of 13

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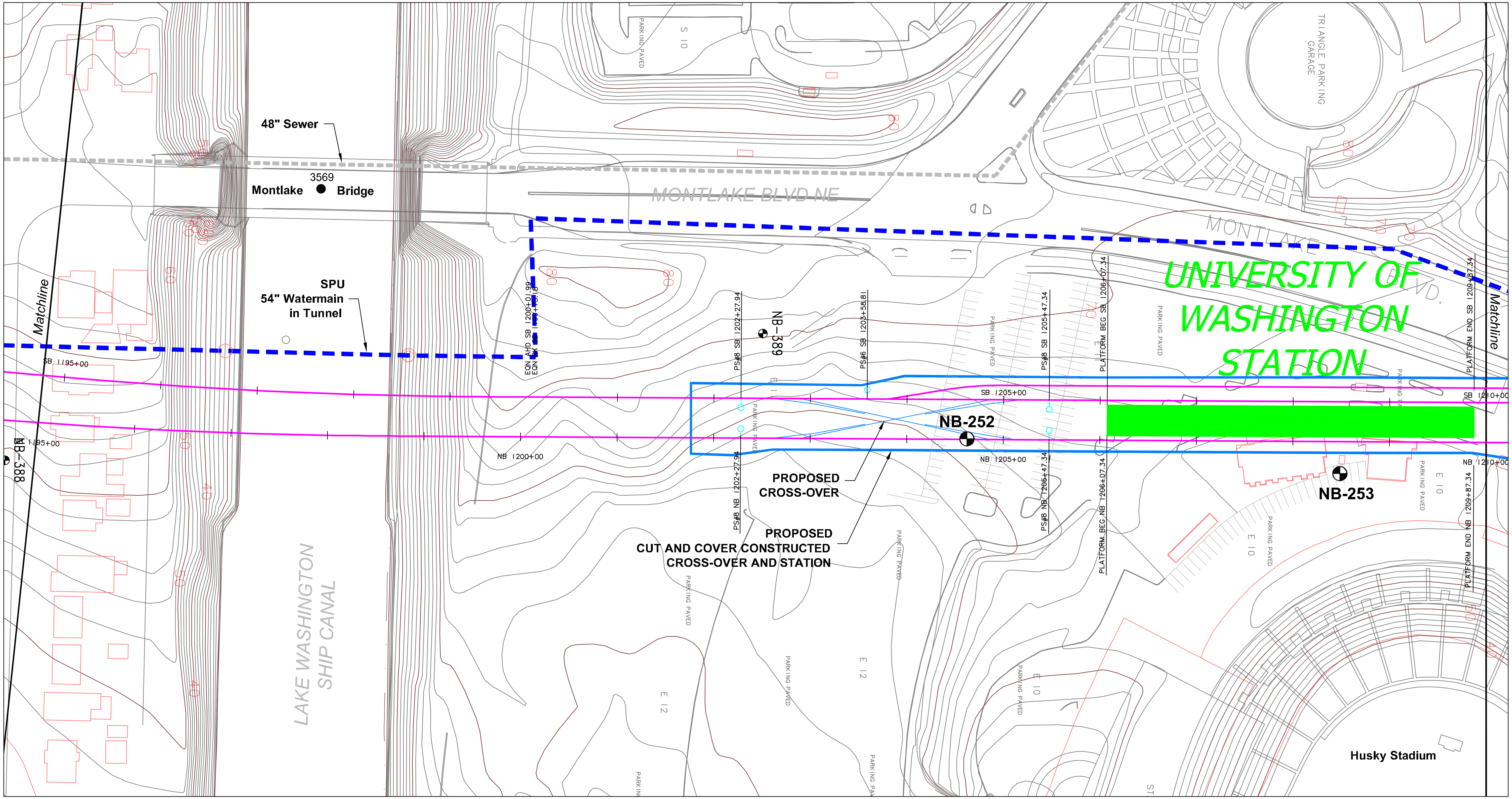
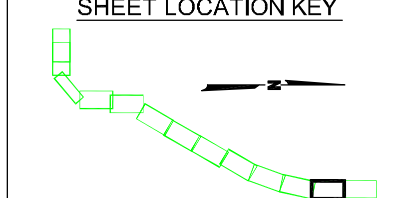


STATIONING BETWEEN MATCHLINES

NB 1210+00 to 1225+00

SHEET LOCATION KEY



0 100 200

Scale in Feet

NOTES

1. Figure based on electronic files provided by PSTC. Alignment "N35_L00_KA.dwg" received 3-7-06.
2. Vertical datum: NAVD88.

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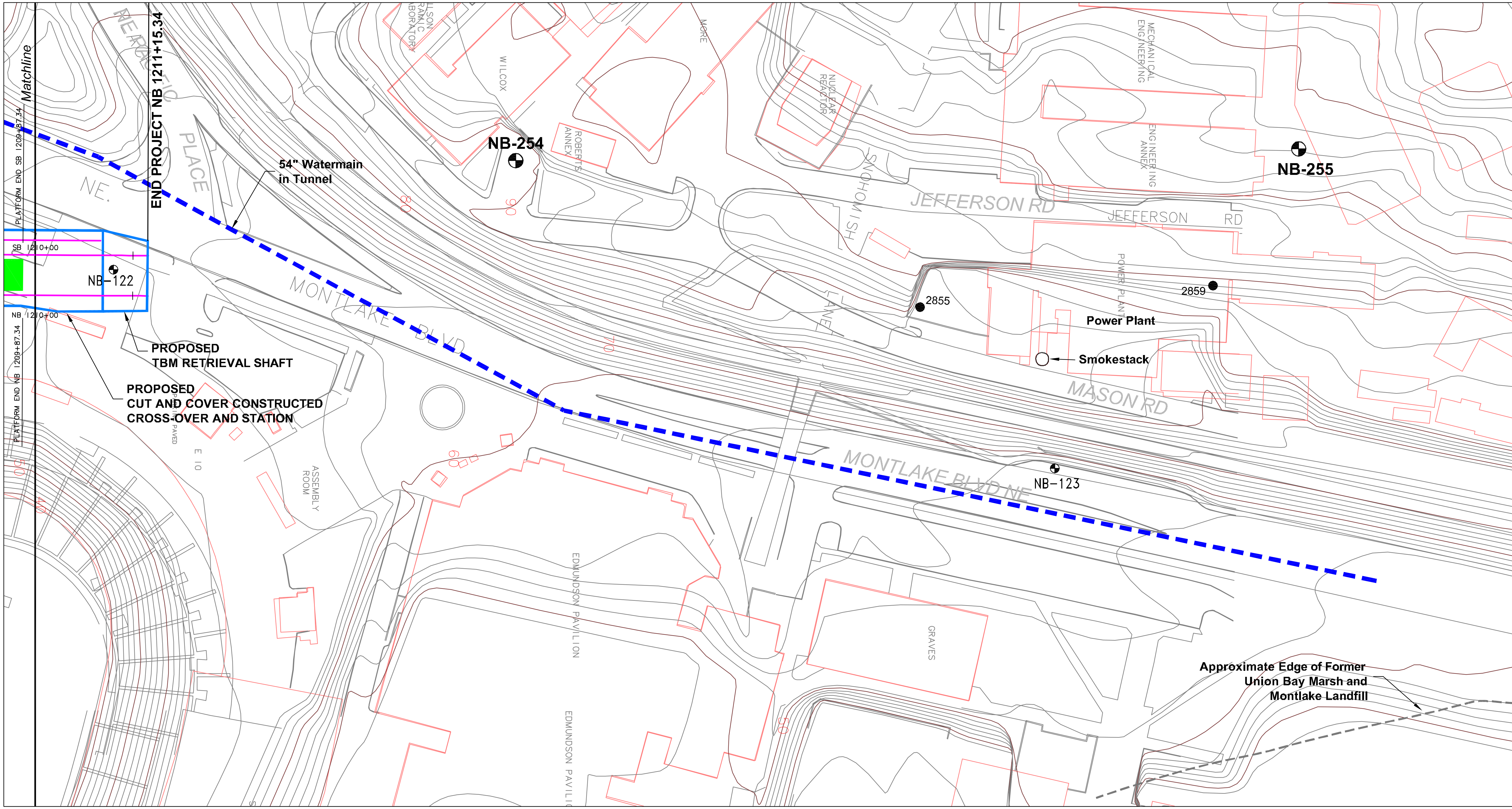
FIG. 2
Sheet 12 of 13

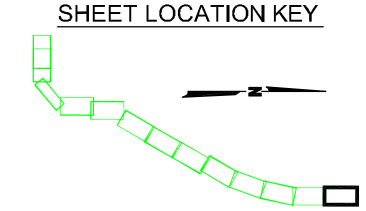
File: J:\211\08109-074\21-1-08109-074 Site Plan.dwg Date: 03-31-2006 Author: SAC



STATIONING BETWEEN MATCHLINES

NB 1210+00 to 1225+00

SHEET LOCATION KEY



0 100 200

Scale in Feet

NOTES

1. Figure based on electronic files provided by PSTC. Alignment "N35_L00_KA.dwg" received 3-7-06.
2. Vertical datum: NAVD88.

LEGEND

NB-282	Current Project Boring Designation
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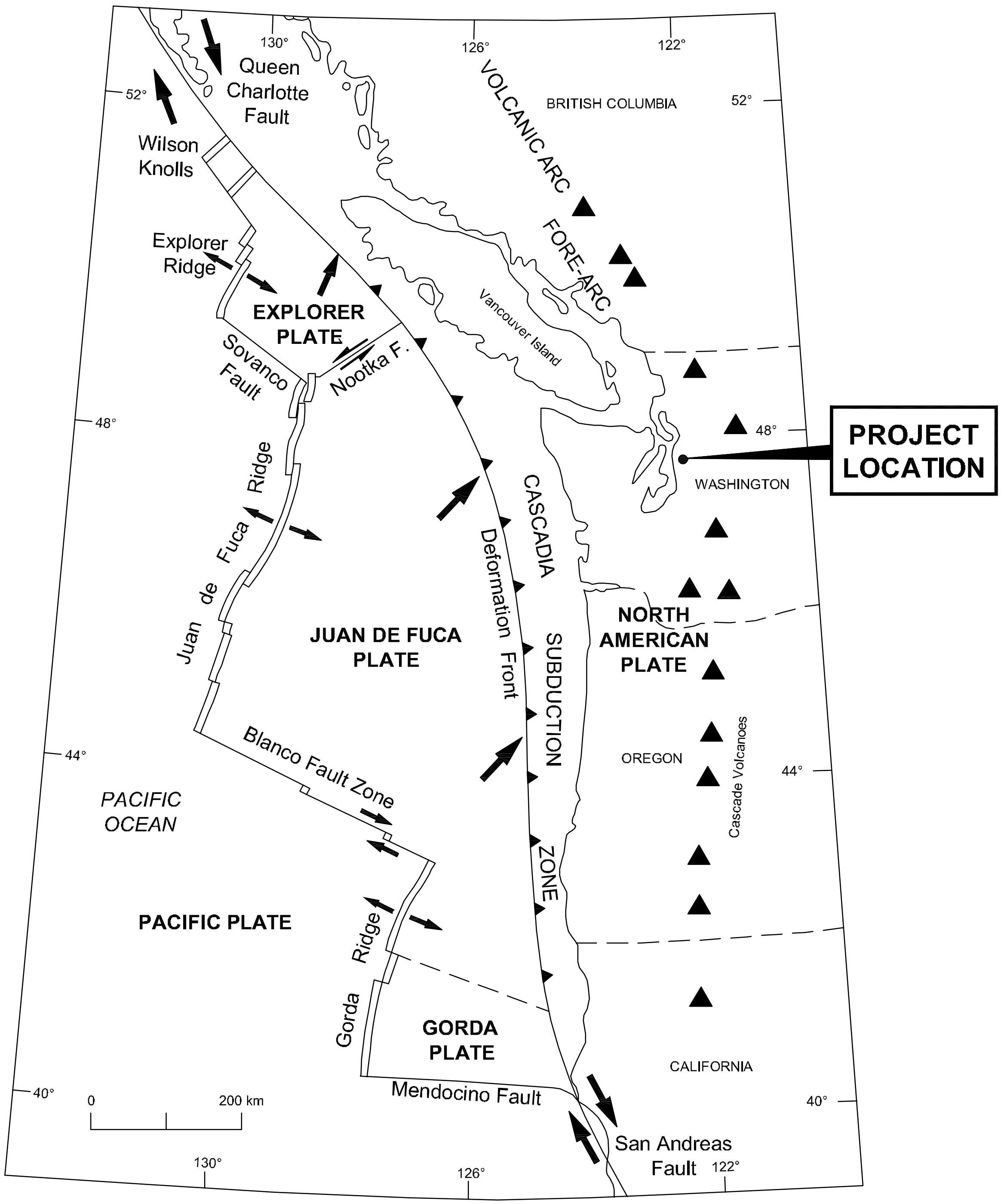
FIG. 2
Sheet 13 of 13

File: J:\211\08109-074\21-1-08109-074 Site Plan.dwg Date: 03-29-2006 Author: SAC



LEGEND

Loading Edge of Deformation Front (Sawtooth on Upper Plate)

Active Volcano

NOTE

Map based on Hyndman and Wang (1993).

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REGIONAL TECTONIC MAP

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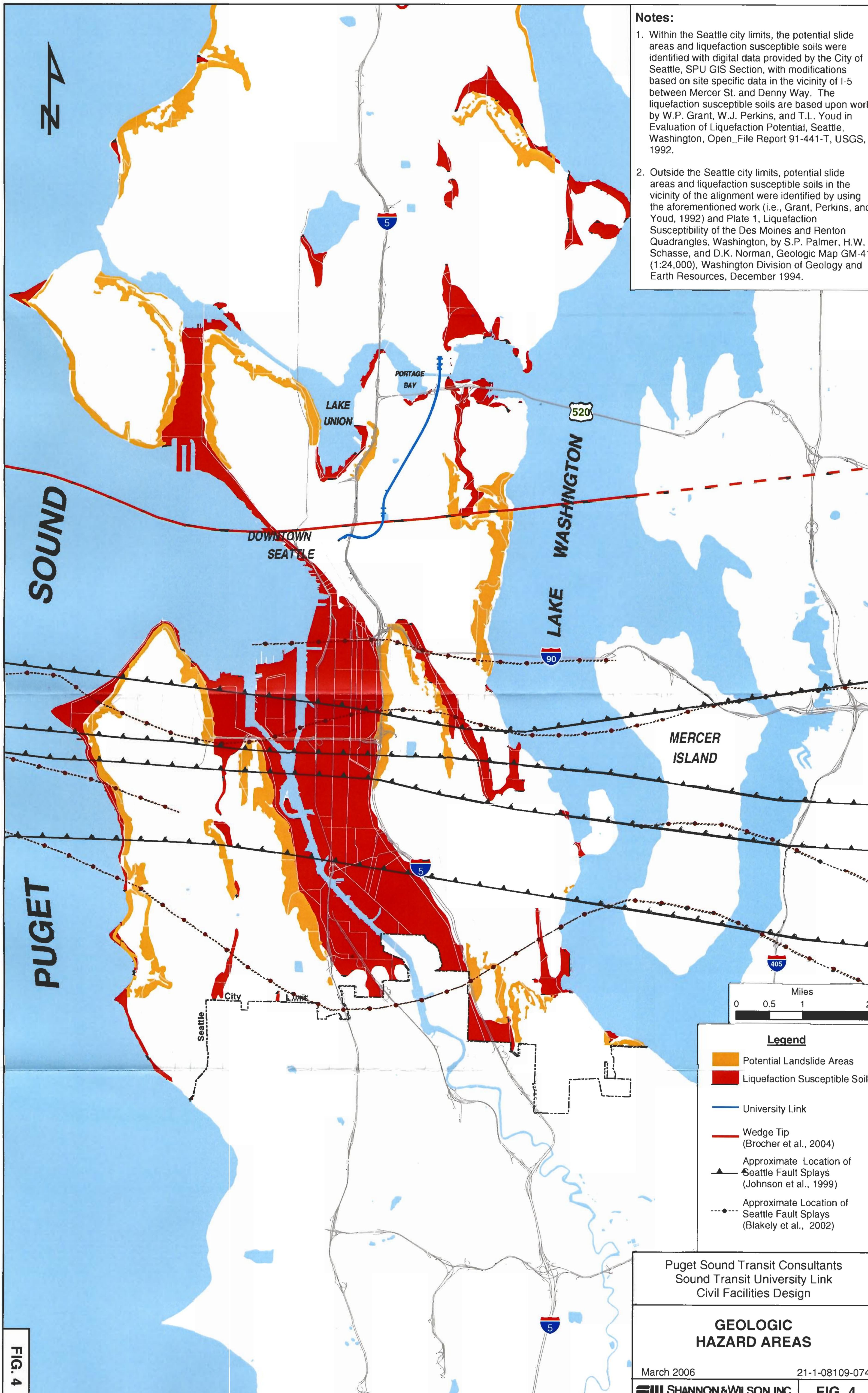
FIG. 3

File: J:\211\08109-074\21-1-08109-074 Fig 03.dwg Date: 03-28-2006 Author: SAC


N
LAKE UNION
PORTAGE BAY
DOWNTOWN SEATTLE
PUGET
SOUND
LAKE
WASHINGTON
MERCER ISLAND
Seattle City Limit
5
520
90
405
Miles
0 0.5 1 2
Notes:
1. Within the Seattle city limits, the potential slide areas and liquefaction susceptible soils were identified with digital data provided by the City of Seattle, SPU GIS Section, with modifications based on site specific data in the vicinity of I-5 between Mercer St. and Denny Way. The liquefaction susceptible soils are based upon work by W.P. Grant, W.J. Perkins, and T.L. Youd in Evaluation of Liquefaction Potential, Seattle, Washington, Open_File Report 91-441-T, USGS, 1992.
2. Outside the Seattle city limits, potential slide areas and liquefaction susceptible soils in the vicinity of the alignment were identified by using the aforementioned work (i.e., Grant, Perkins, and Youd, 1992) and Plate 1, Liquefaction Susceptibility of the Des Moines and Renton Quadrangles, Washington, by S.P. Palmer, H.W. Schasse, and D.K. Norman, Geologic Map GM-41 (1:24,000), Washington Division of Geology and Earth Resources, December 1994.
Legend
Potential Landslide Areas
Liquefaction Susceptible Soils
University Link
Wedge Tip (Brocher et al., 2004)
Approximate Location of Seattle Fault Splays (Johnson et al., 1999)
Approximate Location of Seattle Fault Splays (Blakely et al., 2002)
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GEOLOGIC HAZARD AREAS
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21-1-08109-074
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FIG. 4
FIG. 4

GEOLOGIC EXPLANATION

HOLOCENE DEPOSITS

Unit	Description
Hf	FILL: Fill placed by humans, both engineered and nonengineered Various materials, including debris; cobbles and boulders common; commonly dense or stiff if engineered, but very loose to dense or very soft to stiff if nonengineered
Hh	HYDRAULIC FILL: Fill placed by dredging from river or bay or sluiced into place from adjacent hills Clay and Silt; very soft to medium stiff (from hills); Silt and fine Sand, scattered shells; very loose to medium dense (not from hills)
Hc	COLLUVIUM: Hillside slope accumulations due to gravity emplacement Disturbed, heterogeneous mixture of several soils types, including organic debris; loose or soft
Hls	LANDSLIDE DEPOSITS: Deposits of landslides, normally at and adjacent to the toe of slopes Disturbed, heterogeneous mixture of several soil types; loose or soft, with random dense or hard pockets
Ha	ALLUVIUM: River or creek deposits, normally associated with historic streams, including overbank deposits Sand, silty Sand, gravelly Sand; very loose to very dense
Hp	PEAT DEPOSITS: Depression fillings of organic materials Peat, peaty Silt, organic Silt; very soft to medium stiff
He	ESTUARINE DEPOSITS: Estuary deposits of the ancestral Duwamish River Silty Clay and fine Sand; very soft to very stiff or loose to dense
Hl	LAKE DEPOSITS: Depression fillings of fine-grained soils Silt, clayey Silt, silty Clay; commonly with scattered organics; very soft to stiff or very loose to medium dense
Hb	BEACH DEPOSITS: Deposits along present and former shorelines of Puget Sound and tributary river mouths Silty Sand, sandy Gravel, Sand, scattered fine gravel, organic debris; loose to medium dense
Hrw	REWORKED GLACIAL DEPOSITS: Glacially deposited soils that have been reworked by fluvial or wave action Heterogenious mixture of several soil types; lies on top of glacially overridden soils; loose to dense

QUATERNARY VASHON DEPOSITS

Unit	Description
Qvro	RECESSIONAL OUTWASH DEPOSITS: Glaciofluvial sediment deposited as glacial ice retreated Clean to silty Sand, gravelly Sand, sandy Gravel; cobbles and boulders common; loose to very dense
Qvrl	RECESSIONAL LACUSTRINE DEPOSITS: Glaciolacustrine sediment deposited as glacial ice retreated Fine Sand, Silt, and Clay; dense to very dense, soft to hard
Qvri	ICE-CONTACT DEPOSITS: Heterogeneous soils deposited against or adjacent to ice during the wasting of glacial ice; commonly reworked Stratified to irregular bodies of Gravel, Sand, Silt, and Clay; loose to dense
Qvat	ABLATION TILL: Heterogeneous soils deposited during the wasting of glacial ice; generally not reworked Gravelly silty Sand, silty gravelly Sand, with some clay; cobbles and boulders common; loose to very dense or soft to hard
Qvt	TILL: Lodgment till laid down along the base of the glacial ice Gravelly silty Sand, silty gravelly Sand ("hardpan"); cobbles and boulders common; very dense
Qvd	TILL-LIKE DEPOSITS (DIAMICT): Glacial deposit intermediate between till and outwash; subglacially reworked Silty gravelly Sand, silty Sand, sandy Gravel; highly variable over short distances; cobbles and boulders common; dense to very dense
Qva	ADVANCE OUTWASH: Glaciofluvial sediment deposited as the glacial ice advanced through the Puget Lowland Clean to silty Sand, gravelly Sand, sandy Gravel; dense to very dense
Qvgl	GLACIOLACUSTRINE DEPOSITS: Fined-grained glacial flour deposited in proglacial lake in Puget Lowland Silty clay, Clayey Silt, with interbeds of Silt and fine Sand; locally laminated; scattered organic fragments near base; hard or dense to very dense
Qvgm	GLACIOMARINE DEPOSITS: Till-like deposit with clayey matrix deposited in proglacial lake by icebergs, floating ice, and gravity currents Heterogeneous and variable mixture of of Clay, Silt, Sand, and Gravel; rare shells; cobbles and boulders common; very dense or hard

QUATERNARY PRE-VASHON DEPOSITS

Unit	Description
Qpnf	FLUVIAL DEPOSITS: Alluvial deposits of rivers and creeks Clean to silty Sand, gravelly Sand, sandy Gravel; very dense
Qpnl	LACUSTRINE DEPOSITS: Fine-grained lake deposits in depressions, large and small Fine sandy Silt, silty fine Sand, clayey Silt; scattered to abundant fine organics; dense to very dense or very stiff to hard
Qpnp	PEAT DEPOSITS: Depression fillings of organic materials Peat, peaty Silt, organic Silt; hard
Qpns	PALEOSOL: Buried weathered horizon Clay-rich with various amounts of clastic debris; commonly contains organic material; typically greenish in color; hard or very dense
Qpls	LANDSLIDE DEPOSITS: Heterogeneous deposits of landslide debris Chaotically bedded silt, sand,clay and gravel; may contain wood and other organics; hard or very dense
Qpgo	OUTWASH: Glaciofluvial sediment deposited as the glacial ice advanced through the Puget Lowland Clean to silty Sand, gravelly Sand, sandy Gravel; very dense
Qpgl	GLACIOLACUSTRINE DEPOSITS: Fine-grained glacial flour deposited in proglacial lake in Puget Lowland Silty Clay, clayey Silt, with interbeds of Silt and fine Sand; very stiff to hard or very dense
Qpgt	TILL: Lodgment till laid down along the base of the glacial ice Gravelly silty Sand, silty gravelly Sand ("hardpan"); cobbles and boulders common; very dense
Qpgd	TILL-LIKE DEPOSITS (DIAMICT): Glacial deposit intermediate between till and outwash; subglacially reworked Silty gravelly Sand, silty Sand, sandy Gravel; highly variable over short distance; cobbles and boulders common; very dense
Qpgm	GLACIOMARINE DEPOSITS: Till-like deposit with clayey matrix deposited in proglacial lake by icebergs, floating ice, and gravity currents Heterogeneous and variable mixture of of Clay, Silt, Sand, and Gravel; rare shells; cobbles and boulders common; very dense or hard
	Overprint indicates that Qvgl or Qpgl has Qpnl-like seams and layers

TERTIARY BEDROCK

Unit	Description
Tsi	SILTSTONE: Siltstone, sandy Siltstone, commonly tuffaceous
Tss	SANDSTONE: Sandstone, Silty Sandstone, commonly tuffaceous
Tcs	CLAYSTONE: Claystone, Silty Claystone, sandy Claystone, commonly tuffaceous
Tvc	VOLCANICLASTIC ROCKS: Tuff, Lapilli Tuff, Volcanic Breccia, Agglomerate
Tva	ANDESITE: Andesite and Basalt

NOTE

The description of each geologic unit includes only general information regarding the environment of deposition and basic soil characteristics. For example, cobbles and boulders are only included in the description of those units where they are most prominent. Futher details of each geologic unit are presented in the report.

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GEOLOGIC UNIT DESCRIPTION

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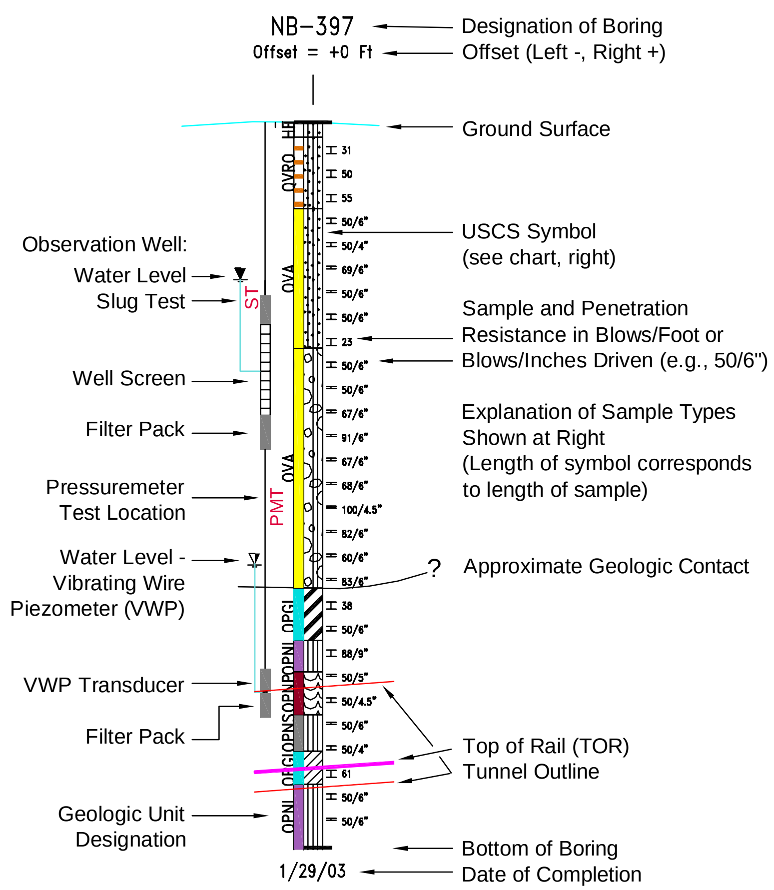
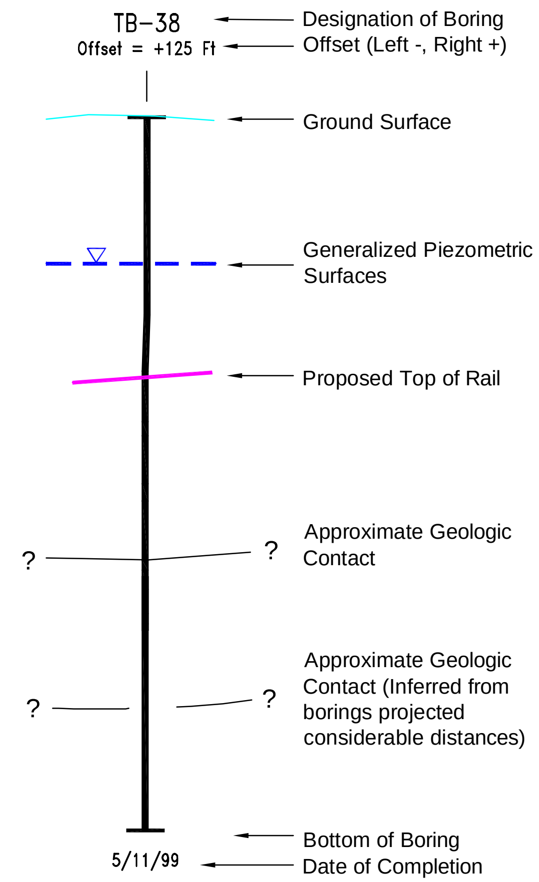
FIG. 5

File: J:\211\08109-074\21-1-08109-074 Fig 05.dwg Date: 03-28-2006 Author: SAC

BORING LOG LEGEND
(Project Borings)



BORING LOG LEGEND
(Non-Project Borings)



UNIFIED SOIL CLASSIFICATION SYSTEM
(From ASTM D 2488-93 & 2487-93)

GW	SM
GP	SC
GW-GM	CL
GP-GM	ML
GM	OL
GC	CH
SW	MH
SP	OH
SW-SM	PT
SP-SM	

1. Dual Symbols (symbols separated by a hyphen, i.e., SP-SM, slightly silty fine SAND) are used for soils with between 5% and 12% fines or when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart.
2. Borderline symbols (symbols separated by a slash, i.e., CL/ML, silty CLAY/clayey SILT; GW/SW, sandy GRAVEL/gravelly SAND) indicate that the soil may fall into one of two possible basic groups, based on ASTM D 2488-93 Visual Manual Classification System. The graphic symbol of only the first group symbol is shown on the profile.

SAMPLE TYPES

- * Sample Not Recovered
- 2" O.D. Split Spoon Sample with 140 lb. Hammer (standard penetration test - SPT)
- 2.5" O.D. Split Spoon Sample with 300 lb. Hammer (non-standard)
- 3" O.D. Split Spoon Sample with 300 lb. Hammer (non-standard)
- Sonic Coring Run
- 3" O.D. Shelby Tube Sample
- Osterberg Sample
- P Pitcher Barrel Sample
- 2.5" O.D. Thin Wall Tube Sample
- G Grab Sample
- Soil Coring Run

NOTES

1. The profiles are constructed from surface elevations based on the North American Vertical Datum 1988 (NAVD88). The geology shown is derived from borings conducted by Shannon & Wilson, Inc. for this study and from borings conducted by Shannon & Wilson and others for other studies. Elevations and geologic contacts should be considered approximate. Variations between the profile and actual conditions are likely to exist.
2. Detailed logs of the current project explorations are presented in Appendix A of the GDR. Water levels shown on current project borings were generally measured in December 2004. Water levels shown on previous project borings were measured at various dates. Groundwater fluctuations should be expected.
3. Tunnel alignment and grades were provided by PSTC on 3-7-06.
4. Piezometric surface lines were inferred between locations of groundwater measurement and are approximate. Water levels may fluctuate seasonally and may have changed since the last reading. Absence of piezometric surface lines along the alignment does not indicate the absence of groundwater; groundwater may be present in areas where no piezometric surface lines are shown.

NOMENCLATURE

Age	GEOLOGIC AGE DESIGNATION		DEPOSITIONAL ENVIRONMENT, GEOLOGIC PROCESS, OR LITHOLOGY		
Present	H = Holocene		f = fill h = hydraulic fill c = colluvium ls = landslide	a = alluvium p = peat e = estuarine	l = lucustrine (lake) b = beach rw = reworked glacial
13,500 yrs BP *	Q = Quaternary	v = Vashon	r = recessional	o = outwash l = lacustrine i = ice contact	
			at = ablation till t = till (lodgment) d = till-like (diamict) a = advance outwash	gm = glaciomarine gl = glaciolacustrine	
15,500 yrs BP *		p = Pre-Vashon 6 or more glacial and interglacial episodes	n = nonglacial (interglacial)	f = fluvial l = lacustrine	p = peat s = soil (paleosol)
			ls = landslide		
			g = glacial	o = outwash l = lacustrine t = till-like (diamict)	d = till-like m = marine
2,000,000 yrs BP	T = Tertiary		si = siltstone ss = sandstone	cs = claystone vc = volcaniclastics	

These radiometric (C^{14}) dates are based on data in Central Puget Lowland. Equivalent calendar years before present are approximately 15,000 and 18,000 yrs BP. These dates may differ from onset and end of Vashon (late Pleistocene) glacial episode in other parts of the Puget Lowland.

NOTE

The nomenclature graphic was created to explain the distinctions among geologic deposits in the Central Puget Lowland for engineering purposes, e.g. engineering properties of geologic deposits. The actual geologic designations and dates, according to internationally accepted stratigraphic rules, may be slightly different.

GEOLOGIC NOMENCLATURE

Each geologic unit has a two- to four-letter abbreviation composed of a leading capital letter signifying geologic age, followed by one or more lowercase letters indicating further breakdown of geologic age, depositional environment or geologic process.

LEGEND

Years BP — Radiocarbon Years Before Present (1950)

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GEOLOGIC PROFILE LEGEND AND NOTES

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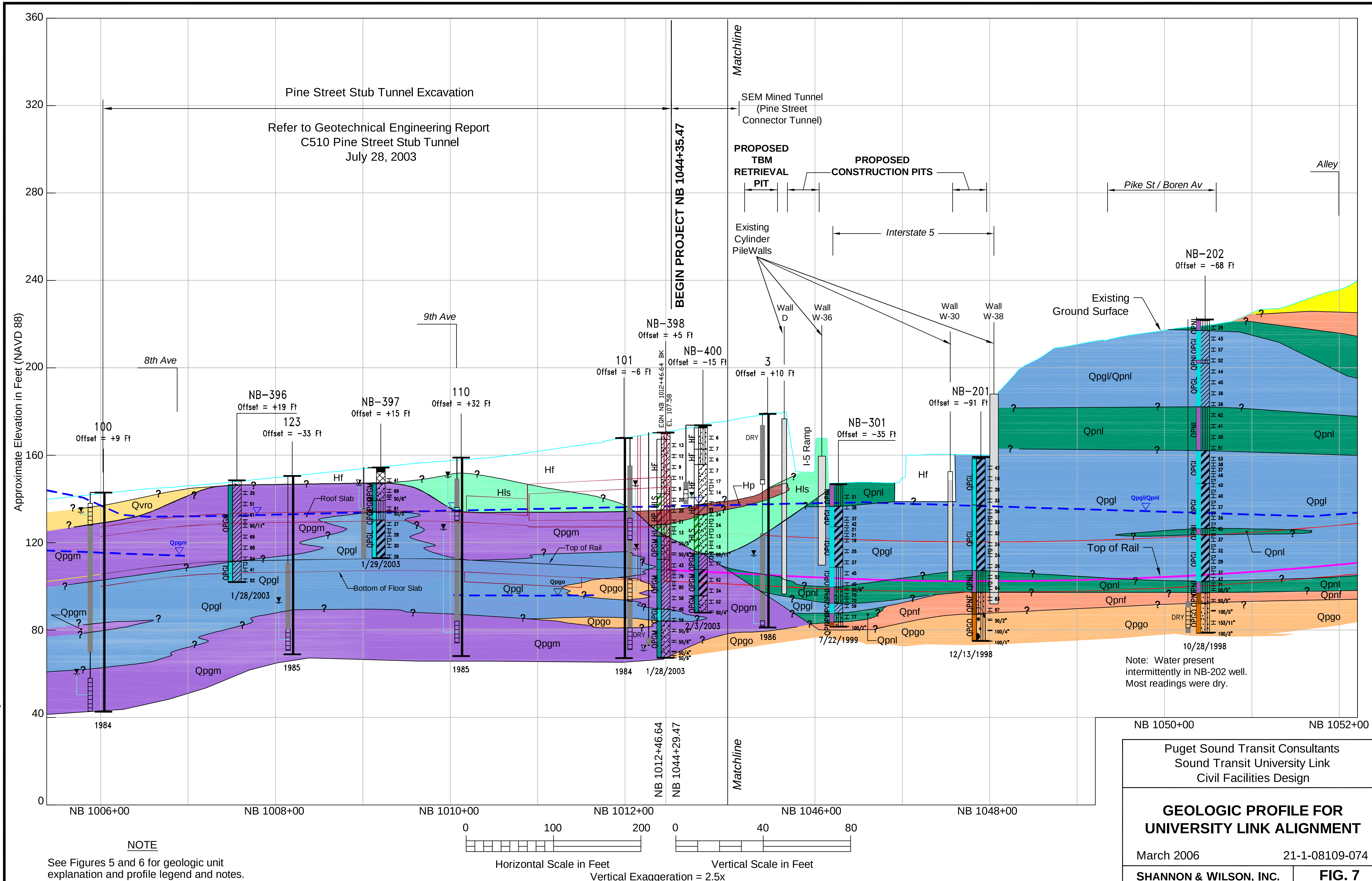
FIG. 6

File: J:\211\08109-074\21-1-08109-074 Fig 06.dwg Date: 03-28-2006 Author: SAC



NOTE

See Figures 5 and 6 for geologic unit explanation and profile legend and notes.

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FIG. 7
Sheet 1 of 13

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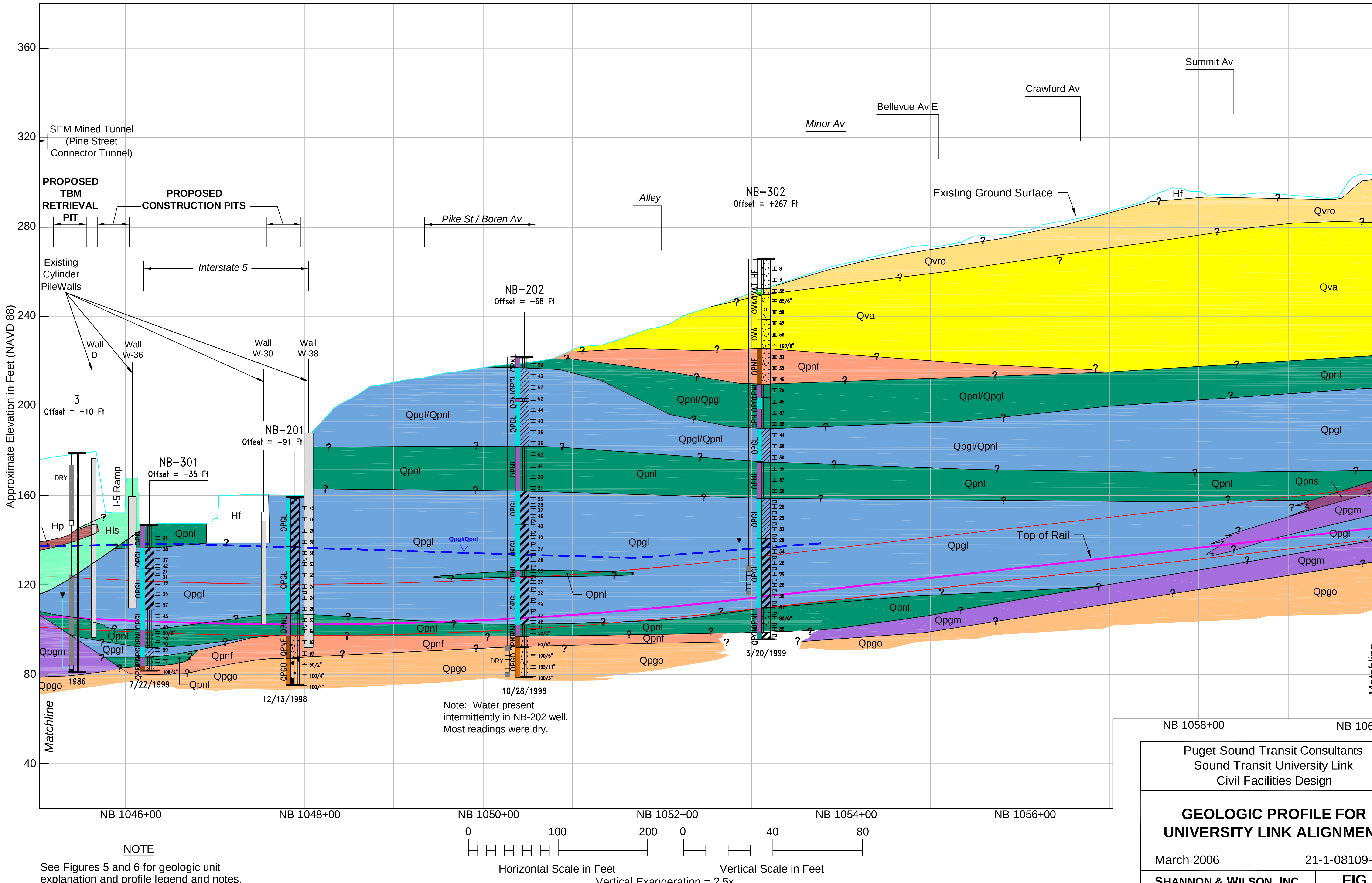

NOTE

See Figures 5 and 6 for geologic unit explanation and profile legend and notes.

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FIG. 7
Sheet 2 of 13

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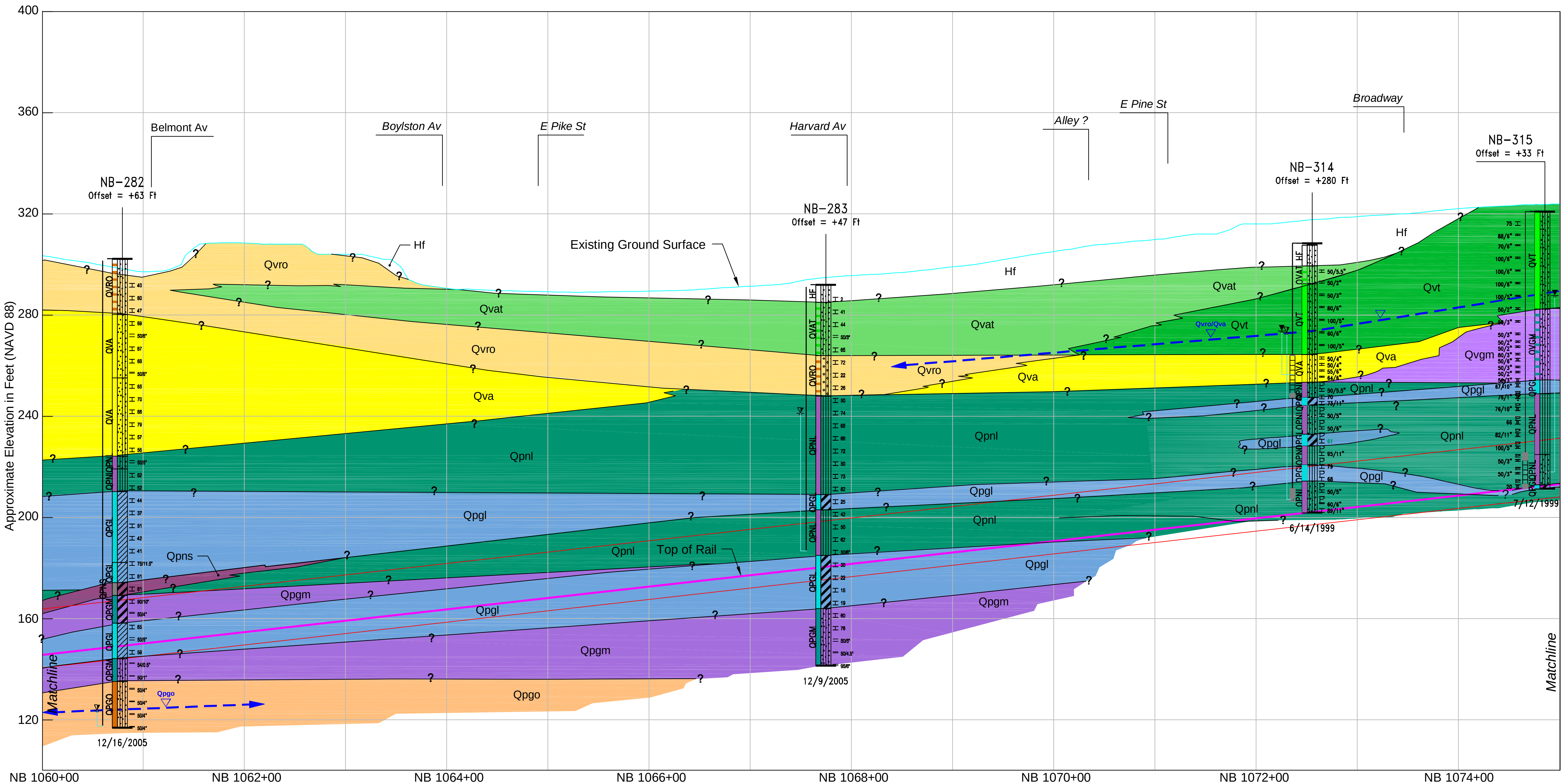

NOTE

See Figures 5 and 6 for geologic unit explanation and profile legend and notes.

0 100 200

Horizontal Scale in Feet

0 40 80

Vertical Scale in Feet

Vertical Exaggeration = 2.5x

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FIG. 7
Sheet 3 of 13

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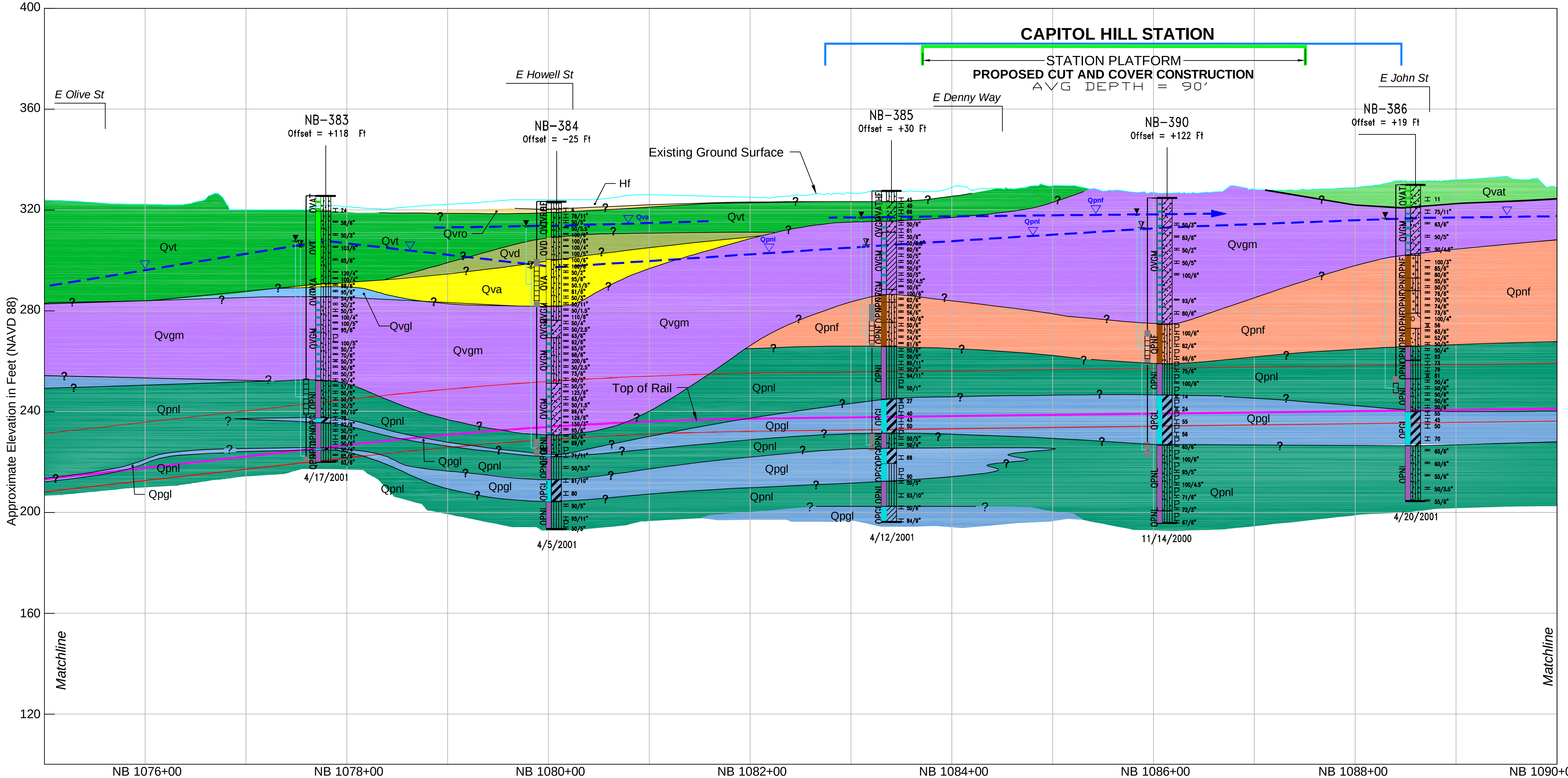

0 100 200
Horizontal Scale in Feet

0 40 80
Vertical Scale in Feet

Vertical Exaggeration = 2.5x

NOTE

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FIG. 7
Sheet 4 of 13

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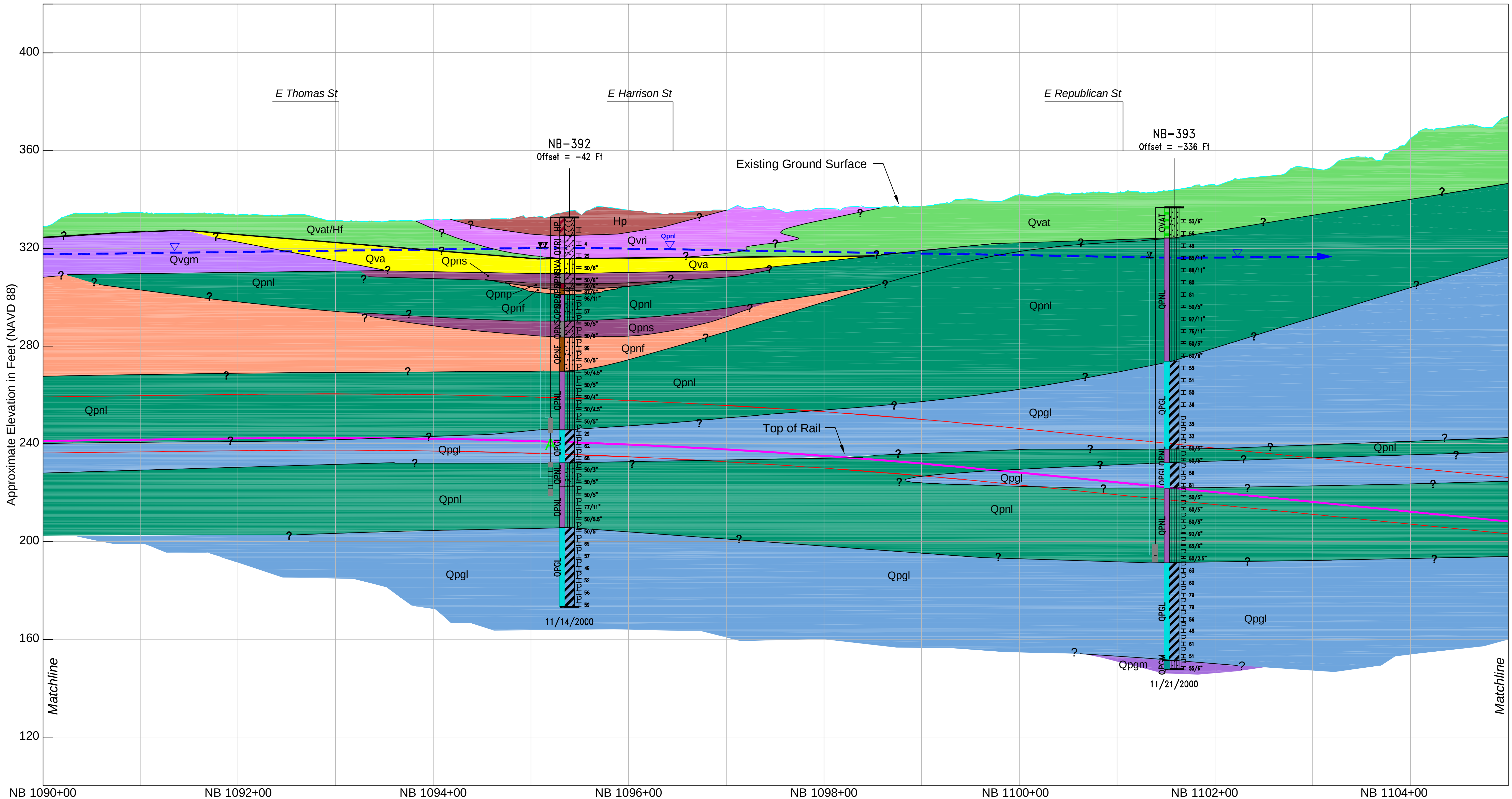

NOTE

See Figures 5 and 6 for geologic unit explanation and profile legend and notes.

0 100 200

Horizontal Scale in Feet

0 40 80

Vertical Scale in Feet

Vertical Exaggeration = 2.5x

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FIG. 7
Sheet 5 of 13

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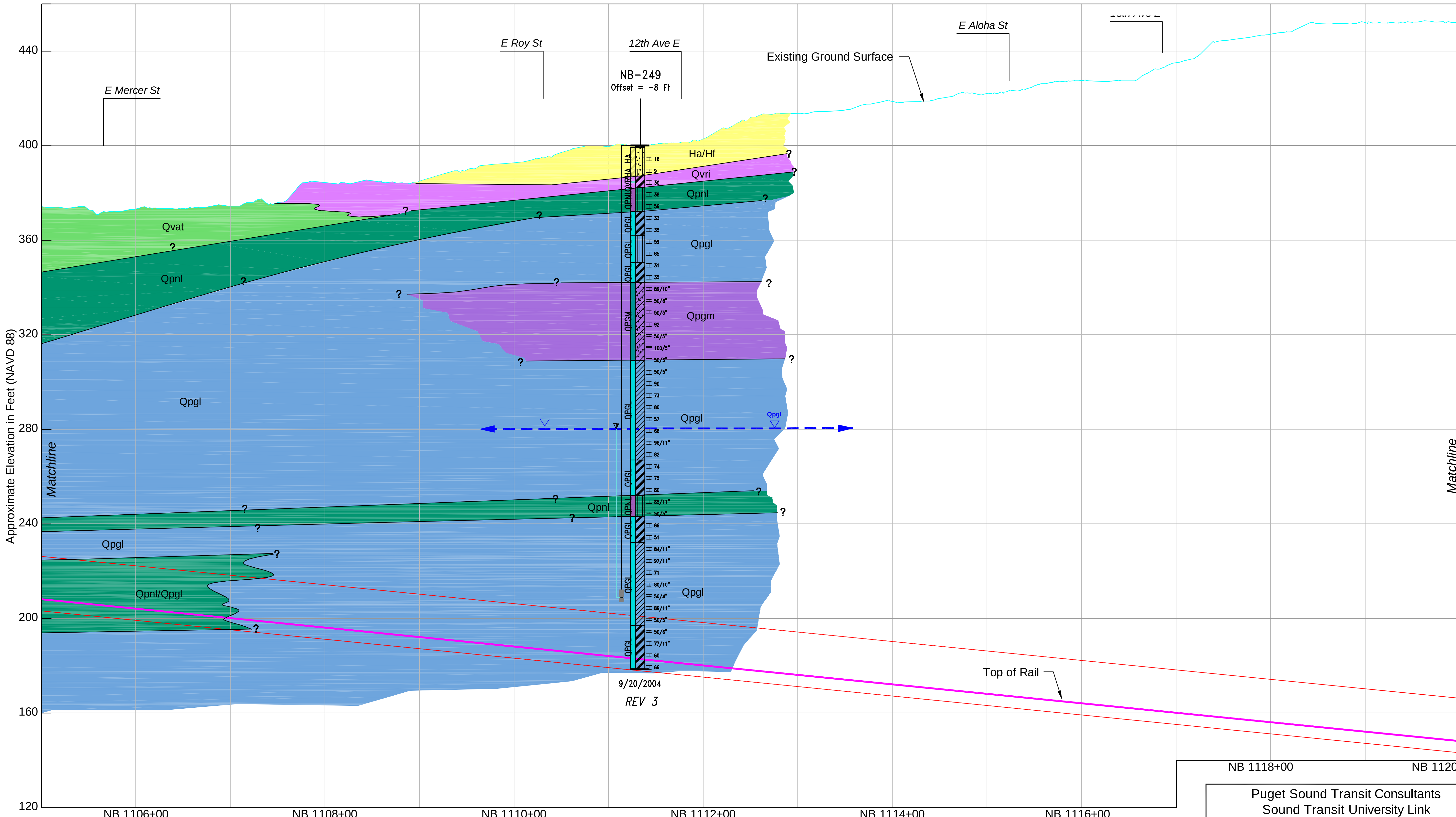

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NOTE

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0 100 200

Horizontal Scale in Feet

0 40 80

Vertical Scale in Feet

Vertical Exaggeration = 2.5x

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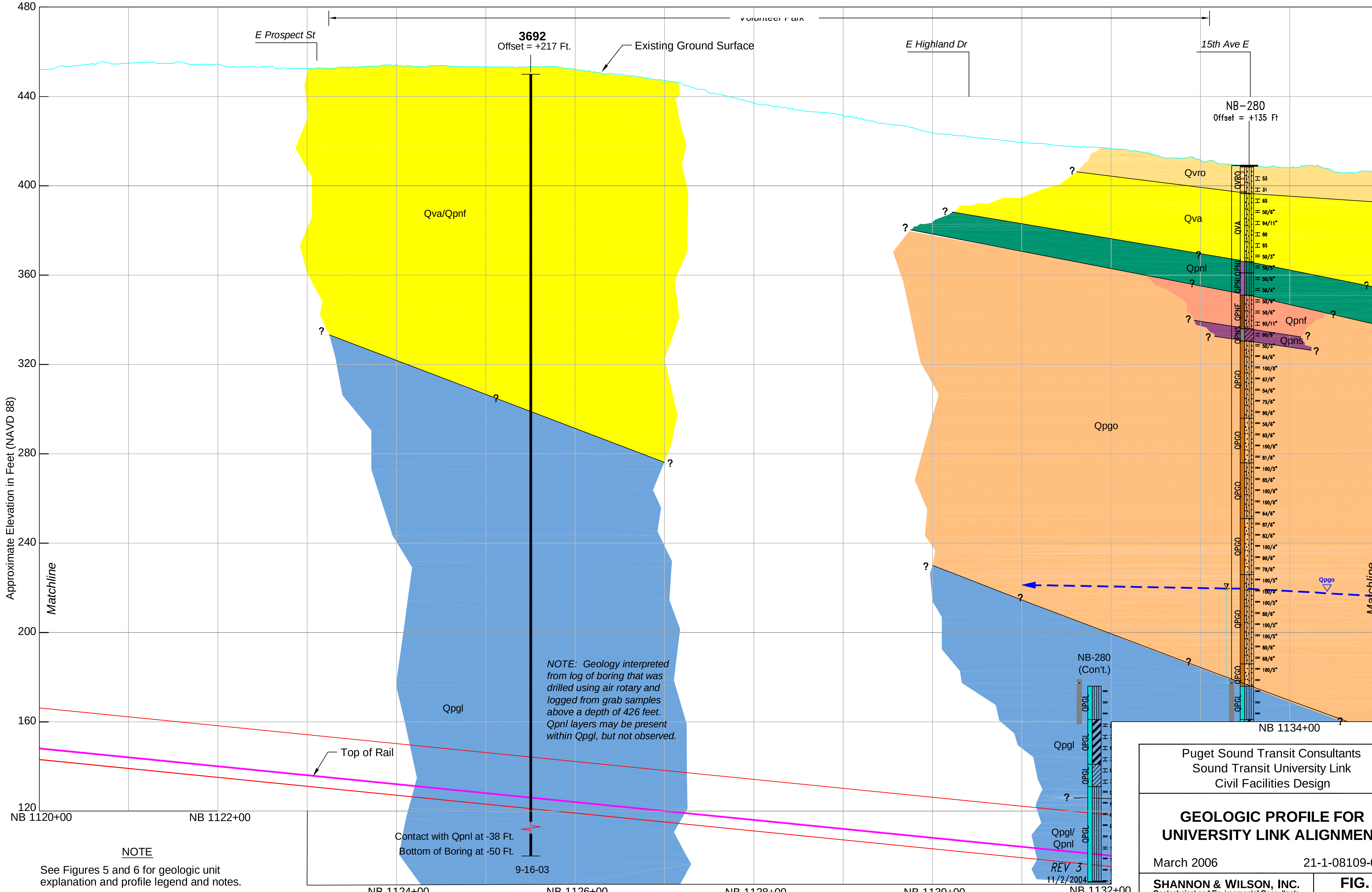
FIG. 7
Sheet 6 of 13



NOTE

See Figures 5 and 6 for geologic unit explanation and profile legend and notes.

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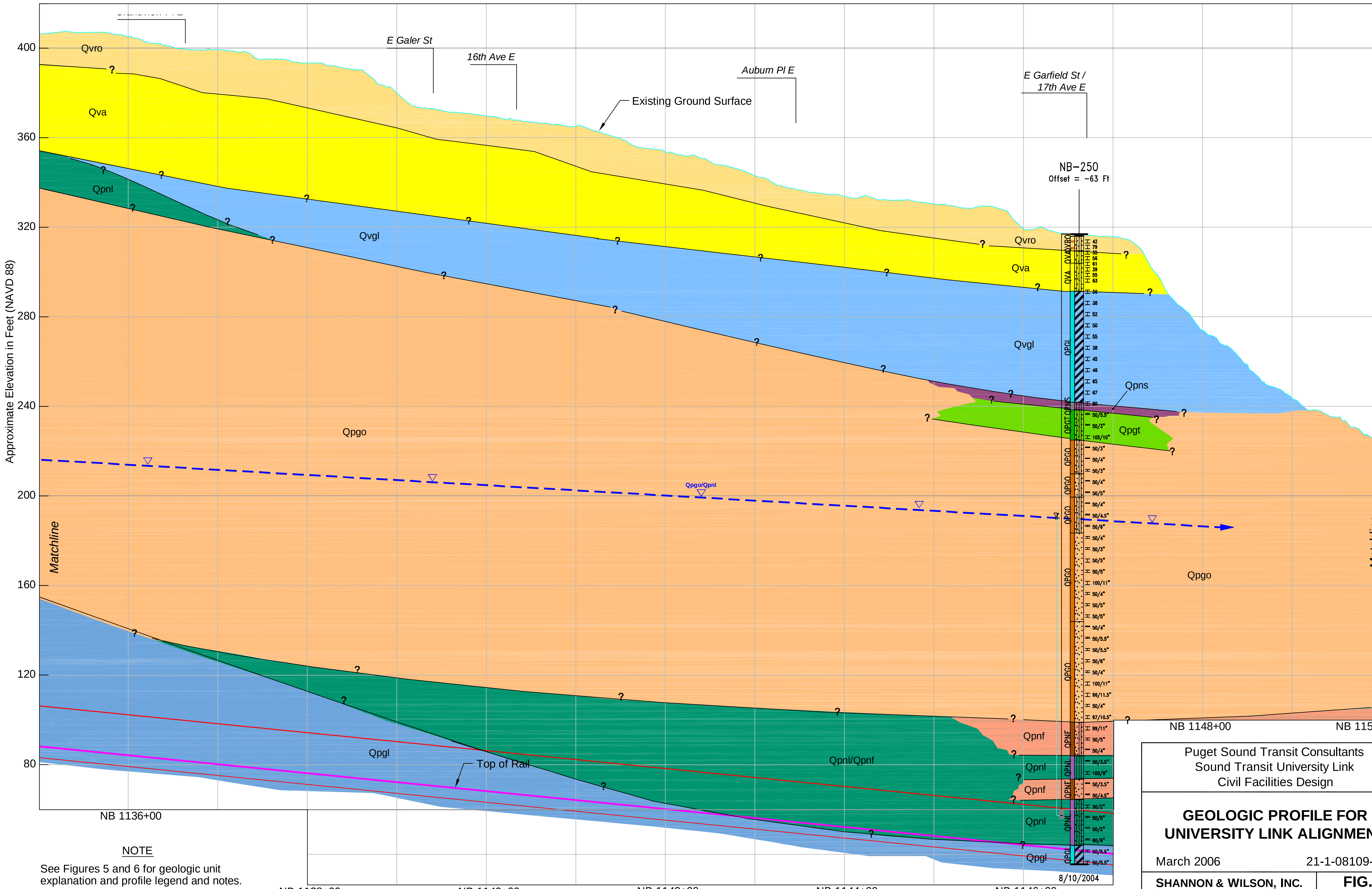
FIG. 7
Sheet 7 of 13



NOTE

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FIG. 7
Sheet 8 of 13

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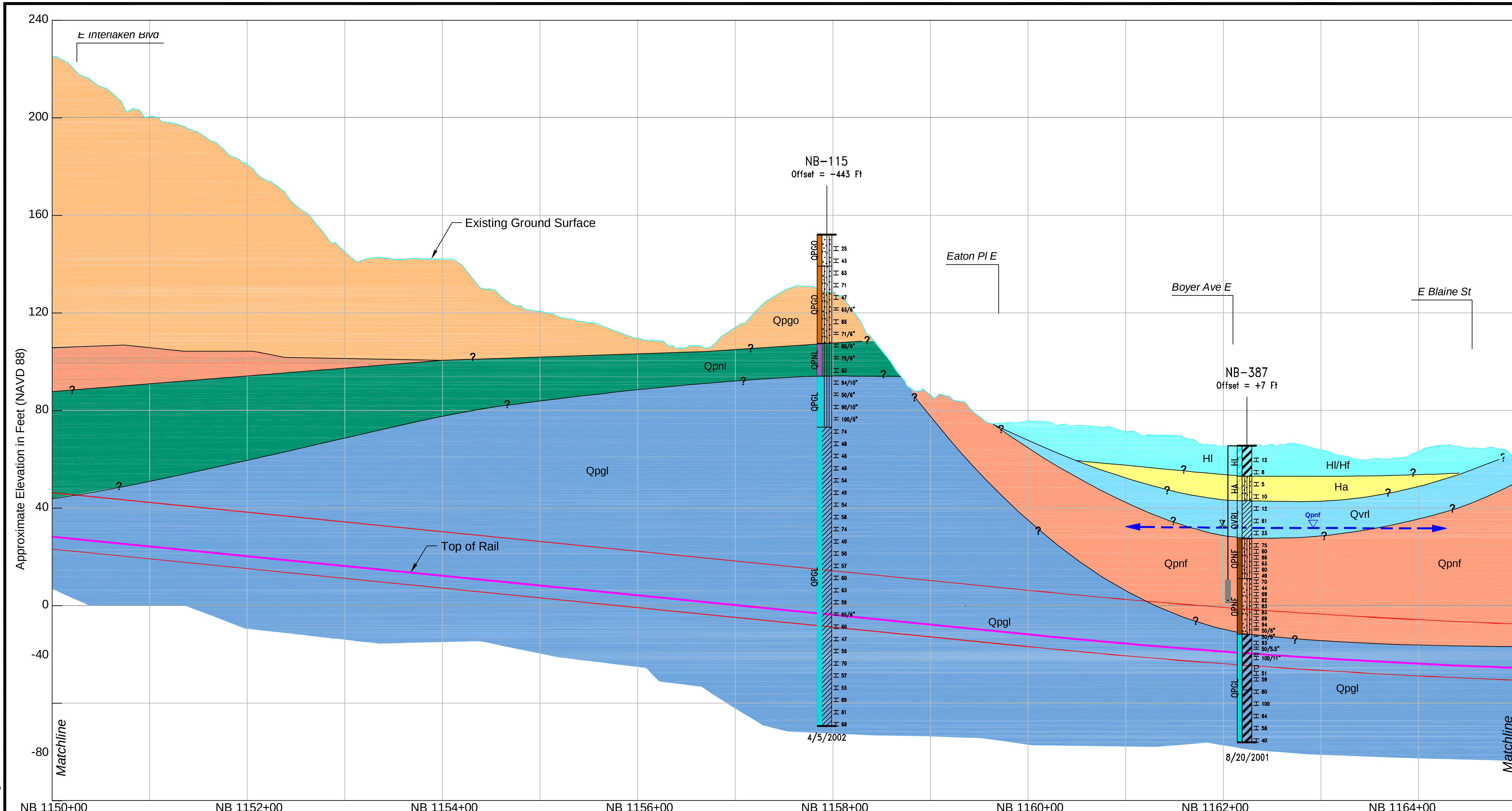

NOTE

See Figures 5 and 6 for geologic unit explanation and profile legend and notes.

0 100 200

Horizontal Scale in Feet

0 40 80

Vertical Scale in Feet

Vertical Exaggeration = 2.5x

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FIG. 7
Sheet 9 of 13

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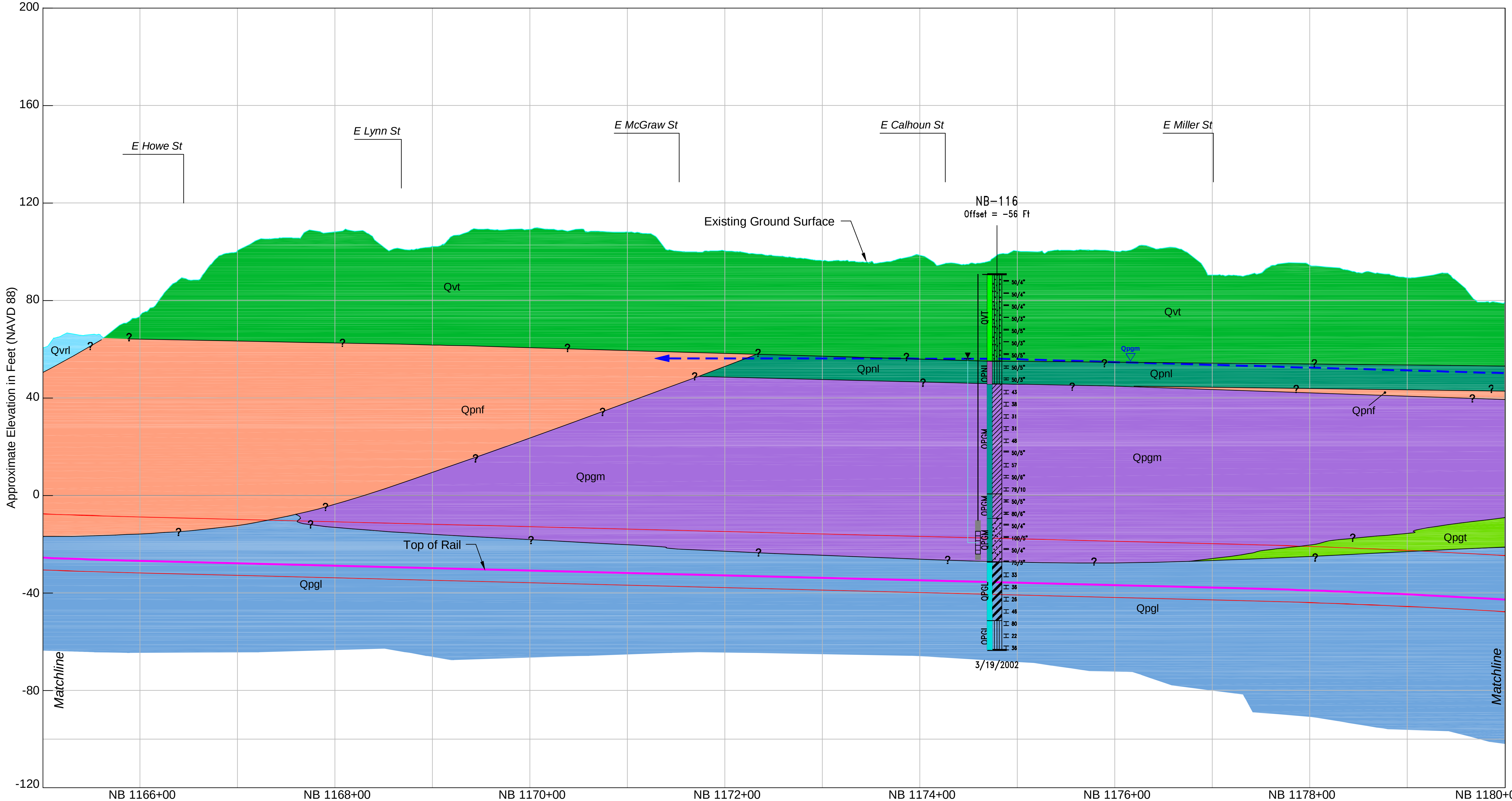

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FIG. 7
Sheet 10 of 13

NOTE

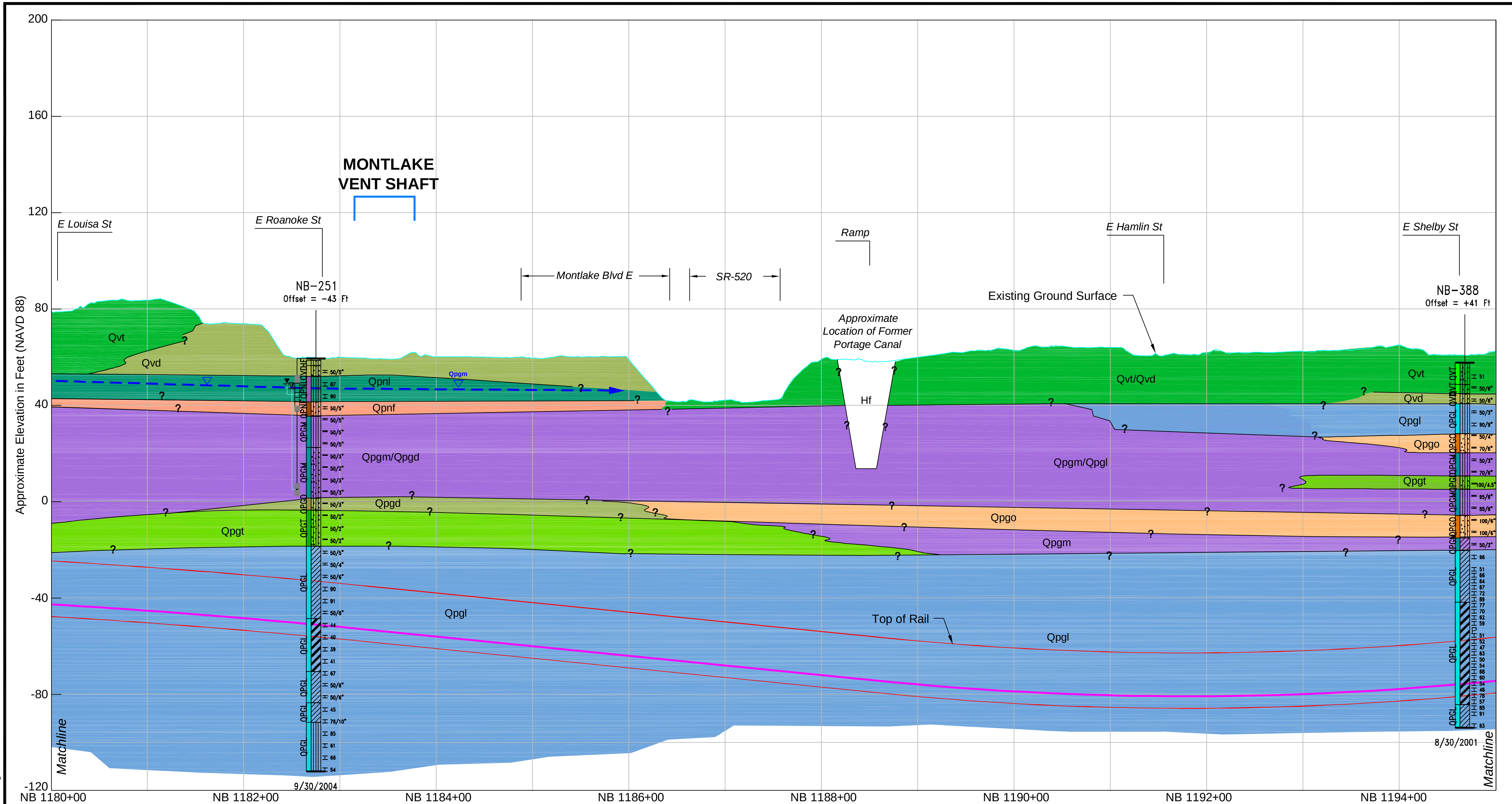
See Figures 5 and 6 for geologic unit explanation and profile legend and notes.

0 100 200
Horizontal Scale in Feet

0 40 80
Vertical Scale in Feet

Vertical Exaggeration = 2.5x

File: J:\211\08109-074\21-1-08109-074 Profile.dwg Date: 03-28-2006 Author: SAC



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NOTE

See Figures 5 and 6 for geologic unit explanation and profile legend and notes.

0 100 200

Horizontal Scale in Feet

0 40 80

Vertical Scale in Feet

Vertical Exaggeration = 2.5x

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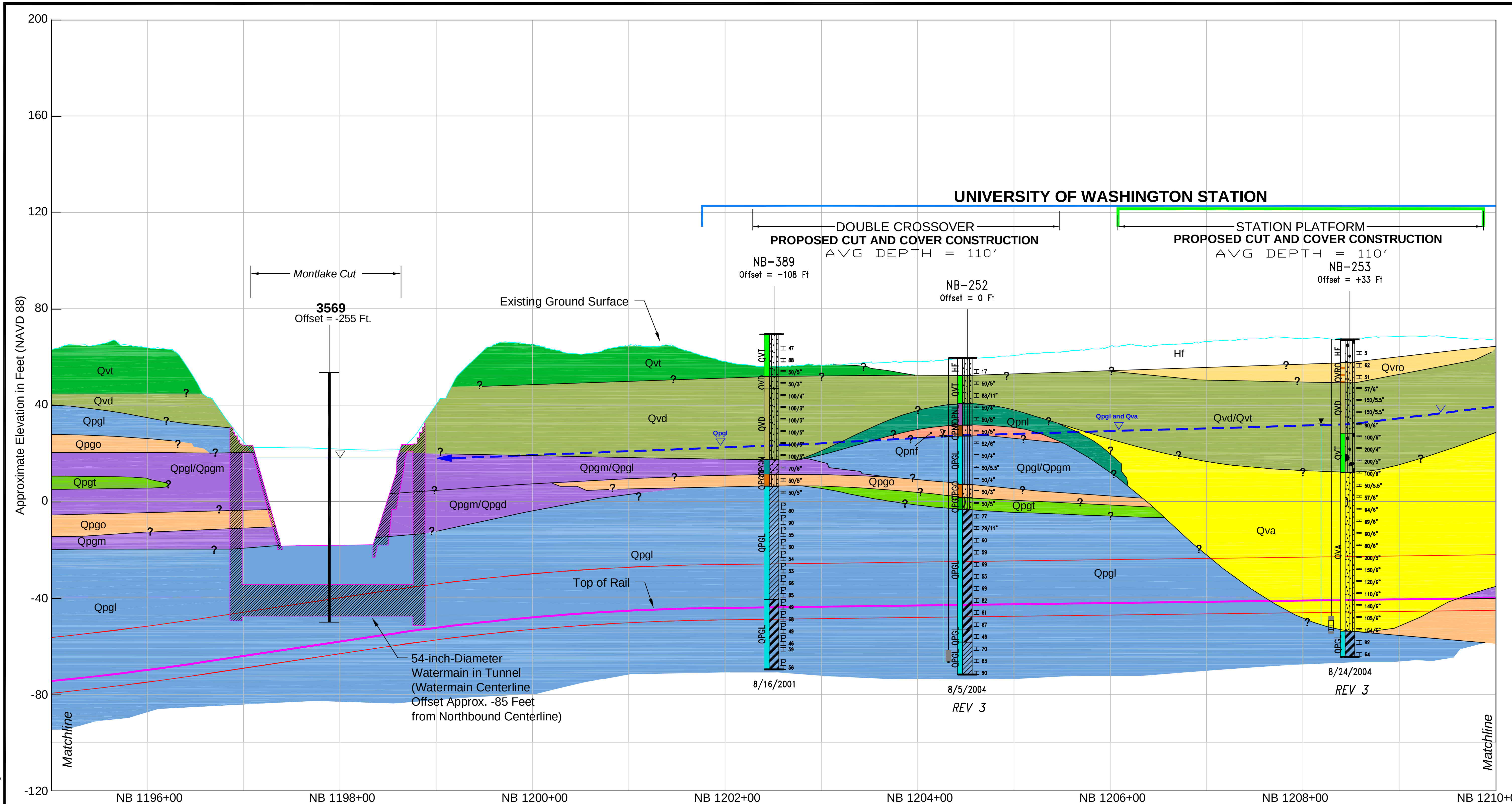
FIG. 7
Sheet 11 of 13



NOTE

See Figures 5 and 6 for geologic unit explanation and profile legend and notes.

0 100 200

Horizontal Scale in Feet

0 40 80

Vertical Scale in Feet

Vertical Exaggeration = 2.5x

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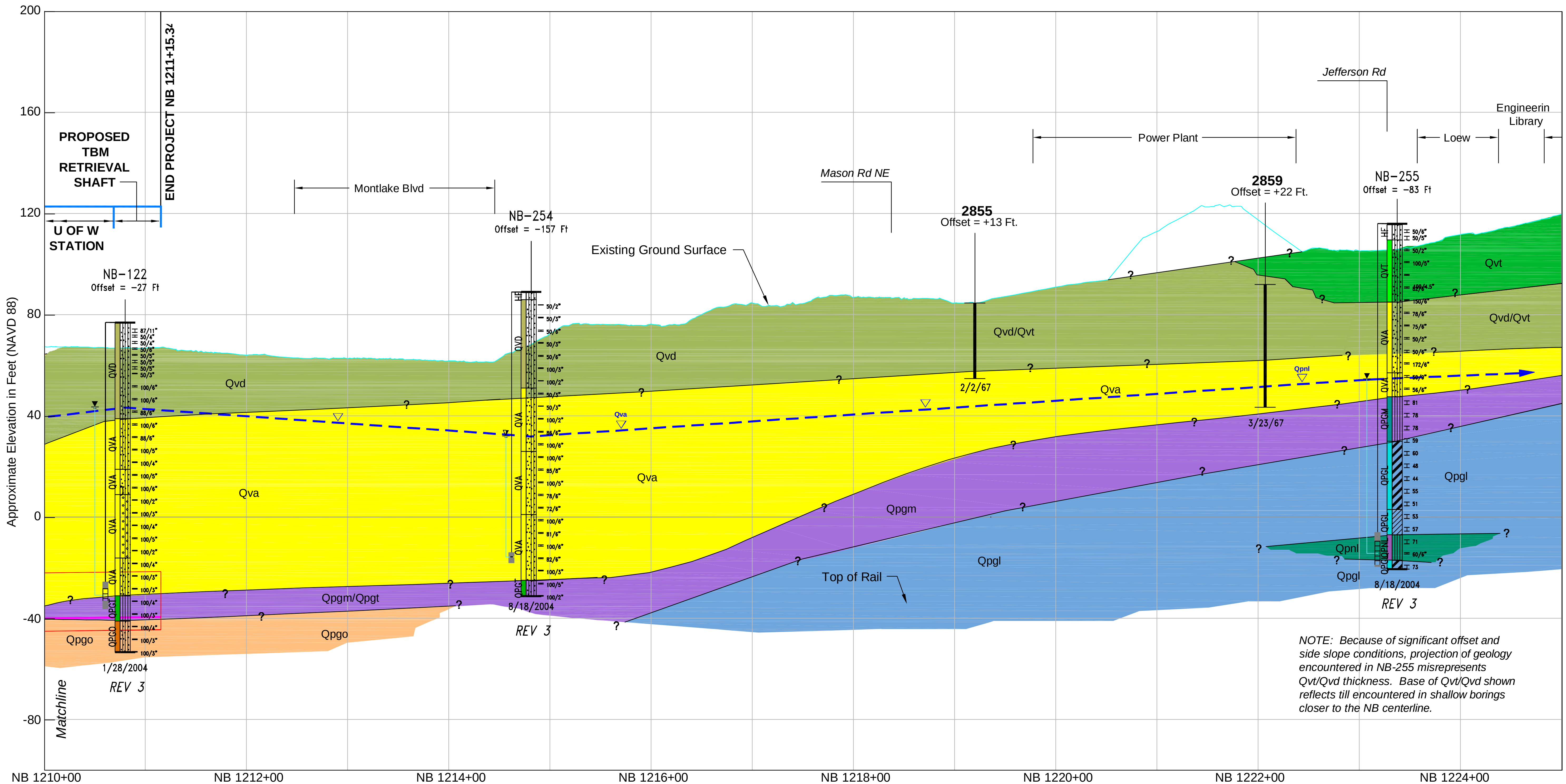
FIG. 7
Sheet 12 of 13

File: J:\211\08109-074\21-1-08109-074 Profile.dwg Date: 03-31-2006 Author: SAC



NOTE

See Figures 5 and 6 for geologic unit explanation and profile legend and notes.

0 100 200

Horizontal Scale in Feet

0 40 80

Vertical Scale in Feet

Vertical Exaggeration = 2.5x

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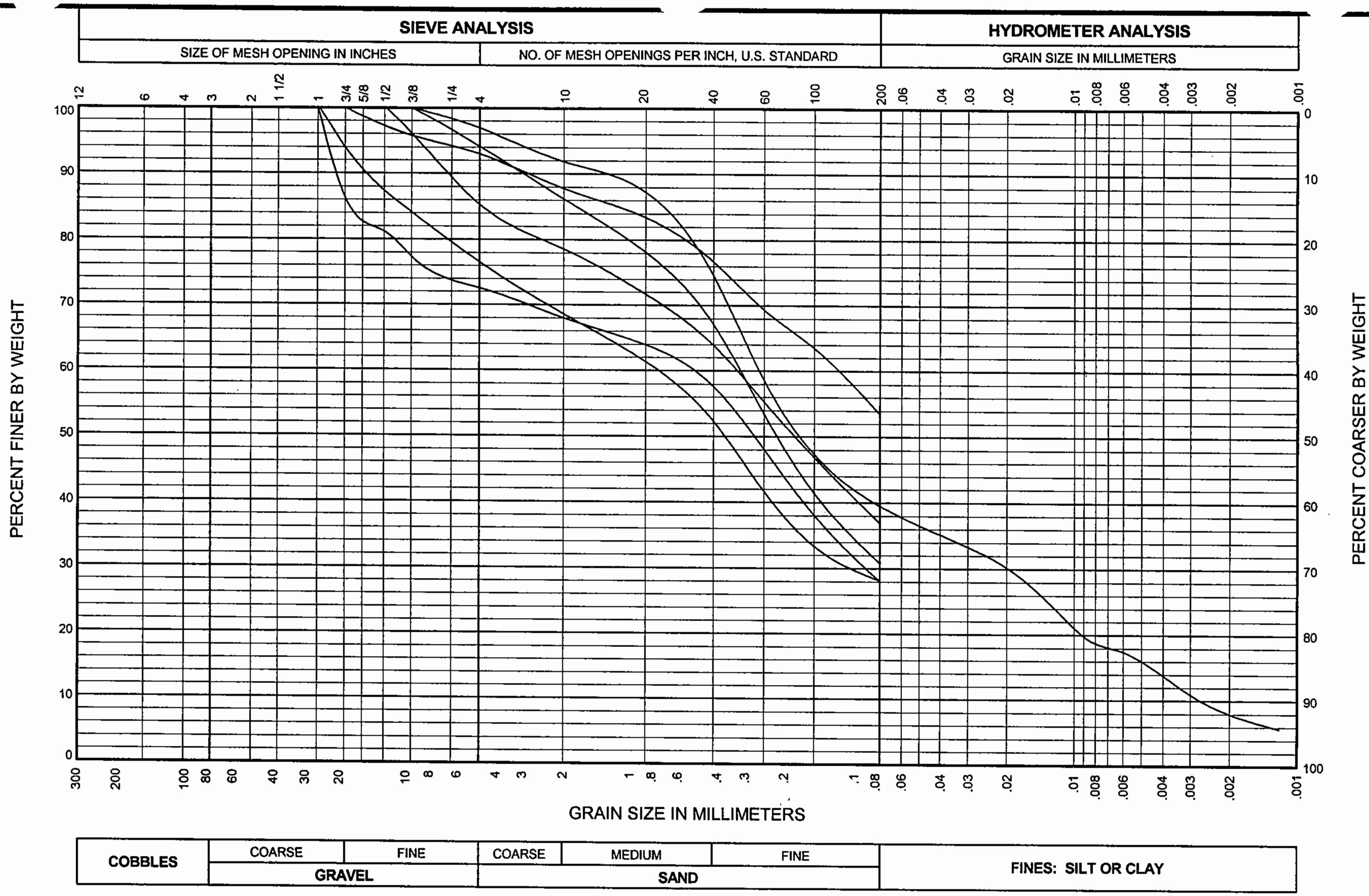
FIG. 7
Sheet 13 of 13

File: J:\211\08109-074\21-1-08109-074 Profile.dwg Date: 03-29-2006 Author: SAC



GSASUM1 W8100.GPJ SHAN_WIL.GDT 3/23/06

FIG. 8

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GRAIN SIZE DISTRIBUTION

Geologic Unit: Hf

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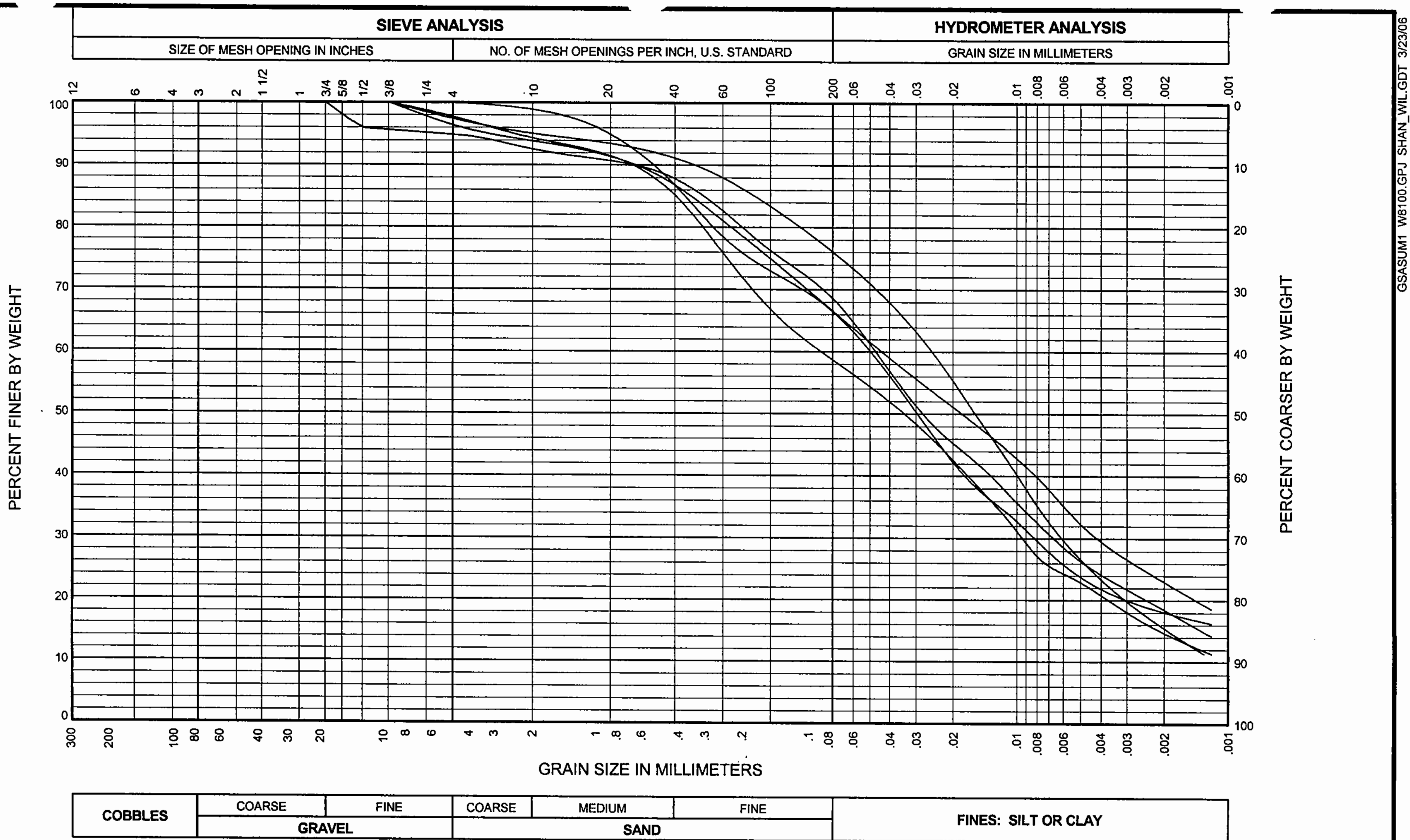
FIG. 8



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GRAIN SIZE DISTRIBUTION

Geologic Unit: Hls

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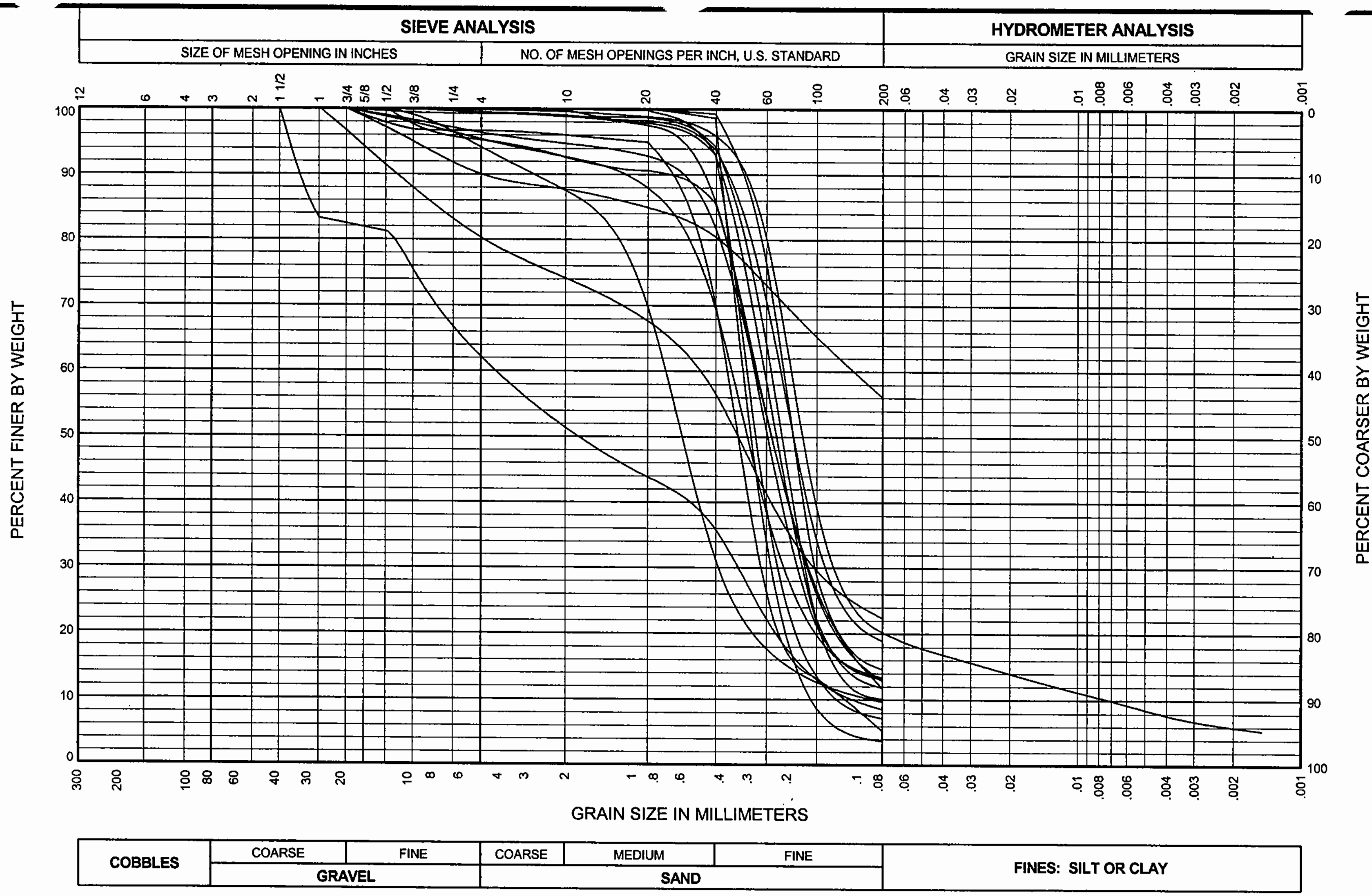
FIG. 9

FIG. 9



GSASUM1 W8100.GPJ SHAN_WIL.GDT 3/23/06

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GRAIN SIZE DISTRIBUTION

Geologic Unit: Qvro

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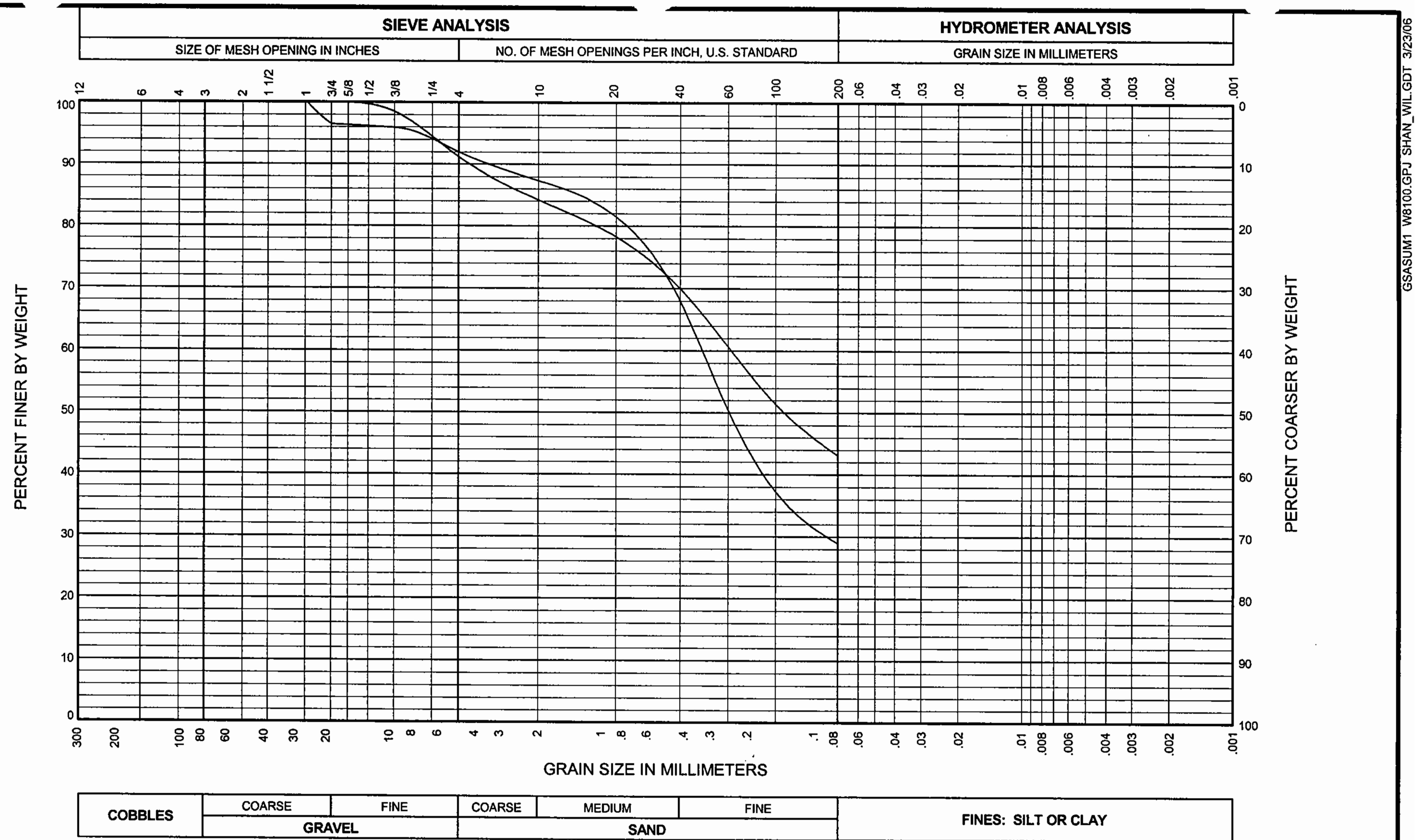
FIG. 10

FIG. 10



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GRAIN SIZE DISTRIBUTION

Geologic Unit: Qvri

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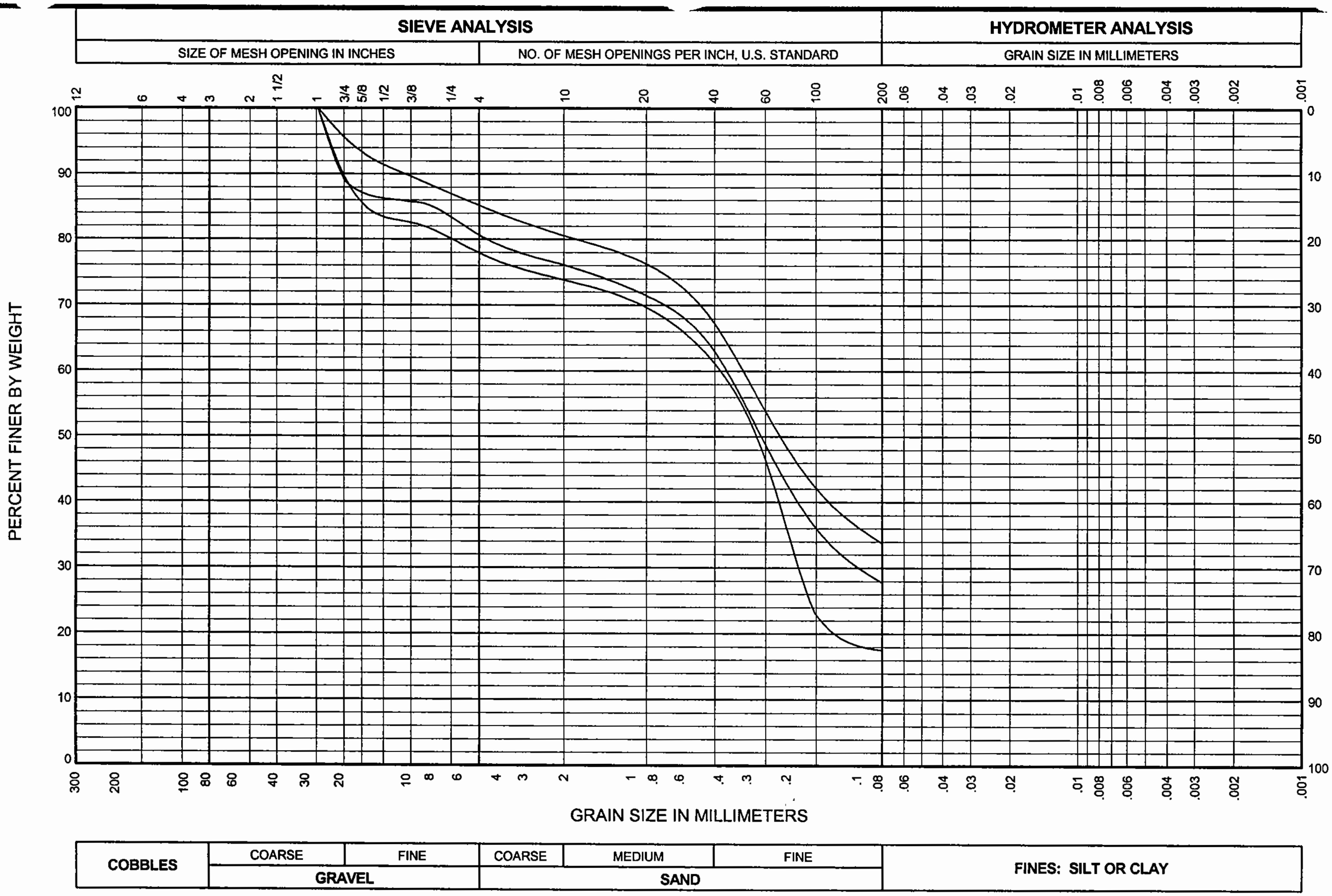
FIG. 11

FIG. 11



GSASUM1 W8100.GPJ SHAN_WIL.GDT 3/23/06

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GRAIN SIZE DISTRIBUTION

Geologic Unit: Qvat

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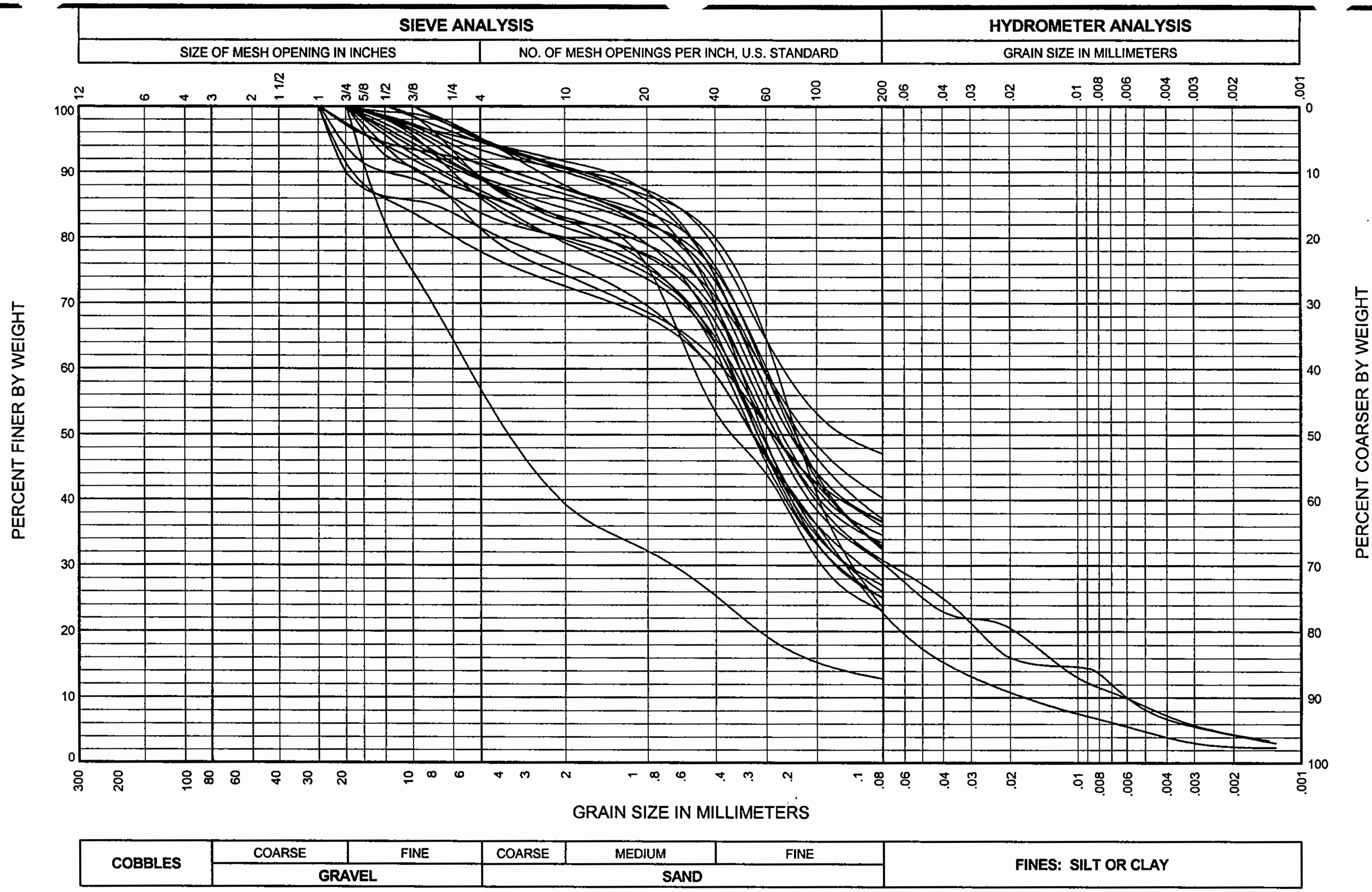
FIG. 12

FIG. 12



GSASUM1 W8100.GPJ SHAN_WIL.GDT 3/23/06

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GRAIN SIZE DISTRIBUTION

Geologic Unit: Qvt

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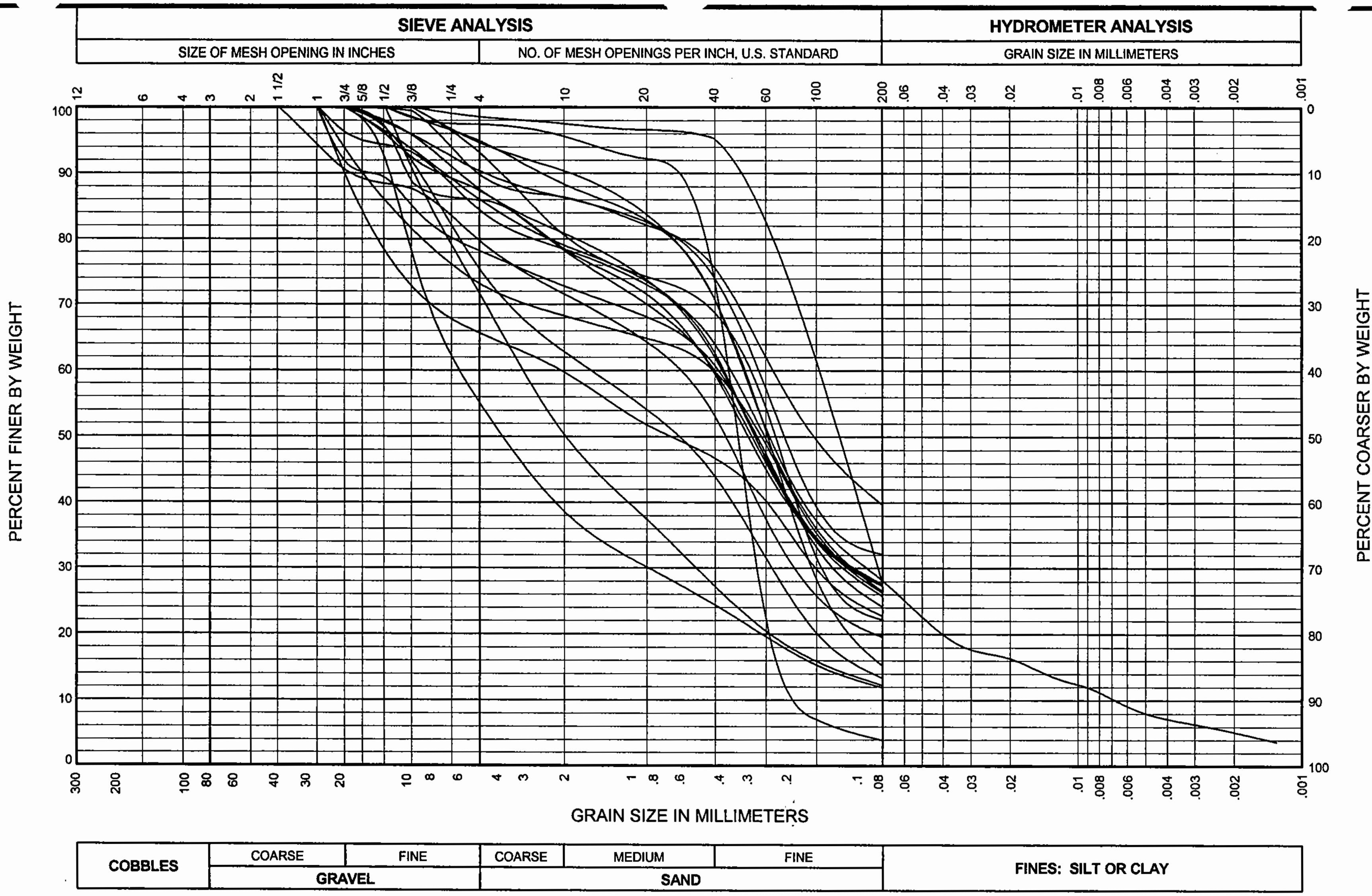
FIG. 13

FIG. 13



GSASUM1 W8100.GPJ SHAN_WIL.GDT 3/23/06

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GRAIN SIZE DISTRIBUTION

Geologic Unit: Qvd

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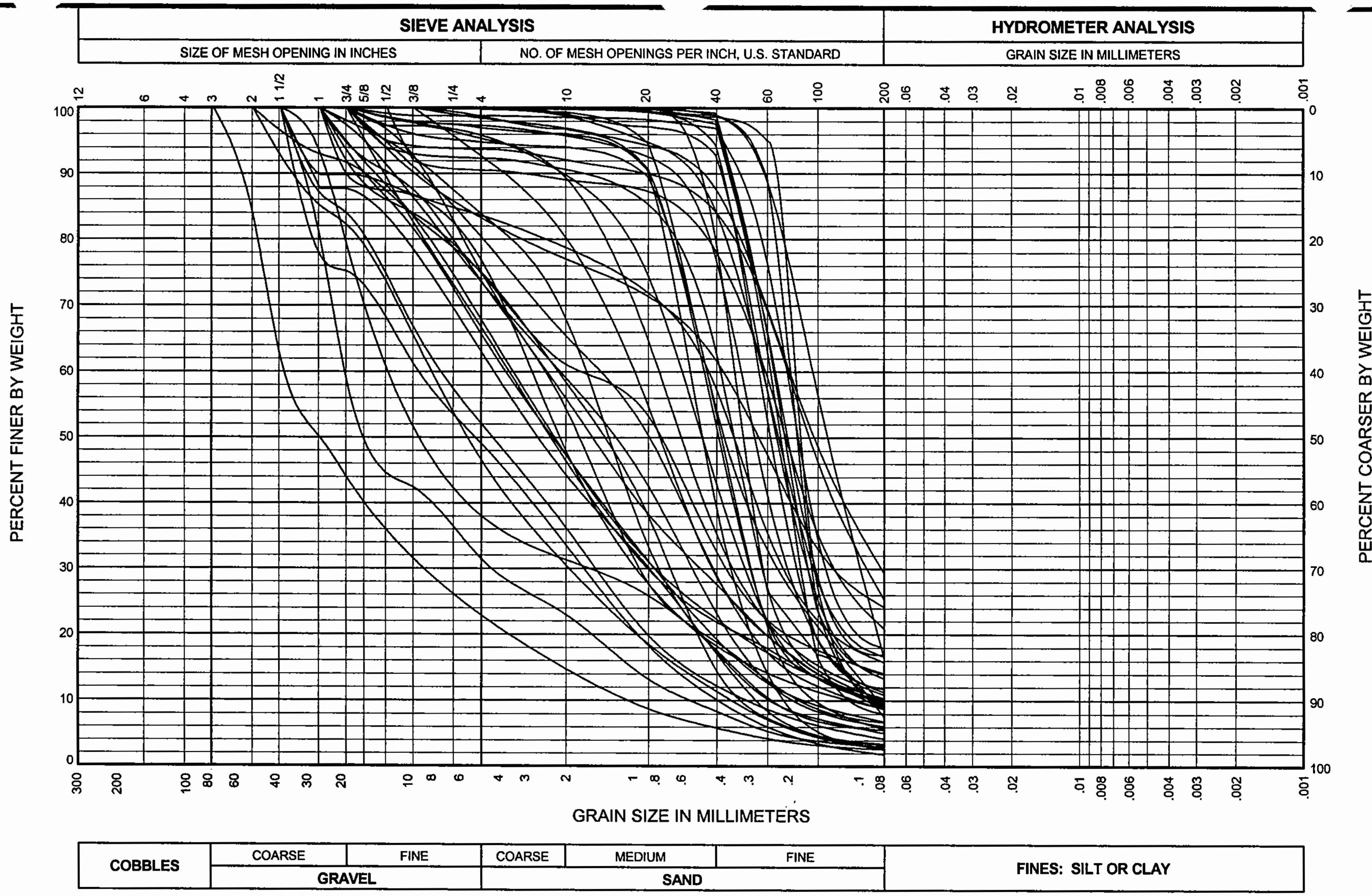
FIG. 14

FIG. 14



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GRAIN SIZE DISTRIBUTION

Geologic Unit: Qva

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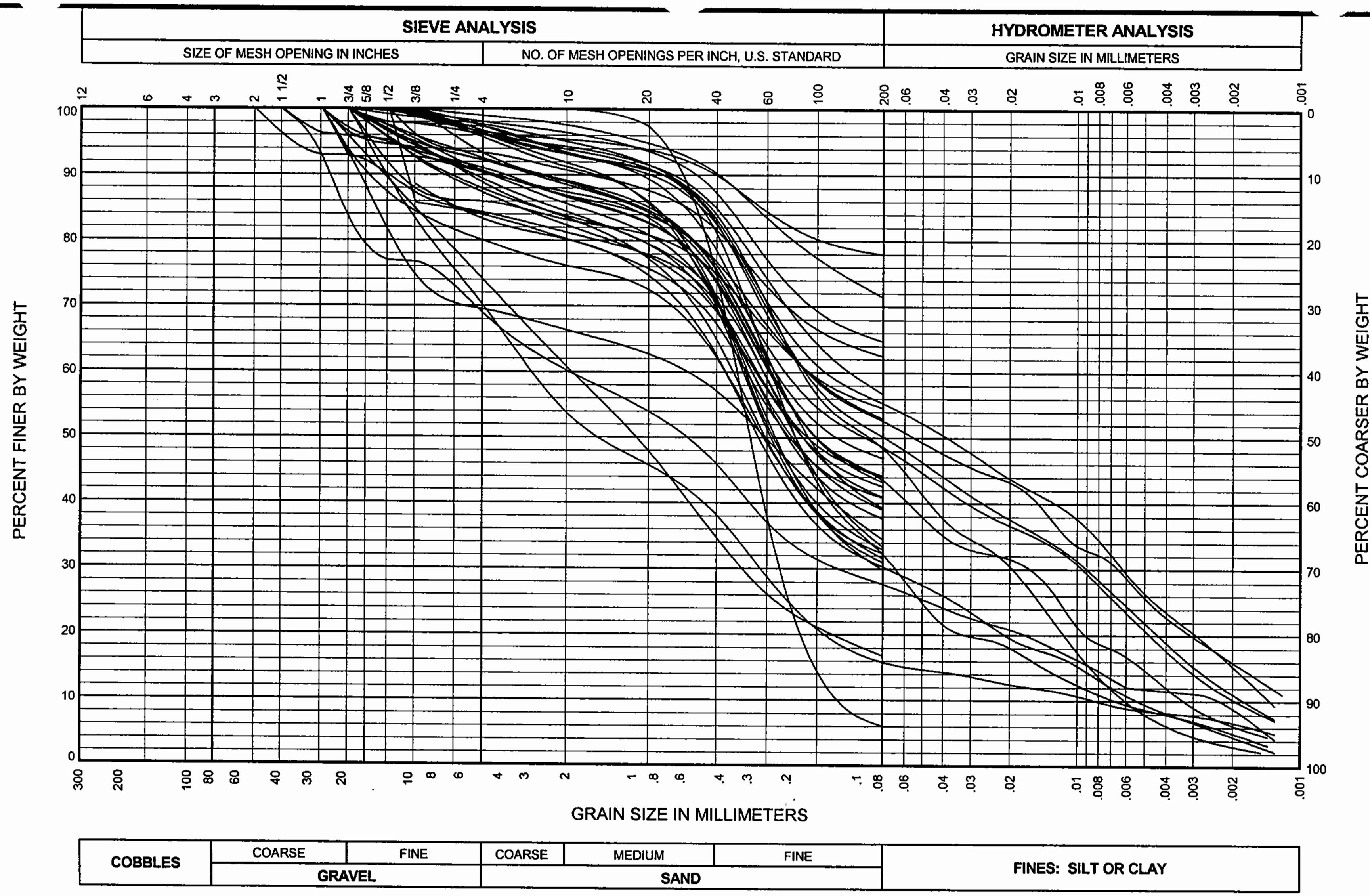
FIG. 15

FIG. 15



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GRAIN SIZE DISTRIBUTION

Geologic Unit: Qvgm

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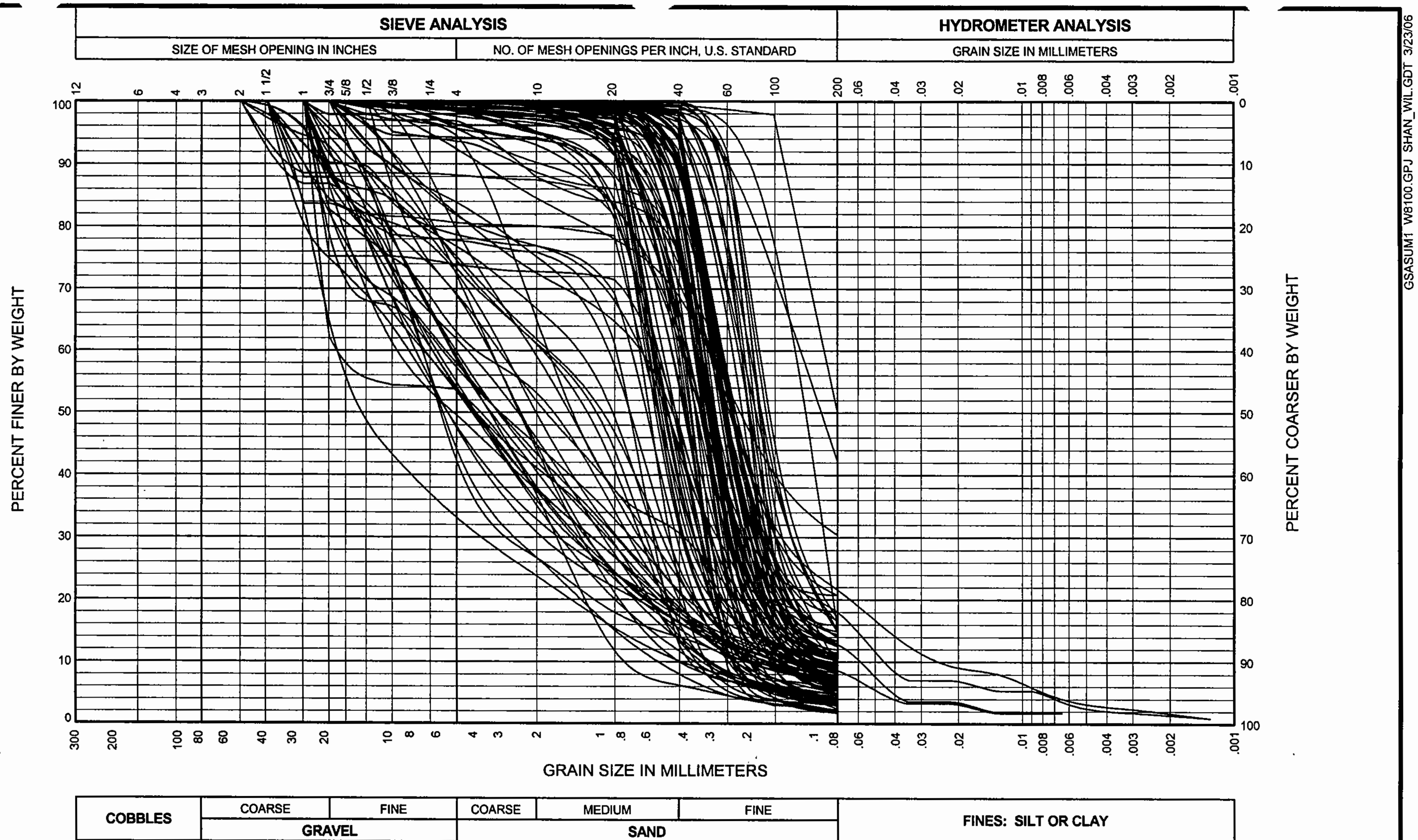
FIG. 16

FIG. 16



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GRAIN SIZE DISTRIBUTION

Geologic Unit: Qpnf

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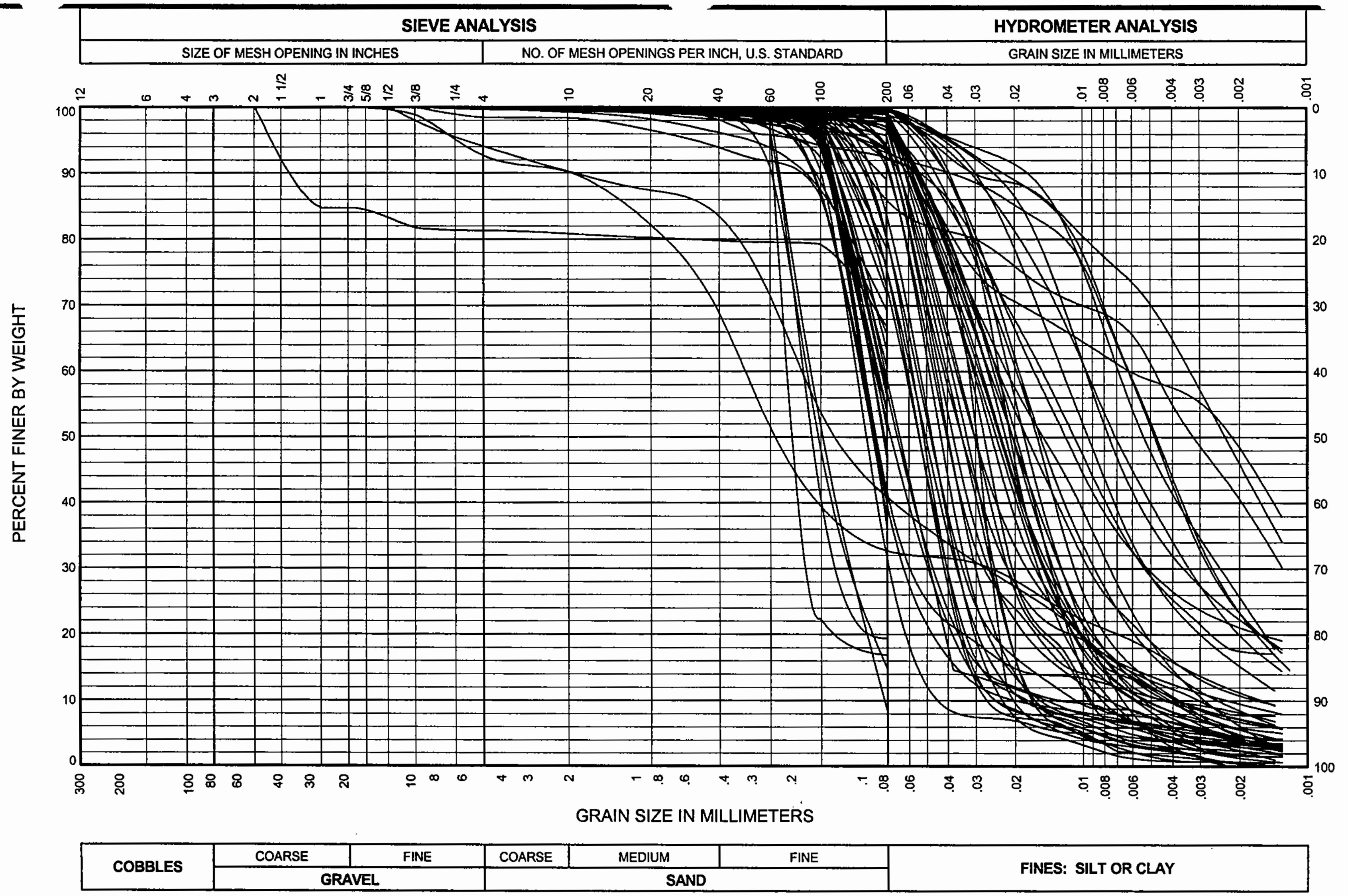
FIG. 17

FIG. 17



GSASUM1 W8100.GPJ SHAN_WIL.GDT 3/23/06

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GRAIN SIZE DISTRIBUTION

Geologic Unit: Qpnl

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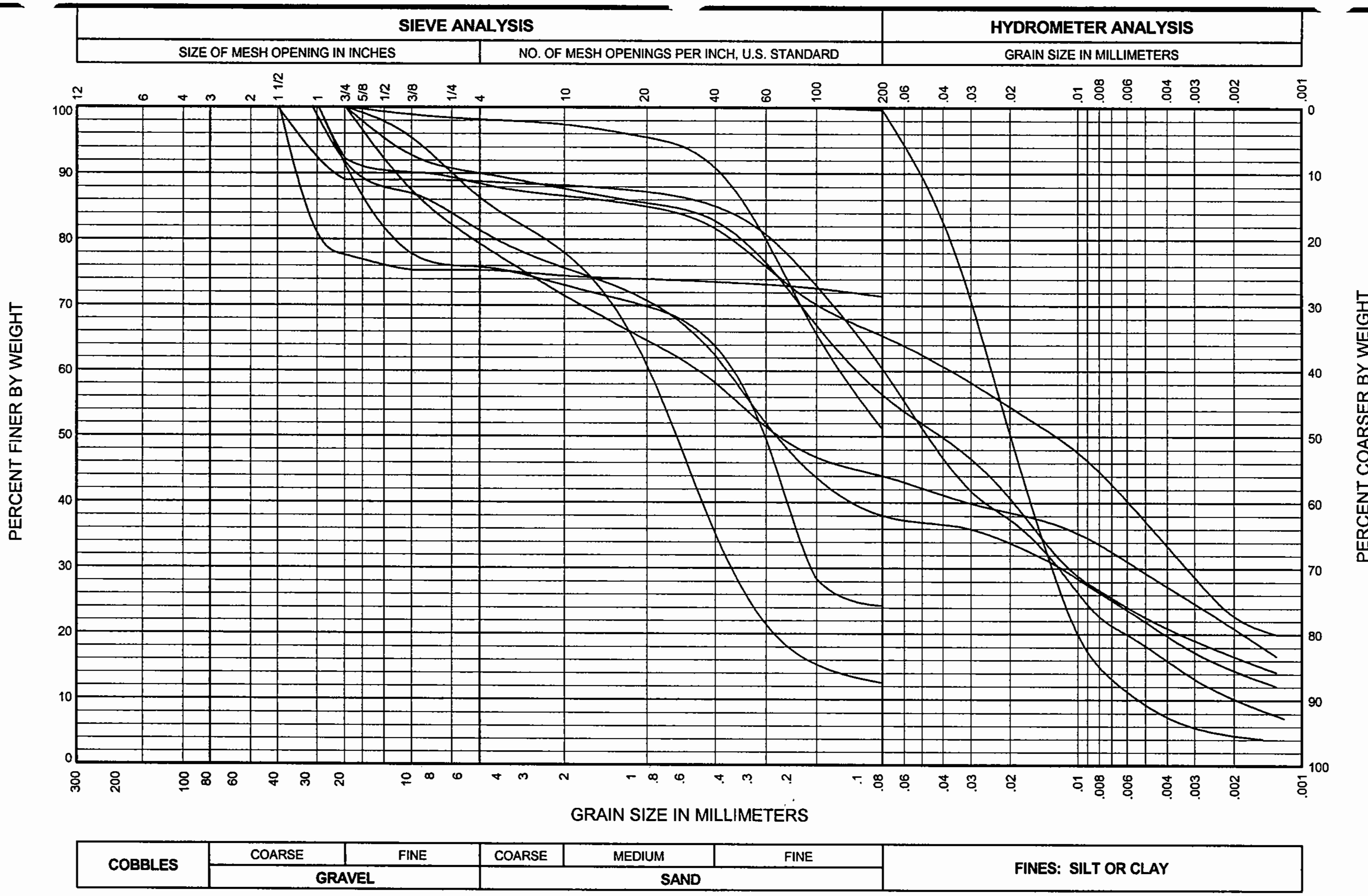
FIG. 18

FIG. 18



GSASUM1 W8100.GPJ SHAN_WIL.GDT 3/23/06

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GRAIN SIZE DISTRIBUTION

Geologic Unit: Qpns

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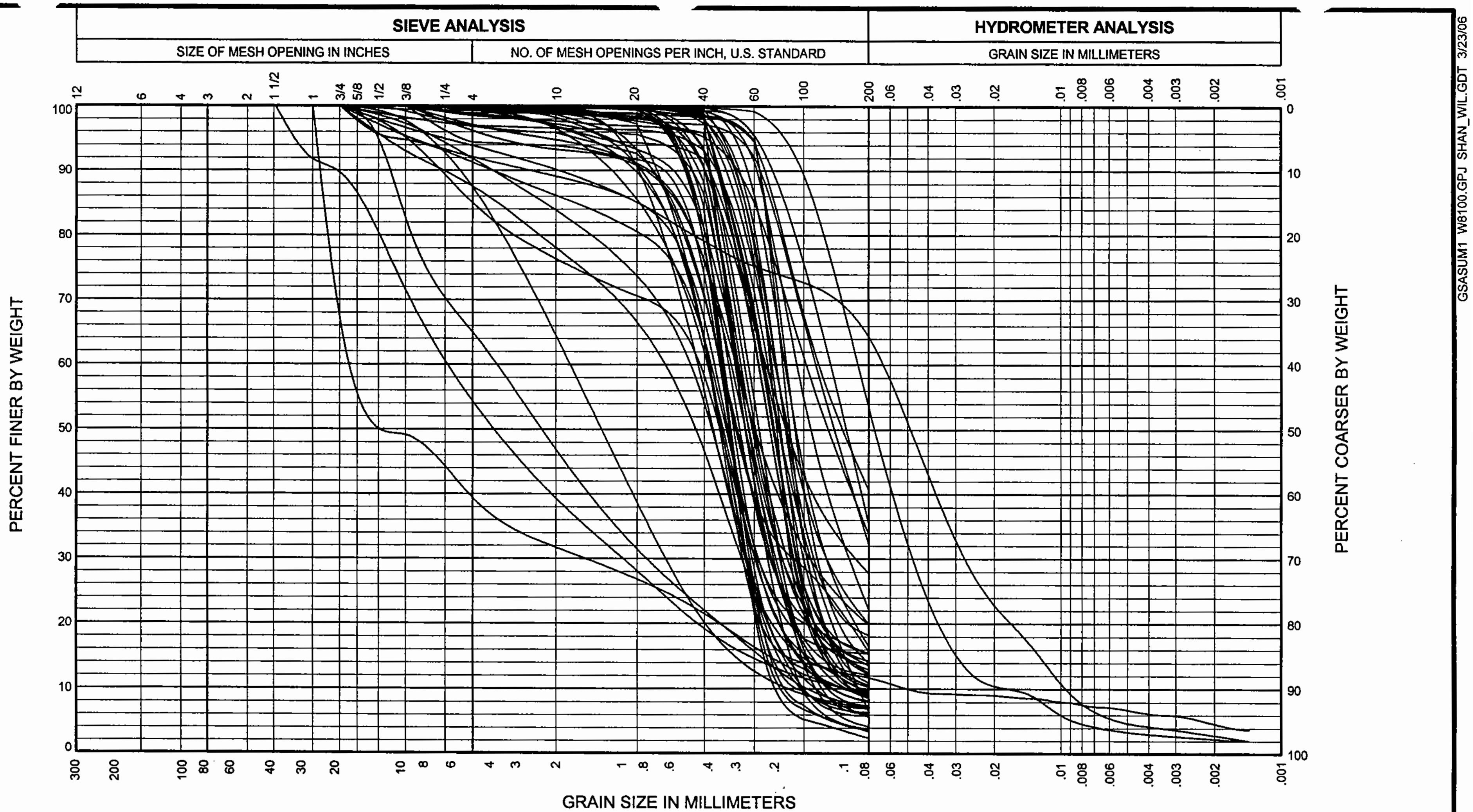
FIG. 19

FIG. 19



COBBLES	GRAVEL		SAND			FINES: SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

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GRAIN SIZE DISTRIBUTION

Geologic Unit: Qpgo

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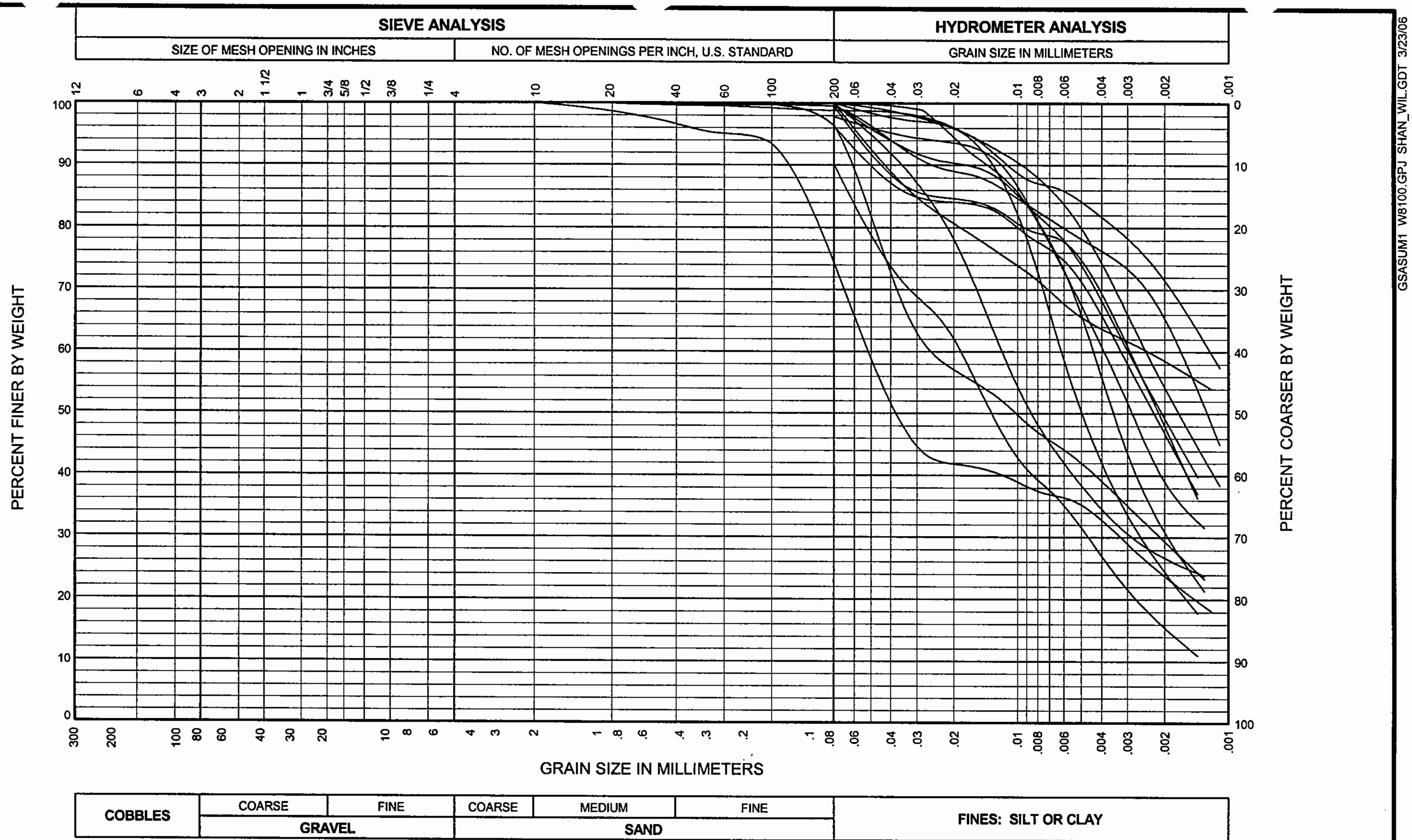
FIG. 20

FIG. 20



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GRAIN SIZE DISTRIBUTION

Geologic Unit: Qpgl

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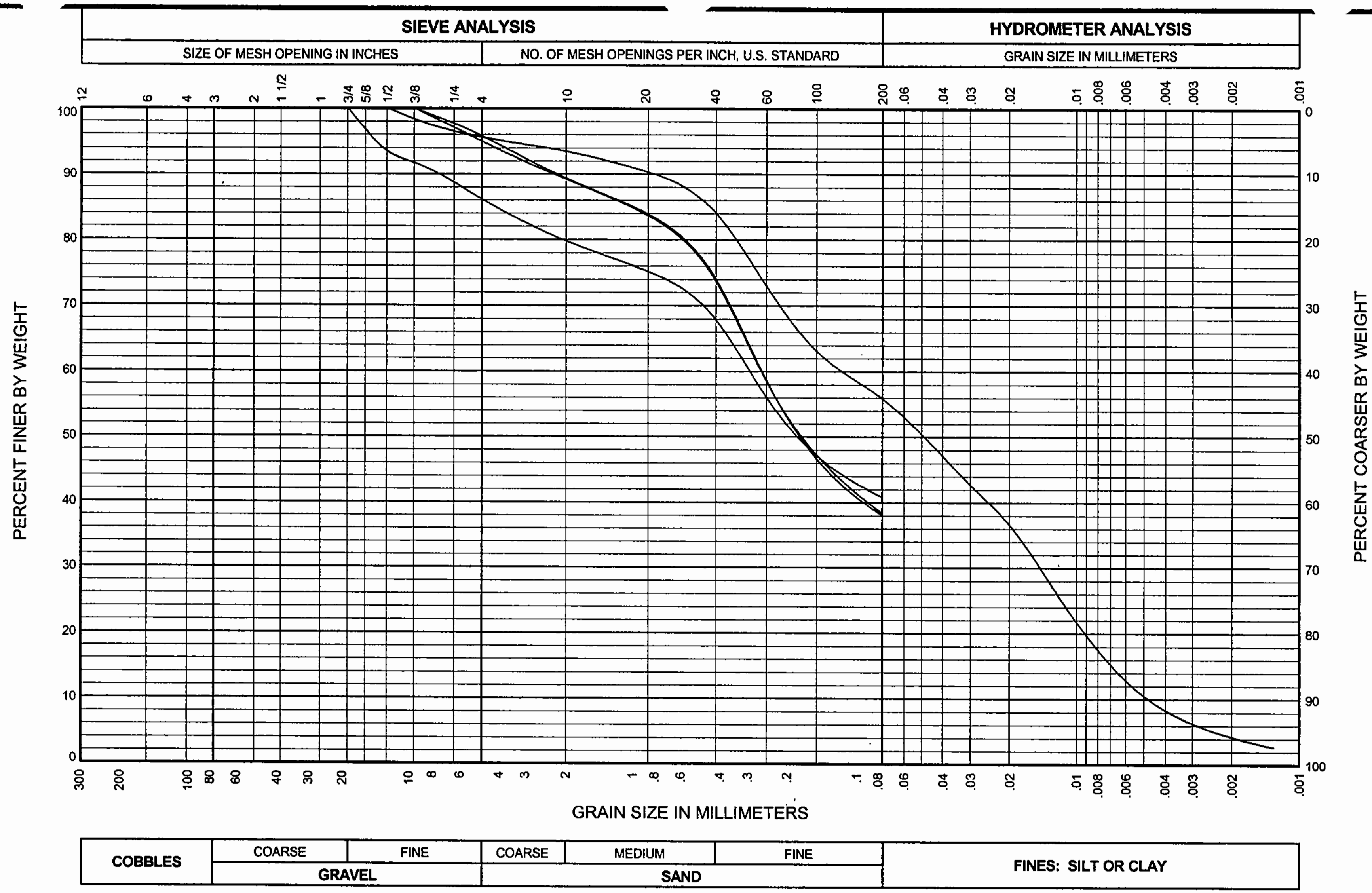
FIG. 21

FIG. 21



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GRAIN SIZE DISTRIBUTION

Geologic Unit: Qpgt

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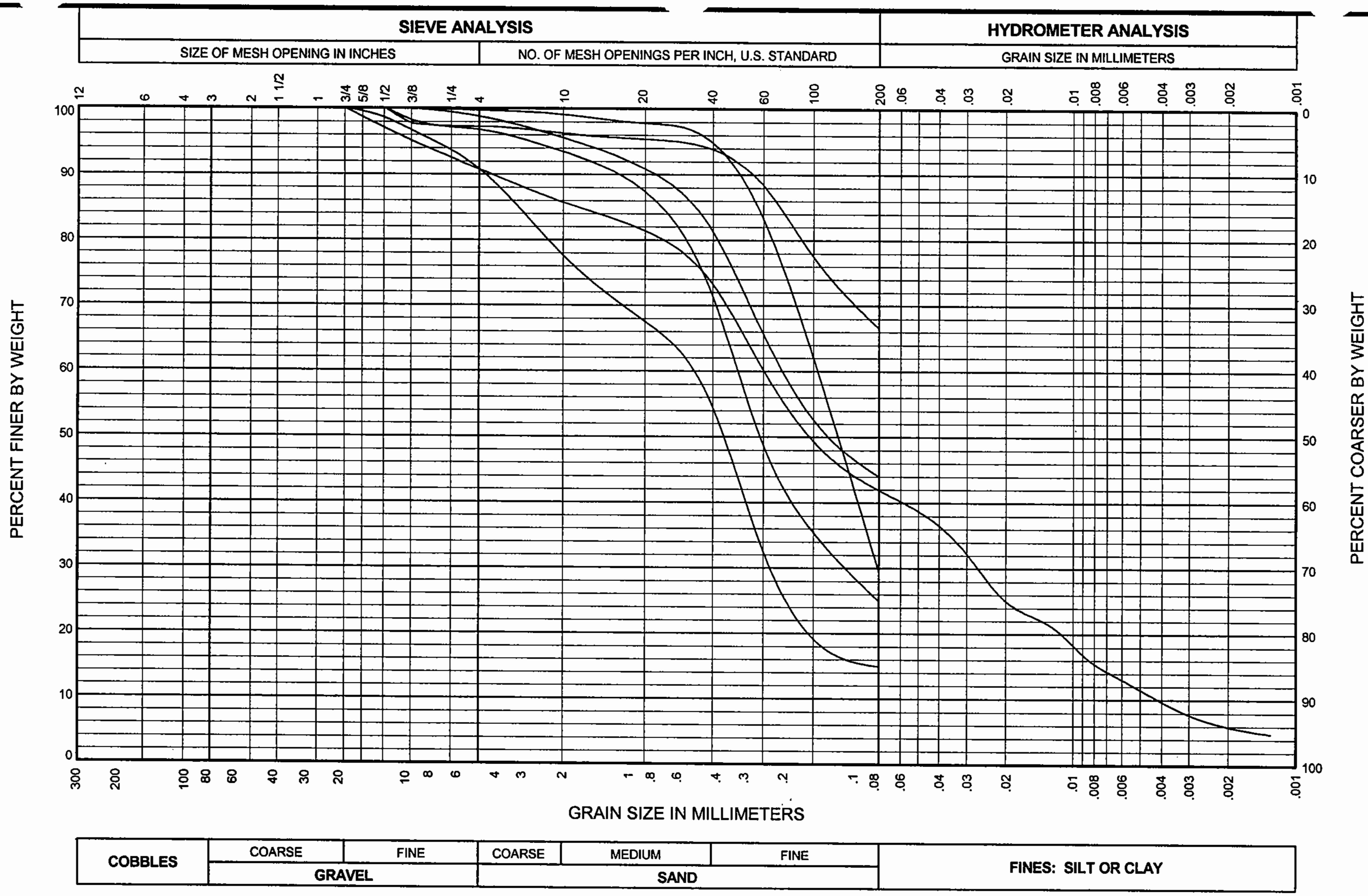
FIG. 22

FIG. 22



GSASUM1 W8100.GPJ SHAN_WIL.GDT 3/23/06

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GRAIN SIZE DISTRIBUTION

Geologic Unit: Qpgd

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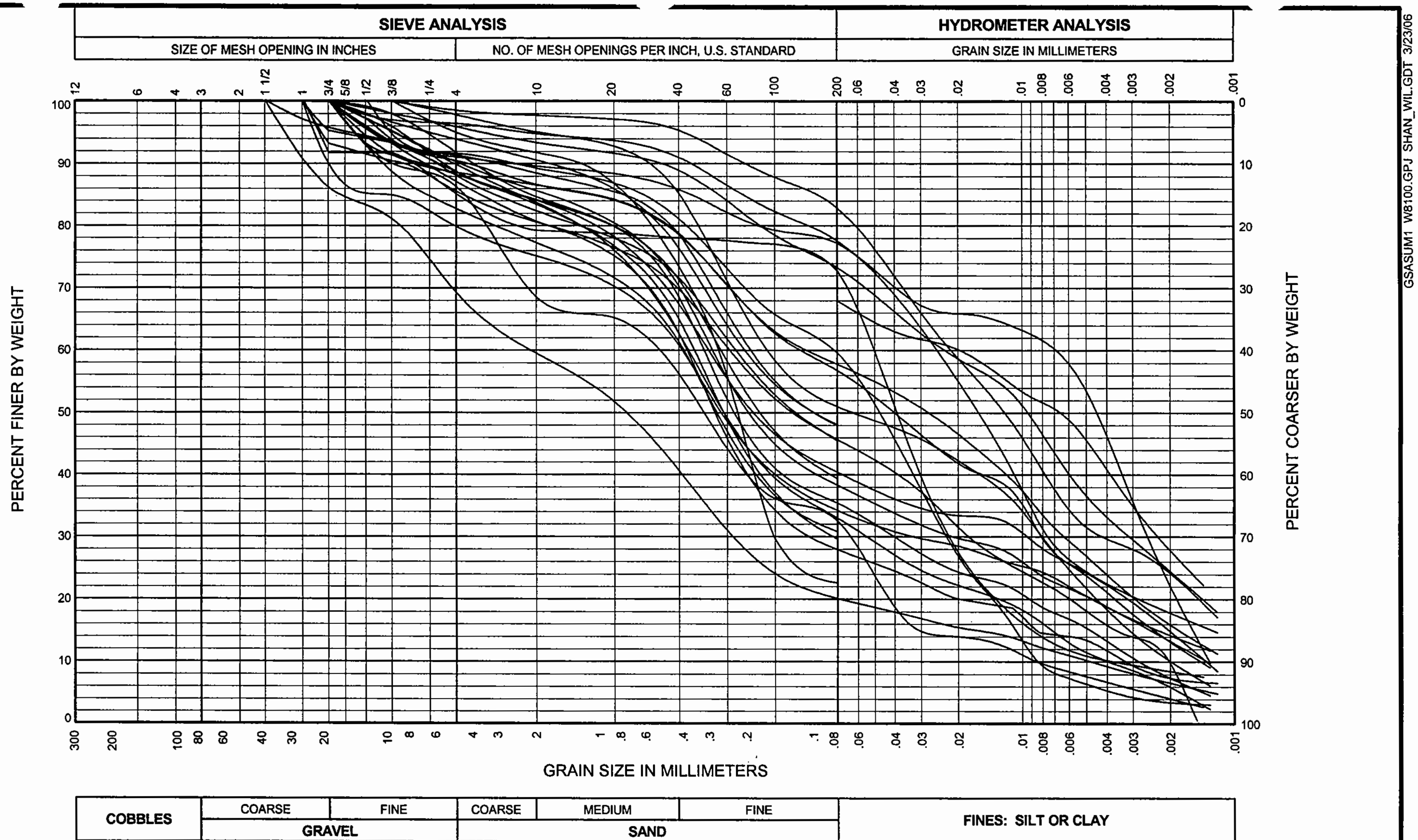
FIG. 23

FIG. 23



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GRAIN SIZE DISTRIBUTION

Geologic Unit: Qpgm

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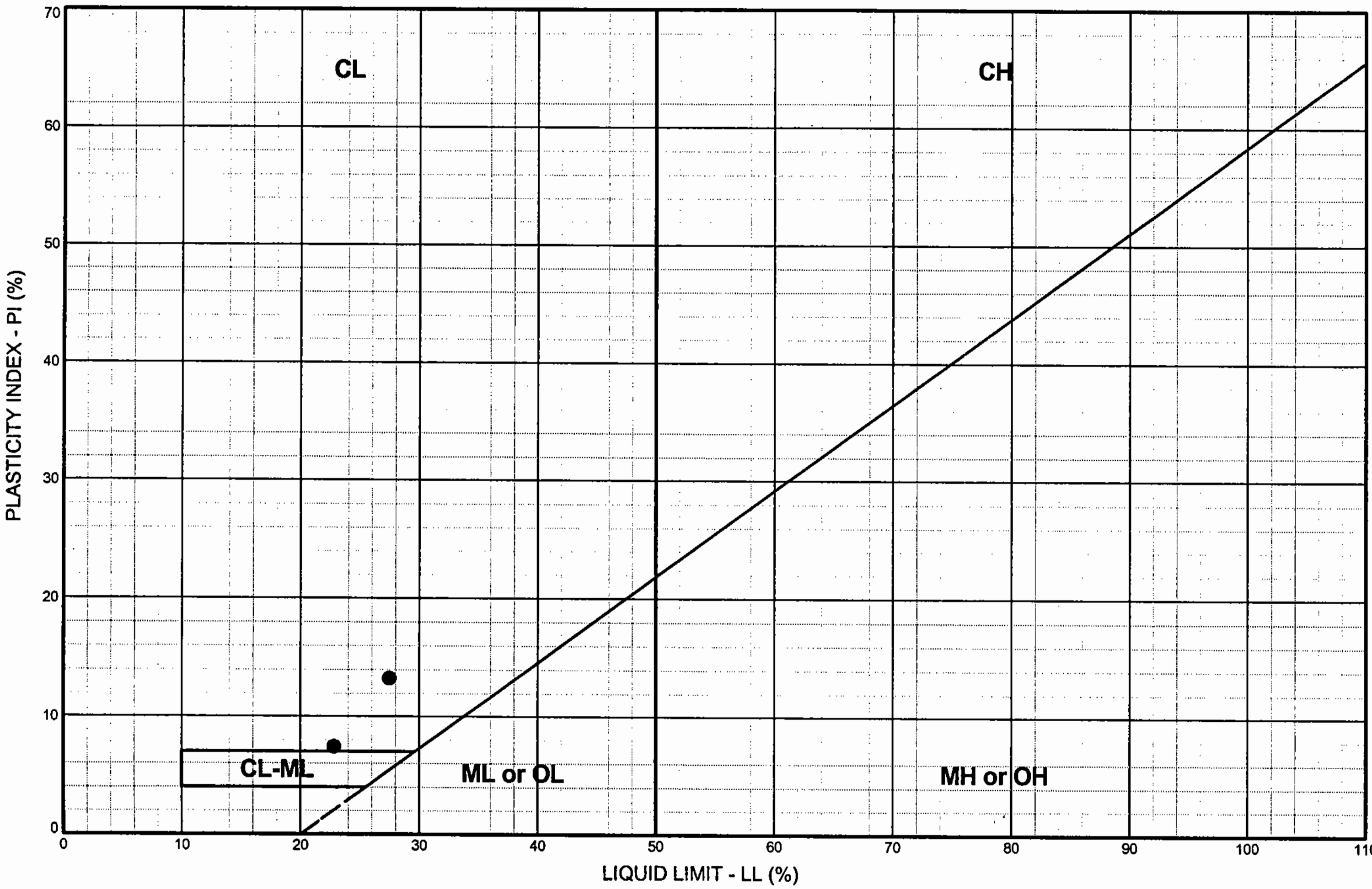
FIG. 24

FIG. 24



LEGEND

CL: Low plasticity inorganic clays; sandy and silty clays

CH: High plasticity inorganic clays

ML or OL: Inorganic and organic silts and clayey silts of low plasticity

MH or OH: Inorganic and organic silts and clayey silts of high plasticity

CL-ML: Silty clays and clayey silts

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PLASTICITY CHART

Geologic Unit: Hf

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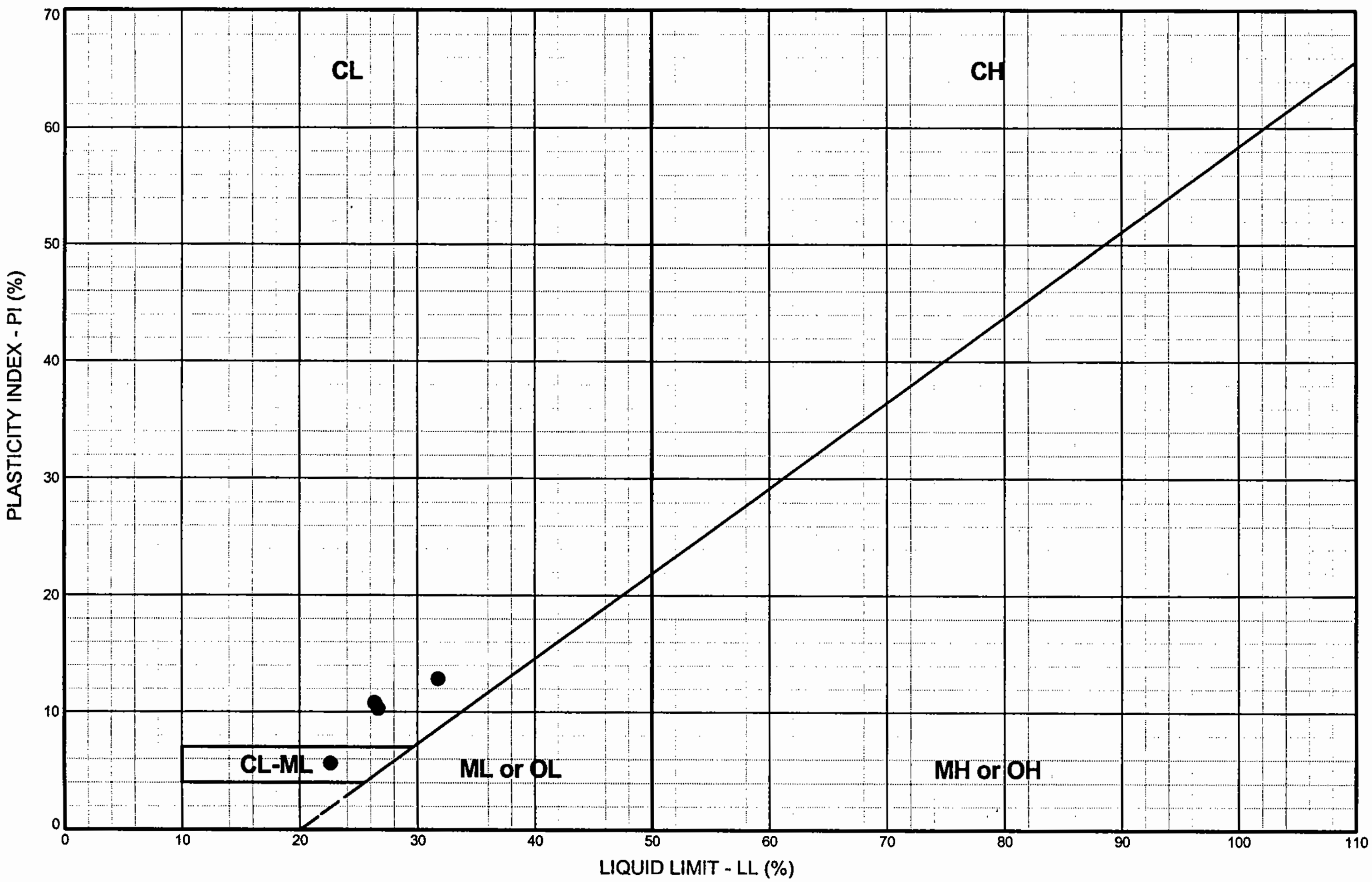
FIG. 25



LEGEND

CL: Low plasticity inorganic clays; sandy and silty clays

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CL-ML: Silty clays and clayey silts

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PLASTICITY CHART

Geologic Unit: Hls

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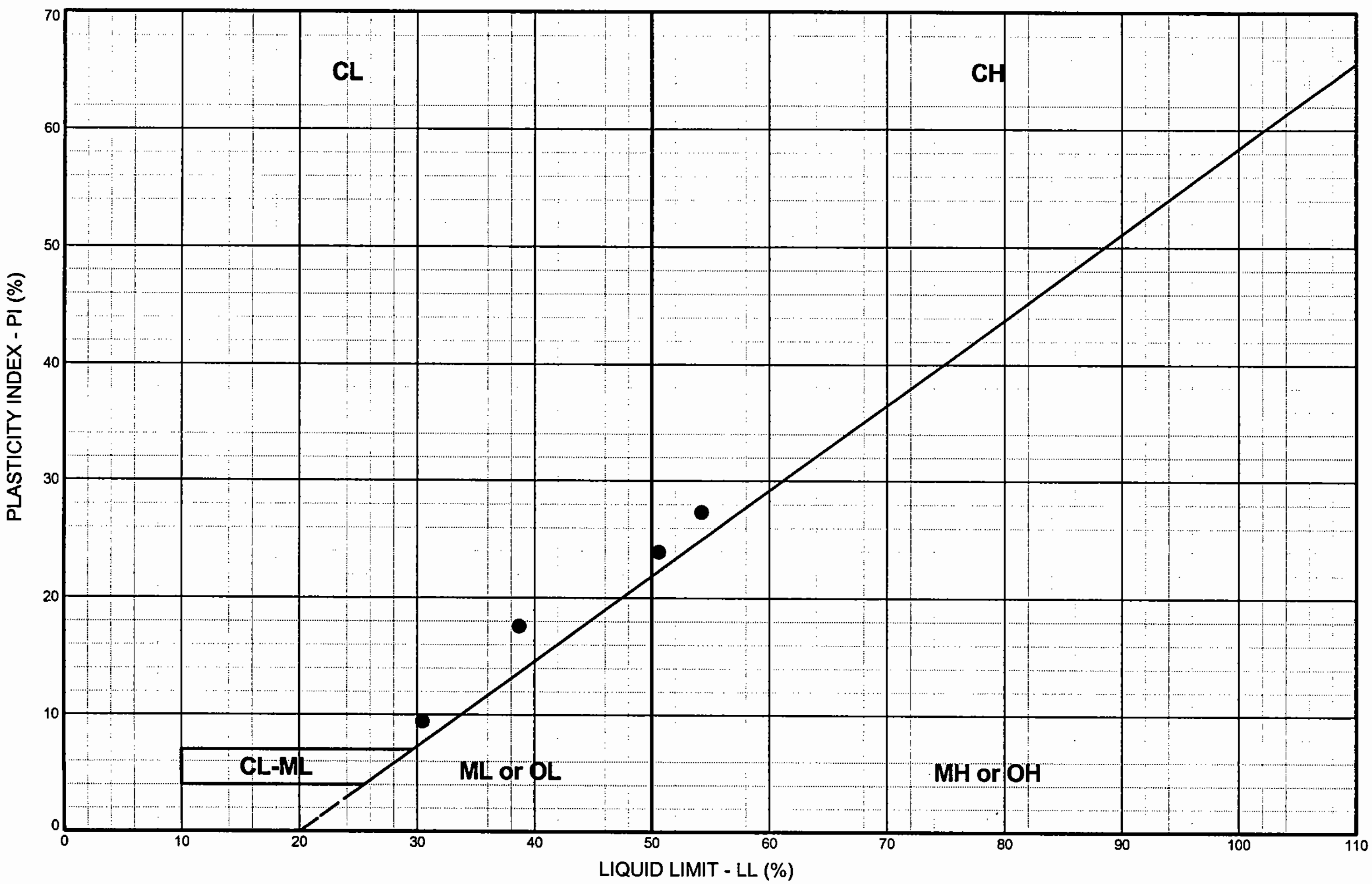
FIG. 26



LEGEND

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MH or OH: Inorganic and organic silts and clayey silts of high plasticity

CL-ML: Silty clays and clayey silts

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PLASTICITY CHART

Geologic Unit: Qvrl

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FIG. 27

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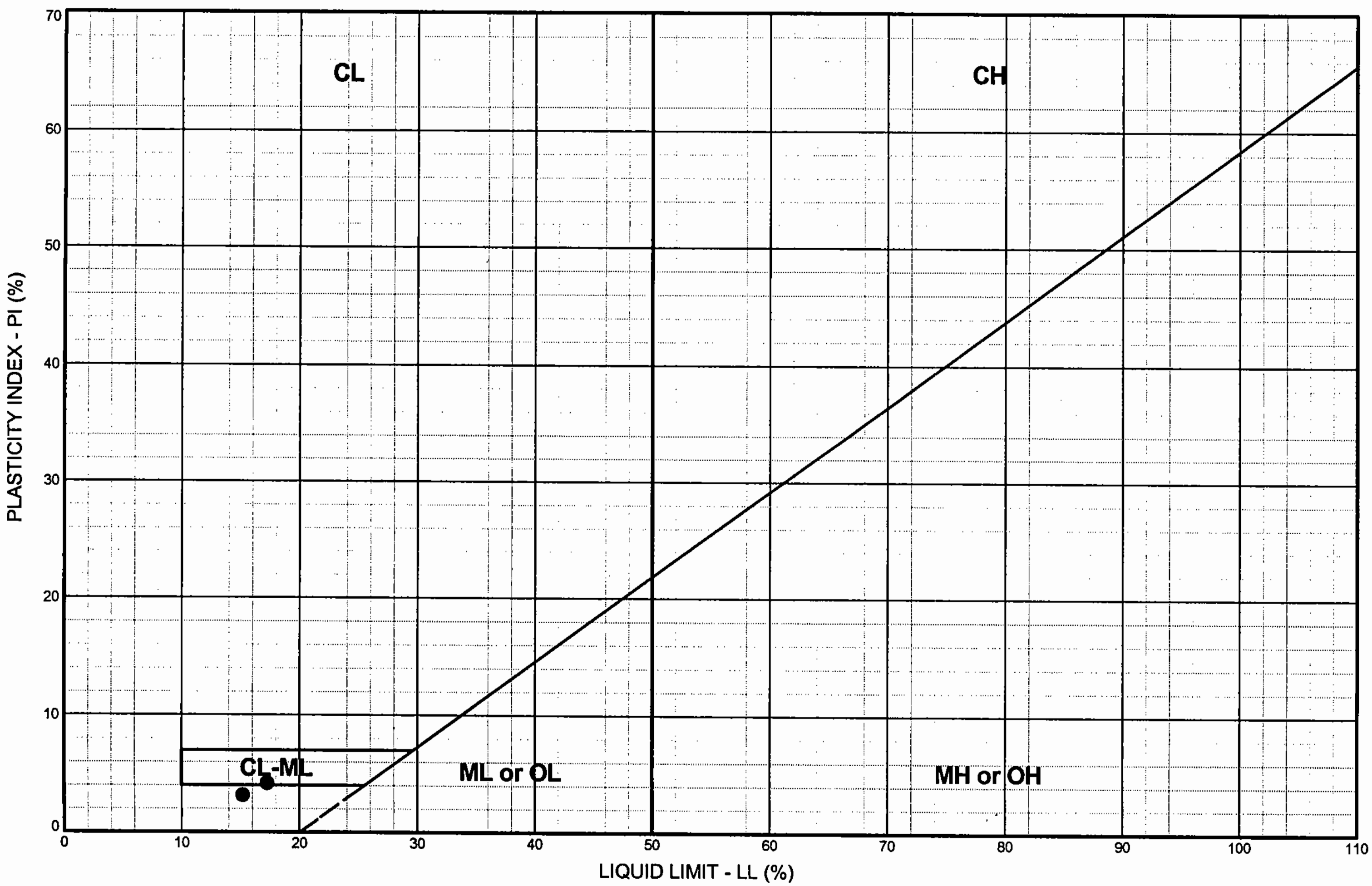

LEGEND

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MH or OH: Inorganic and organic silts and clayey silts of high plasticity

CL-ML: Silty clays and clayey silts

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PLASTICITY CHART

Geologic Unit: Qvt

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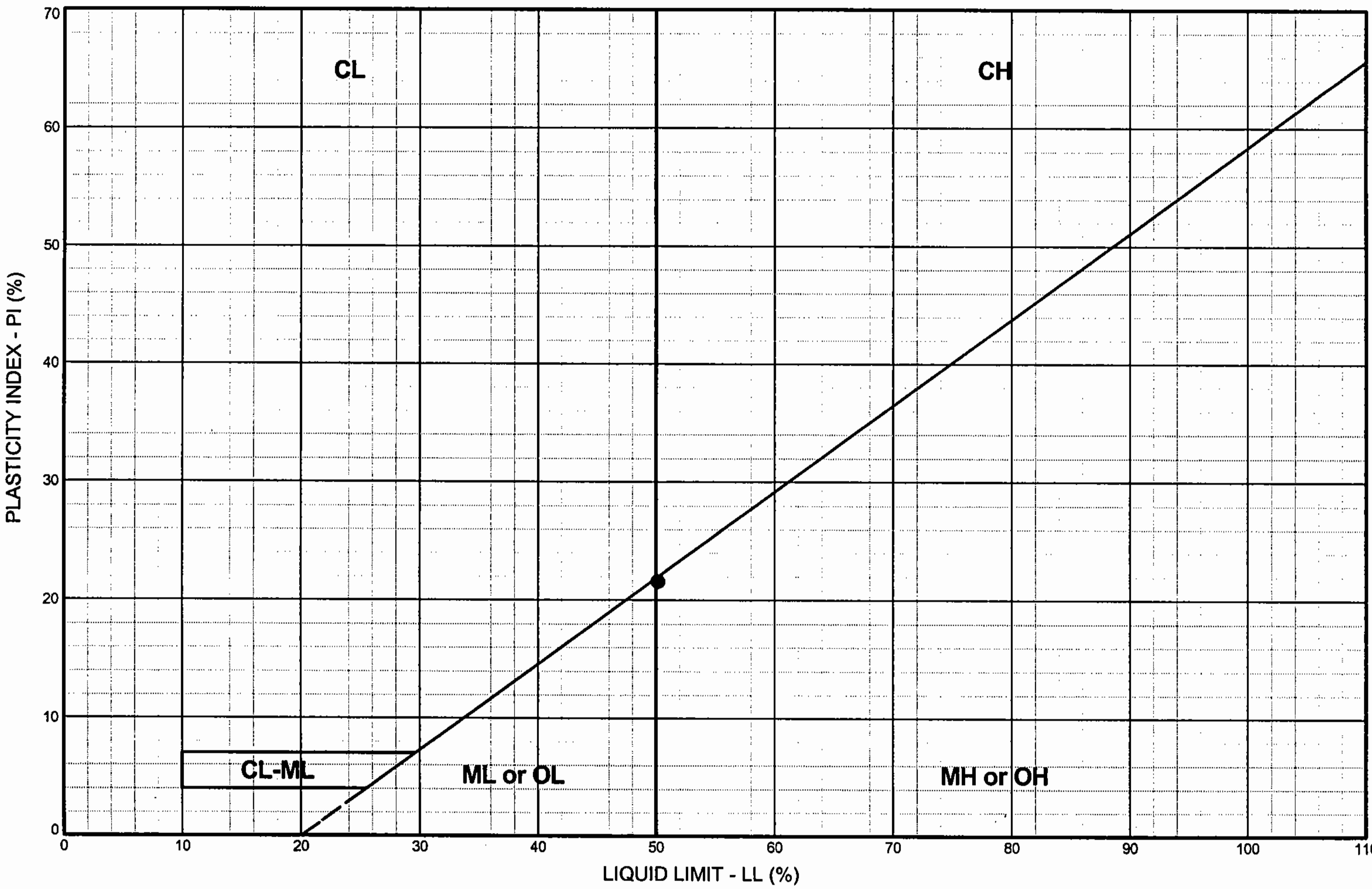
FIG. 28



LEGEND

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CH: High plasticity inorganic clays

ML or OL: Inorganic and organic silts and clayey silts of low plasticity

MH or OH: Inorganic and organic silts and clayey silts of high plasticity

CL-ML: Silty clays and clayey silts

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PLASTICITY CHART

Geologic Unit: Qvgl

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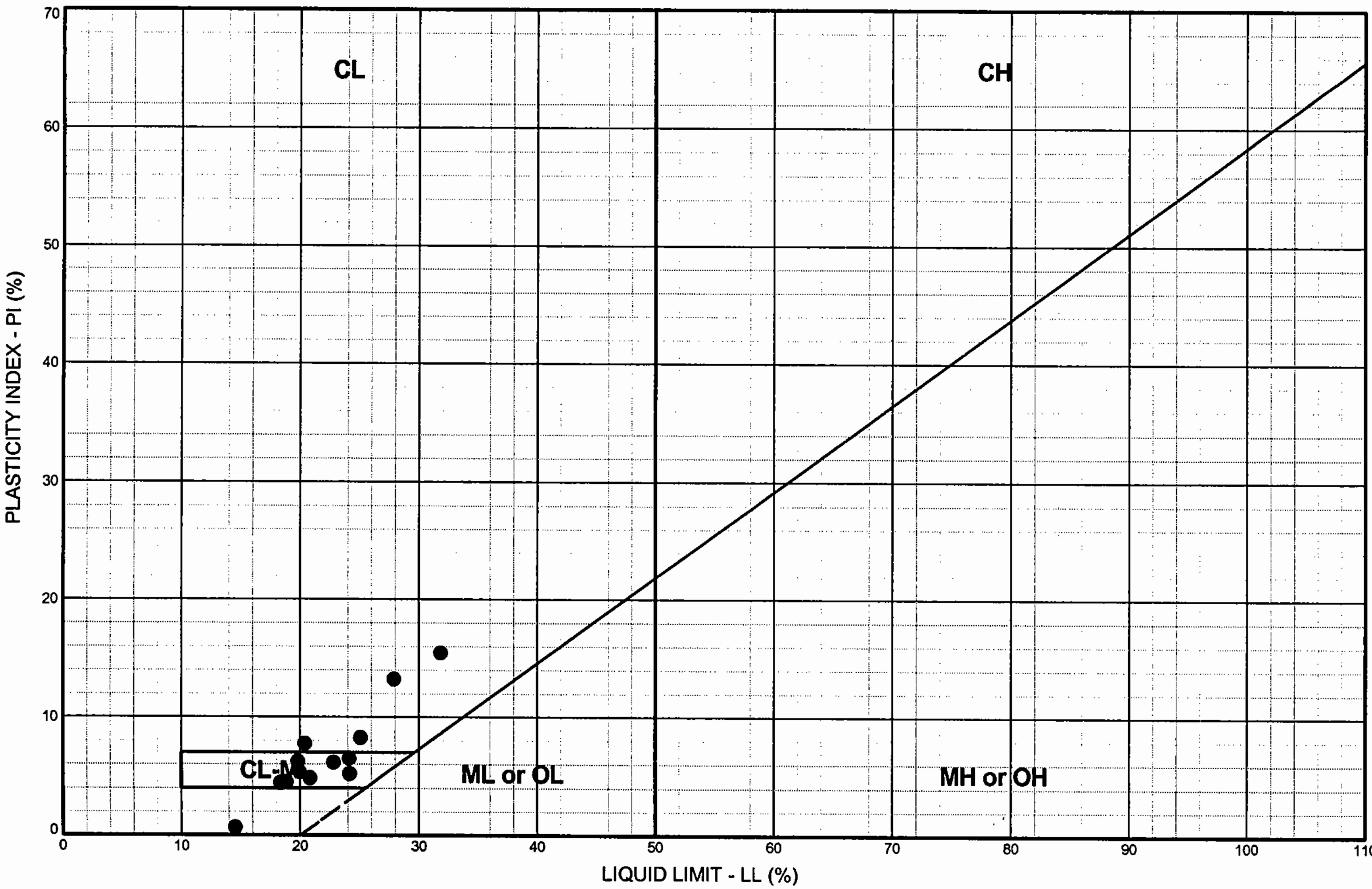
FIG. 29

FIG. 29



LEGEND

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ML or OL: Inorganic and organic silts and clayey silts of low plasticity

MH or OH: Inorganic and organic silts and clayey silts of high plasticity

CL-ML: Silty clays and clayey silts

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PLASTICITY CHART

Geologic Unit: Qvgm

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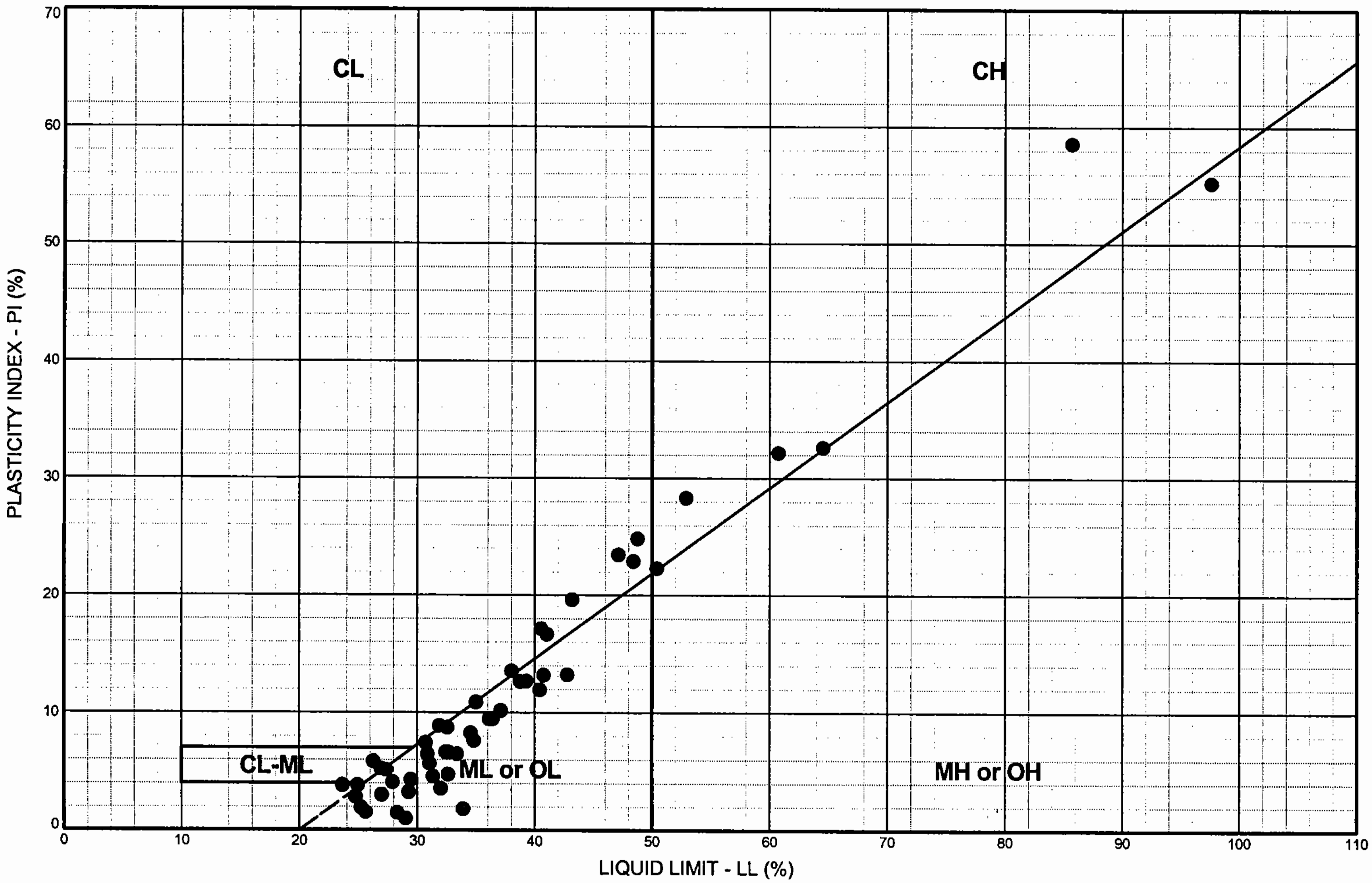
FIG. 30

FIG. 30



LEGEND

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MH or OH: Inorganic and organic silts and clayey silts of high plasticity

CL-ML: Silty clays and clayey silts

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PLASTICITY CHART

Geologic Unit: Qpnl

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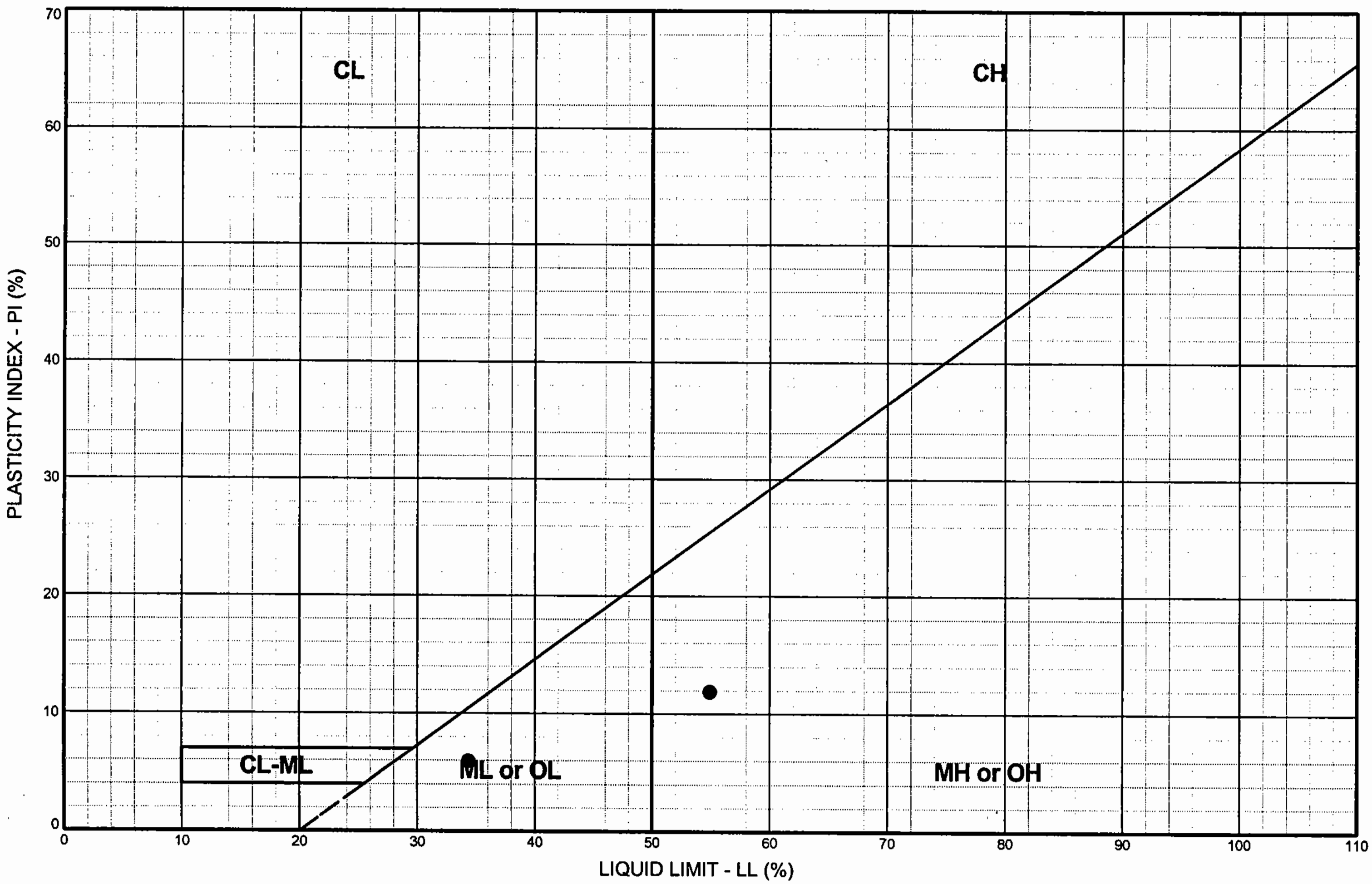
FIG. 31

FIG. 31



LEGEND

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CH: High plasticity inorganic clays

ML or OL: Inorganic and organic silts and clayey silts of low plasticity

MH or OH: Inorganic and organic silts and clayey silts of high plasticity

CL-ML: Silty clays and clayey silts

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PLASTICITY CHART

Geologic Unit: Qpnp

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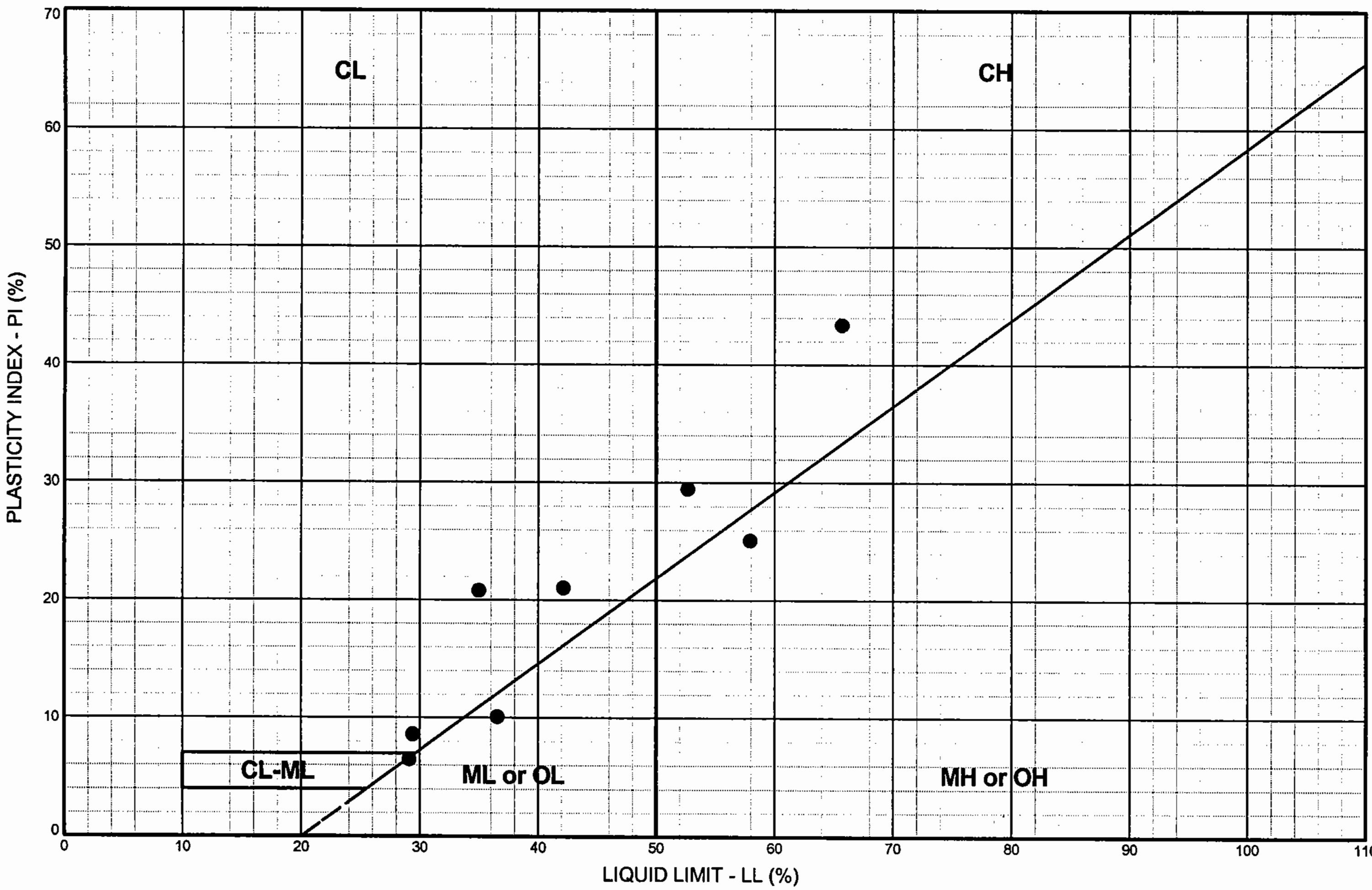
FIG. 32



LEGEND

CL: Low plasticity inorganic clays; sandy and silty clays

CH: High plasticity inorganic clays

ML or OL: Inorganic and organic silts and clayey silts of low plasticity

MH or OH: Inorganic and organic silts and clayey silts of high plasticity

CL-ML: Silty clays and clayey silts

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PLASTICITY CHART

Geologic Unit: Qpns

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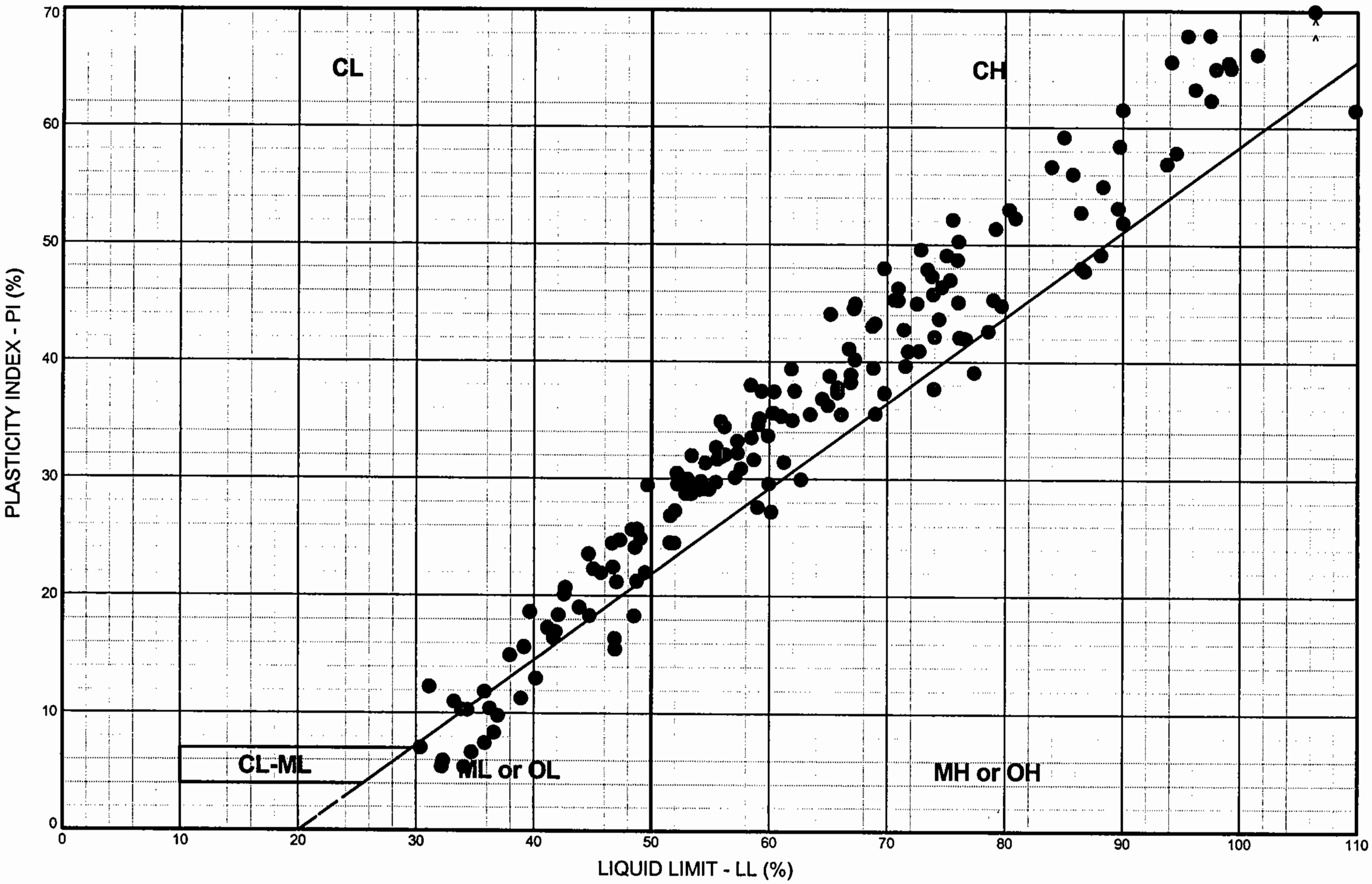
FIG. 33

FIG. 33



LEGEND

CL: Low plasticity inorganic clays; sandy and silty clays

CH: High plasticity inorganic clays

ML or OL: Inorganic and organic silts and clayey silts of low plasticity

MH or OH: Inorganic and organic silts and clayey silts of high plasticity

CL-ML: Silty clays and clayey silts

ATTSUM1 W8100.GPJ SHAN_WIL.GDT 3/23/06

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PLASTICITY CHART
Geologic Unit: Qpgl

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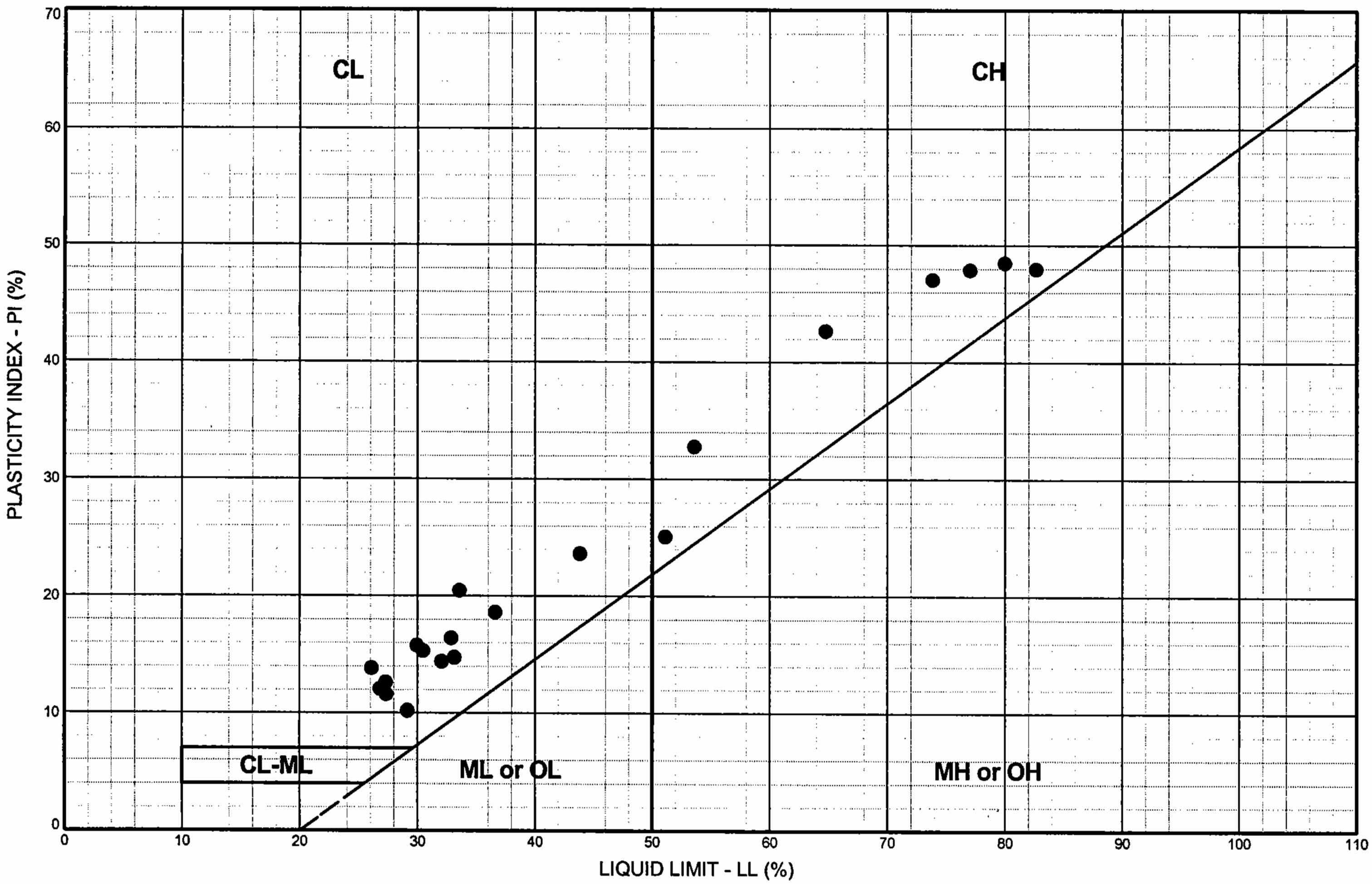
FIG. 34



LEGEND

CL: Low plasticity inorganic clays; sandy and silty clays

CH: High plasticity inorganic clays

ML or OL: Inorganic and organic silts and clayey silts of low plasticity

MH or OH: Inorganic and organic silts and clayey silts of high plasticity

CL-ML: Silty clays and clayey silts

ATTSUM1 W8100.GPJ SHAN_WIL.GDT 3/23/06

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PLASTICITY CHART

Geologic Unit: Qpgm

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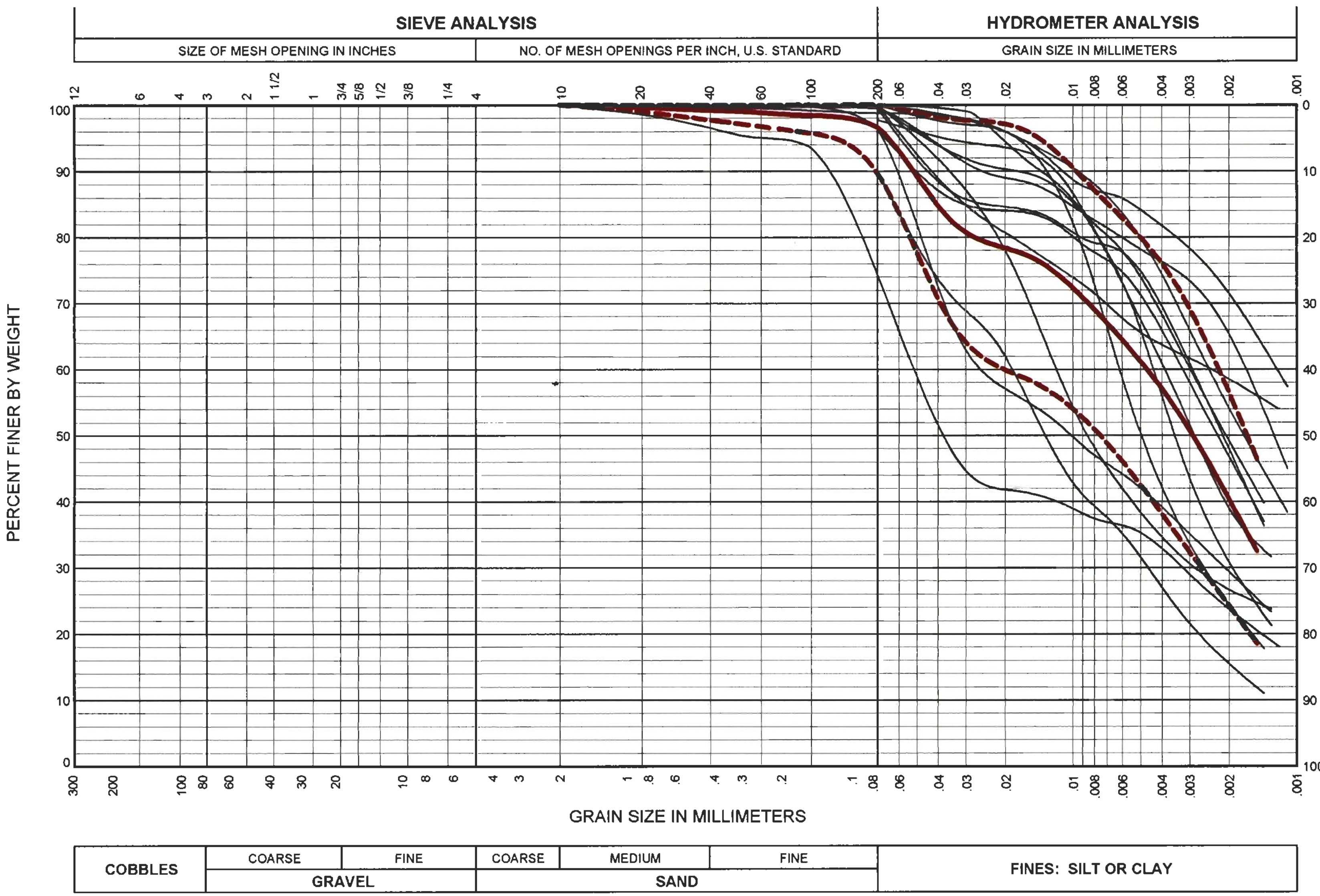
FIG. 35



GSASUM_MANY W8100.GPJ SHAN_WL.GDT 3/22/06

— — — Average - 1 Standard Deviation
——— Average
— — — Average +1 Standard Deviation

Note: Group 2 consists of Qpgl and Qvgl geologic units.

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GRAIN SIZE DISTRIBUTION

GP 2: Cohesive SILT and CLAY

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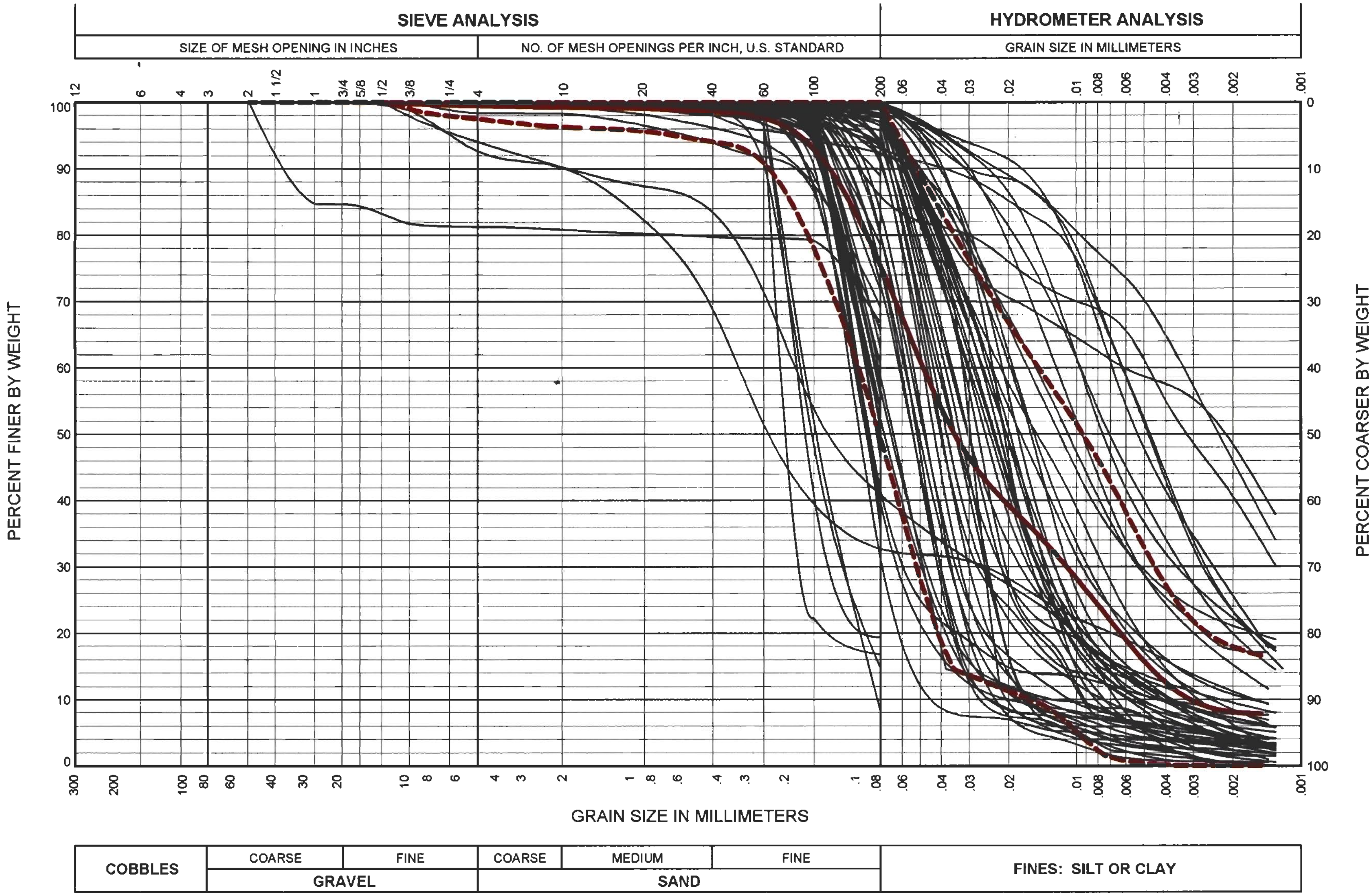
FIG. 36

FIG. 36



GSASUM_MANY W8100.GPJ SHAN_WL.GDT 2/24/06

Average - 1 Standard Deviation
Average
Average +1 Standard Deviation

Note: Group 3 consists of Qpnl and Qpnp geologic units.

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GRAIN SIZE DISTRIBUTION **GP 3: Cohesionless SILT and SAND**
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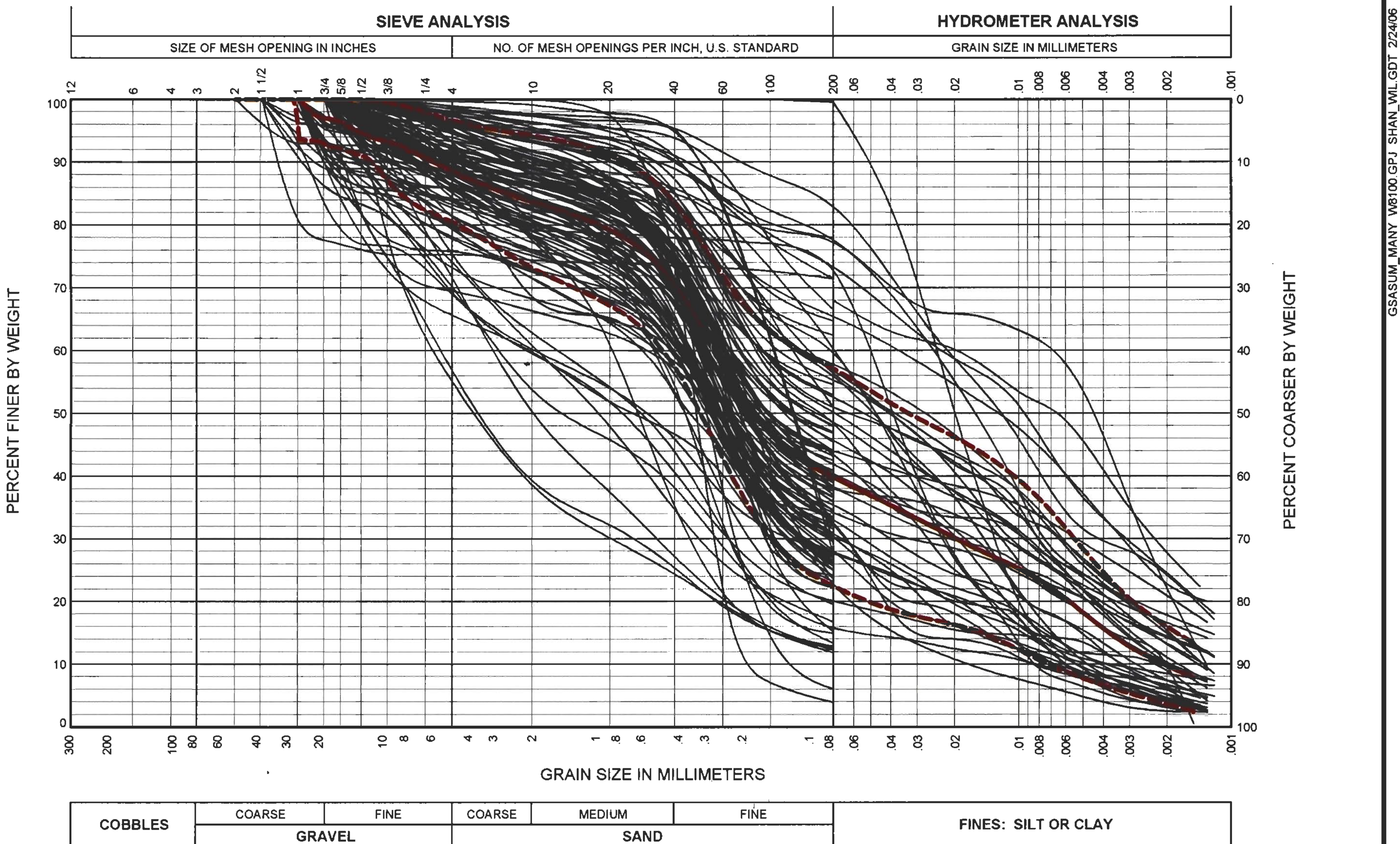
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants — **FIG. 37**

FIG. 37



Average - 1 Standard Deviation
Average
Average +1 Standard Deviation

Note: Group 4 consists of Qpgm, Qpgd, Qvd, Qvt, Qvgm, Qpns, and Qpgt geologic units.

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GRAIN SIZE DISTRIBUTION

GP 4: Till-like deposits

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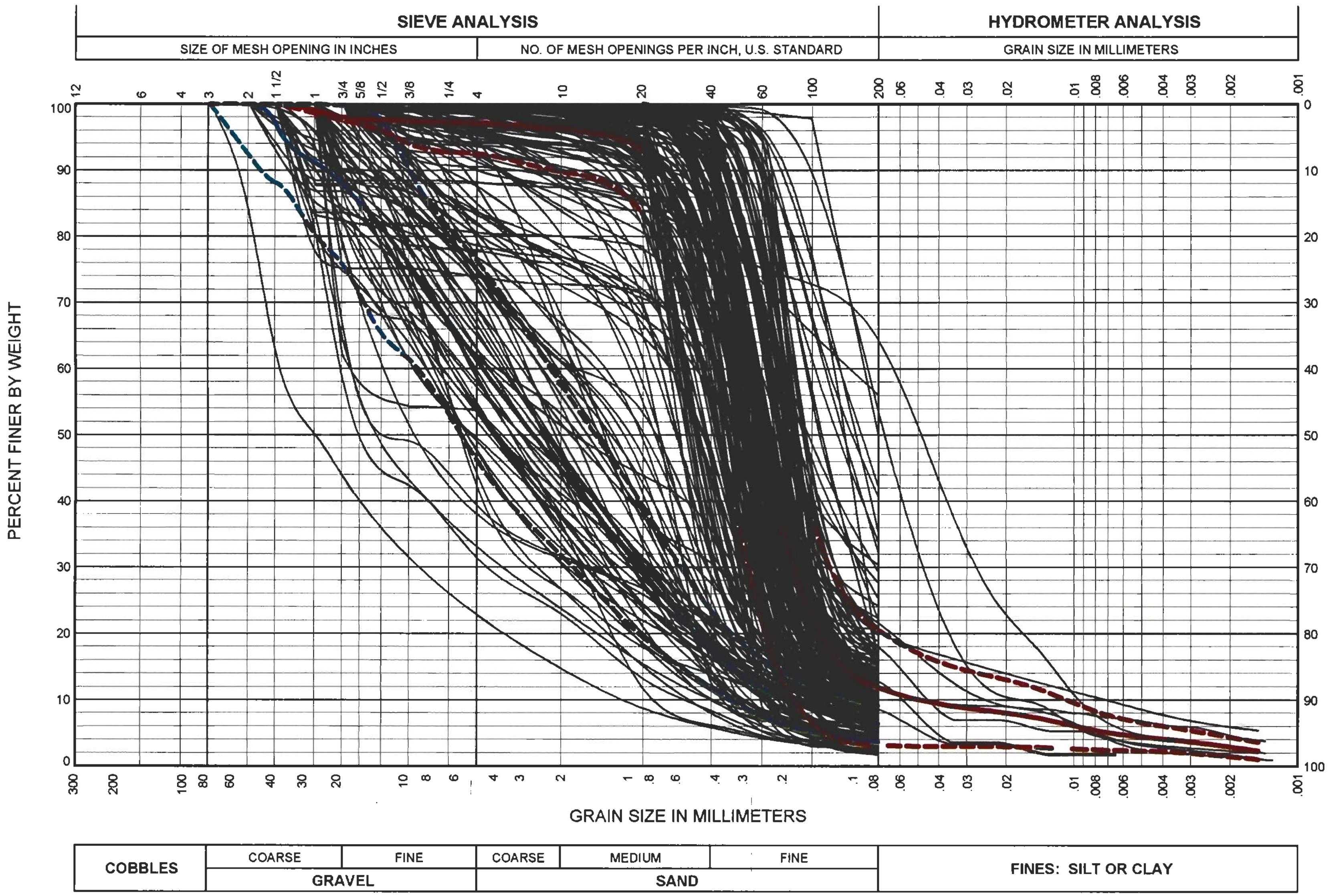
FIG. 38



Average - 1 Standard Deviation
Average
Average +1 Standard Deviation
Average - 1 Standard Deviation
Average
Average +1 Standard Deviation

Note: Group 5 consists of Qvro, Qpgo, Qpnf, and Qva geologic units. Group 5a represents the finer fraction, and Group 5b represents the coarser fraction.

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GRAIN SIZE DISTRIBUTION

GP 5: Cohesionless SAND & GRAVEL

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FIG. 39

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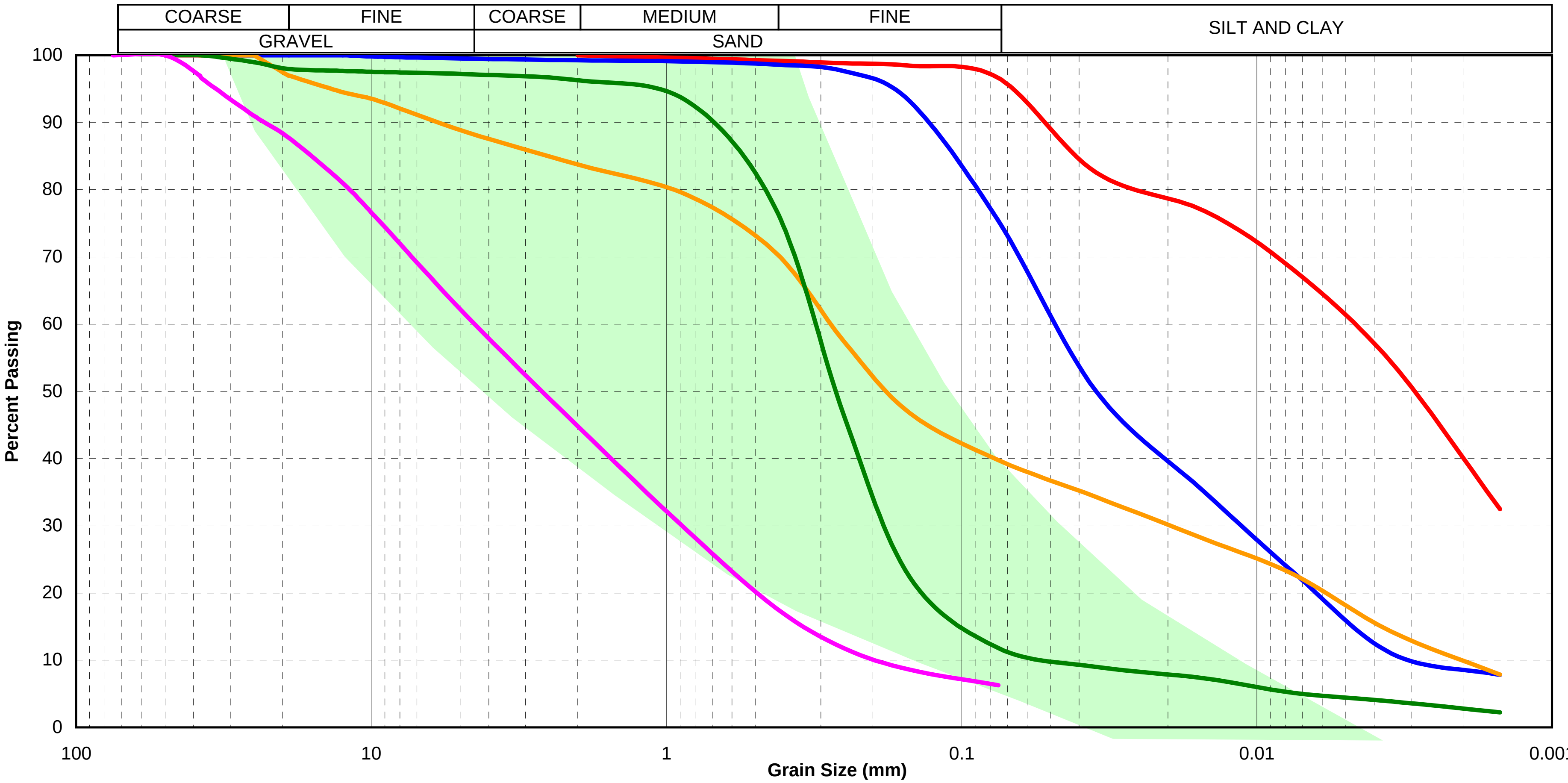
FIG. 39



LEGEND:

- GP 2 Avg: Cohesive SILT and CLAY
- GP 3 Avg: Cohesionless SILT AND SAND
- GP 4 Avg: Till-Like Deposits
- GP 5a Avg: Cohesionless SAND and GRAVEL
- GP 5b Avg: Cohesionless SAND and GRAVEL
- Range for Slurry

NOTES:

1. Slurry gradation limits from: *Maidl, B, Herrenknecht, M., and Anheuser, L., 1996, "Mechanised Shield Tunneling", Berlin, Ernst and Sohn.*
2. See Figures 36 to 39 for grain size analyses and definitions of different soil groups

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SLURRY TBM LIMITS from Herrenknecht

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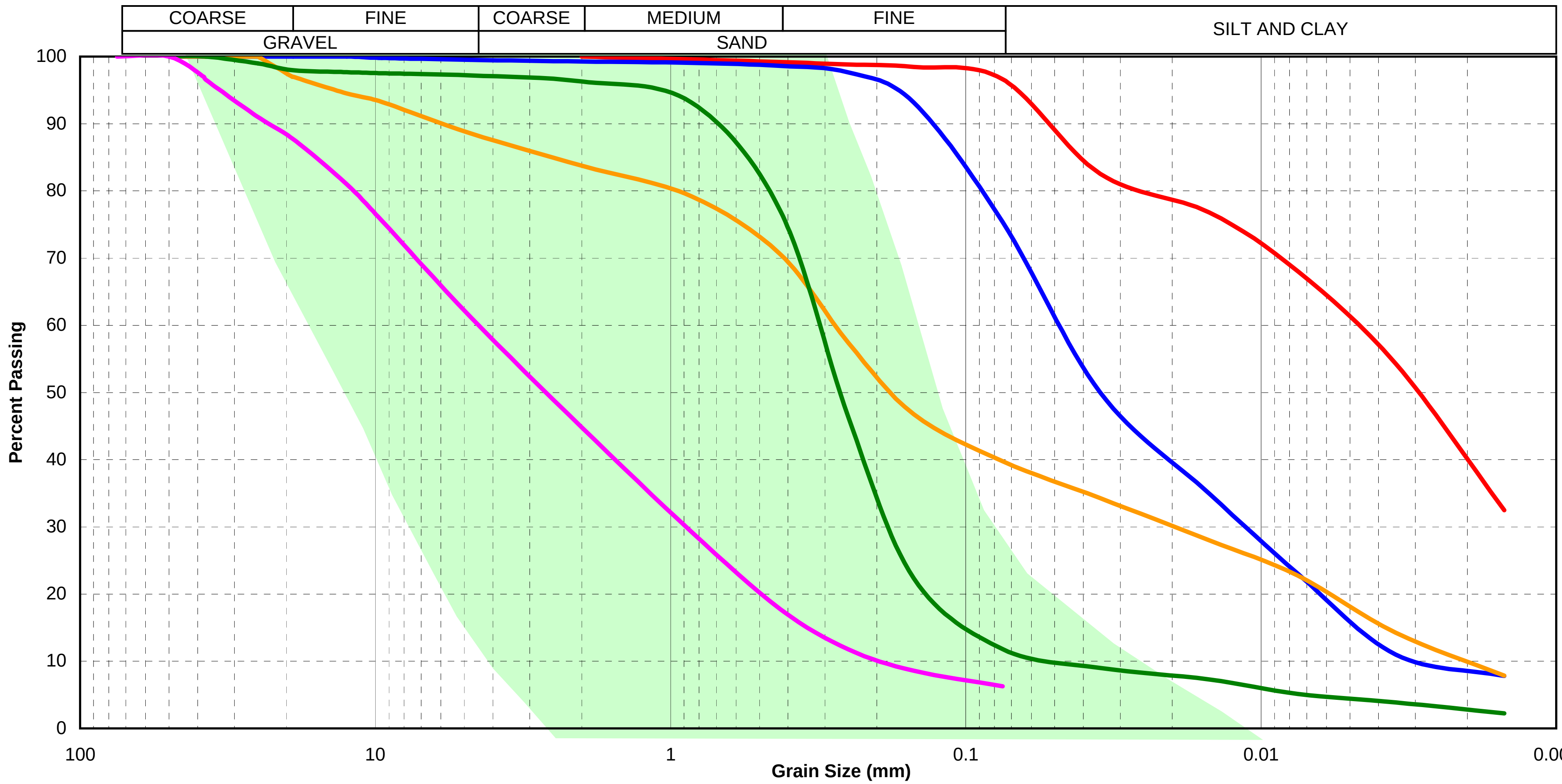
FIG. 40

FIG. 40



LEGEND:

- GP 2 Avg: Cohesive SILT and CLAY
- GP 3 Avg: Cohesionless SILT and SAND
- GP 4 Avg: Till-Like Deposits
- GP 5a Avg: Cohesionless SAND and GRAVEL
- GP 5b Avg: Cohesionless SAND and GRAVEL
- Range for Slurry

NOTES:

1. Slurry gradation limits from:
   Langmaack, L., 2002, "Soil Condition for TBM - Chances and Limits." Proceedings, Conference on Underground Works: Living Structures, Toulose, AFTES, Paris.
2. See Figures 36 through 39 for grain size analyses and definitions of different soil groups

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SLURRY TBM LIMITS
from Langmaack

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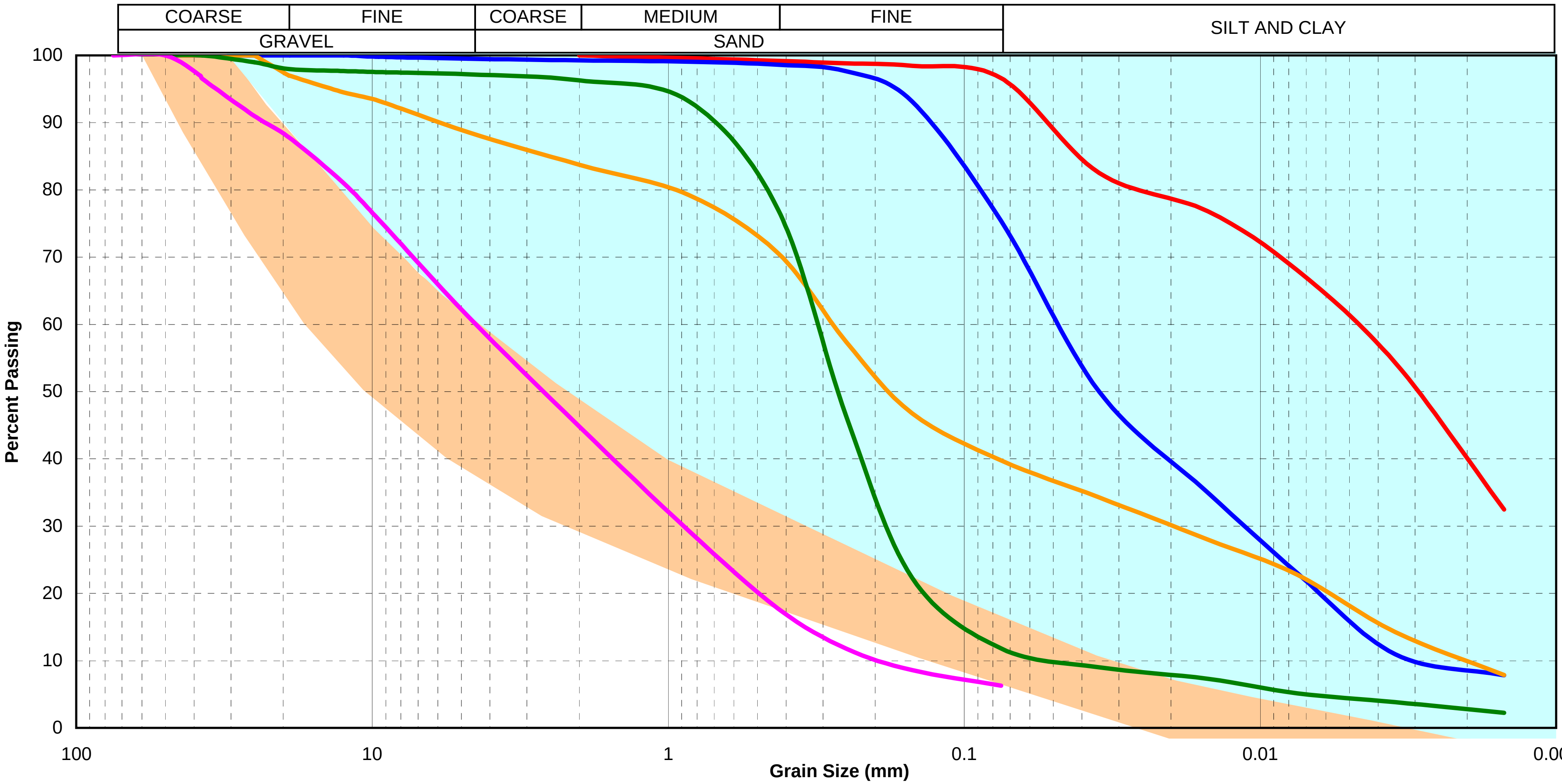
FIG. 41

FIG. 41



LEGEND:

GP 2 Avg: Cohesive SILT and CLAY
GP 3 Avg: Cohesionless SILT and SAND
GP 4 Avg: Till-Like Deposits
GP 5a Avg: Cohesionless SAND and GRAVEL
GP 5b Avg: Cohesionless SAND and GRAVEL
Range for EPB (no water pressure)
Range for EPB

NOTES:

1. EPB gradation limits from:
Maidl, B, Herrenknecht, M., and Anheuser, L., 1996, "Mechanised Shield Tunneling", Berlin, Ernst and Sohn.
2. See Figures 38 to 41 for grain size analyses and definitions of different soil groups
3. EPB = Earth Pressure Balance

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EPB TBM LIMITS
from Herrenknecht

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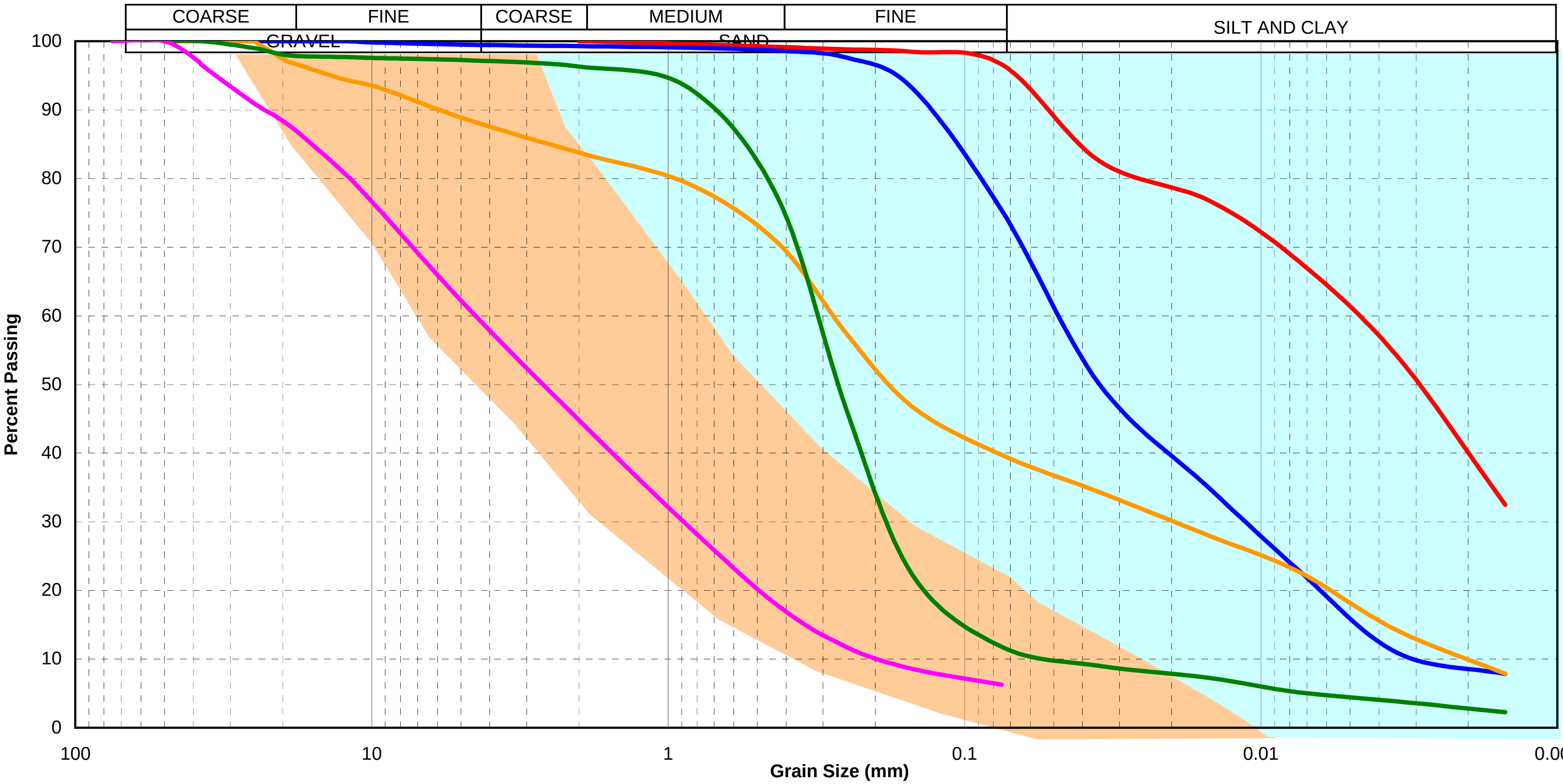
FIG. 42

FIG. 42

4/3/2006-Figs40-43_updated (Univ Lnk).xls-Figs-man



LEGEND:

- GP 2 Avg: Cohesive SILT and CLAY
- GP 3 Avg: Cohesionless SILT and SAND
- GP 4 Avg: Till-Like Deposits
- GP 5a Avg: Cohesionless SAND and GRAVEL
- GP 5b Avg: Cohesionless SAND and GRAVEL
- Range for EPB (with conditioning)
- Range for EPB

NOTES:

1. EPB gradation limits from: *Langmaack, L., 2003, "Europe and Asia: Applications of new TBM" Conditioning Additives," presented at Bauma Tradeshow, available online at www.degussa-ugc.com/NR/rdonlyres.*
2. See Figures 36 to 39 for grain size analyses and definitions of different soil groups
3. EPB = Earth Pressure Balance

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EPB TBM LIMITS
from Langmaack

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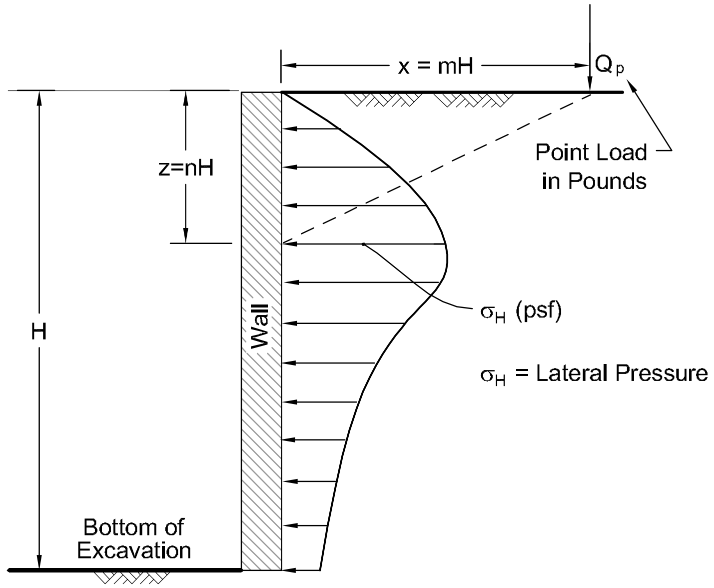
FIG. 43

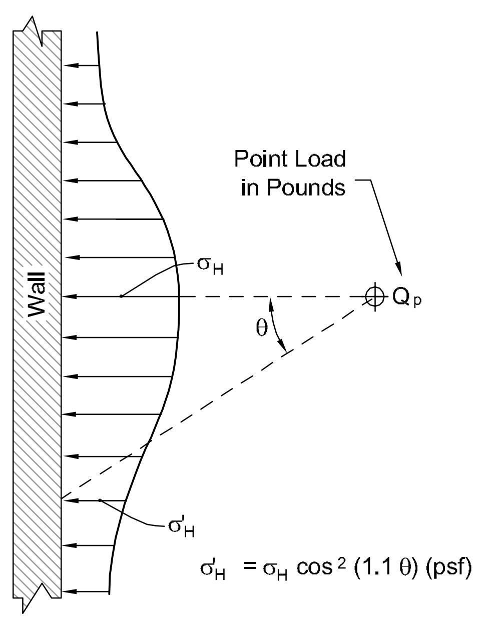
FIG. 43



ELEVATION VIEW

For $m \le 0.4$: $\sigma_H = 0.28 \dfrac{Q_p}{H^2} \dfrac{n^2}{(0.16 + n^2)^3}$ (psf)

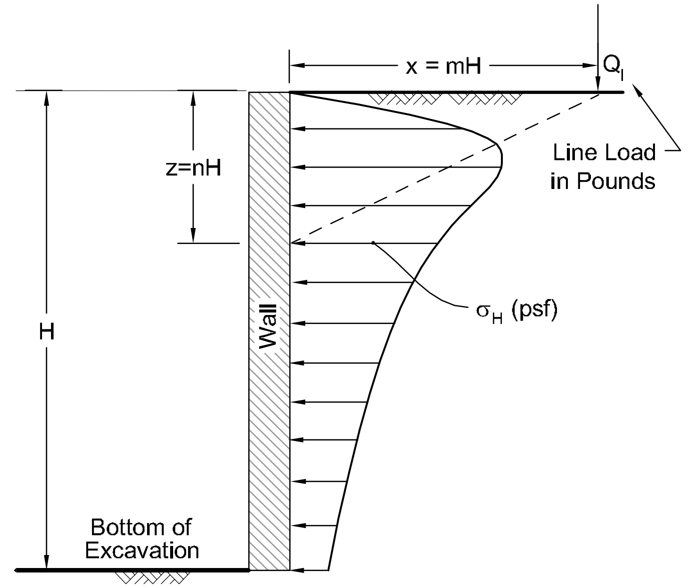
For $m > 0.4$: $\sigma_H = 1.77 \dfrac{Q_p}{H^2} \dfrac{m^2 n^2}{(m^2 + n^2)^3}$ (psf)



PLAN VIEW

A) LATERAL PRESSURE DUE TO POINT LOAD i.e. SMALL ISOLATED FOOTING OR WHEEL LOAD

(NAVFAC DM 7.2, 1986)



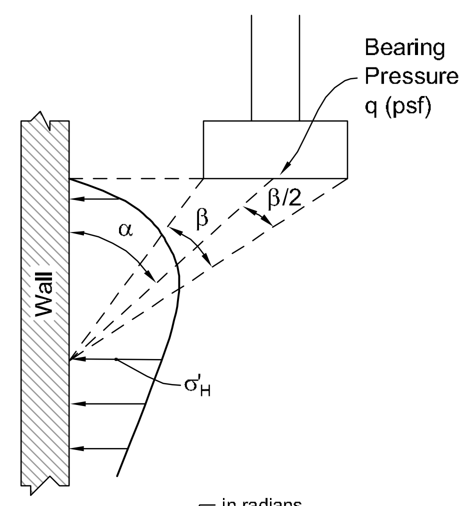
ELEVATION VIEW

For $m \le 0.4$: $\sigma_H = 0.20 \dfrac{Q_l}{H} \dfrac{n}{(0.16 + n^2)^2}$ (psf)

For $m > 0.4$: $\sigma_H = 1.28 \dfrac{Q_l}{H} \dfrac{m^2 n}{(m^2 + n^2)^2}$ (psf)

B) LATERAL PRESSURE DUE TO LINE LOAD i.e. NARROW CONTINUOUS FOOTING PARALLEL TO WALL

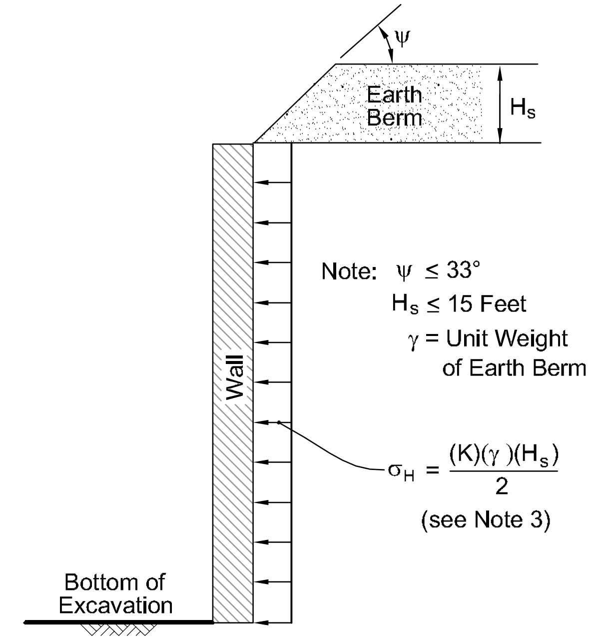
(NAVFAC DM 7.2, 1986)



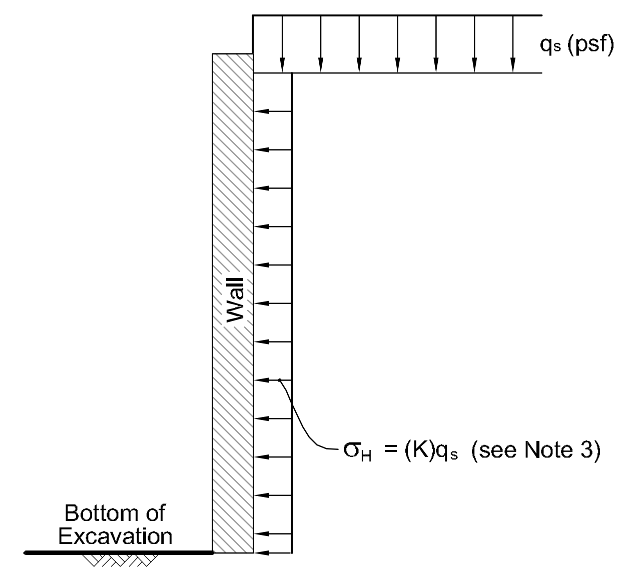
$\sigma_H = \dfrac{2q}{\pi} (\beta - \sin\beta \cos 2\alpha)$ (psf) — in radians

C) LATERAL PRESSURE DUE TO STRIP LOAD

(derived from Fang, *Foundation Engineering Handbook*, 1991)



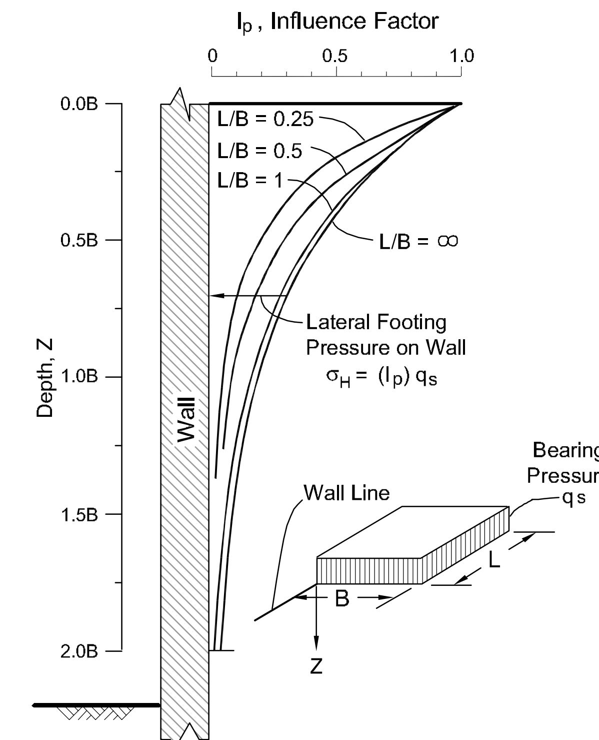
EARTH BERM



UNIFORM SURCHARGE

D) LATERAL PRESSURE DUE TO EARTH BERM OR UNIFORM SURCHARGE

(derived from Poulous and Davis, *Elastic Solutions for Soil and Rock Mechanics*, 1974; and Terzaghi and Peck, *Soil Mechanics in Engineering Practice*, 1967)



E) LATERAL PRESSURE DUE TO ADJACENT SPREAD FOOTING

(derived from NAVFAC DM 7.2, 1986; and Sandhu, *Earth Pressure on Walls Due to Surcharge*, 1974)

NOTES

1. Figures are not drawn to scale.
2. Applicable surcharge pressures should be added to appropriate permanent wall lateral earth and water pressure.
3. For K, use Ka under active pressure conditions and Ko under at-rest pressure conditions. See Table 16 for recommended Ka and Ko values.

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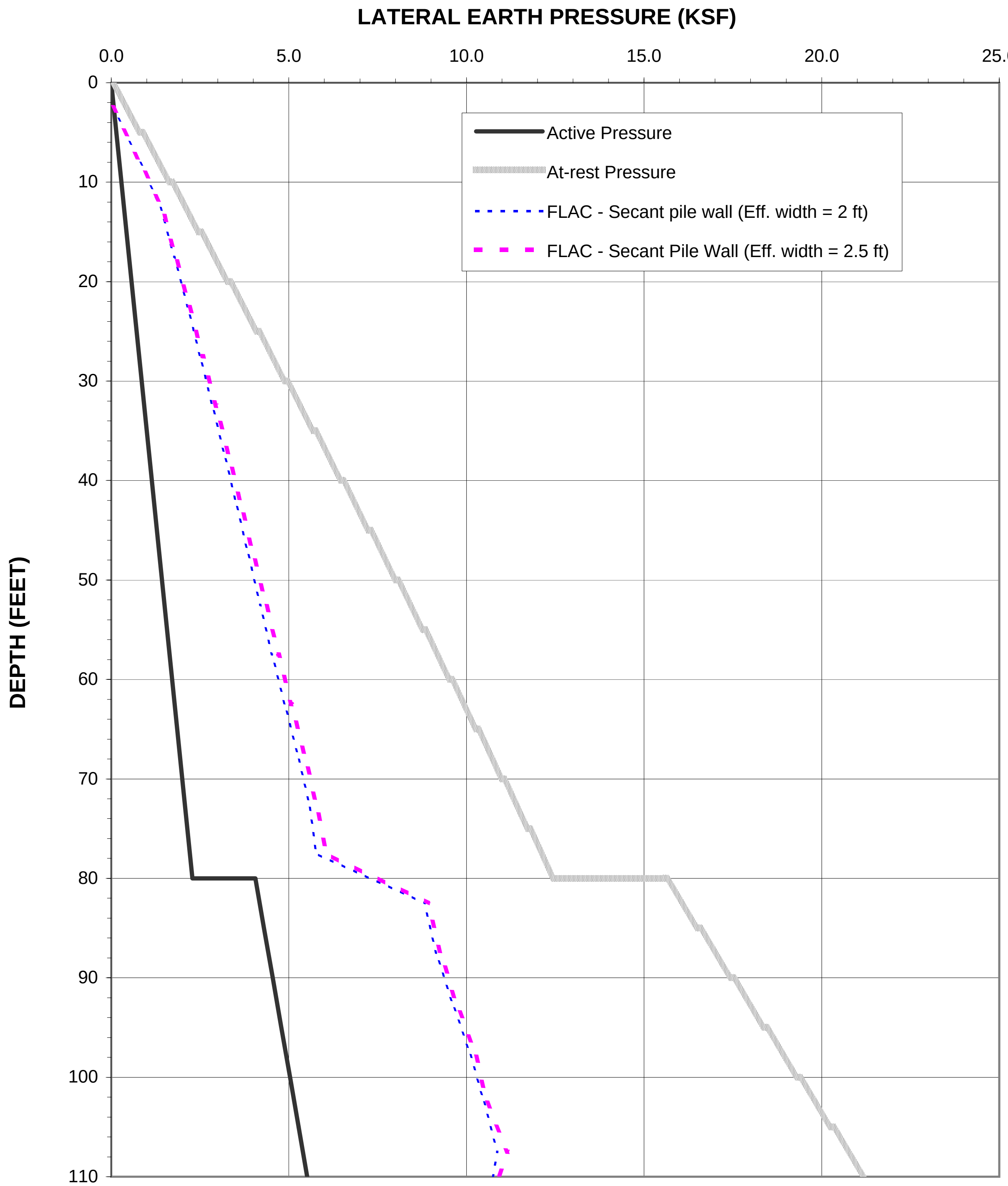
RECOMMENDED SURCHARGE LOADING FOR TEMPORARY AND PERMANENT WALLS

March 2006 | 21-1-08109-074

SHANNON & WILSON, INC. Geotechnical and Environmental Consultants | **FIG. 44**

lmm



NOTES

1. The proposed diameter of the vent shaft at Roanoke Street is 22 feet. The lateral earth pressures presented above are for secant pile walls only because of the relatively small diameter of the shaft.
2. Recommended (FLAC) lateral earth pressures presented above were determined by performing numerical analyses using the two-dimensional finite difference soil-structure interaction program (FLAC) developed by Itasca Consulting Group (1998). The soil properties presented in Table 14 of this report were used in the analyses.
3. Active and at-rest earth pressures presented above were determined using Rankine's earth pressure theory. Rankine's theory does not consider soil arching around circular structures; therefore, we do recommend that the FLAC pressures be used to estimate lateral loads on deep shafts.

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RECOMMENDED LATERAL EARTH PRESSURES
MONTLAKE VENT SHAFT

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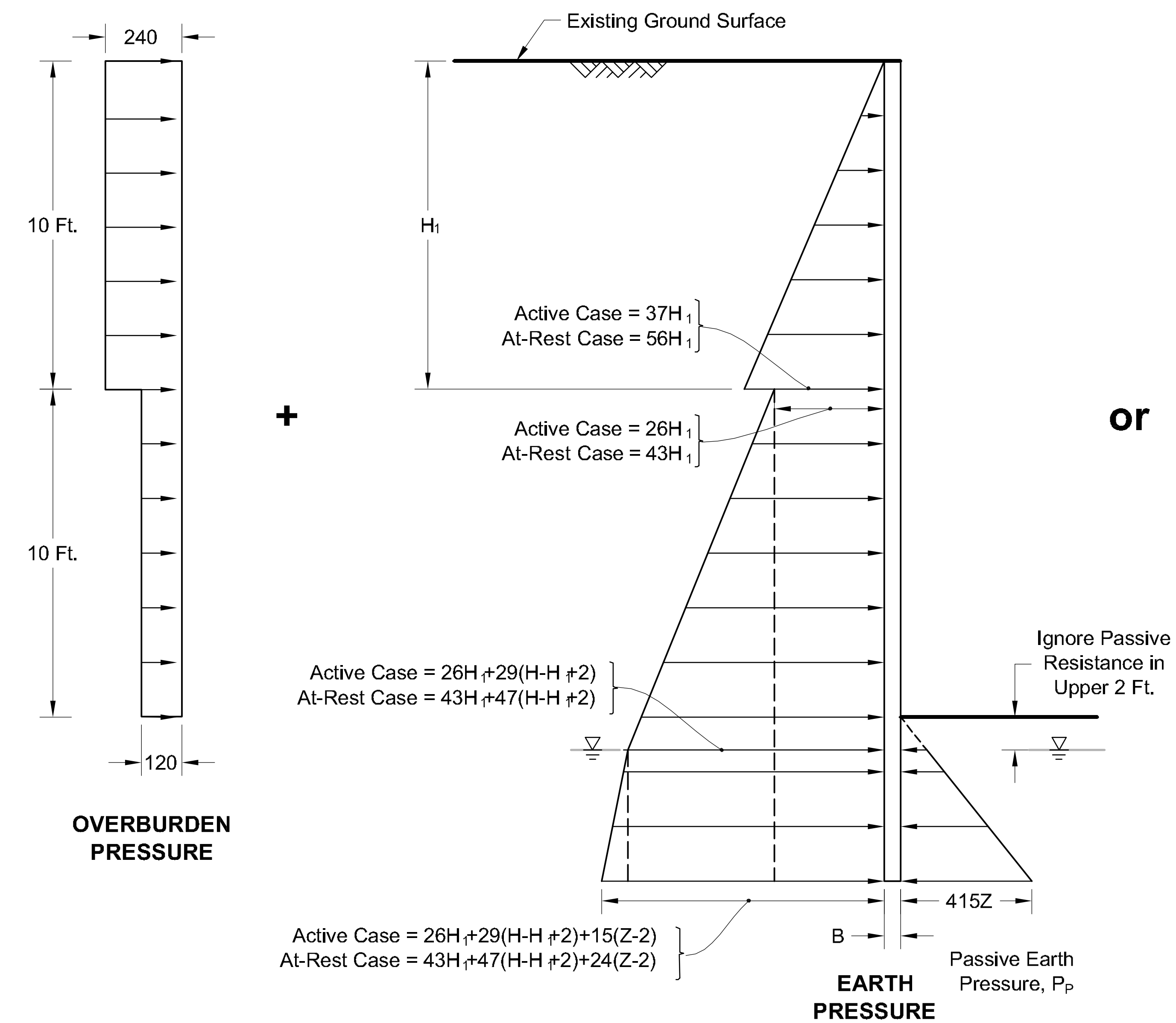
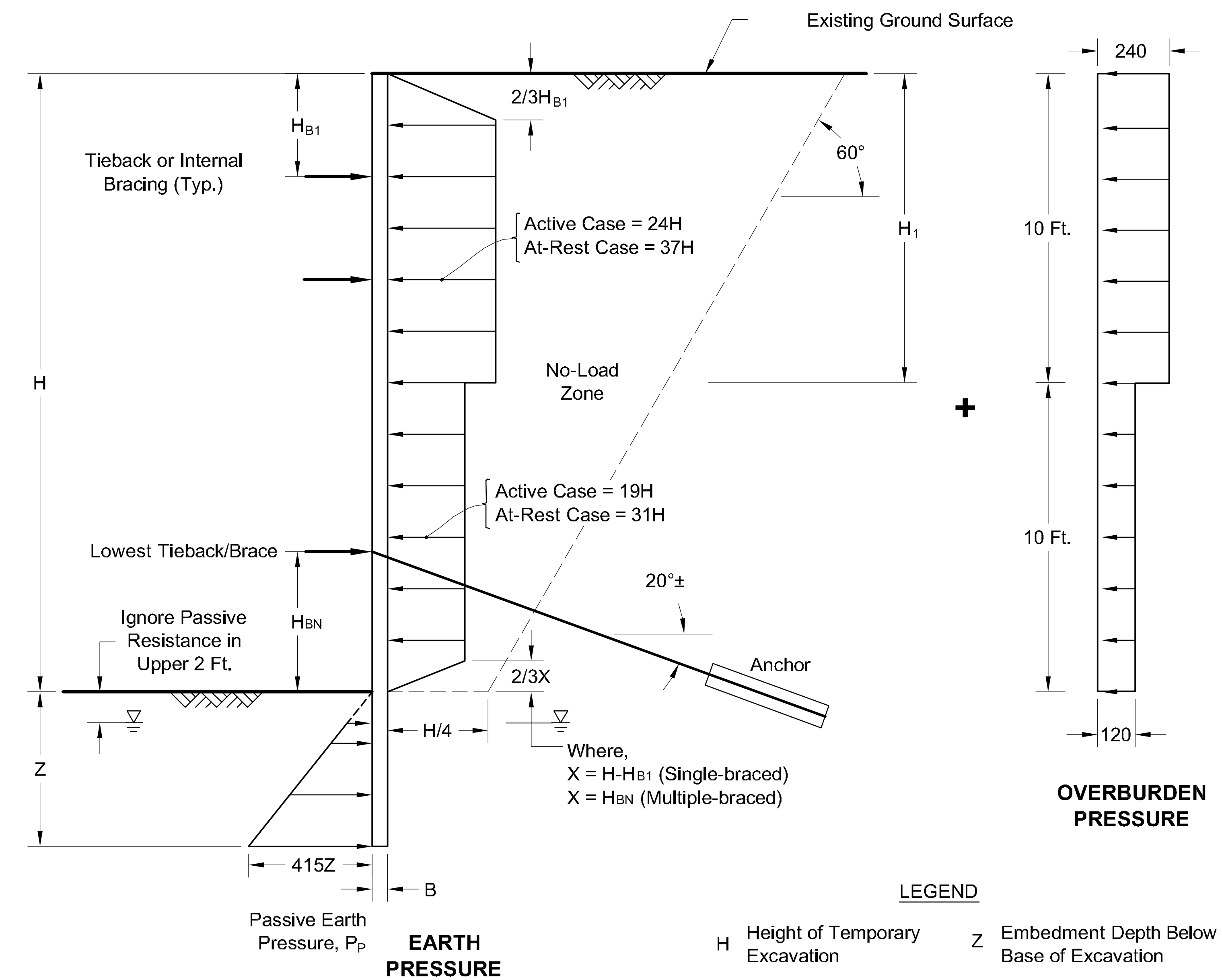
FIG. 45

RECOMMENDED EARTH PRESSURES FOR CANTILEVER WALL



or

RECOMMENDED EARTH PRESSURES FOR SINGLE AND MULTIPLE TIEBACK/BRACED WALL



LEGEND

H	Height of Temporary Excavation	Z	Embedment Depth Below Base of Excavation
H_{B1}	Depth to Uppermost Level of Tieback/Brace from Ground Surface	B	Soldier Pile Width in Feet
H_{BN}	Height of Lowest Tieback/Brace from Base of Excavation	H_1	Depth to Bottom of Group 1 (Refer to Table 10 for description of soil groups.)

NOTES

1. Earth pressures are in psf, Z, B, and H in feet.
2. P_P should be computed as acting on 2 times pile diameter B or pile spacing, whichever is smaller. Active and at-rest pressures above the excavation bottom are assumed to act over the pile spacing. Below the excavation bottom, active and-at rest pressures may be assumed to act over the width of the soldier pile diameter.
3. Determine penetration Z based on moment equilibrium at lowest tieback level.
4. Use 80% of the above pressures for computing moment in piles.
5. Locate anchors behind no-load zone.
6. Use 50% of the above pressures for design of lagging.
7. Allowable vertical pile capacity:
   Unit Skin friction: 1.5 ksf; Unit End bearing: 30 ksf
8. Lateral pressure is based on an assumed traffic surface surcharge of 600 psf acting over a limited influence area. More severe construction equipment loading requires special analysis. Refer to Figure 44 for lateral pressures due to other types of surcharge loading.
9. It is assumed that the site is dewatered prior to and during construction so that hydrostatic pressures do not act on walls above the subgrade level. The groundwater level is assumed to be lowered to at least 2 feet below the bottom of the excavation.
10. Based on available subsurface conditions, contamination may be encountered during excavation and/or dewatering.
11. Based on available subsurface information, H_1 is assumed to be approximately 10 feet.
12. Earth pressures assume horizontal backslope. If a sloping ground surface exists, the provided earth pressures should be increased depending on the slope angle.
13. Provided earth pressures do not include pressure increase due to seismic loading.

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RECOMMENDATIONS FOR DESIGN OF TEMPORARY WALLS AT CAPITOL HILL STATION

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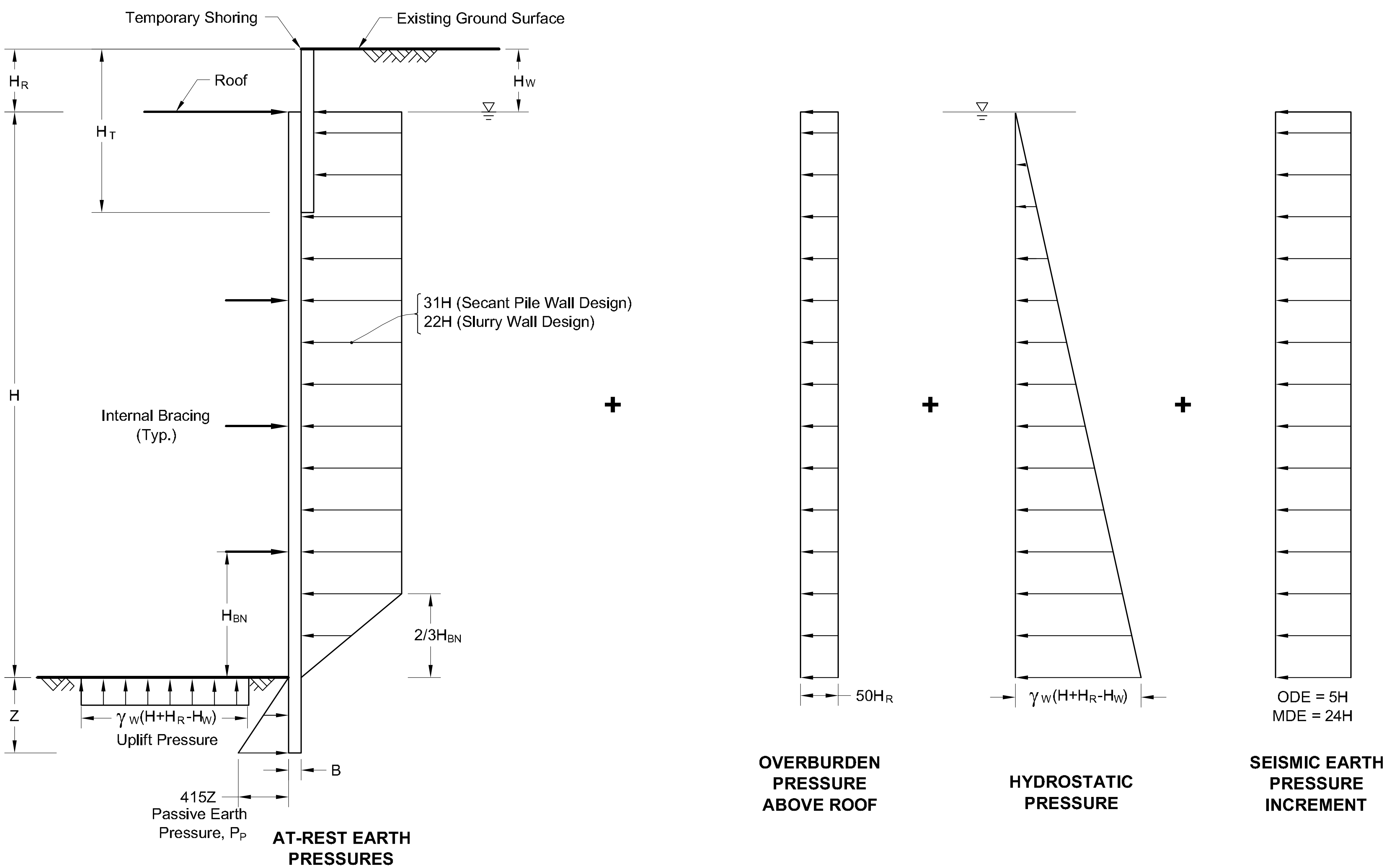
FIG. 46

File: J:\211\08109-074\Earth Pressure Diagrams\21-1-08113-270 Fig 46.dwg Date: 03-29-2006 Author: SAC



LEGEND

H, H_T	Height of Excavation (Permanent and Temporary, Respectively)
H_R	Depth of Roof Below Ground Surface
H_W	Depth to Groundwater Table
H_{BN}	Height of Lowest Brace from Base of Excavation
Z	Embedment Depth Below Base of Excavation
B	Slurry Wall or Secant Pile Width in Feet
ODE	Operating Design Earthquake (PGA = 0.18g)
MDE	Maximum Design Earthquake (PGA = 0.77g)
γ_W	Unit Weight of Water

NOTES

1. Earth pressures are in psf; Z, B, and H are in feet.

2. Lateral earth pressures assume permanent walls are internally braced.

3. Lateral pressure due to overburden is based on an assumed soil weight above roof acting over an infinite influence area. Refer to Figure 44 for additional lateral pressures due to other types of surcharge loading.

4. Per PSTC preliminary design drawings, the roof is assumed to be the top level of bracing for the permanent structure.

5. Recommendations for design of temporary shoring wall are included on Figure 46.

6. Based on information available at this time, H_W was assumed to be 10 feet. This value is an approximation and should be evaluated after additional subsurface information becomes available.

7. The provided seismic earth pressure increments are based on a seismic earth pressure determination method developed by Zhang et. al. (1998). Refer to report for full reference. A deflection approach should be considered for seismic design in future phases.

8. For design of tiedowns to resist uplift forces, the recommended allowable unit skin friction is 1.5 ksf for tiedowns installed with a continuous flight auger. If pressure-grouted micropiles are used, an ultimate load transfer of 8 kips per lineal foot is recommended. The uplift resistance should be at least 10 percent greater than the calculated uplift force.

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RECOMMENDATIONS FOR DESIGN OF PERMANENT WALLS AT CAPITOL HILL STATION

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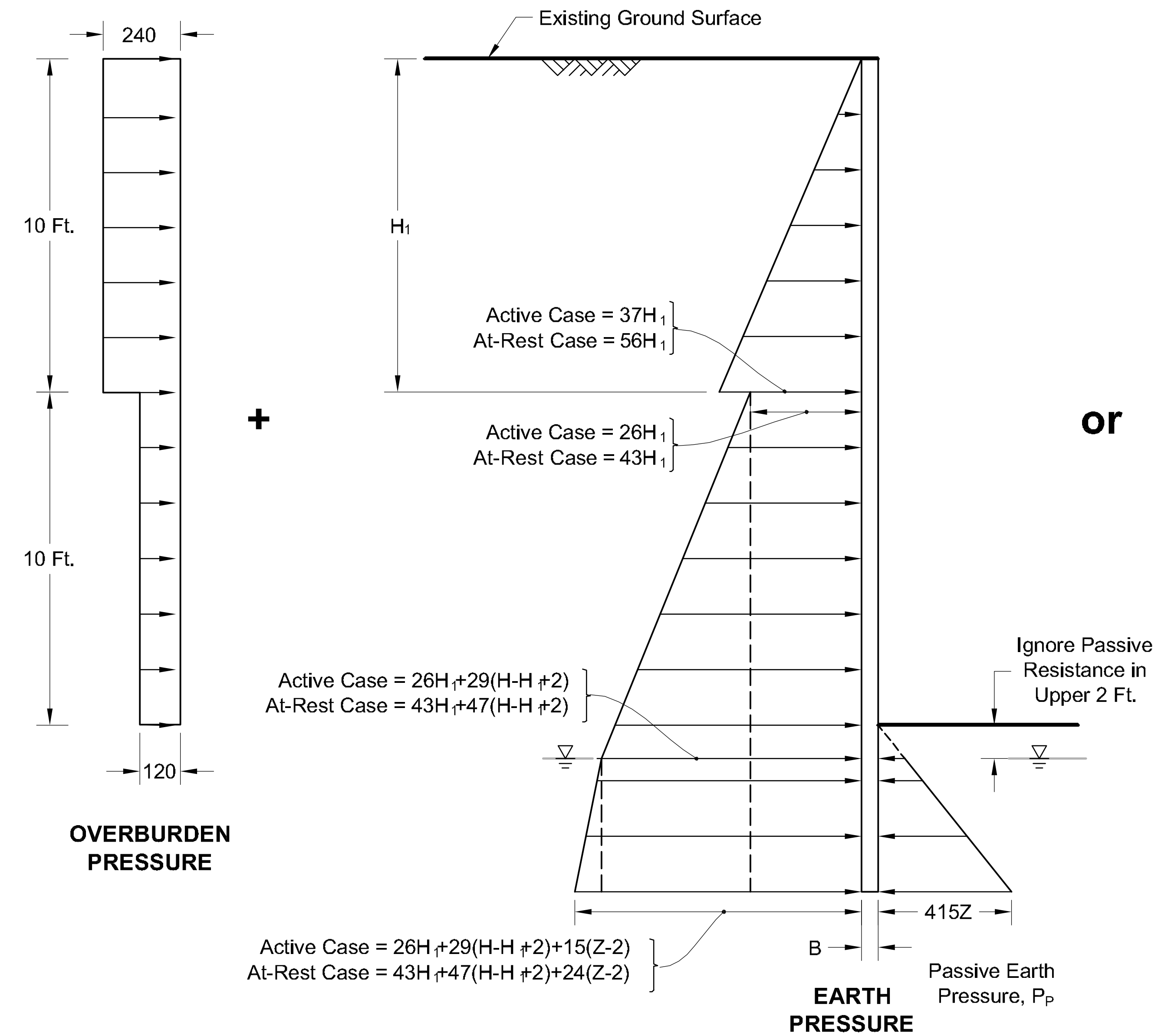
FIG. 47

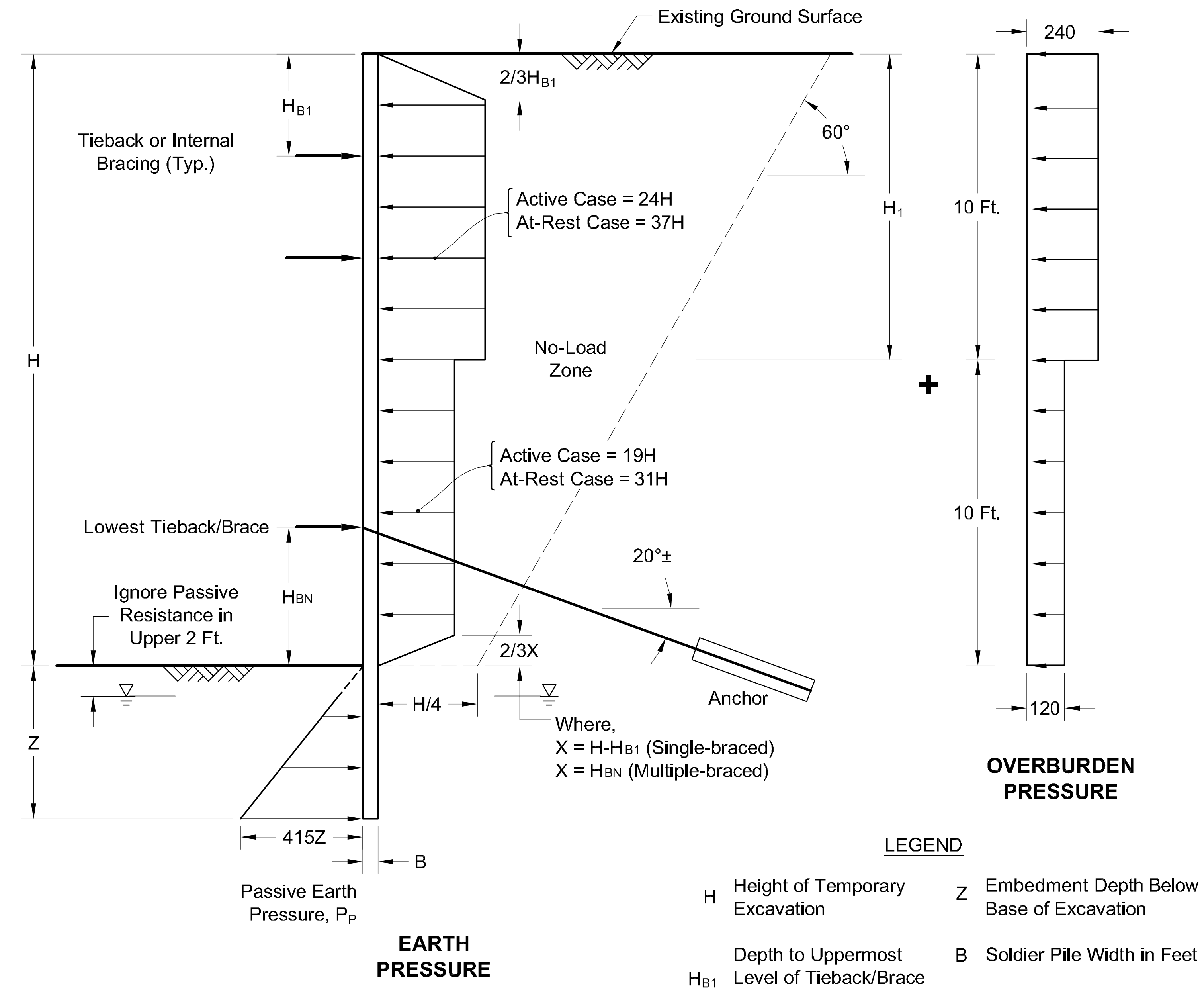
File: J:\211\08109-074\Earth Pressure Diagrams\21-1-08113-270 Fig 47.dwg Date: 03-29-2006 Author: SAC

RECOMMENDED EARTH PRESSURES FOR CANTILEVER WALL



or

RECOMMENDED EARTH PRESSURES FOR SINGLE AND MULTIPLE TIEBACK/BRACED WALL



LEGEND

H	Height of Temporary Excavation	Z	Embedment Depth Below Base of Excavation
H_{B1}	Depth to Uppermost Level of Tieback/Brace from Ground Surface	B	Soldier Pile Width in Feet
H_{BN}	Height of Lowest Tieback/Brace from Base of Excavation	H_1	Depth to Bottom of Group 1 (Refer to Table 10 for description of soil groups.)

NOTES

1. Earth pressures are in psf, Z, B, and H in feet.
2. P_P should be computed as acting on 2 times pile diameter B or pile spacing, whichever is smaller. Active and at-rest pressures above the excavation bottom are assumed to act over the pile spacing. Below the excavation bottom, active and at-rest pressures may be assumed to act over the width of the soldier pile diameter.
3. Determine penetration Z based on moment equilibrium at lowest tieback level.
4. Use 80% of the above pressures for computing moment in piles.
5. Locate anchors behind no-load zone.
6. Use 50% of the above pressures for design of lagging.
7. Allowable vertical pile capacity:
   Unit Skin friction: 1.5 ksf; Unit End bearing: 30 ksf
8. Lateral pressure is based on an assumed traffic surface surcharge of 600 psf acting over a limited influence area. More severe construction equipment loading requires special analysis. Refer to Figure 44 for lateral pressures due to other types of surcharge loading.
9. It is assumed that the site is dewatered prior to and during construction so that hydrostatic pressures do not act on walls above the subgrade level. The groundwater level is assumed to be lowered to at least 2 feet below the bottom of the excavation.
10. Based on available subsurface conditions, contamination may be encountered during excavation and/or dewatering.
11. Based on available subsurface information, H_1 is assumed to be approximately 10 feet.
12. Earth pressures assume horizontal backslope. If a sloping ground surface exists, the provided earth pressures should be increased depending on the slope angle.
13. Provided earth pressures do not include pressure increase due to seismic loading.

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RECOMMENDATIONS FOR DESIGN OF TEMPORARY WALLS AT UNIVERSITY OF WASHINGTON STATION AND CROSSOVER

March 2006 21-1-08109-074

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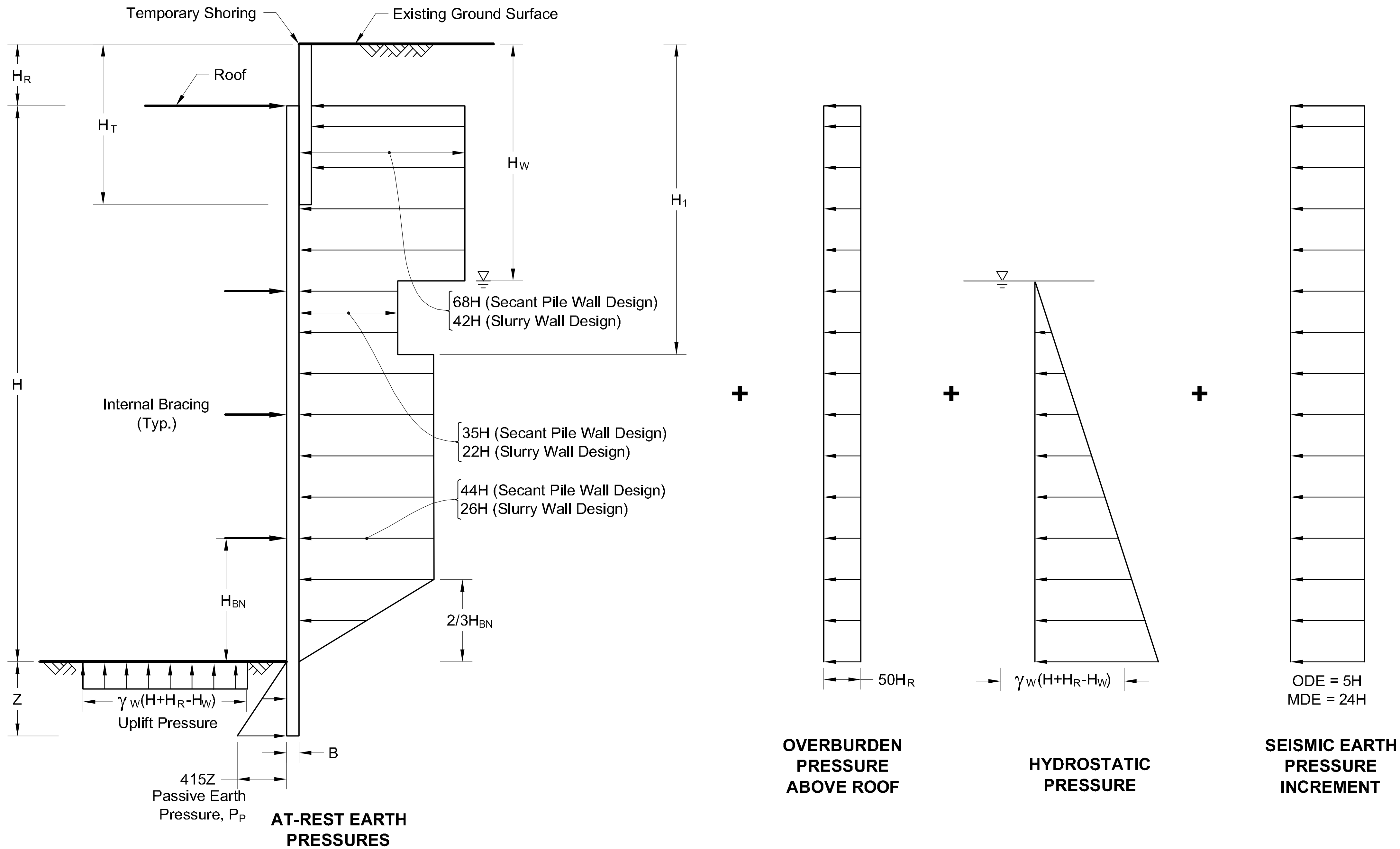
FIG. 48

File: J:\211\08109-074\Earth Pressure Diagrams\21-1-08113-270 Fig 48.dwg Date: 03-29-2006 Author: SAC



LEGEND

H, H_T	Height of Excavation (Permanent and Temporary, Respectively)
H_R	Depth of Roof Below Ground Surface
H_W	Depth to Groundwater Table
H_1	Depth to Top of Group 2 or Group 5 Soils/ Bottom of Group 4 Soils (Refer to Table 10 for description of soil groups.)
H_{BN}	Height of Lowest Brace from Base of Excavation
Z	Embedment Depth Below Base of Excavation
B	Slurry Wall or Secant Pile Width in Feet
ODE	Operating Design Earthquake (PGA = 0.18g)
MDE	Maximum Design Earthquake (PGA = 0.77g)
γ_W	Unit Weight of Water

NOTES

1. Earth pressures are in psf; Z, B, and H are in feet.
2. Lateral earth pressures assume permanent walls are internally braced.
3. Lateral pressure due to overburden is based on an assumed soil weight above roof acting over an infinite influence area. Refer to Figure 44 for additional lateral pressures due to other types of surcharge loading.
4. Per PSTC preliminary design drawings, the roof is assumed to be the top level of bracing for the permanent structure.
5. Recommendations for design of temporary shoring wall are included on Figure 48.
6. Based on information available at this time, H_W was assumed to be 30 feet. This value is an approximation and should be evaluated after additional subsurface information becomes available.
7. Based on available subsurface information at this time, H_1 is assumed to be approximately 60 feet.
8. The provided seismic earth pressure increments are based on a seismic earth pressure determination method developed by Zhang et. al. (1998). Refer to report for full reference. A deflection approach should be considered for seismic design in future phases.
9. For design of tiedowns to resist uplift forces, the recommended allowable unit skin friction is 1.5 ksf for tiedowns installed with a continuous flight auger. If pressure-grouted micropiles are used, an ultimate load transfer of 8 kips per lineal foot is recommended. The uplift resistance should be at least 10 percent greater than the calculated uplift force.

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RECOMMENDATIONS FOR DESIGN OF PERMANENT WALLS AT UNIVERSITY OF WASHINGTON STATION AND CROSSOVER

March 2006 21-1-08109-074

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FIG. 49

File: J:\211\08109-074\Earth Pressure Diagrams\21-1-08113-270 Fig 49.dwg Date: 03-29-2006 Author: SAC

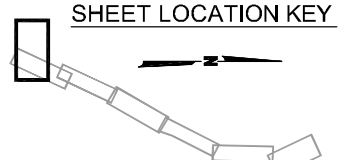
STATIONING BETWEEN MATCHLINES

NB 1020+00 to 1065+00

0 200 400

Scale in Feet

SHEET LOCATION KEY



LEGEND

Zone of Influence

0.2-inch Contour Interval

1.0-inch Contour Interval

NOTES

1. Figure based on electronic files provided by PSTC. Alignment "N35_L00_KA.dwg" received 3-7-06.
2. Vertical datum: NAVD88.

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SETTLEMENT CONTOURS
UNIVERSITY LINK ALIGNMENT

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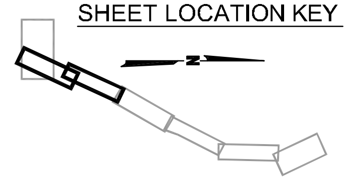
FIG. 50
Sheet 1 of 4

File: J:\211\08109-074\21-1-08109-074 Sediment Contours.dwg Date: 03-29-2006 Author: SAC

STATIONING BETWEEN MATCHLINES

NB 1065+00 to 1085+00
NB 1085+00 to 1115+00

SHEET LOCATION KEY



0 200 400

Scale in Feet

LEGEND

Zone of Influence

0.2-inch Contour Interval

1.0-inch Contour Interval

NOTES

1. Figure based on electronic files provided by PSTC. Alignment "N35_L00_KA.dwg" received 3-7-06.

2. Vertical datum: NAVD88.

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SETTLEMENT CONTOURS
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FIG. 50
Sheet 2 of 4

File: J:\211\08109-074\21-1-08109-074 Sediment Contours.dwg Date: 03-29-2006 Author: SAC

STATIONING BETWEEN MATCHLINES
NB 1115+00 to 1145+00 and NB 1145+00 to 1175+00

SHEET LOCATION KEY

0 200 400
Scale in Feet

LEGEND

Zone of Influence

0.2-inch Contour Interval

1.0-inch Contour Interval

NOTES

1. Figure based on electronic files provided by PSTC. Alignment "N35_L00_KA.dwg" received 3-7-06.

2. Vertical datum: NAVD88.

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SETTLEMENT CONTOURS
UNIVERSITY LINK ALIGNMENT

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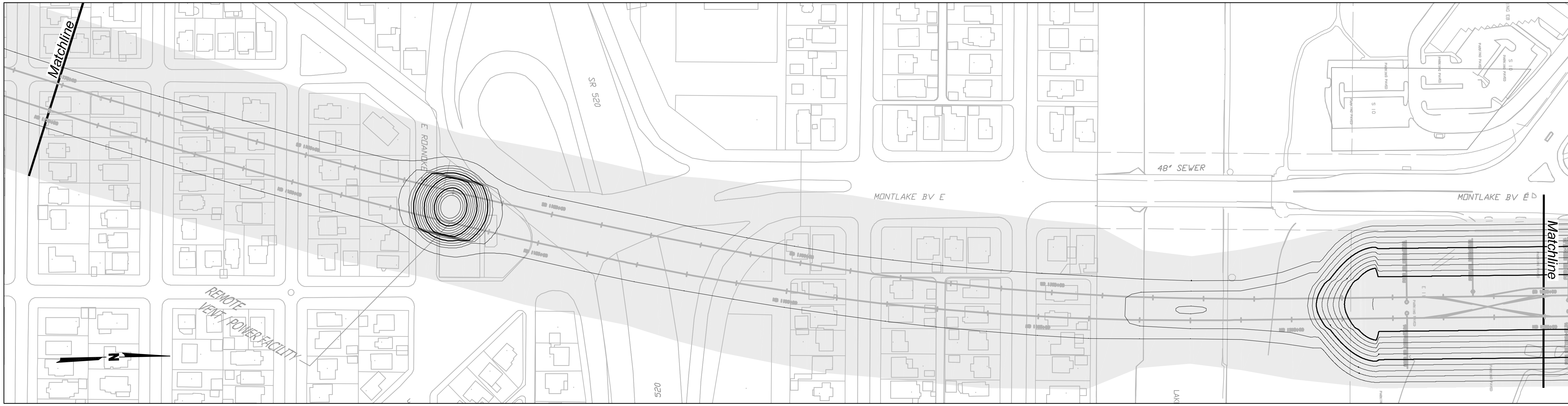
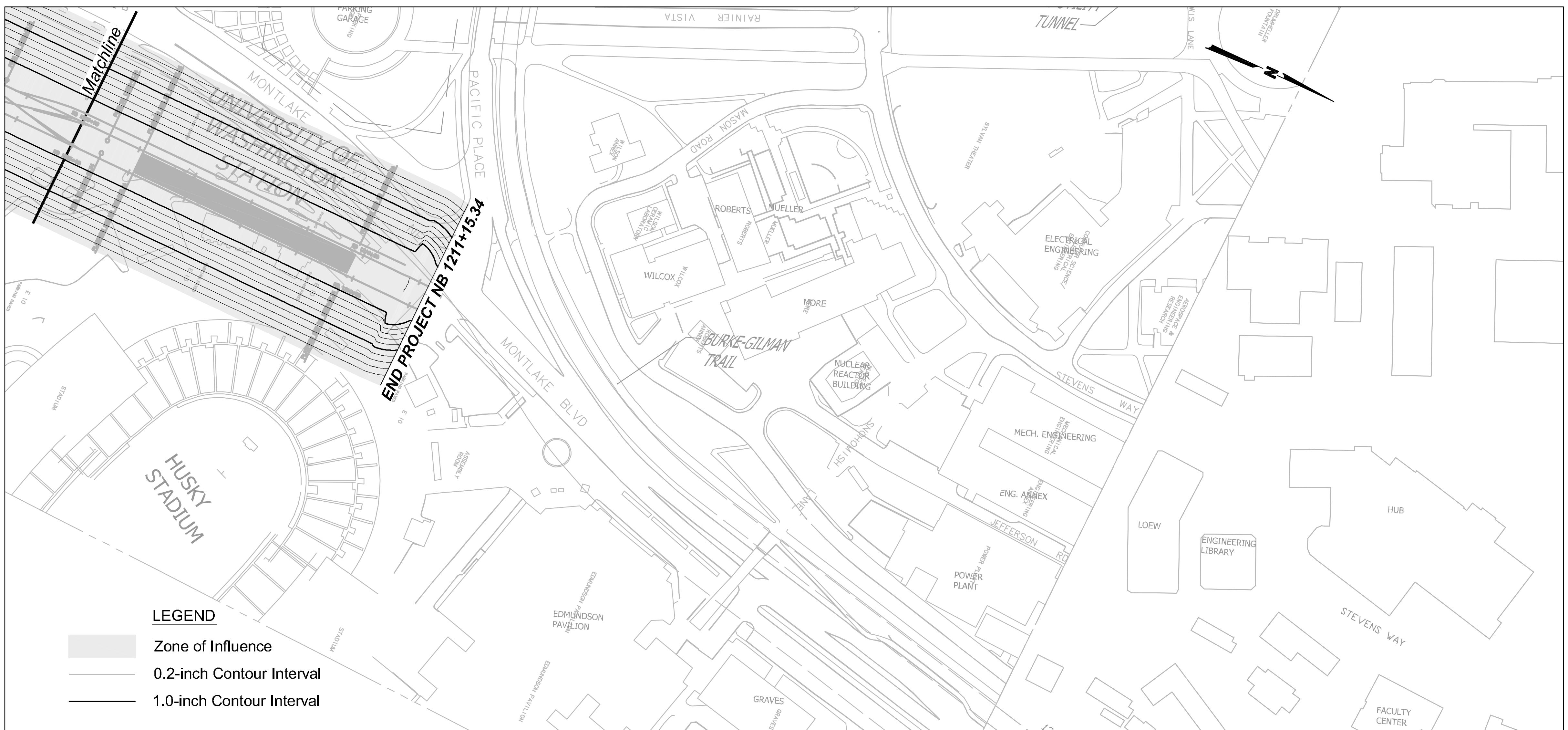
FIG. 50
Sheet 3 of 4

File: J:\211\08109-074\21-1-08109-074 Sediment Contours.dwg Date: 03-29-2006 Author: SAC





LEGEND

- Zone of Influence
- 0.2-inch Contour Interval
- 1.0-inch Contour Interval

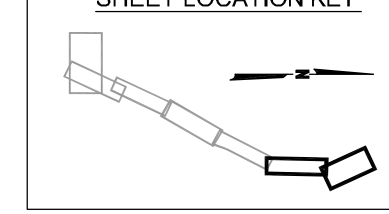
NOTES

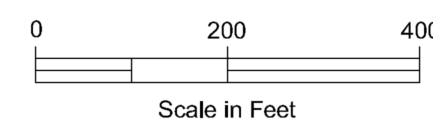
1. Figure based on electronic files provided by PSTC. Alignment "N35_L00_KA.dwg" received 3-7-06.
2. Vertical datum: NAVD88.

STATIONING BETWEEN MATCHLINES

NB 1175+00 to 1205+00
and NB 1205+00 to 1230+00

SHEET LOCATION KEY





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SETTLEMENT CONTOURS
UNIVERSITY LINK ALIGNMENT

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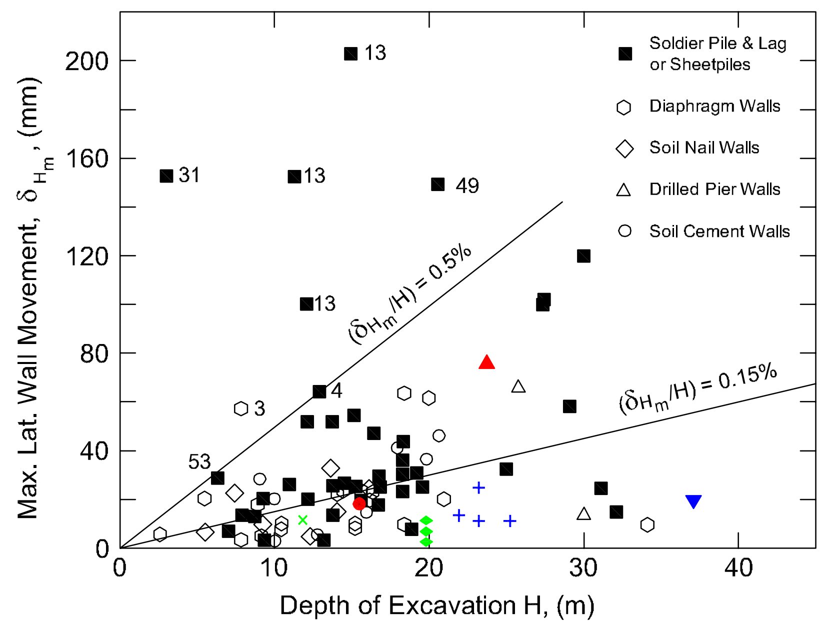
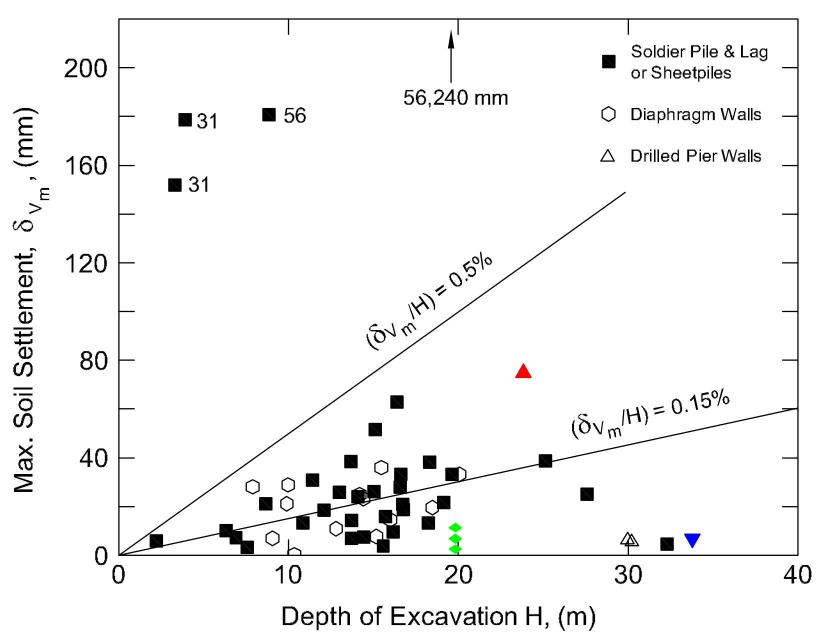
FIG. 50
Sheet 4 of 4

File: J:\211\08109-074\21-1-08109-074 Sediment Contours.dwg Date: 03-29-2006 Author: SAC



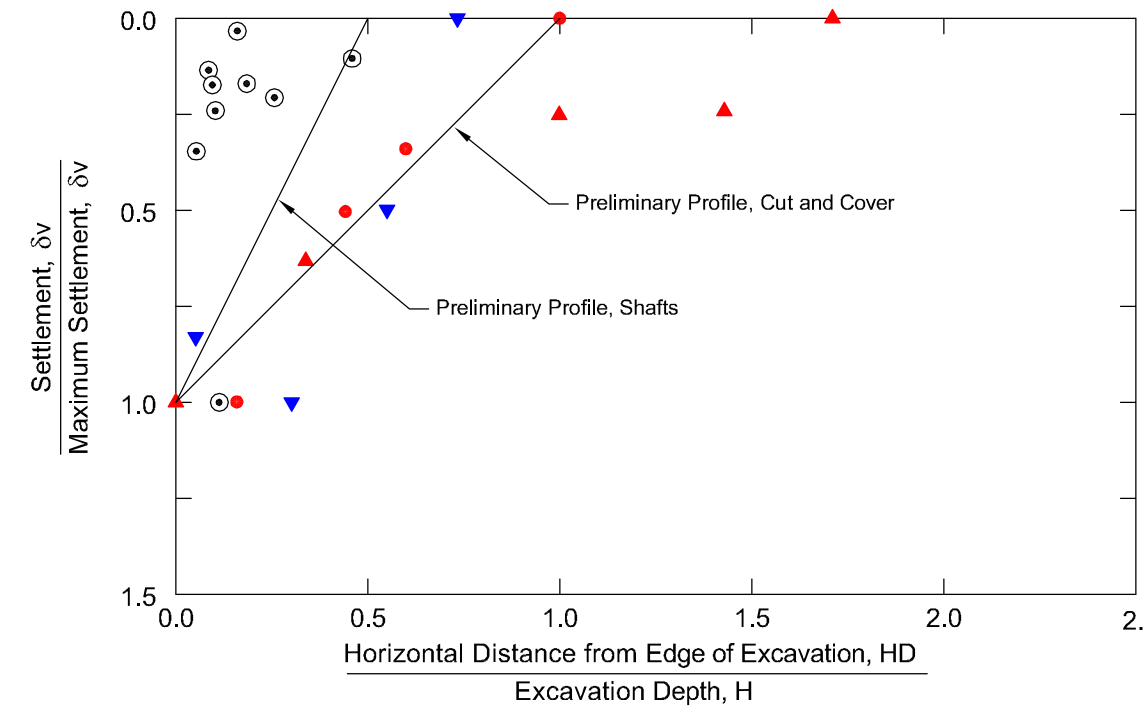
Observed Maximum Lateral Movements for In Situ Walls in Stiff Clays, Residual Soils, and Sands

FIGURE A



Observed Maximum Soil Settlements for In Situ Walls in Stiff Clays, Residual Soils, and Sands

FIGURE B



Settlement Profiles and Zones of Influence

FIGURE C

LEGEND

- DSTP, Borst et. al. (1990)
- Pacific First Center, Winter (1990)
- Bank of California, Clough et. al. (1972)
- Seattle First National Bank, Shannon and Strazer (1970)*
- Columbia - Seafirst Center, Grant et.al. (1984)
- Benaroya Hall, Mitchell and Nykamp (1998)
- Beacon Hill Test Shaft, (unpublished)

* The maximum vertical and horizontal movements reportedly occurred at the top of the soldier piles. The maximum deformations include movements that occurred due to inadequate vertical bearing capacity of the soldier piles.

NOTE

Figures A and B are based on figures from Clough, G.W. and O'Rourke, T.D., "Construction Induced Movements of Insitu Walls", in Design & Performance of Earth Retaining Structures , Special Publication No. 25, June 1990,used with permission of American Society of Civil Engineers.

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EXCAVATION-INDUCED GROUND MOVEMENTS

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FIG. 51