Discipline Report

Geology and Soils

Prepared by:
Shannon & Wilson, Inc.

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Seattle Department of Transportation
Agreement No. T01-34

Draft EIS

Magnolia Bridge Replacement

City of Seattle

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Purpose

The purpose of this project is to replace the existing Magnolia Bridge structure, approaches, and related arterial connections with facilities that maintain convenient and reliable vehicular and non-motorized access between the Magnolia community and the rest of the City of Seattle. The bridge provides an important link to the Magnolia community in Seattle (see Figure 1 and Figure 2). Since the existing bridge also provides the only public vehicular access to the land between North Bay, also referred to as Terminal 91, Smith Cove Park, Elliott Bay Marina, and U.S. Navy property, the project purpose also includes maintenance of access to these areas.

Need

Structural Deficiencies

The City of Seattle has identified the Magnolia Bridge as an important bridge that should remain standing following a "design" seismic event (an earthquake with a peak ground acceleration of 0.3g that is anticipated to happen every 475 years and may measure 7.5 on the Richter Scale). Even with the repairs completed following the February 2001 earthquake, the existing bridge is susceptible to severe damage and collapse from an earthquake that is less severe than the "design" seismic event.

The original bridge was constructed in 1929 and has been modified, strengthened, and repaired several times. The west end of the bridge was damaged by a landslide in 1997, requiring repair and replacement of existing bridge columns and bracing, the construction of six additional supports, and a retaining wall north of the bridge to stabilize the bluff from further landslides. Repairs after the 2001 earthquake included replacement of column bracing at 27 of the 81 bridge supports. A partial seismic retrofit of the single-span bridge structure over 15th Avenue West was completed in 2001. The other spans were not upgraded.

Inspections of the bridge conclude that the concrete structure is showing signs of deterioration. The concrete is cracking and spalling at many locations, apparently related to corrosion of the reinforcing steel. The bridge requires constant maintenance in order to maintain its load capacity, but there does not appear to be any immediate load capacity problem. The existing foundations have insufficient capacity to handle the lateral load and uplift forces that would be generated by a "design" seismic event. The existing foundations to not extend below the soils that could liquefy during a "design" seismic event. If the soils were to liquefy, the foundations would loose their vertical load carrying ability and the structure would collapse.

System Linkage

There are three roadway connections from the Magnolia community, of over 20,000 residents, to the rest of Seattle. As the southernmost of the three connections, the Magnolia Bridge is the most direct route for much of south and west Magnolia to downtown Seattle and the regional freeway system.

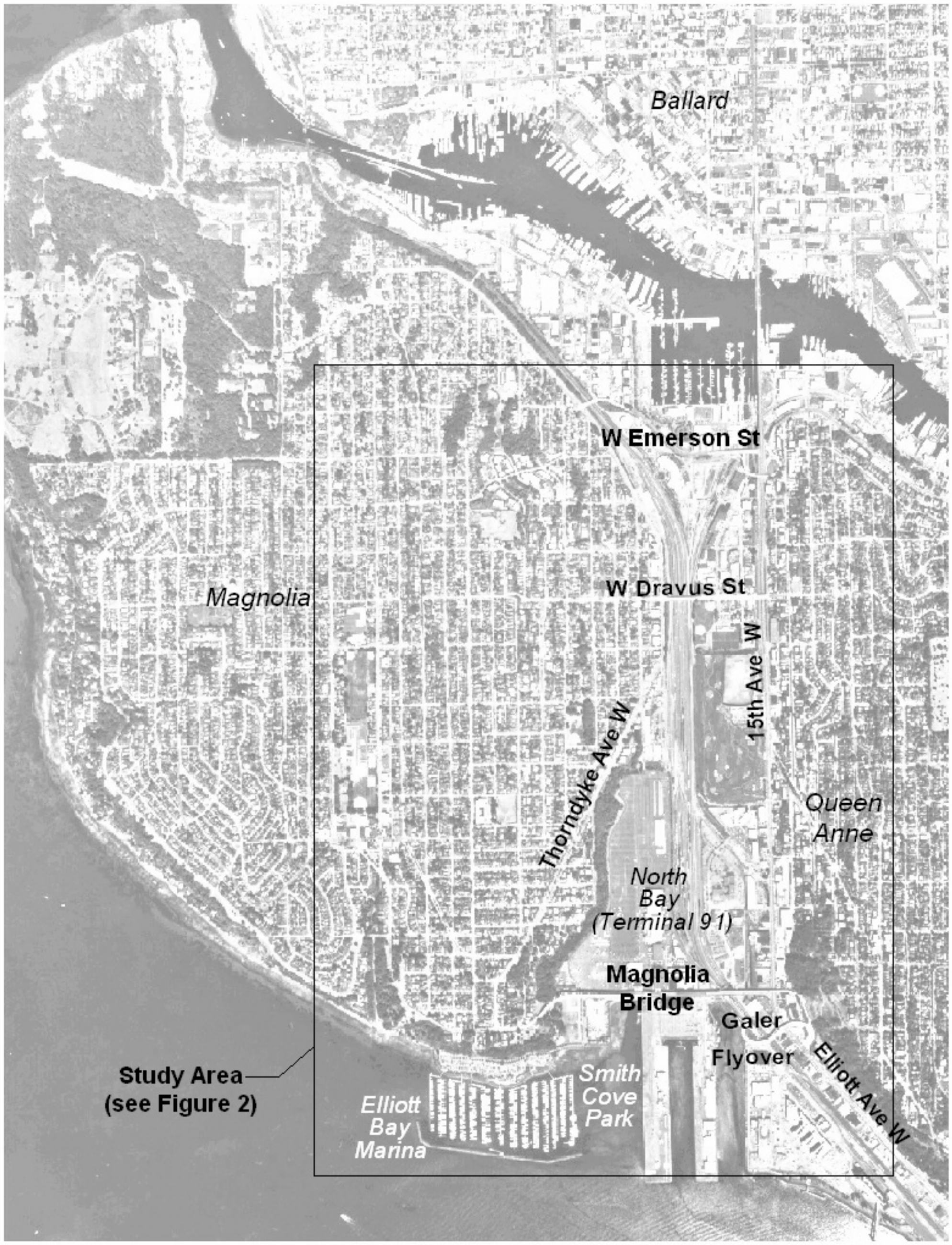


Figure 1
Vicinity Map

In meetings with the public and the Seattle Fire Department, the importance of this route for emergency services has been emphasized. The loss of use of this bridge in 1997 and again in 2001 demonstrated to the City that the remaining two bridges do not provide acceptable operation. During the bridge closure following the February 2001 earthquake, the City addressed community concerns about reduced emergency response time to medical facilities outside of Magnolia by 24-hour stationing of paramedics at Fire Station 41 (2416 34th Avenue West).

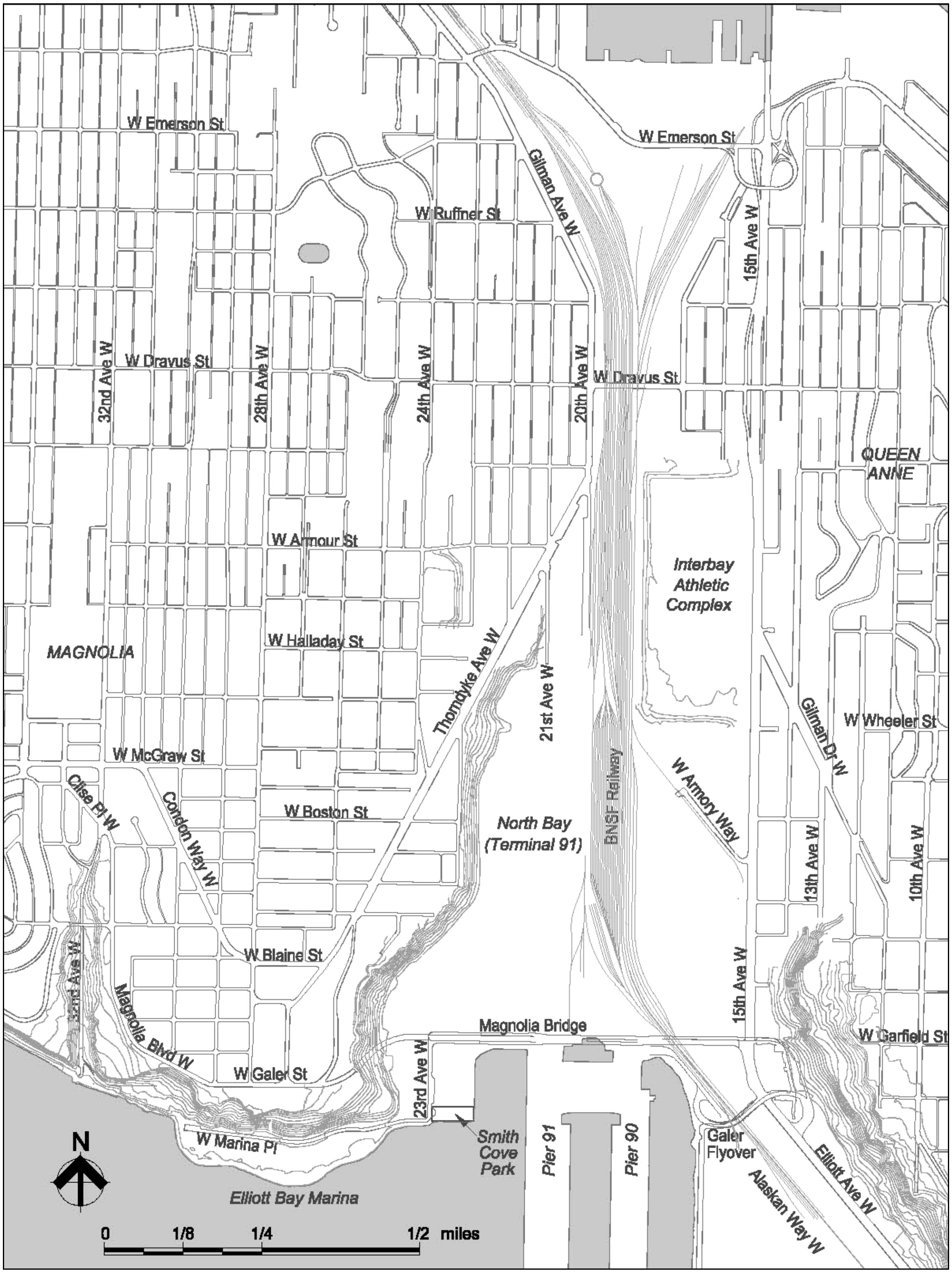


Figure 2
Study Area

Traffic Capacity

The three Magnolia community connections to the 15th Avenue West corridor are adequate for the present volume of traffic. Each of the three connections carries about 30 to 35 percent of the 60,100 daily vehicle trips (2001 counts) in and out of the Magnolia community. Loss of the use of the Magnolia Bridge for several months after the February 2001 earthquake, and in 1997 following the landslide at the west end of the bridge, resulted in lengthy 15 to 30 minute delays and increased trip lengths for many of the users of the Magnolia Bridge. These users were required to use one of the two remaining bridges at West Dravus Street and West Emerson Street. Travel patterns in the Magnolia community changed substantially resulting in negative impacts on local neighborhood streets. The increase of traffic through the West Dravus Street and West Emerson Street connections also resulted in congestion and delay for the regular users of these routes. Losing the use of any one of these three bridges would result in redirected traffic volumes that would overwhelm the capacity of the remaining two bridges.

Modal Interrelationships

The Magnolia Bridge carries three of the four local transit routes serving Magnolia and downtown Seattle destinations. The topography of the east side of Magnolia, East Hill, would make access to the 15th Avenue West corridor via the West Dravus Street bridge a circuitous route for transit. Use of the West Emerson Street connection to 15th Avenue West would add significant distance and travel time for most trips between Magnolia and downtown Seattle.

The Magnolia Bridge has pedestrian facilities connecting the Magnolia neighborhood to Smith Cove Park and Elliott Bay Marina as well as to 15th Avenue West/Elliott Avenue West. These facilities need to be maintained. The Elliott Bay multi-use trail connects Magnolia with downtown Seattle through Myrtle Edwards Park. The trail passes under the Magnolia Bridge along the west side of the BNSF rail yard, but there are no direct connections to the bridge.

Bicycle facilities on the Magnolia Bridge need to be maintained or improved. Even with the steep (about 6.3 percent) grade, bicyclists use the Magnolia Bridge in both directions. There are no bike lanes on the bridge, so bicyclists use the traffic lanes and sidewalks. Once bicyclists cross the bridge, they must either travel with motor vehicles on Elliott Avenue West or find a way back to the Elliott Bay Trail using local east-west streets such as the Galer Flyover.

Transportation Demand

The existing Magnolia Bridge provides automobile access for Port of Seattle North Bay (Terminal 91) to and from the Elliott Avenue West/15th Avenue West. Truck access between Terminal 91 and Elliott Avenue West/15th Avenue West is accommodated via the Galer Flyover. Future planned expansion of the Amgen facility on Alaskan Way West and redevelopment of underutilized portions of North Bay and other areas of Interbay will increase demand for traffic access to the Elliott Avenue West/15th Avenue West corridor. The Port of Seattle has a master planning process underway (July 2003) for its North Bay property (Terminal 91) and the Washington National Guard property east of the BNSF Railway between West Garfield Street and West Armory Way. This area contains 82 acres available for redevelopment. There are also 20 or more acres of private property available for redevelopment east of the BNSF Railway between West Wheeler Street and West Armory Way. Redevelopment of the North Bay property will include public surface streets with connections to the replacement for the Magnolia Bridge. Forecasts of future (year 2030) traffic demand indicate that the access provided by the Galer Flyover and West Dravus Street would be inadequate. The capacity provided by the existing Magnolia Bridge or its replacement would also be needed.

Legislation

Seattle Ordinance 120957, passed in October 2002, requires the Magnolia Bridge Replacement Study: identify possible additional surface roads from Magnolia to the waterfront (avoiding 15th Avenue West and the railroad tracks); obtain community input on the proposed roads; and identify the cost for such road and include it in the total cost developed in the Magnolia Bridge Replacement Study.

Description of Alternatives

An alignment study process was implemented to help identify the specific bridge replacement alternatives to be studied in the EIS. Twenty-five concepts were developed and screened against the project goals and objectives. This resulted in nine alignment alternatives, identified as A through I, that merited further analysis. These nine went through an extensive public review and comment process as well as project screening criteria and prioritization. Initially, the top four priority alternatives, A, B, D, and H, were identified to be studied in the EIS. Early on, Alternative B was eliminated because it became clear that it violated City shoreline policies and Federal section 4(f) criteria. Following detailed traffic analysis, Alternative H was eliminated because two key intersections were predicted to function at a level of service F and could not be mitigated. The next priority, Alternative C, was then carried forward for analysis in the EIS.

Independent of this project, a new north-south surface street will be constructed on Port of Seattle property connecting 21st Avenue West at the north end of North Bay with 23rd Avenue West near Smith Cove Park. In addition, a southbound ramp will be added to the Galer Flyover to accommodate eastbound to southbound Elliott Avenue West traffic movements. The Galer Flyover ramp has been identified as a needed improvement for expected future development of property west of the railroad tracks. New surface streets through the Port of Seattle property will be located through the Port's master planning process for the North Bay property. The north-south surface street and ramp are assumed to exist in any build alternative, but are not part of this environmental process.

Typical sections and plans of the build and no-build alternatives are located at the end of this section.

No Build Alternative

The No Build Alternative, shown in Figure 3 and Figure 5, would maintain the existing bridge structure in place with the existing connections at the east and west ends. Long-term strategies for maintaining the existing structure would be required for the No Build alternative. To keep the existing bridge in service for over ten years, the following would need to be accomplished:

- An in-depth inspection of the bridge would be required to determine needed repairs and a long-term maintenance program.
- Concrete repairs would be required. These repairs could include injection of cracks with epoxy grout, repair of spalled concrete, and replacement of deficient concrete and grout.
- Preservation measures to slow corrosion of the reinforcement would be required. These measures could include a cathodic protection system.
- Any structural elements that lack the capacity to carry a tractor-trailer truck with a 20-ton gross trailer weight would need to be identified, modeled, and strengthened.

Alternative A

Alternative A would replace the existing bridge with a new structure immediately south of the existing bridge as shown in Figure 4 and Figure 6. The alternative would construct a signalized elevated intersection (Alternative A – Intersection) in the bridge's mid-span to provide access to the waterfront and the Port of Seattle North Bay property from both the east and the west. Connections at the east and west ends of the bridge would be similar to the existing bridge.

An optional half-diamond interchange (Figure 7 Alternative A - Ramps) could be constructed in lieu of the elevated intersection to provide access to the waterfront and the Port of Seattle North Bay property to and from the east only.

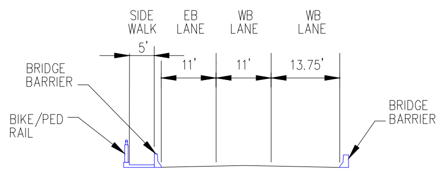
Alternative C

Alternative C would provide 2,200 feet of surface roadway within the Port of Seattle North Bay property between two structures as shown in Figure 4 and Figure 8. The alternative would descend from Magnolia Bluff on a structure running along the toe of the slope. The alignment would reach the surface while still next to the bluff, before turning east to an intersection with the north-south surface street. The alignment would continue east from the intersection, turning south along the west side of the rail yard. The alignment would rise on fill and structure, turning east to cross the railroad tracks and connect to 15th Avenue West.

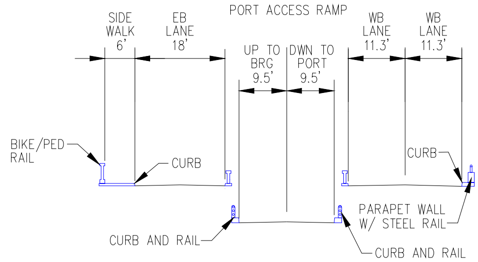
Alternative D

Alternative D would construct a new bridge in the form of a long arc north of the existing bridge, as shown in Figure 4 and Figure 9. Connections at the east and west ends of the bridge would be similar to the existing bridge. This alternative would construct a signalized elevated intersection (Alternative D – Intersection) in the bridge mid-span to provide access to the waterfront and Port of Seattle North Bay property from both the east and the west.

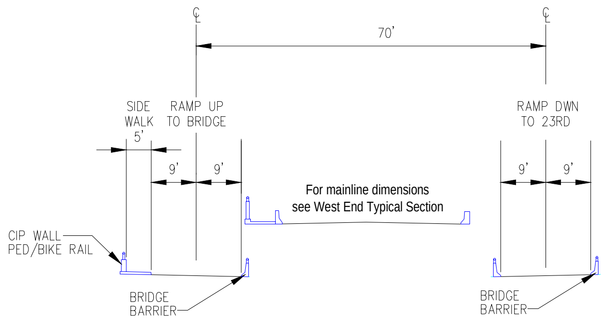
An optional half-diamond interchange (Figure 10 Alternative D - Ramps) could be constructed in lieu of the elevated intersection to provide access to the waterfront and the Port of Seattle North Bay property to and from the east only.



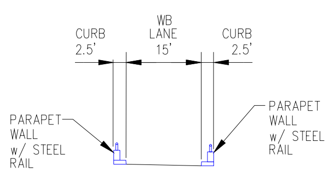
Bridge West End



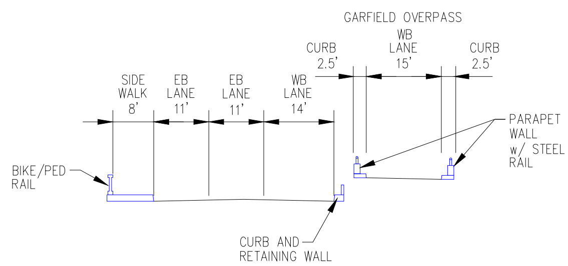
Ramp to Port Access



Ramps to 23rd Avenue West



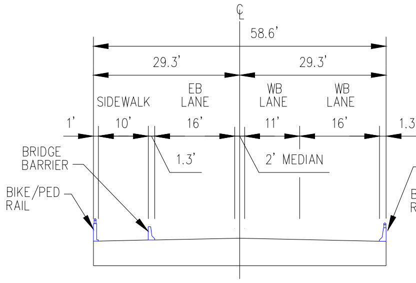
Garfield Overpass



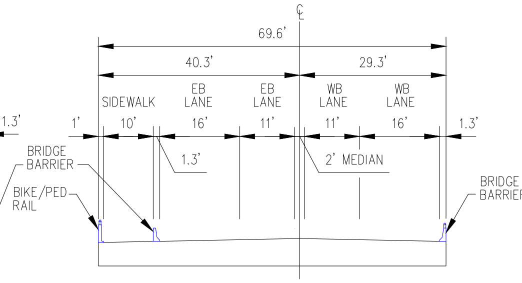
15th Avenue West Connection
Eastbound Off-Ramp
Westbound On-Ramp

NOTE:
Dimensions are approximate and obtained from construction plans and aerial photographs. The information shown has not been field verified.

Figure 3
Typical Sections – No Build Alternative

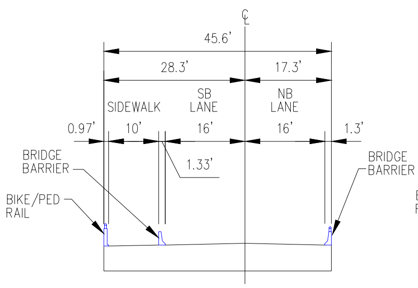


West End



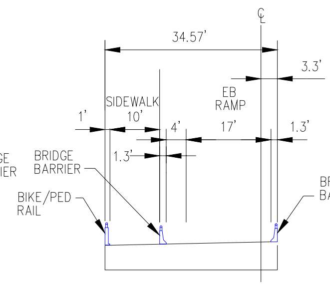
East End

Typical Bridge Structure

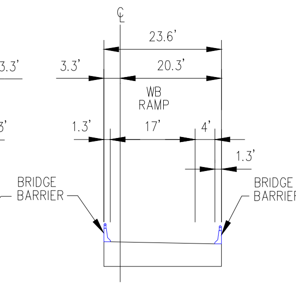


T-Ramp

Typical A & D Intersection Option

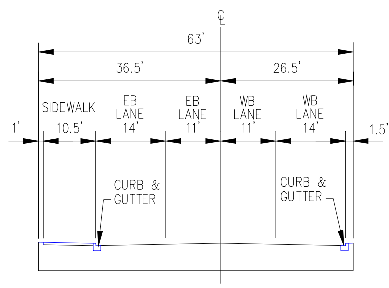


On-Ramp

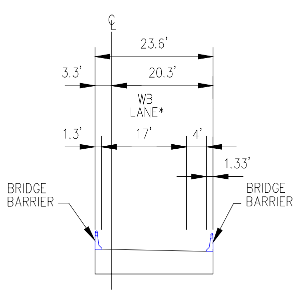


Off-Ramp

Typical A & D Ramp Option

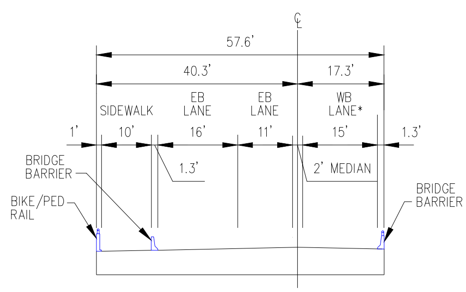


Typical Alternative C Surface Road



Garfield Overpass

* 15' Alternative C
19' Alternative D



15th Avenue West Connection
Eastbound Off-Ramp
Westbound On-Ramp

* 16' Alternative D

Figure 4
Typical Sections – Build Alternatives

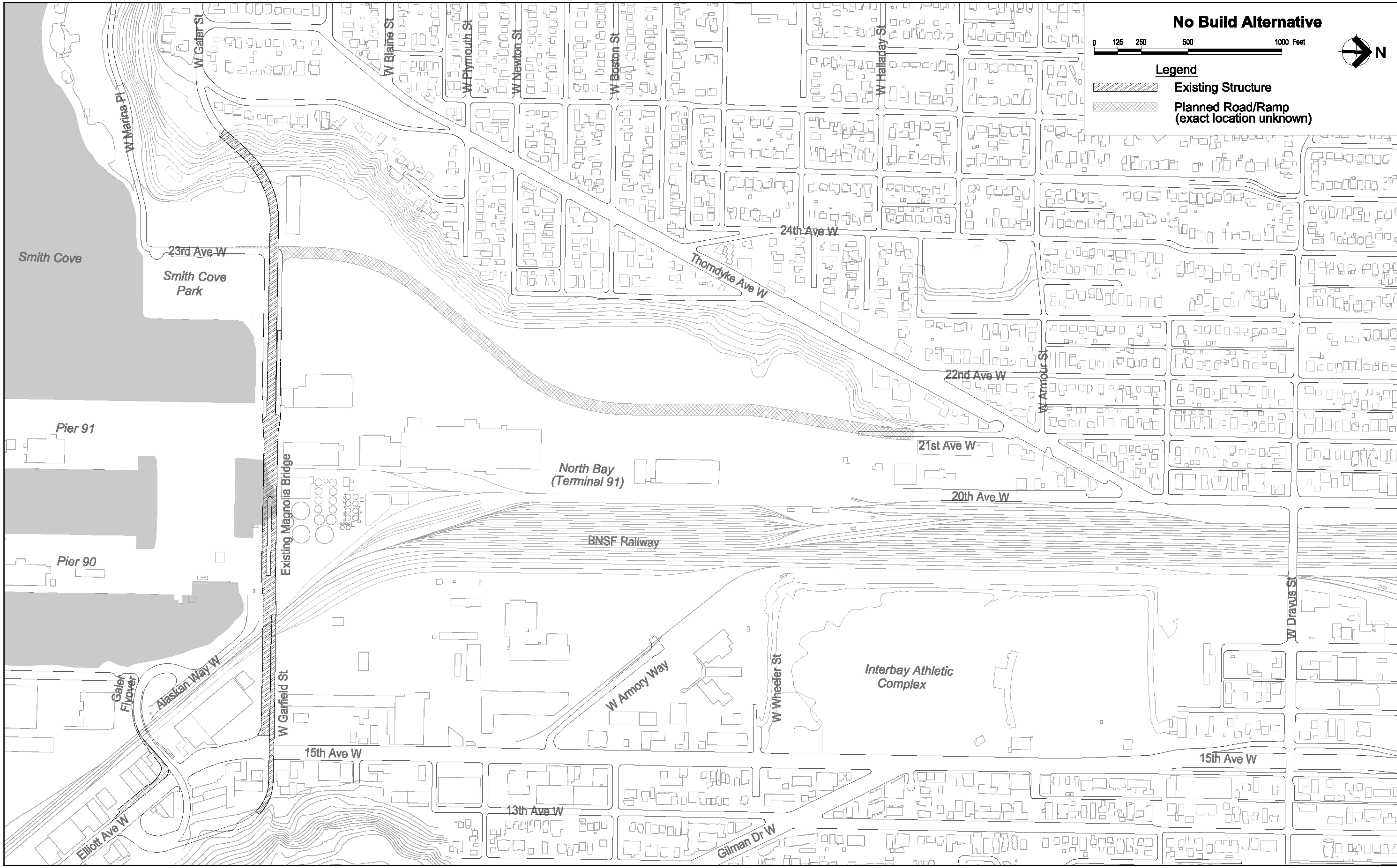


Figure 5 No Build Alternative

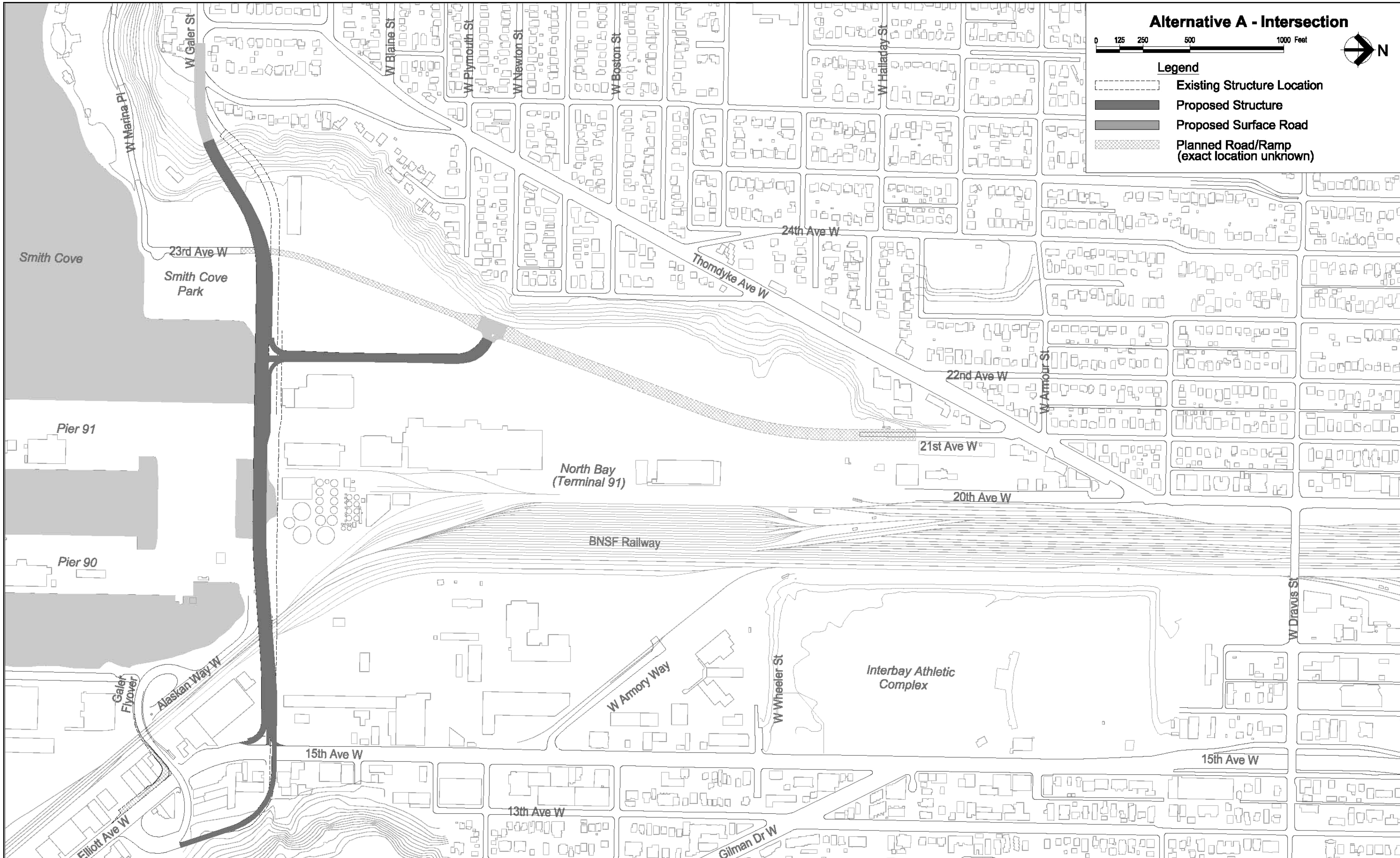


Figure 6 Alternative A - Intersection

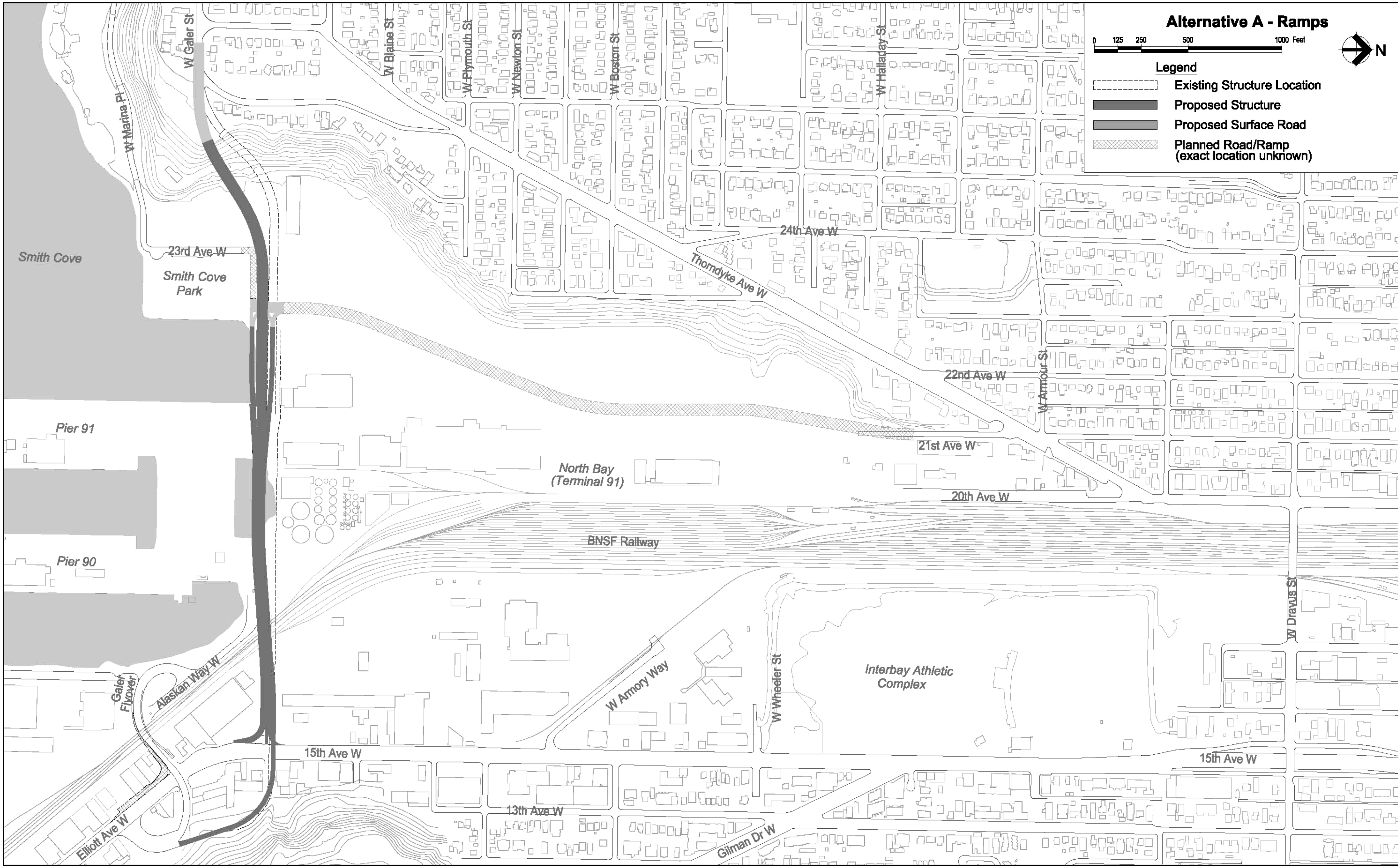


Figure 7 Alternative A - Ramps

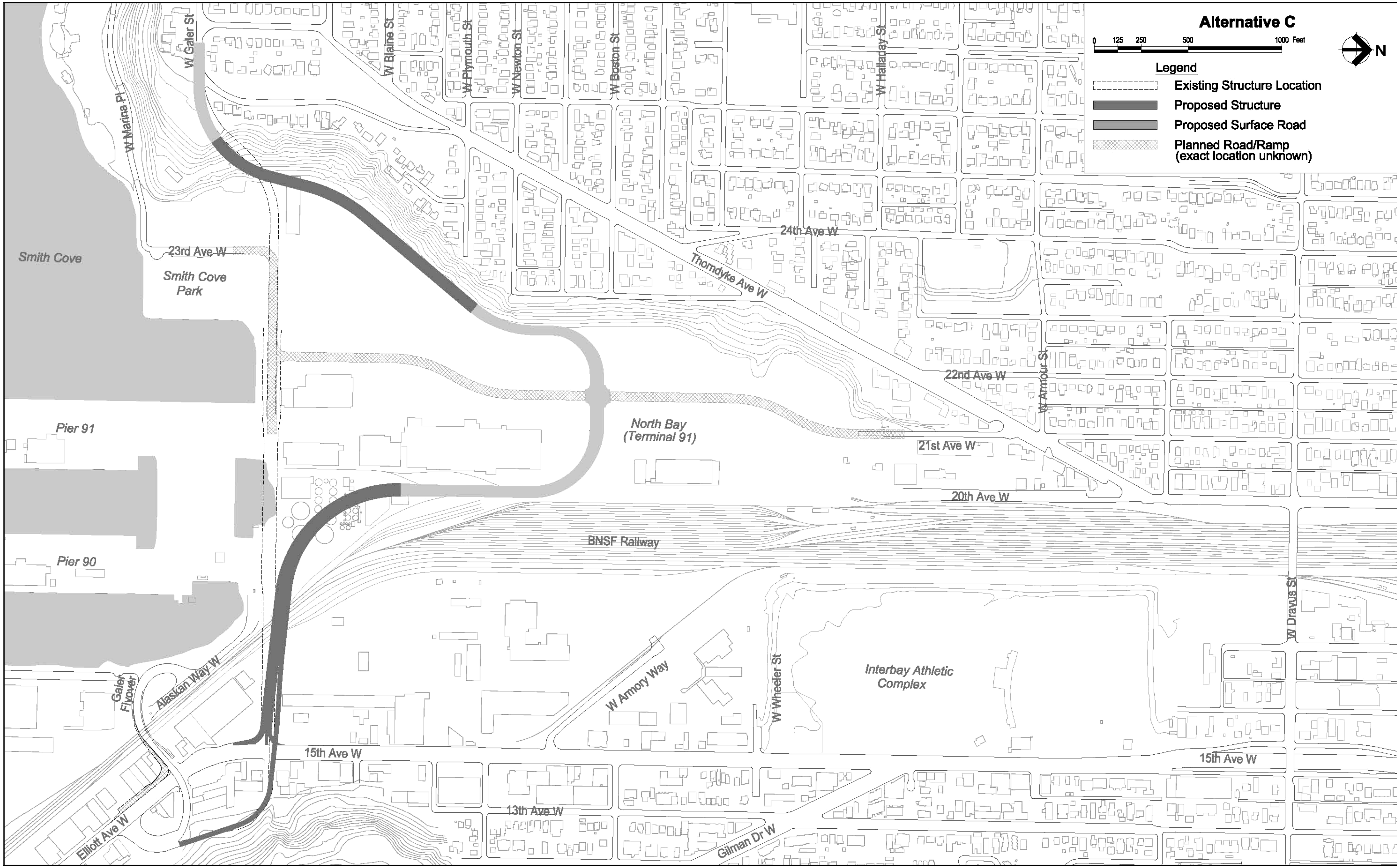


Figure 8 Alternative C

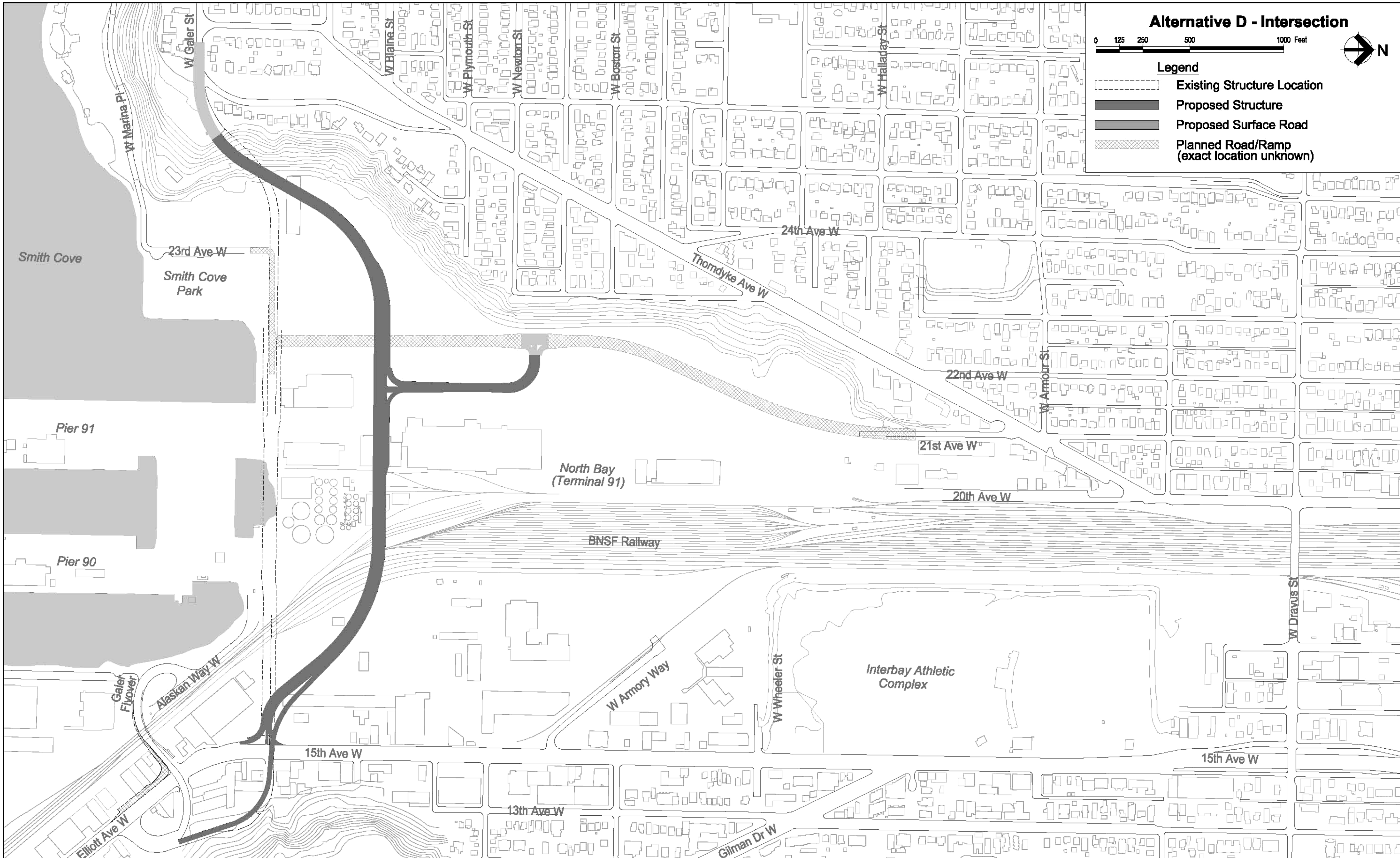


Figure 9 Alternative D - Intersection

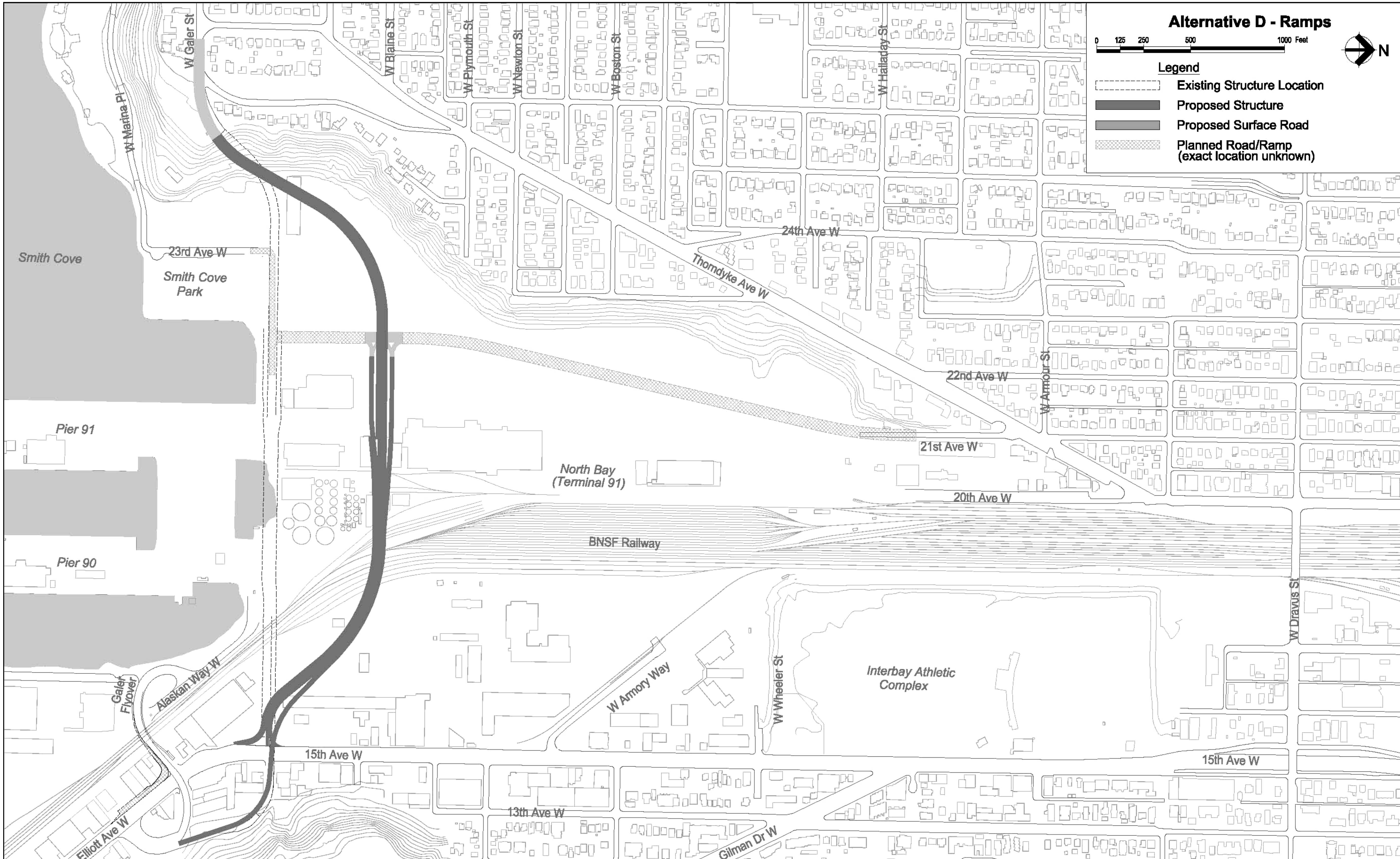


Figure 10 Alternative D - Ramps

Methods

Information about the geologic surface and subsurface conditions along the build alternative alignments (affected environment) was evaluated by reviewing existing available subsurface information; by performing a geologic field slope reconnaissance; and by performing subsurface explorations. Available subsurface information was collected from files maintained by the City of Seattle, the City of Seattle Department of Planning and Development (DPD), the Seattle-area Geologic Mapping project office, and the Port of Seattle. A geologic slope reconnaissance of the western approaches for the build alternatives was also performed to identify major geologic surface features such as landslide scarps, seepage, and erosional evidence. Information from published geologic maps and other documents was also reviewed. Available information regarding existing building foundations was collected from several City, County, and Port of Seattle sources.

The information collected from the data review, geologic field slope reconnaissance, and subsurface explorations was used to develop a description of the affected environment including geology, location of critical geologic areas, and general topographic setting. A description of the affected environment based on these studies is presented later in this report.

Based on the No Build Alternative and the build alternatives (Alternatives A, C, and D), geologic and geotechnical impacts were assessed related to cuts and fills, retaining walls, bridge foundations, landslides, liquefaction, lateral spreading, construction, and utilities. Mitigation measures for these impacts are proposed and are included in this report.

The information collected from the literature and data review, field reconnaissance, and field explorations was used to develop a description of the affected environment. This description includes the general topographic setting; geology; location of critical geologic areas (such as landslides, groundwater levels, glacial soil, etc); location of regional faults and other geologic hazards; and other miscellaneous but pertinent geologic data related to the proposed alternatives. The following sections describe each of these issues in more detail. The project vertical datum is NAVD88. The site and exploration plans are presented on Figures 11, 12, and 13 for Alternatives A, C, and D, respectively.

Project Area Description and Topographic Setting

A study area topographic map is shown on Figure 14. Alternatives A, C, and D are located in the area between West Boston Street to the north and Piers 90 and 91 to the south. Alternatives A, C, and D would connect to West Galer Street at their west ends, similar to the existing alignment, and they would use the existing Magnolia Bridge on-ramp alignment just east of 15th Avenue West.

The majority of Alternative A would be parallel to, run immediately south of, and be within about 50 feet of, the existing bridge structure. Alternatives C and D would be a maximum of approximately 1,800 and 570 feet north of the existing bridge, respectively.

The existing topography is relatively flat from east to west, until the alignments reach the toe of Magnolia Bluff. From the toe of the bluff's slope, the ground surface rises to the Magnolia surface streets. The maximum ground slope up Magnolia Bluff at the centerline of the alignments is approximately 1.9 Horizontal to 1 Vertical (1.9H:1V) for Alternatives A and C, and 3.5H:1V for Alternative D. The elevation gain up Magnolia Bluff is approximately 150 feet. Queen Anne Hill lies east of the three alignments, just beyond their eastern approaches; this hill is about 80 feet high in the vicinity of the three alignments.

Residential, commercial, City of Seattle Parks and Recreation, National Guard Armory, and Port of Seattle properties comprise most of the development within the project area. Information regarding existing buildings' foundations within approximately 200 feet of each alternative alignment is presented in Table 1. The majority of the proposed alignments are already paved. The Magnolia Bluff hillside is generally vegetated with deciduous trees, predominantly alder and maple along with other species, and undergrowth, much of which is Himalayan blackberry and ivy.

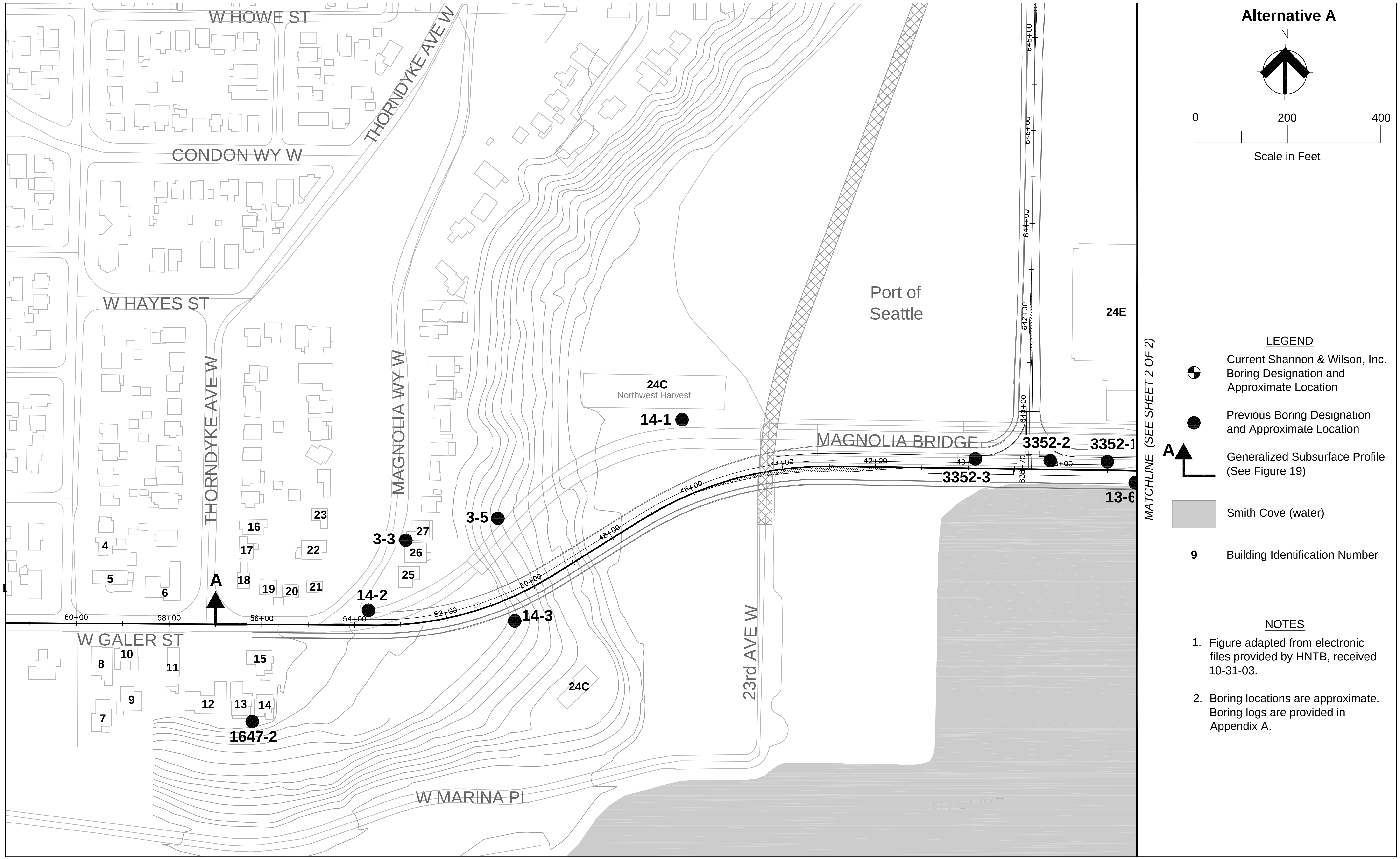


Figure 11, Sheet 1 of 2 - Site and Exploration Plan - Alternative A

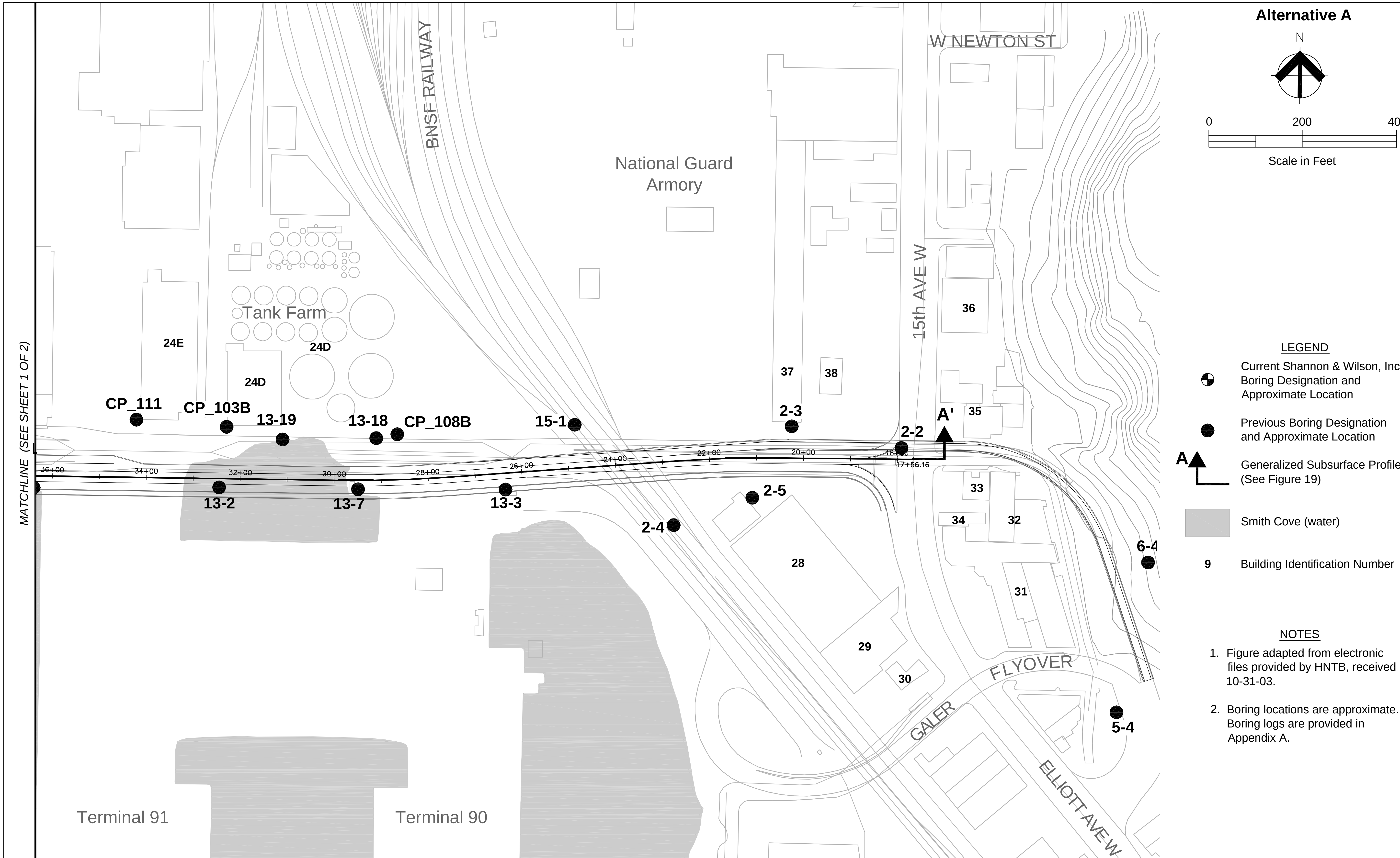


Figure 11, Sheet 2 of 2 - Site and Exploration Plan - Alternative A

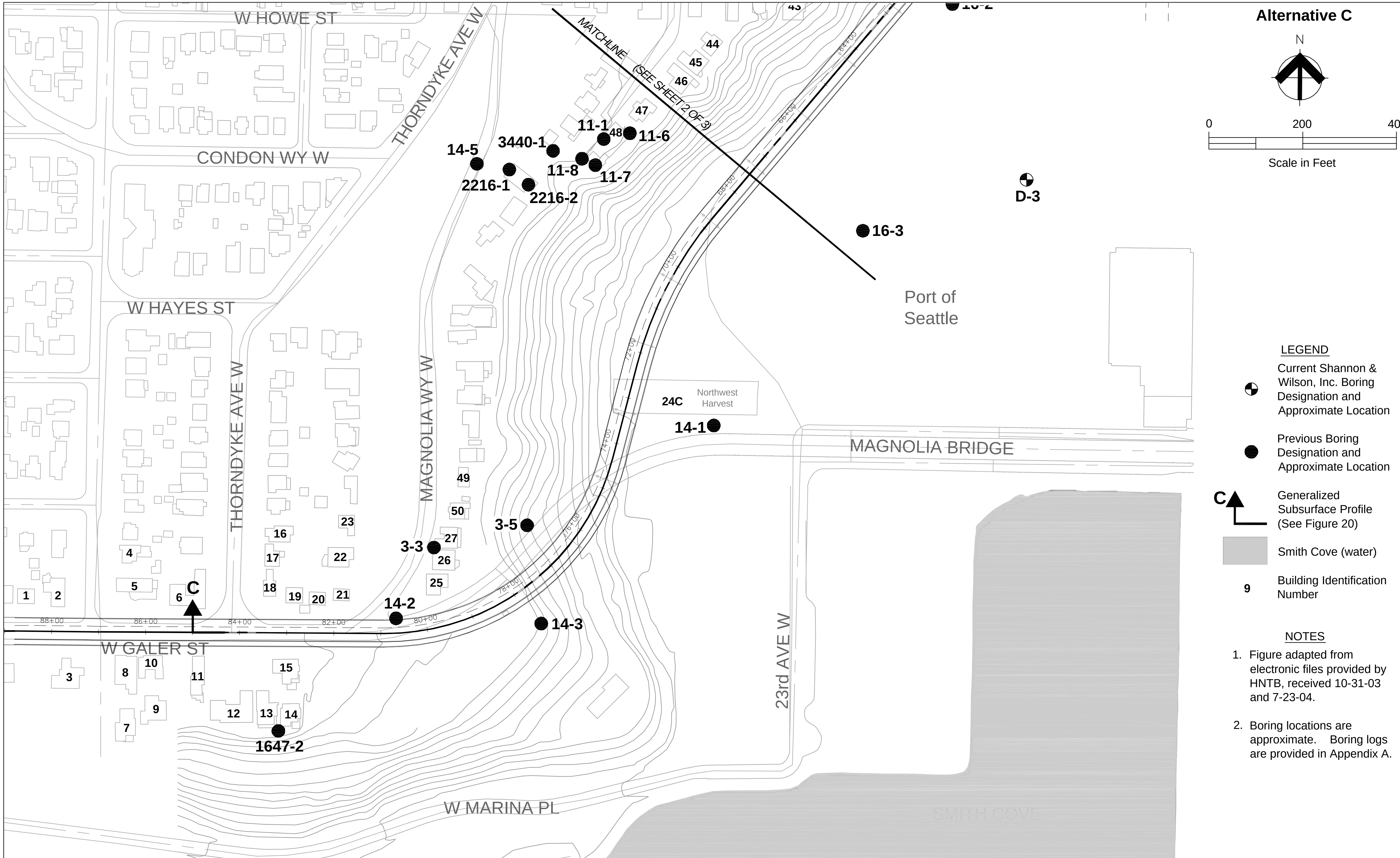


Figure 12, Sheet 1 of 3 - Site and Exploration Plan - Alternative C

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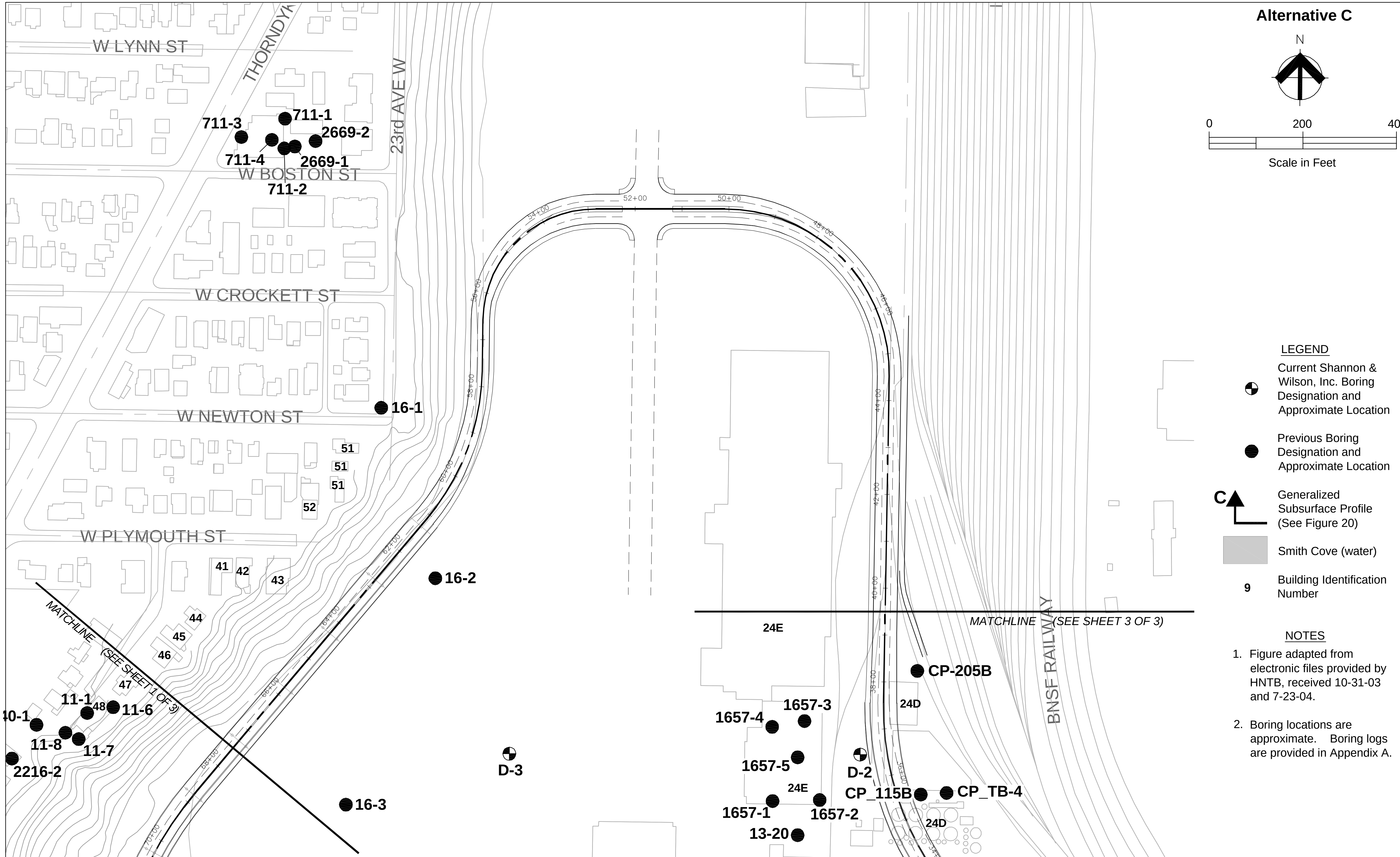

Figure 12, Sheet 2 of 3 - Site and Exploration Plan - Alternative C

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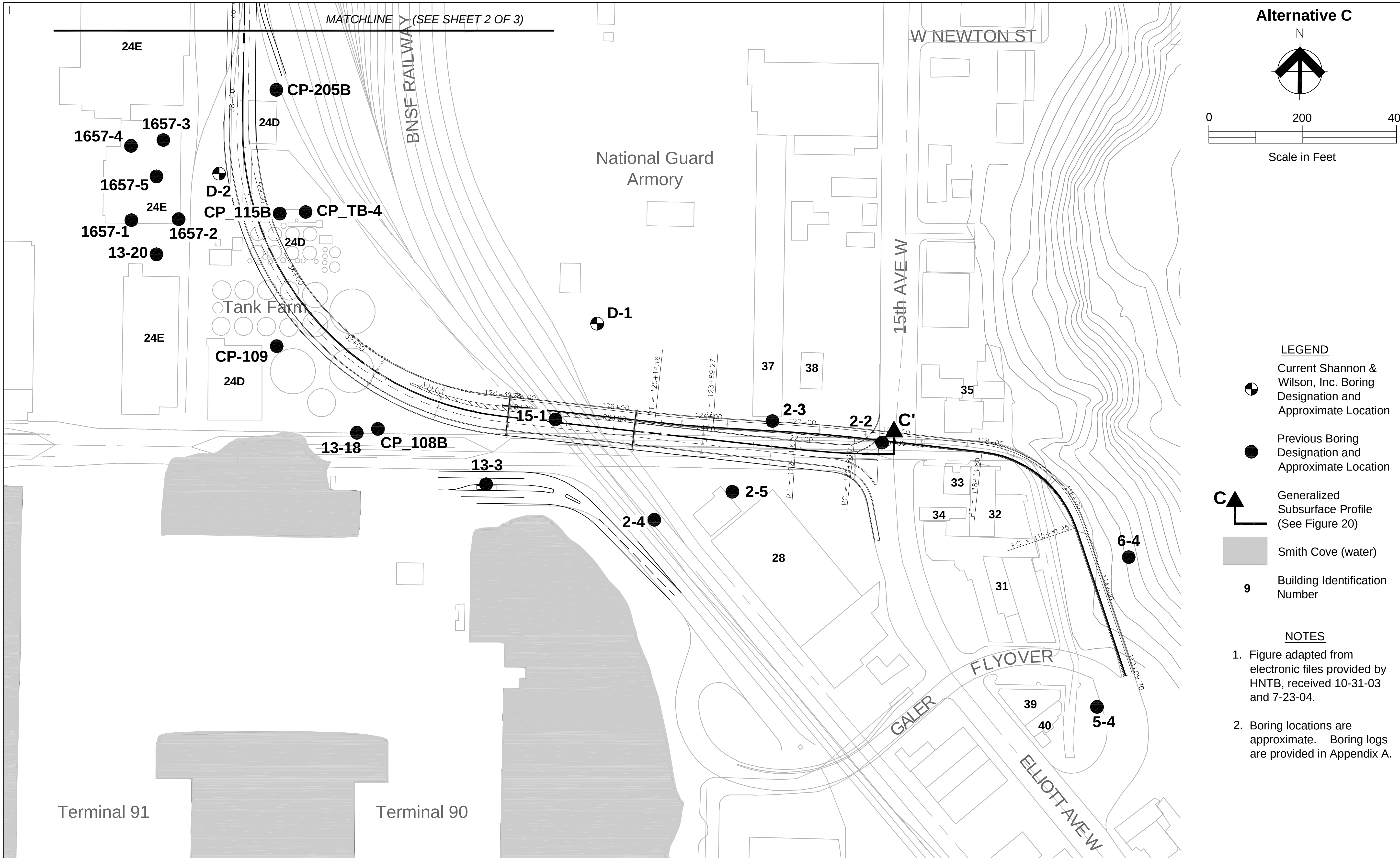

Figure 12, Sheet 3 of 3 - Site and Exploration Plan - Alternative C

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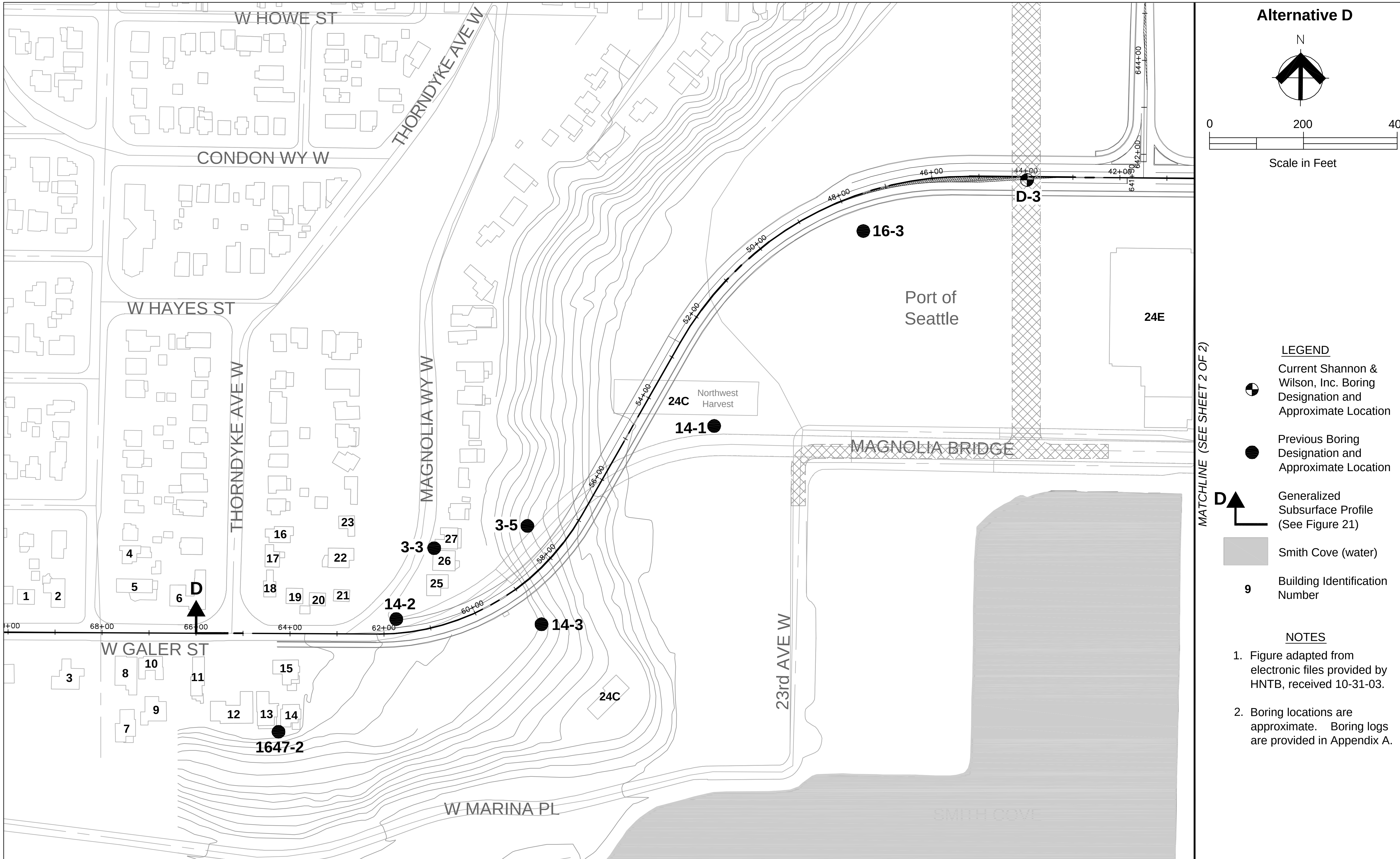

Figure 13, Sheet 1 of 2 - Site and Exploration Plan - Alternative D

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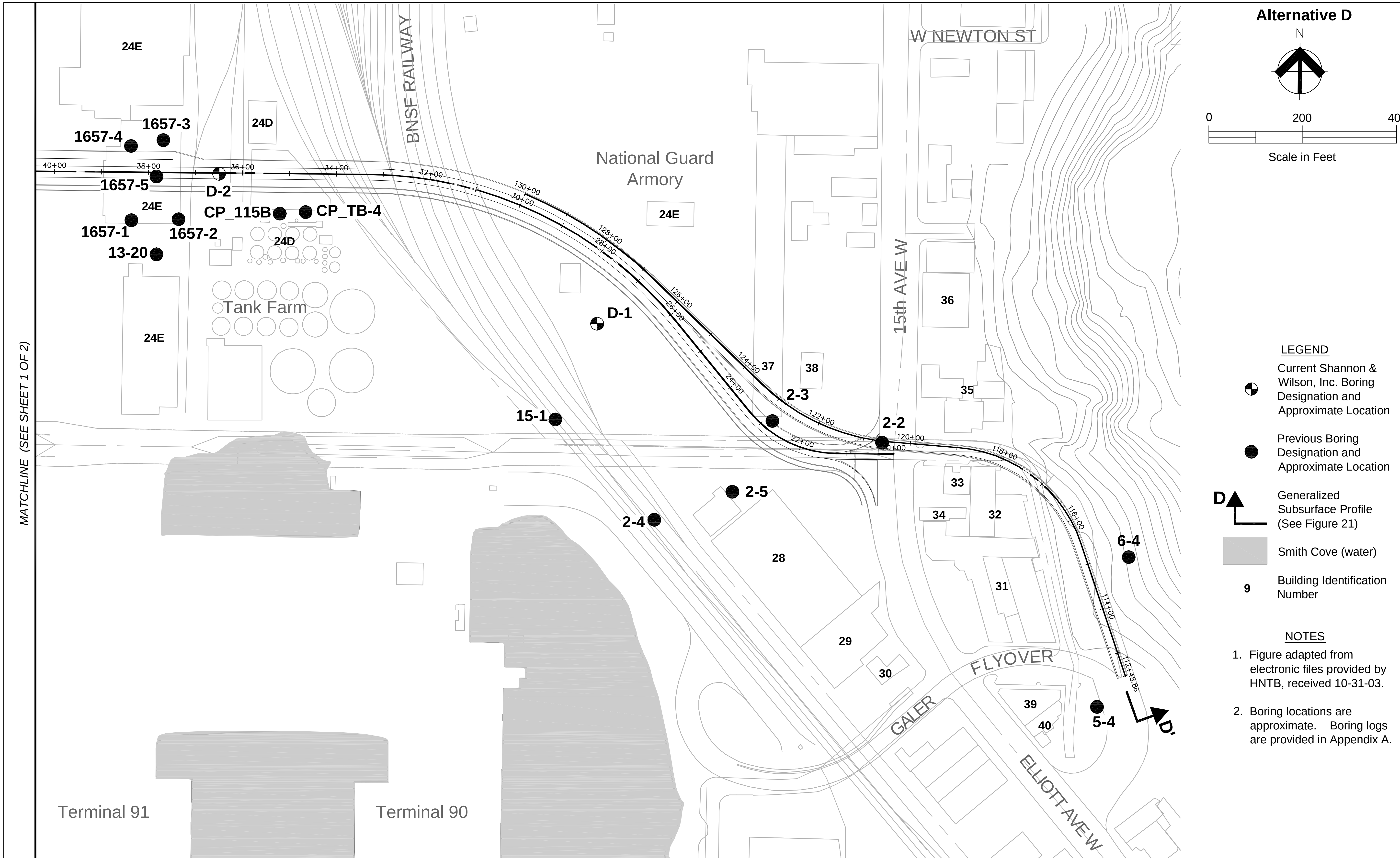

Figure 13, Sheet 2 of 2 - Site and Exploration Plan - Alternative D

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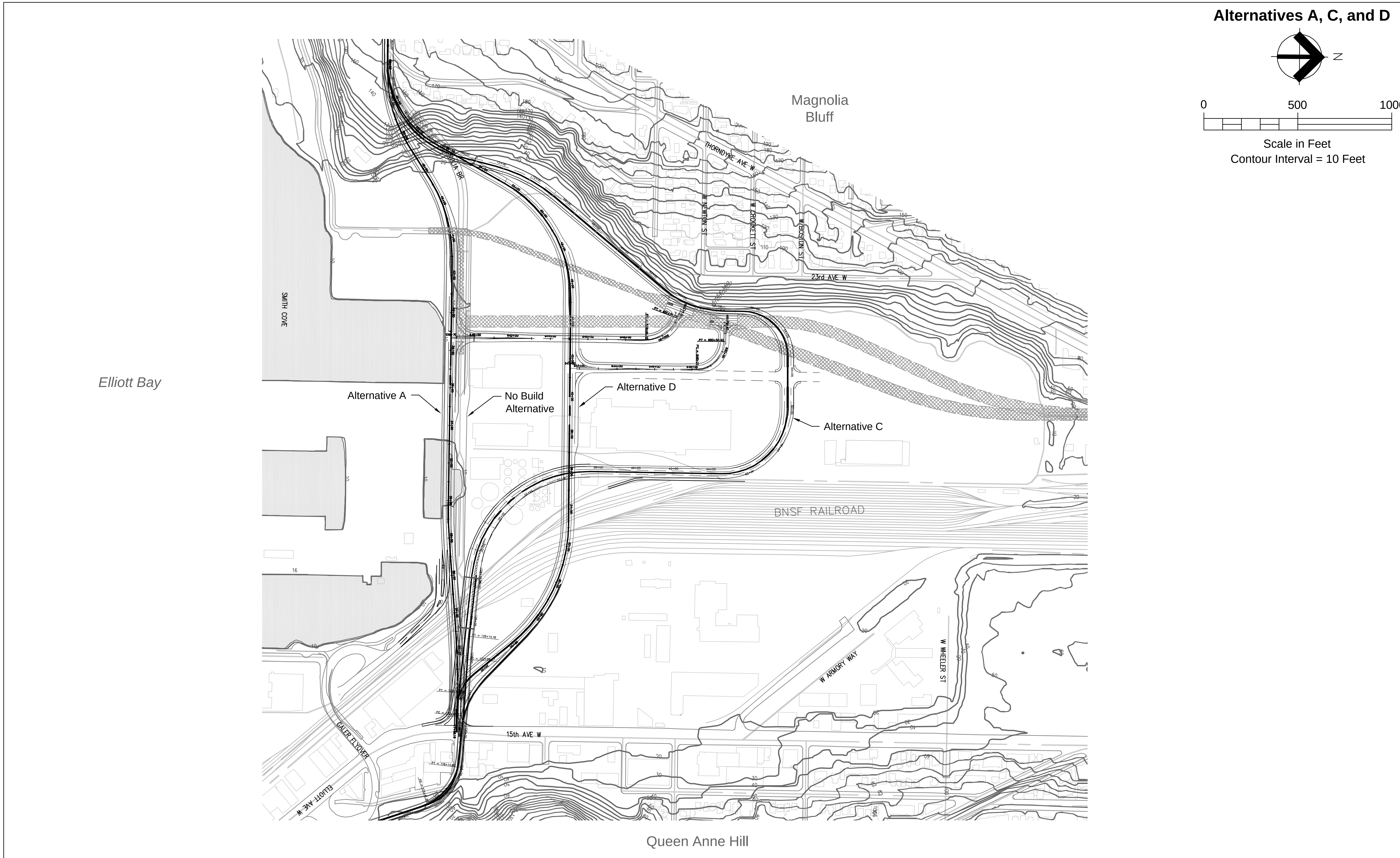

Figure 14 - Project Area Topography

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Table 1
Existing Building Foundations

ID NO.	SITE NAME/BUSINESS NAME/TYPE	ADDRESS	FOUNDATION TYPE	PARCEL NUMBER	ALIGNMENT	COMMENTS	SOURCES
4	Single family residence	1512 28th Ave W	Unknown (footings likely)	5037300060	A, C, D	Built in 1938	Archive records, tax assessor records, DPD files, King County Website
5	Single family residence	2720 W Galer St	Building (1985); footings (2,000 psf)	5037300065	A, C, D	Built in 1909	Archive records, tax assessor records, DPD files, King County Website
6	Single family residence	2700 W Galer St	Unknown (footings likely)	5037300075	A, C, D	Built 1955	Archive records, tax assessor records, DPD files, King County Website
7	Single family residence	1452 28th Ave W	Addition/remodel (1982; footings 2,000 psf); landslide report on file	5553300453	A, C, D	Built 1982	Archive records, tax assessor records, DPD files, King County Website
8	Single family residence	2719 W Galer St	Building (1995); footings (2,000 psf)	5553300375	A, C, D	Built in 1943; Remodeled/Rebuild in 1996	Archive records, tax assessor records, DPD files, King County Website
9	Single family residence	2709 W Galer St	Unknown (footings likely)	5553300381	A, C, D	Built 1953	Archive records, tax assessor records, DPD files
10	Single family residence	2715 W Galer St	Unknown (footings likely)	5553300380	A, C, D		King County Website
11	Single family residence	2703 W Galer St	Footings (2,000 psf), residence includes retaining wall (including residences 2619, 2625, and 2703)	5553300389	A, C, D	Built 1985	Archive records, tax assessor records, DPD files, King County Website
12	Single family residence	2625 W Galer St	Addition/renovation (1995); existing property; footings (2,000 psf), residence includes retaining wall (including residences 2619, 2625, and 2703)	5553300395	A, C, D	Built 1953	Archive records, tax assessor records, DPD files, King County Website
13	Single family residence	2619 W Galer St	Footings likely, residence includes retaining wall (including residences 2619, 2625, and 2703)	5553300405	A, C, D	Built 1953	Archive records, tax assessor records, DPD files, King County Website

Table 1 (cont.)
Existing Building Foundations

ID NO.	SITE NAME/BUSINESS NAME/TYPE	ADDRESS	FOUNDATION TYPE	PARCEL NUMBER	ALIGNMENT	COMMENTS	SOURCES
14	Single family residence	2617 W Galer St	Footings (4,000 psf)	5553300407	A, C, D	Built 1987	Archive records, tax assessor records, DPD files, King County Website
15	Single family residence	2615 W Galer St	Footings	5553300406	A, C, D	Built 1987	Archive records, tax assessor records, DPD files, King County Website
16	Single family residence	1516 Thorndyke Ave W	Unknown (footings likely)	5037300185	A, C, D	Built 1951	Archive records, tax assessor records, DPD files, King County Website
17	Single family residence	1512 Thorndyke Ave W	Unknown (footings likely)	5037300190	A, C, D	Built 1926	Archive records, tax assessor records, DPD files, King County Website
18	Single family residence	1502 Thorndyke Ave W	Unknown (footings likely)	5037300200	A, C, D	Built 1940	Archive records, tax assessor records, DPD files, King County Website
19	Single family residence	2612 W Galer St	Unknown (footings likely)	5037300195	A, C, D	Built 1940	Archive records, tax assessor records, DPD files, King County Website
20	Single family residence	2608 W Galer St	Unknown (footings likely)	5037300220	A, C, D	Built 1940	Archive records, tax assessor records, DPD files, King County Website
21	Single family residence	2600 W Galer St	Unknown (footings likely)	5037300215	A, C, D	Built 1940	Archive records, tax assessor records, DPD files, King County Website
22	Single family residence	1511 Magnolia Way W	Unknown (footings likely)	5037300235	A, C, D	Built 1941	Archive records, tax assessor records, DPD files, King County Website
23	Single family residence	1517 Magnolia Way W	Unknown (footings likely)	5037300241	A, C, D	Built 1947	Archive records, tax assessor records, DPD files, King County Website
24A	Port of Seattle property	2001 W Garfield St	Unknown	2325039012	A, C, D	Labeled Bldg 49 in DPD records; zoned commercial; has one building built in 1942	DPD Parcel Records, King County Website

ID NO.	SITE NAME/BUSINESS NAME/TYPE	ADDRESS	FOUNDATION TYPE	PARCEL NUMBER	ALIGNMENT	COMMENTS	SOURCES
24B	Port of Seattle property	2001 W Garfield St	Unknown	2325039013	A, C, D	Labeled Bldg 54 in DPD records; zoned commercial; has one building built in 1942	Archives files on the POS, King County Website
24C	Port of Seattle property (Northwest Harvest)	2001 W Garfield St	Unknown	2325039107	A, C, D	Bldg 50 (Boiler House) is located on this property, built in 1942, based on parcel number 2325039015 records	Archives files on the POS, King County Website
24D	Port of Seattle property	2001 W Garfield St	Unknown	7666201530	A, C, D	Auto processing buildings/facilities; truck scales; storage yard; BNSF railroad tracks	Archive records, tax assessor records, DPD files, King County Website
24E	Port of Seattle property	2001 W Garfield St	Unknown	7666201146	A, C, D	Tank Farm, fuel pump station, storage yard, auto processing facilities, warehouses, car wash	Archive records, tax assessor records, DPD files, King County Website
25	Single family residence	1500 Magnolia Way W	Unknown (footings likely)	5037300305	A, C, D	Built 1953	Archive records, tax assessor records, DPD files, King County Website
26	Single family residence	1512 Magnolia Way W	Addition/renovation; footings	5037300300	A, C, D	Built 1952	Archive records, tax assessor records, DPD files, King County Website
27	Single family residence	1518 Magnolia Way W	Addition (1998); footings	5037300295	A, C, D	Built 1951	Archive records, tax assessor records, DPD files, King County Website
28	Part of Staples Office Supply store	1523 15th Ave W	Staples - Building A(2001); footings; U-Rent - Building B; footings likely	7666201685	A, C, D	Seattle Tide Lands Plat, Block 134, Lot 3/No address given in DPD	Archive records, tax assessor records, DPD files, King County Website
28	Staples Office Supply store	1523 15th Ave W	Staples - Building A(2001); footings; U-Rent - Building B; footings likely	7666201690	A, D		Archive records, tax assessor records, DPD files, King County Website
28	Alexander U-Rent store	1523 15th Ave W	Staples - Building A(2001); footings; U-Rent - Building B; footings likely	7666201695	A, D	Present: Retail Store; Occupying the same building as Staples Office Supply. DPD has property as vacant	Archive records, tax assessor records, DPD files, King County Website

Table 1 (cont.)
Existing Building Foundations

ID NO.	SITE NAME/BUSINESS NAME/TYPE	ADDRESS	FOUNDATION TYPE	PARCEL NUMBER	ALIGNMENT	COMMENTS	SOURCES
29	Vacant office/warehouse building	1515 15th Ave W	Unknown	7666201700	A, D	1990s?-present: whse., office bldgs. vacant; 1946 to 1993: Turner and Pease operated a frozen food plant here.	Archive records, tax assessor records, DPD files, King County Website
30	Precision Motorworks	1501 Elliott Ave W	Unknown	7666201705	A, D	? - present: Precision Motor-works; 1958 - ?	Archive records, tax assessor records, DPD files, King County Website
31	Builders Hardware Supply	1524 15th Ave W	Building (1930); footings; additional building (1971); (footings likely)	7666201660	A, C, D	1960 to present: Builders Hardware Store (BHS); 1940-1960 Restaurant located on this parcel	Archive records, tax assessor records, DPD files, King County Website
31	Builders Hardware Supply	1502 15th Ave W (to 1516?)	Building (1930); footings; additional building (1983); (footings likely)	7666201665	A, C, D	Present: BHS; 1931-1941?: Shell Service Station; 1942-1949: Fentron Steel & Iron had a whse. located here; After 1949 to ?: NW Builders Inc. (same as BHS?) had a whse., factory, and store located here	Archive records, tax assessor records, DPD files, King County Website
32	Part of Builders Hardware Supply, owner: Winkler Family Partnership or the Bedrock Stoneyard?	1401-1409 W Garfield St	Unknown	7666201640	A, C, D	This is currently part of BHS store. 1953 to ?: Michigan Sales and Service operated a service garage here; 1932 to 1953: Fentron Steel and Iron Works, Inc. had a plant here.	Archive records, tax assessor records, DPD files, King County Website
33	The Bedrock Stoneyard	1415 W Garfield St	Footings	7666201641	A, C, D	Present: Vacant building; Formerly the U.S. Post Office was located here and from 1940-1960: Best Lock Company	Archive records, tax assessor records, DPD files, King County Website
34	Lighthouse Uniforms (retail)	1532 15th Ave W	Footings	7666201650	A, C, D	Built 1956	Archive records, tax assessor records, DPD files, King County Website

Table 1 (cont.)
Existing Building Foundations

ID NO.	SITE NAME/BUSINESS NAME/TYPE	ADDRESS	FOUNDATION TYPE	PARCEL NUMBER	ALIGNMENT	COMMENTS	SOURCES
35	SPCC (Formerly Rudd Paint Company)	1602 15th Ave W	Unknown	3657700060	A, C, D	1911 - ? One bldg with an apartment, barbershop, and café located at 1604 Elliott Ave W. was washed out in mudslide in 1930s?; Replaced by a restaurant/café in 19?? To ?; Rudd Paint & Varnish from ?	Archive records, tax assessor records, King County Website
36	Commercial/retail	1630 15th Ave W	Building (1964); footings; may be demolished; no information for new building	3657700015	A, D	Present: Occupied by SPCC; 1946? -?: Rudd Paint Store; 1929 - 1946?: A two story factory (furniture?)	Archive records, tax assessor records, DPD files, King County Website
37	Dilapidated warehouse on vacant lot	1819 15th Ave W	Unknown	7666201560	A, C, D	1956 to Present: Property owner: Tsubota Steel & Pipe Co., north-south trending property is vacant with a corrugated metal shed (built in 1965) and a lady bug shop located on northern portion of lot; 1947 to 1956: war surplus store (1910-1914 15th Ave W); 1901 to 1956: service station.	Archive records, tax assessor records, DPD files
38	Neon electric sign company occupies lot	1617 15th Ave W	Unknown	7666201601	A, C, D	Formerly Evergreen Trailway Garage was located here, built in 1956 for service and repair of autos/buses	Archive records, tax assessor records, DPD files, King County Website
1	Single family residence	2810 W Galer St	Addition/renovation (1997); footings (2,000 psf)	2021201085	D	Built in 1942	DPD files, King County Website
2	Single family residence	1503 28th Ave W	Addition (1988); footings	2021201070	D	Built in 1951	DPD files, King County Website
3	Single family residence	2807 W Galer St	Unknown (footings likely)	5553300195	D	Built 1915	Archive records, tax assessor records, DPD files, King County Website
39	Albert Lee Appliances	1470 Elliott Ave W	Unknown	7666201775	D		King County Website

Table 1 (cont.)
Existing Building Foundations

ID NO.	SITE NAME/BUSINESS NAME/TYPE	ADDRESS	FOUNDATION TYPE	PARCEL NUMBER	ALIGNMENT	COMMENTS	SOURCES
40	Maytag Appliance store	1460 Elliott Ave W	Footings (2,500 psf)	7666201780	D	Built 1968	DPD files, King County Website
41	Fourplex	2333 W Plymouth St	Footings (4,000 psf)	2771604860	C	Built in 1959	tax assessor records, DPD files
42	Apartment	2327 W Plymouth St	Unknown (footings likely)	2771604865	C	Built in 1958	tax assessor records
43	Condominium	2321 W Plymouth St	Footings (400 psf)	6835500000	C	Built in 1965	tax assessor records, DPD files
44	Single-family residence	2311 W Howe St	Unknown (footings likely)	3547900350	C	Built in 1963	tax assessor records
45	Single-family residence	1820 Amherst Pl W	Unknown (footings likely)	3547900370	C	Built in 1964	tax assessor records
46	Single-family residence	1818 Amherst Pl W	Footings (2,000 psf); 1991 hot tub structure addition on footings	3547900360	C	Built in 1965	tax assessor records, DPD files
47	Single-family residence	1812 Amherst Pl W	Unknown (footings likely)	3547900380	C	Built in 1940	tax assessor records
48	Single-family residence	1800 Amherst Pl W	Footings for original construction, augercast piling foundation repair in 1990	3547900405	C	Built in 1962	tax assessor records, DPD files
49	Single-family residence	1528 Magnolia Way W	Unknown (footings likely)	2325039040	C	Built in 1939	tax assessor records
50	Single-family residence	1524 Magnolia Way W	Footings; 1999 addition on footings	2325039100	C	Built in 1927	tax assessor records, DPD files
51	2 rectories and 1 detached garage	2301 W Newton St	Unknown (footings likely)	2771604405	C	Built in 1940	tax assessor records
52	3 apartment buildings	2323 W Newton St	Unknown (footings likely)	2771604390	C	Built in 1958	tax assessor records

Table 1 (cont.)
Existing Building Foundations

Notes:

1. Unknown means information is currently unavailable.
2. DPD = City of Seattle Department of Planning and Development
3. SFR = Single family residence
4. POS = Port of Seattle
5. DOD = Department of Defense
6. BNSF = Burlington Northern Santa Fe Railway
7. whse. = warehouse
8. bldg = building
9. mfg = manufacturing
10. psf = pounds per square foot
11. BHS = Builders Hardware Store/Supply
12. For a discussion of structures that may be demolished due to construction, refer to the Social and Economic Discipline Report.
13. Under "foundation type," listings such as "Addition/remodel (1965)" indicate that in 1965 an addition and remodel were completed on the property. Listings such as "Footings (2,500 psf)" indicate that the structure is supported on shallow footings with a design bearing pressure of 2,500 psf.

Geologic Conditions

The geologic conditions were interpreted from information obtained from the current and previous subsurface explorations, geologic maps of the area, and a geologic site reconnaissance, as described previously. A preliminary geologic map of each build alternative is presented in Figures 15, 16, and 17. A summary of the geologic units is presented in Figure 18. The following sections include a description of the regional and site geology, and the soil and groundwater conditions encountered along the alignments. The generalized subsurface conditions along Alternative Alignments A, C, and D are shown on the profiles presented on Figures 19, 20, and 21, respectively.

The proposed alternatives extend across a north-trending topographic trough called Interbay. The trough is bounded on both sides by glacial uplands; Magnolia on the west and Queen Anne Hill on the east. While the uplands are comprised of very dense and hard glacial soils laid down during the advance and retreat of several glaciations, the intervening topographic swale/trough of Interbay is comprised of loose to dense glacial recessional outwash, beach deposits, and very soft to stiff estuarine deposits laid down since the last retreat of glacial ice approximately 13,000 years ago. Since the late nineteenth century, the Interbay area (specifically Smith Cove) has been filled with various materials.

The subsurface geology encountered along the three proposed build alignments includes pre-Vashon deposits, Vashon glacial deposits, and overlying Holocene (post-Vashon) deposits. 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 deposits encountered along the project alignments and for interpreting stratigraphic correlation between borings. It also provides a framework for anticipating subsurface conditions that may not have been disclosed directly by the exploration program but which may reasonably be expected based on past local experience with similar geologic units.

Project 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 a series of north-trending ridges separated by deeply cut ravines and broad valleys. These ridges and valleys are the result of glacial scouring and subglacial erosion. In general, the ground surface elevation is within 500 feet of sea level.

During the past 3 million years (Pleistocene Epoch), fluctuating climates have caused the waxing and waning of glacial ice in the Puget Lowland. Geologists now believe that the Puget Sound area has been subjected to six or more major glaciations during the Pleistocene Epoch (2 million years ago to about 10,000 years ago), which filled the Puget Lowland to significant depths with a complex sequence of glacial and nonglacial sediments. These glaciers originated in the coastal mountains of British Columbia. The maximum southward advance of the ice was about halfway between Olympia and Centralia (about 60 miles south of Seattle). During the most recent ice advance into the central Puget Lowland (Vashon Stade of

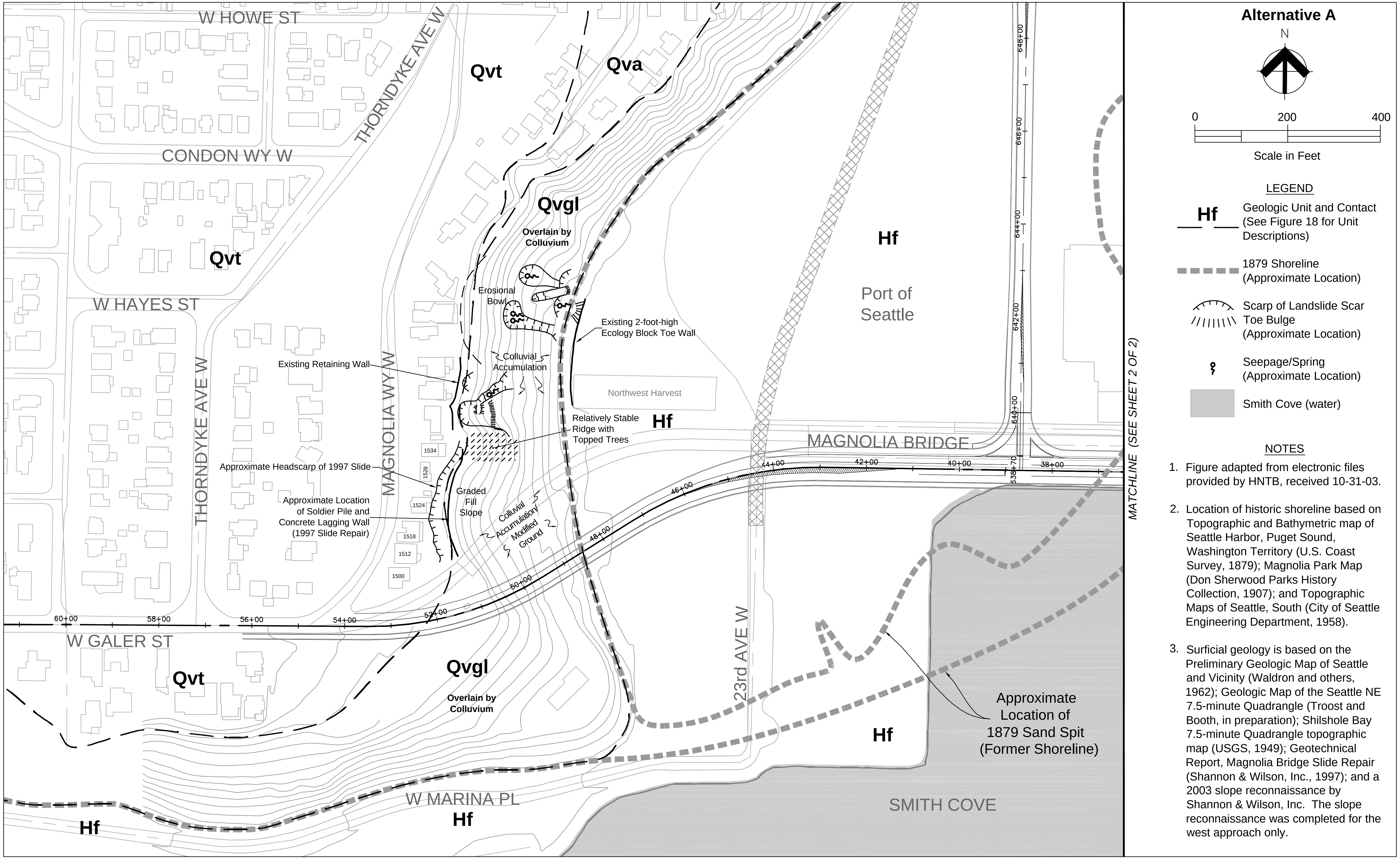

Figure 15, Sheet 1 of 2 - Preliminary Geologic Map - Alternative A

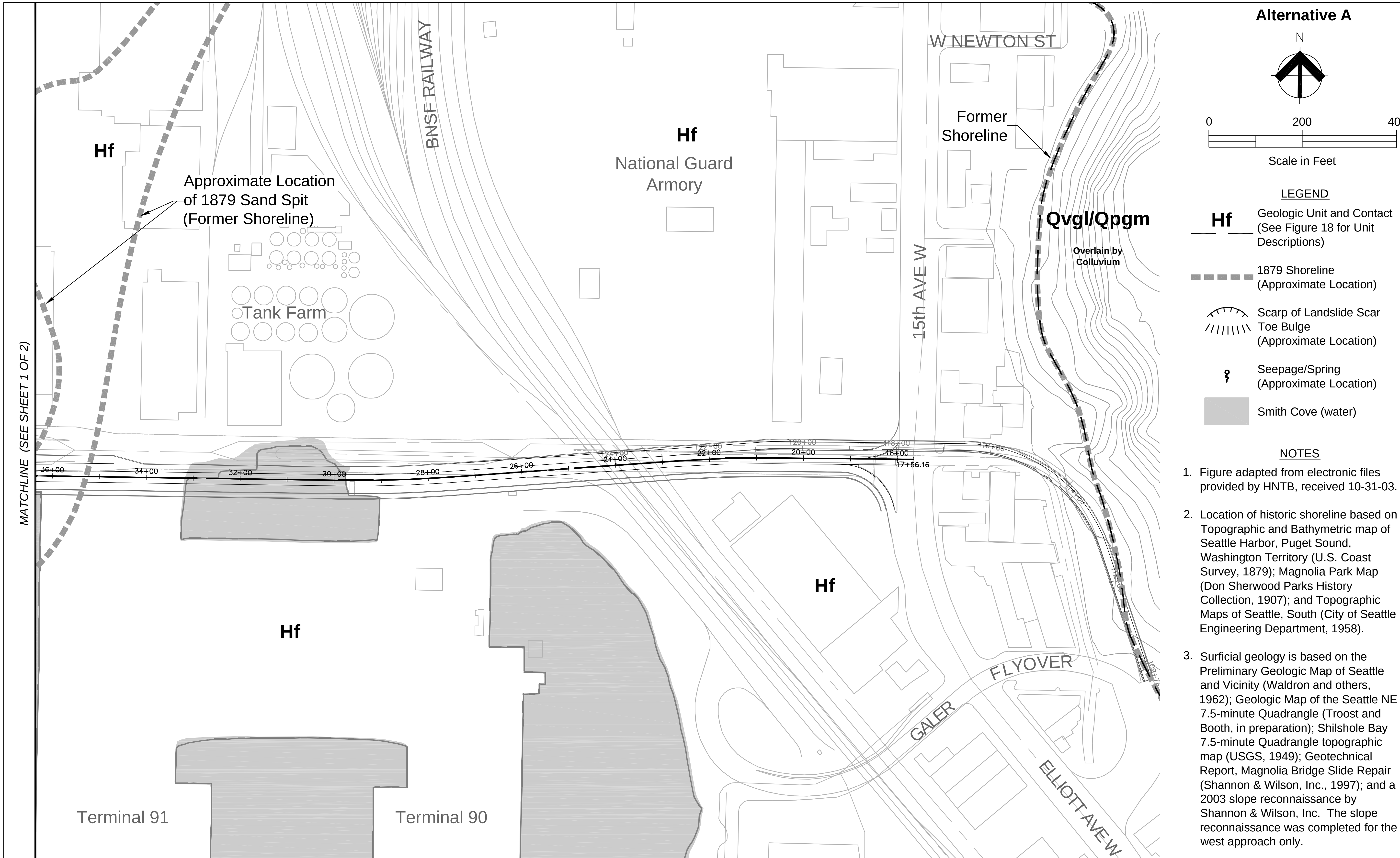

Figure 15, Sheet 2 of 2 - Preliminary Geologic Map - Alternative A

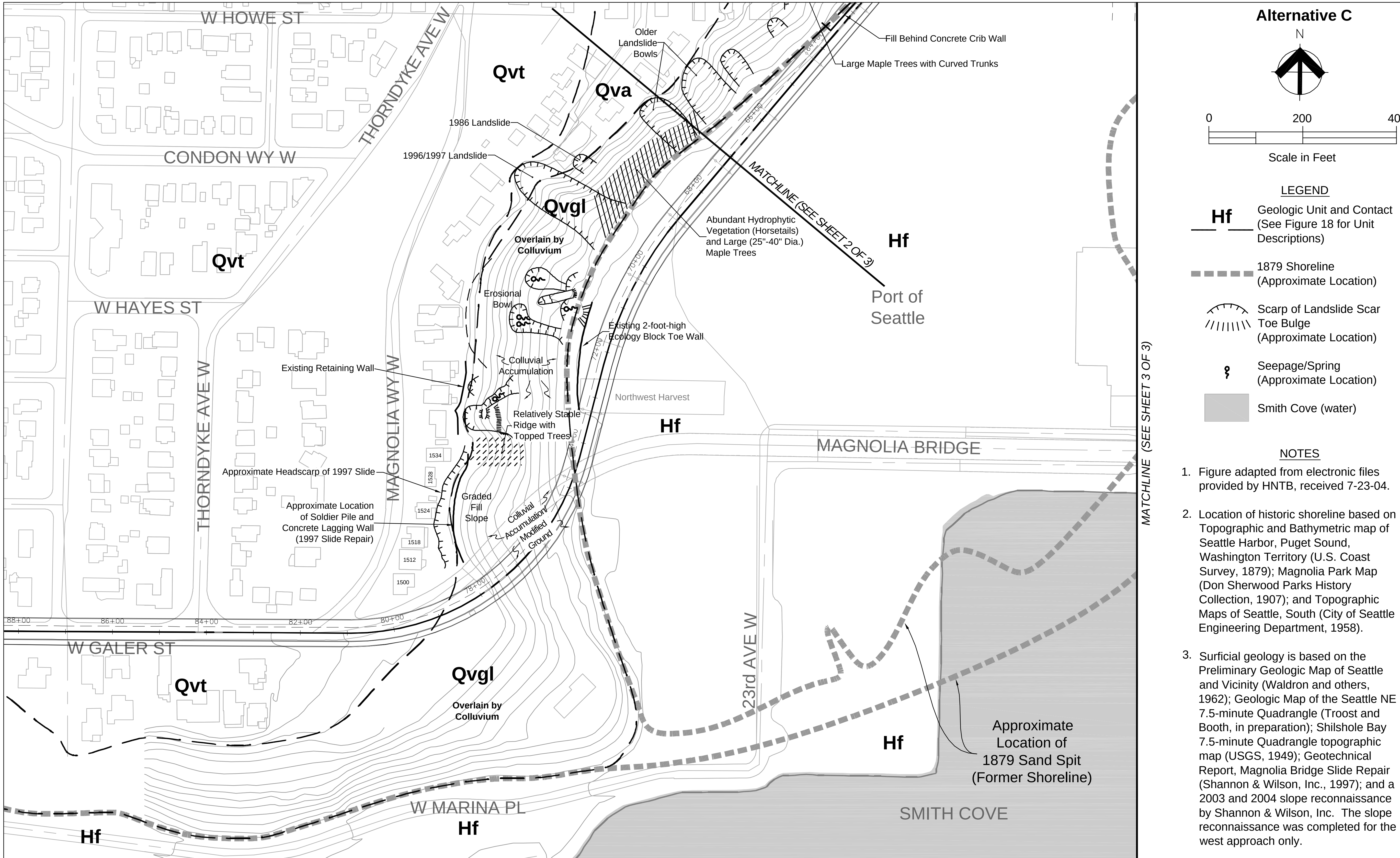

Figure 16, Sheet 1 of 3 - Preliminary Geologic Map - Alternative C

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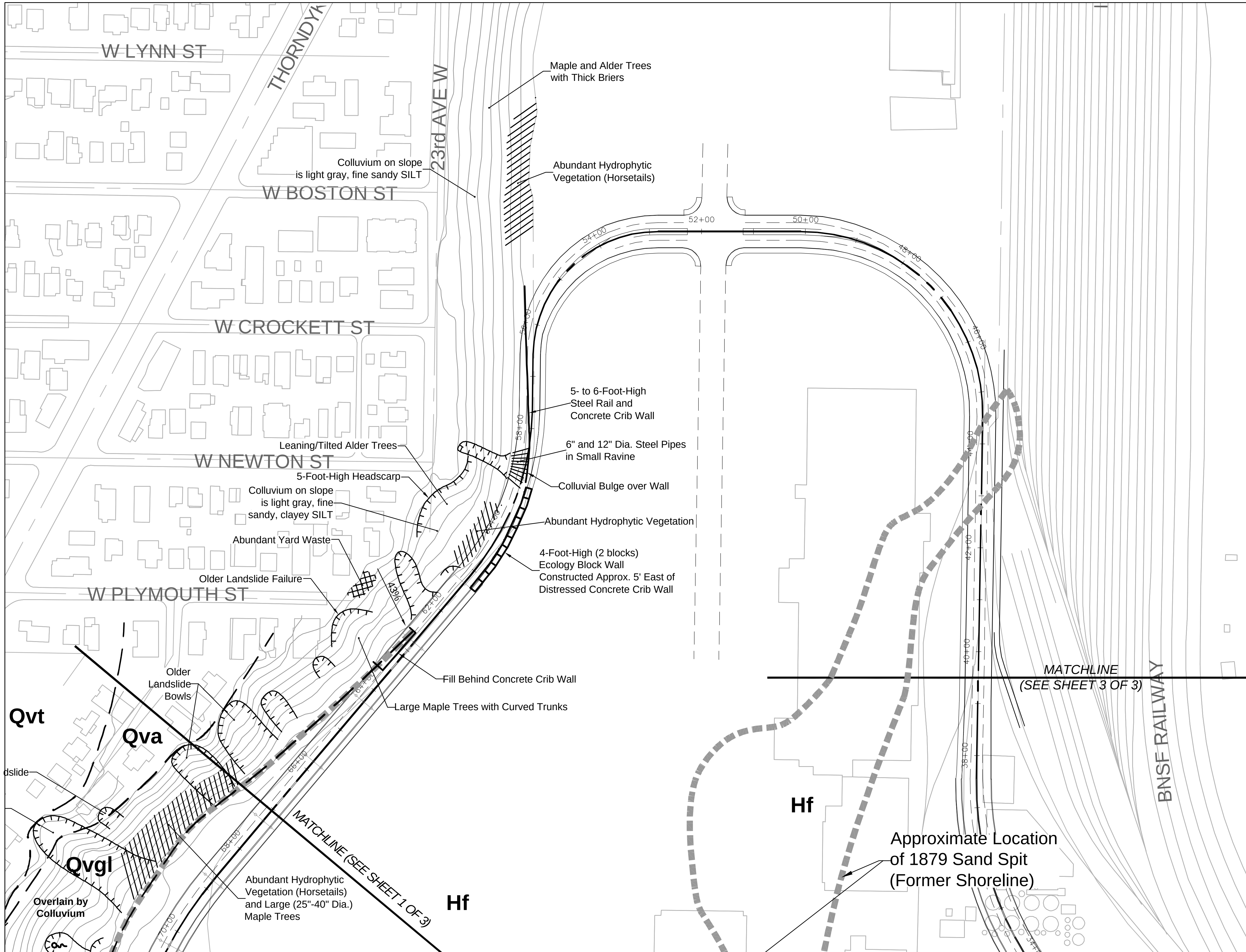

Alternative C

N

0 200 400

Scale in Feet

LEGEND

Hf — Geologic Unit and Contact (See Figure 18 for Unit Descriptions)

1879 Shoreline (Approximate Location)

Scarp of Landslide Scar
Toe Bulge
(Approximate Location)

Seepage/Spring (Approximate Location)

Smith Cove (water)

NOTES

1. Figure adapted from electronic files provided by HNTB, received 7-23-04.
2. Location of historic shoreline based on Topographic and Bathymetric map of Seattle Harbor, Puget Sound, Washington Territory (U.S. Coast Survey, 1879); Magnolia Park Map (Don Sherwood Parks History Collection, 1907); and Topographic Maps of Seattle, South (City of Seattle Engineering Department, 1958).
3. Surficial geology is based on the Preliminary Geologic Map of Seattle and Vicinity (Waldron and others, 1962); Geologic Map of the Seattle NE 7.5-minute Quadrangle (Troost and Booth, in preparation); Shilshole Bay 7.5-minute Quadrangle topographic map (USGS, 1949); Geotechnical Report, Magnolia Bridge Slide Repair (Shannon & Wilson, Inc., 1997); and a 2003 and 2004 slope reconnaissance by Shannon & Wilson, Inc. The slope reconnaissance was completed for the west approach only.

Figure 16, Sheet 2 of 3 - Preliminary Geologic Map - Alternative C

File: J:\211\09759-008\G&S Discipline Report (1-05)\21-1-09759-008 fig. 16.dwg Date: 03-21-2005 Author: SAC

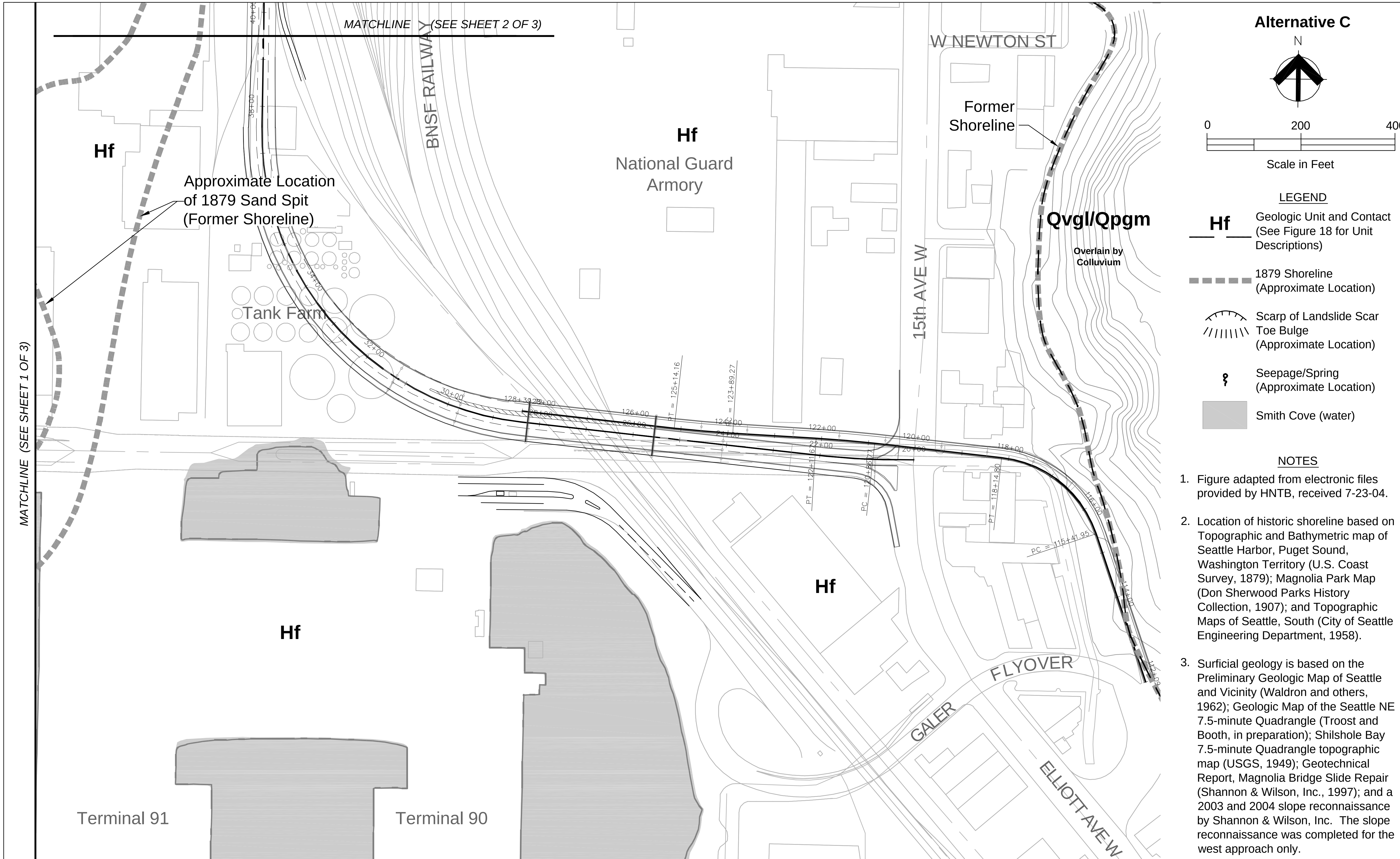

Figure 16, Sheet 3 of 3 - Preliminary Geologic Map - Alternative C

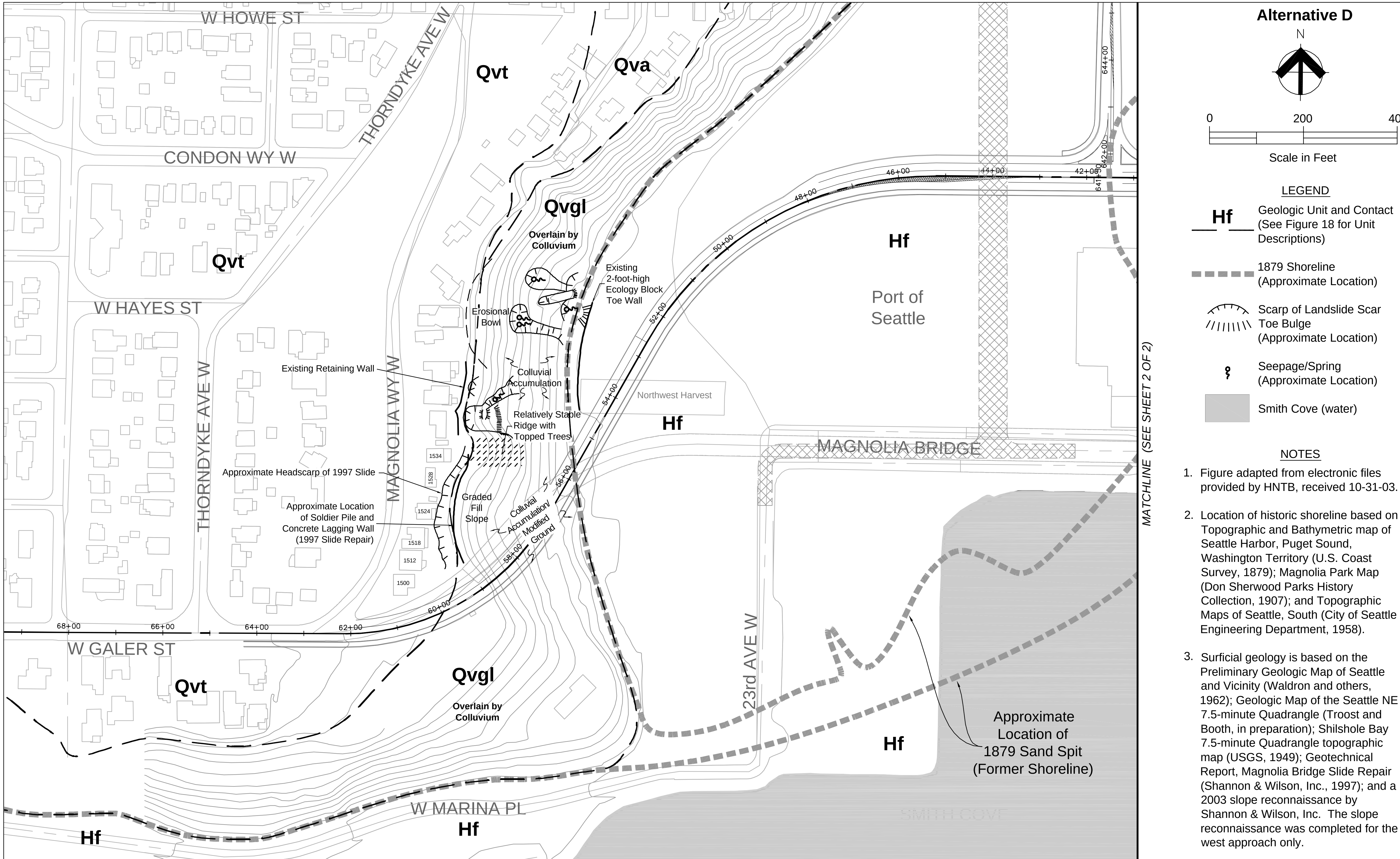

Figure 17, Sheet 1 of 2 - Preliminary Geologic Map - Alternative D

File: J:\21\1\09759-008\G&S Discipline Report (1-05)\21-1-09759-008 fig. 17.dwg Date: 02-07-2005 Author: SAC

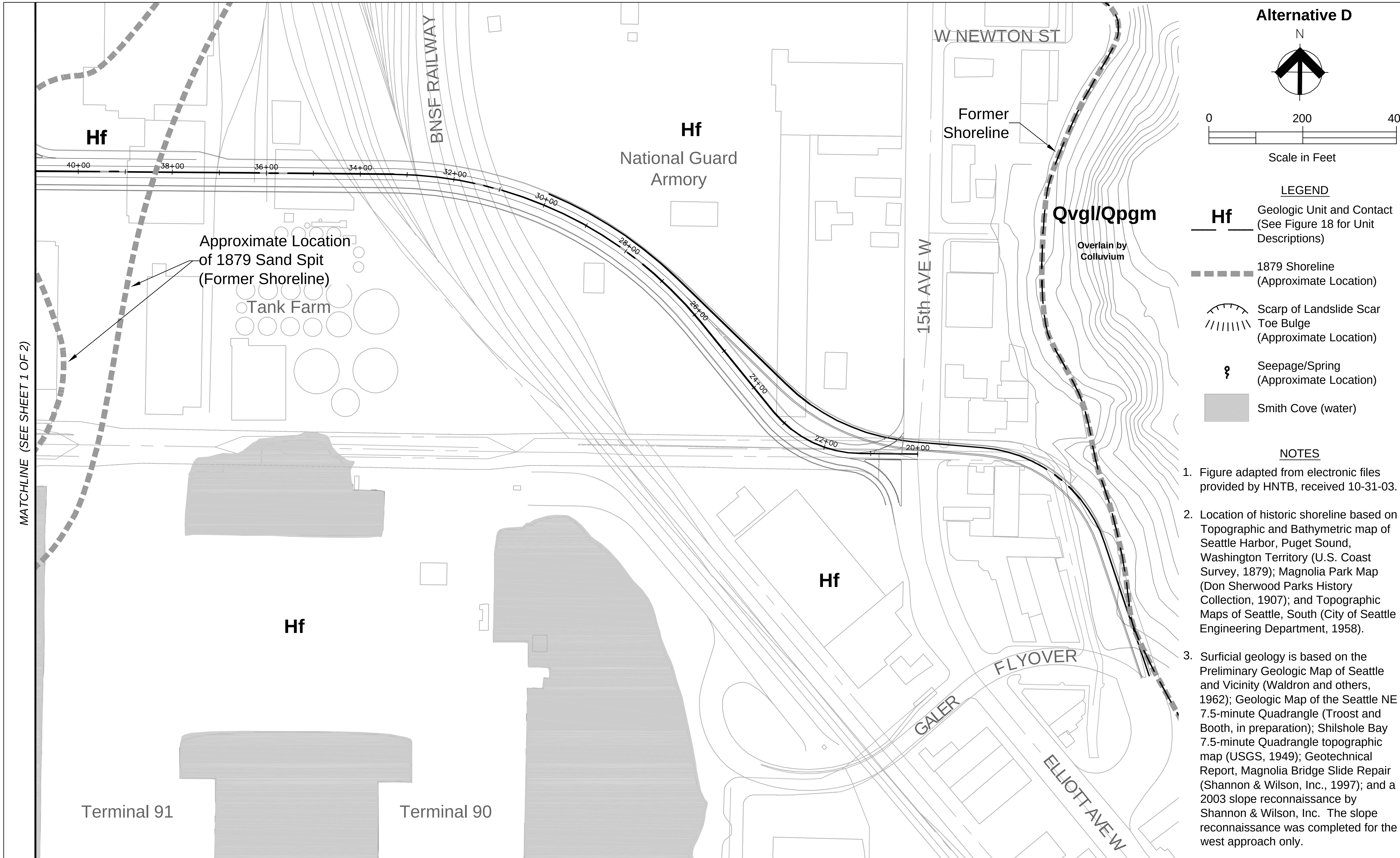

Figure 17, Sheet 2 of 2 - Preliminary Geologic Map - Alternative D

GEOLOGIC UNITS

HOLOCENE DEPOSITS

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.

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.

He ESTUARINE DEPOSITS: Estuary deposits of intertidal zones associated with rivers and streams located along the present and former Puget Sound shoreline.
Clayey Silt, silty Clay, Silt, and fine Sand; very soft to very 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 and shell debris; loose to dense.

QUATERNARY VASHON DEPOSITS

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.

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.

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: Fine-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.

QUATERNARY PRE-VASHON DEPOSITS

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.

Qpnm MUDFLOW DEPOSITS: Distal deposits of mass movements such as landslides or lahars.
Stratified or irregular bodies of a heterogeneous mixture of Gravel, Sand, Silt, and Clay; pumice, obsidian and ash common; rare organics (charcoal); 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.

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.

NOMENCLATURE

GEOLOGIC AGE DESIGNATION		DEPOSITIONAL ENVIRONMENT, GEOLOGIC PROCESS, OR LITHOLOGY		
H = Holocene		f = fill ls = landslide	e = estuarine b = beach	Present
Q = Quaternary	v = Vashon	r = recessional	o = outwash at = ablation till	10,000 yrs BP *
		t = till (lodgment) a = advance outwash gl = glaciolacustrine		
	p = Pre-Vashon 6 or more glacial and interglacial episodes	n = nonglacial (interglacial)	l = lacustrine m = mudflow	15,000 yrs BP *
		g = glacial	l = lacustrine o = outwash m = marine t = till (lodgment)	
				2,000,000 yrs BP

* *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.*

NOTES

1. The description of each geologic unit includes only general information regarding the environment of deposition and basic soil characteristics.
2. 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.
3. 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.

LEGEND

Glacially Overridden Soil Units Below Line

Years BP — Radiocarbon Years Before Present (1950)

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Figure 18 - Geologic Unit Explanation

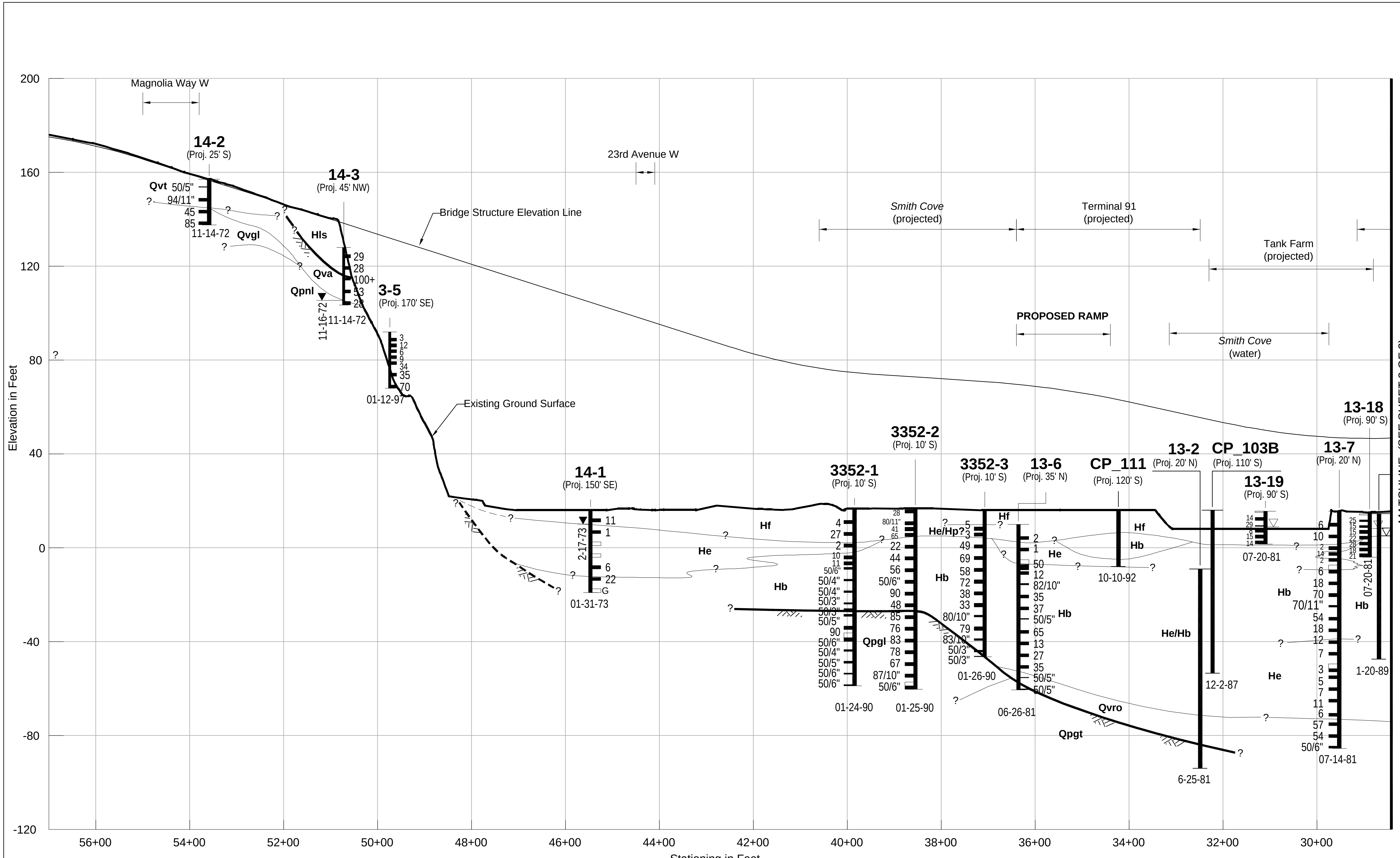

Figure 19, Sheet 1 of 2 - Generalized Subsurface Profile - Alternative A

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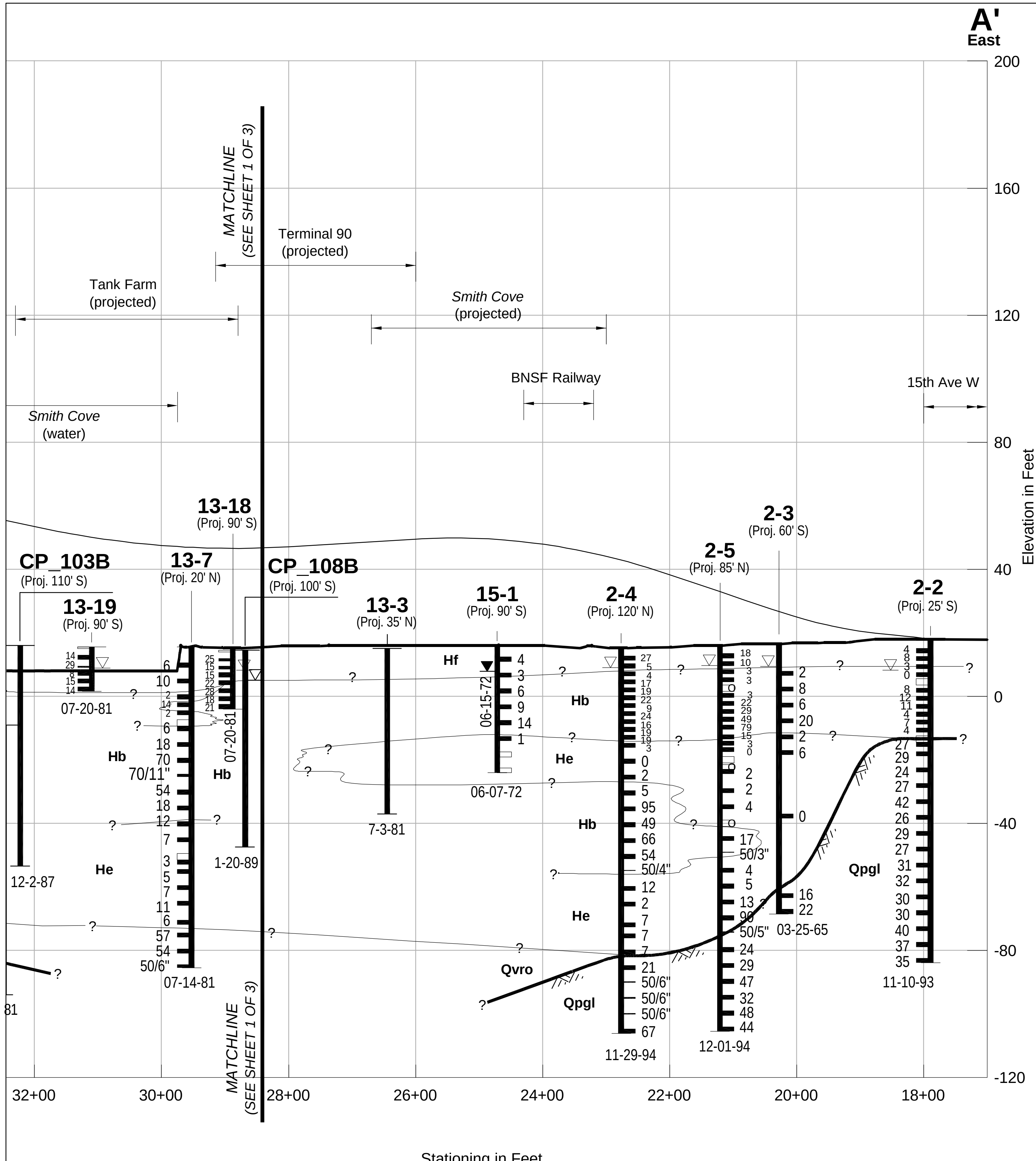
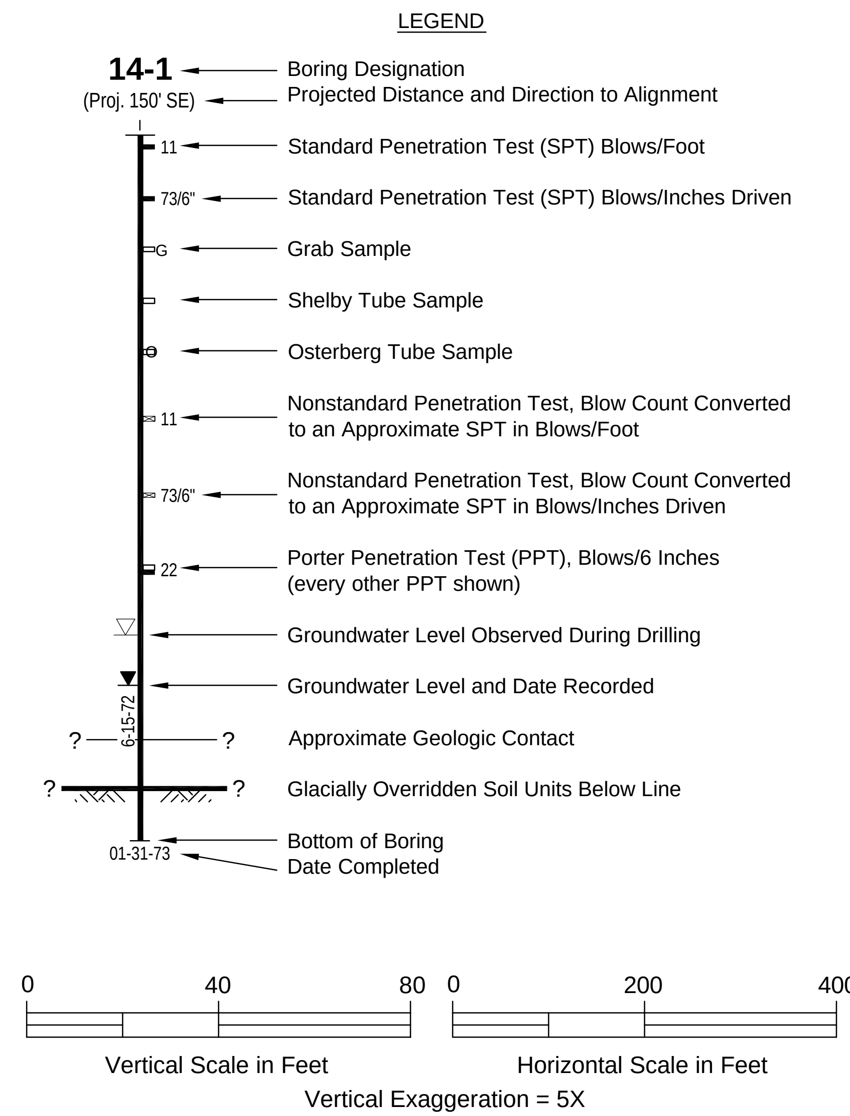




NOTES

1. This subsurface profile is generalized from materials observed in soil borings. Variations may exist between profile and actual conditions. Contact lines are dashed where questionable.
2. Figure adapted from electronic files provided by HNTB, received 10-31-03.
3. Datum: NAVD 88

Figure 19, Sheet 2 of 2 - Generalized Subsurface Profile - Alternative A

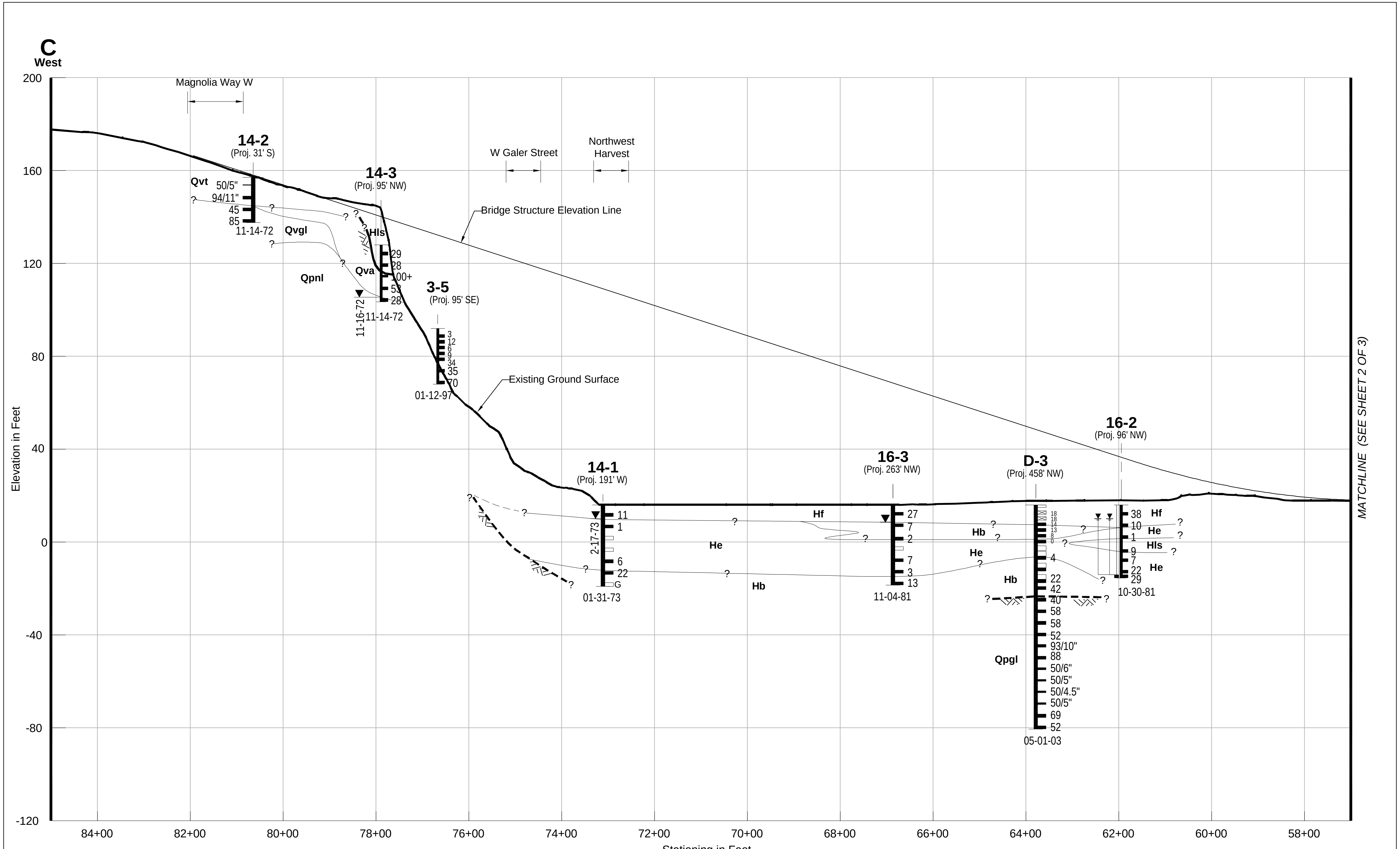


Figure 20, Sheet 1 of 3 - Generalized Subsurface Profile - Alternative C

File: J:\21\09759-008\G&S Discipline Report (1-05)\21-1-09759-008 fig. 20.dwg Date: 02-07-2005 Author: SAC

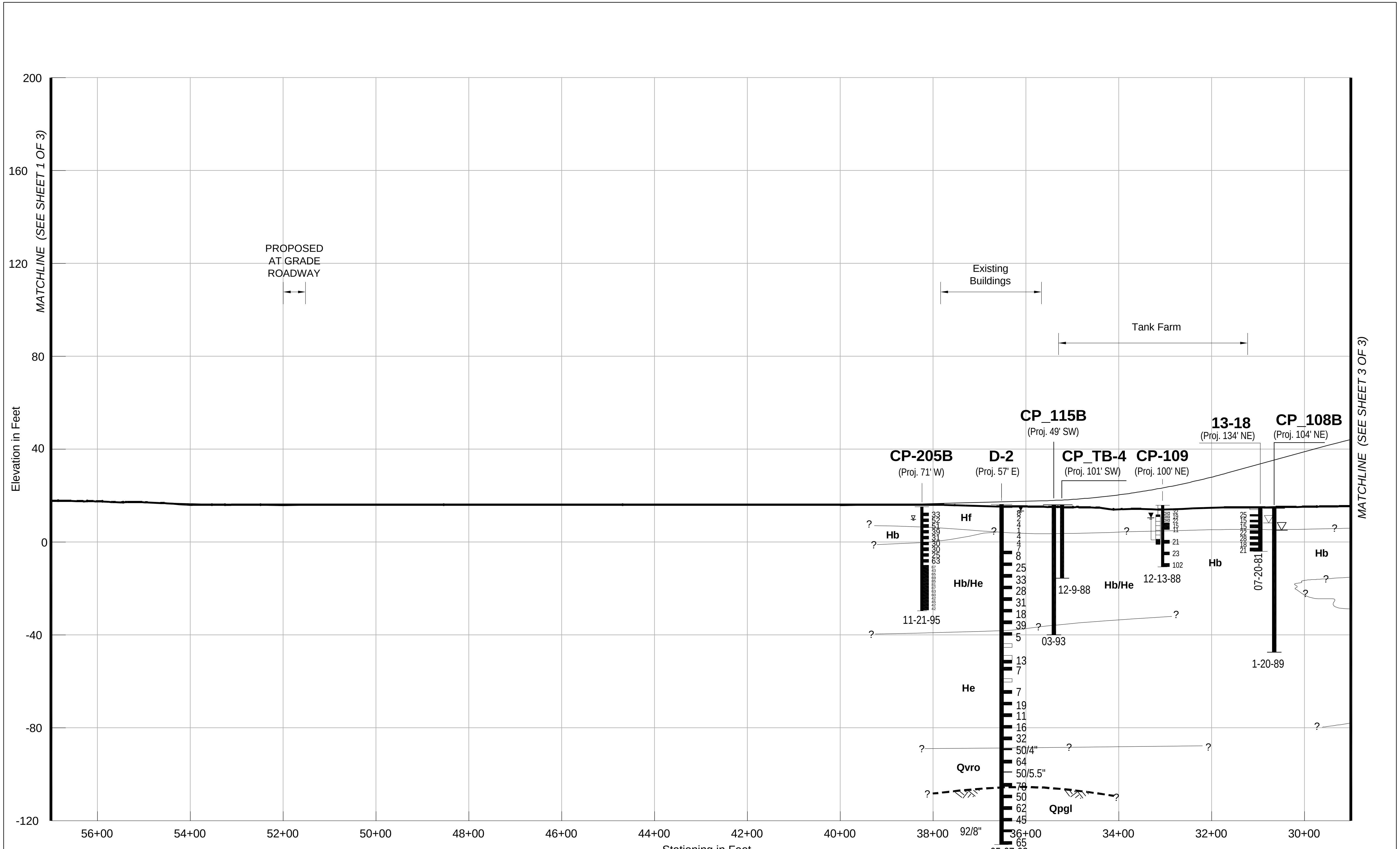
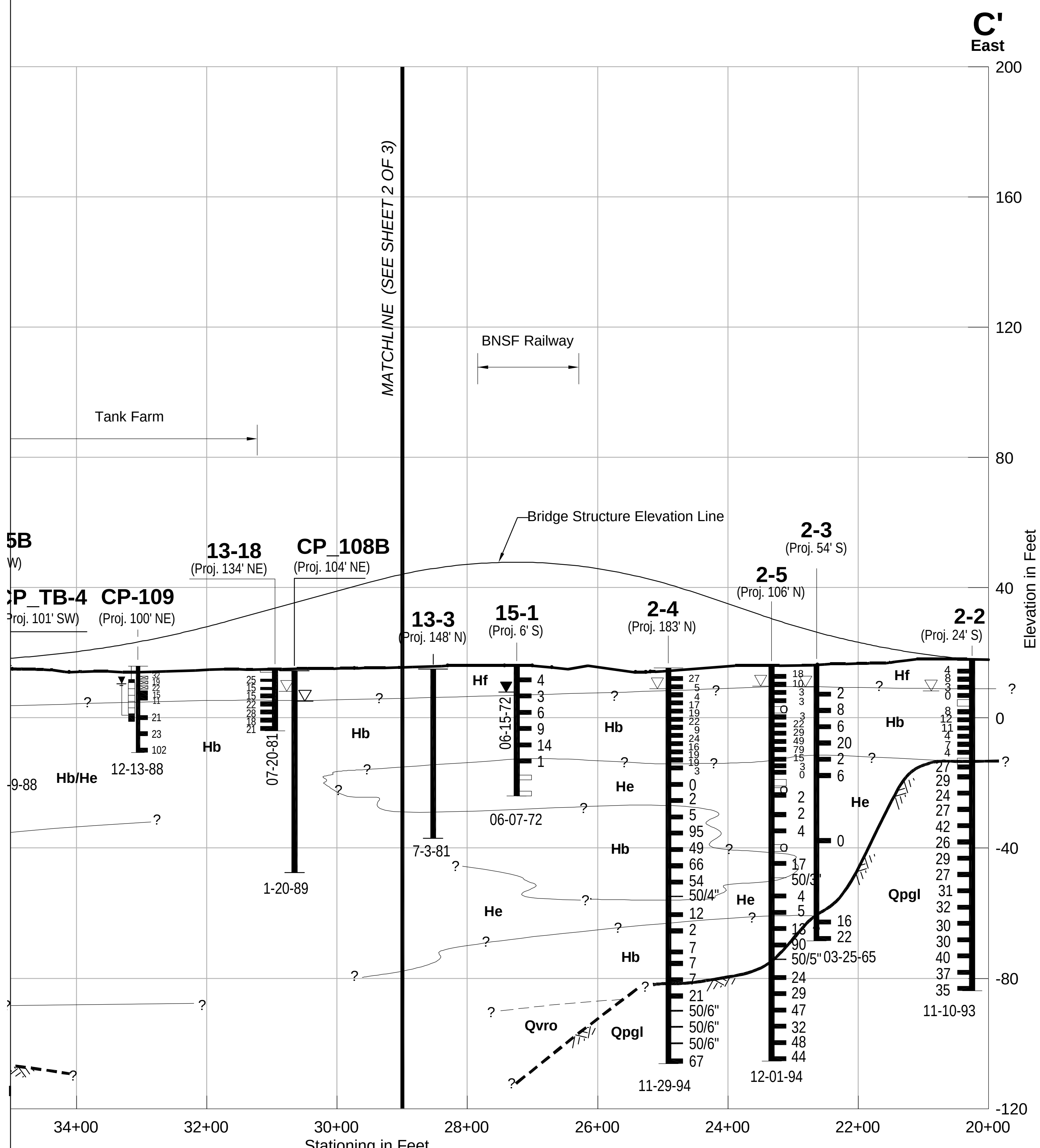
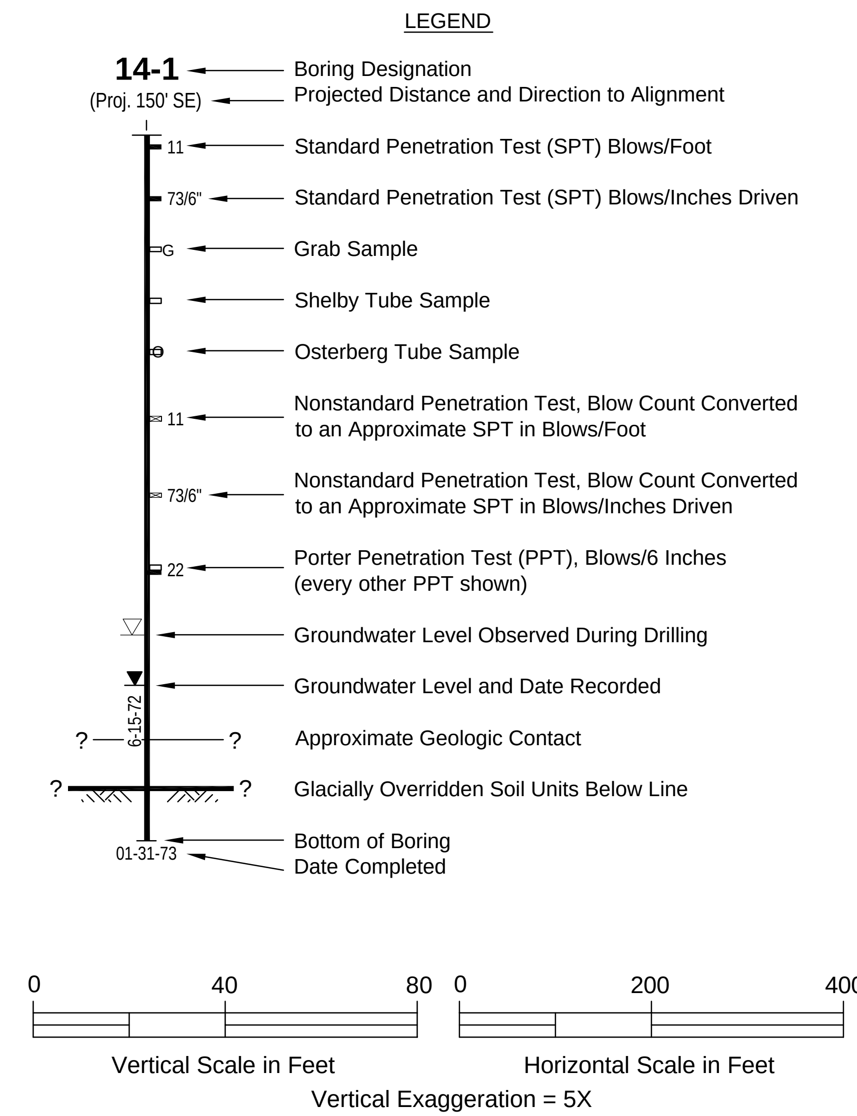


Figure 20, Sheet 2 of 3 - Generalized Subsurface Profile - Alternative C





NOTES

1. This subsurface profile is generalized from materials observed in soil borings. Variations may exist between profile and actual conditions. Contact lines are dashed where questionable.
2. Figure adapted from electronic files provided by HNTB, received 10-31-03 and 7-23-04.
3. Datum: NAVD 88
4. The blow counts for CP-205B were assumed to be SPTs.
5. The top three samples in CP-109 were taken using a 3-inch-diameter split spoon. The log did not describe the hammer weight or drop height; therefore, the blow counts could not be converted to SPTs.

Figure 20, Sheet 3 of 3 - Generalized Subsurface Profile - Alternative C

File: J:\211109759-008\G&S Discipline Report (1-05)\21-1-09759-008 fig. 20.dwg Date: 02-07-2005 Author: SAC

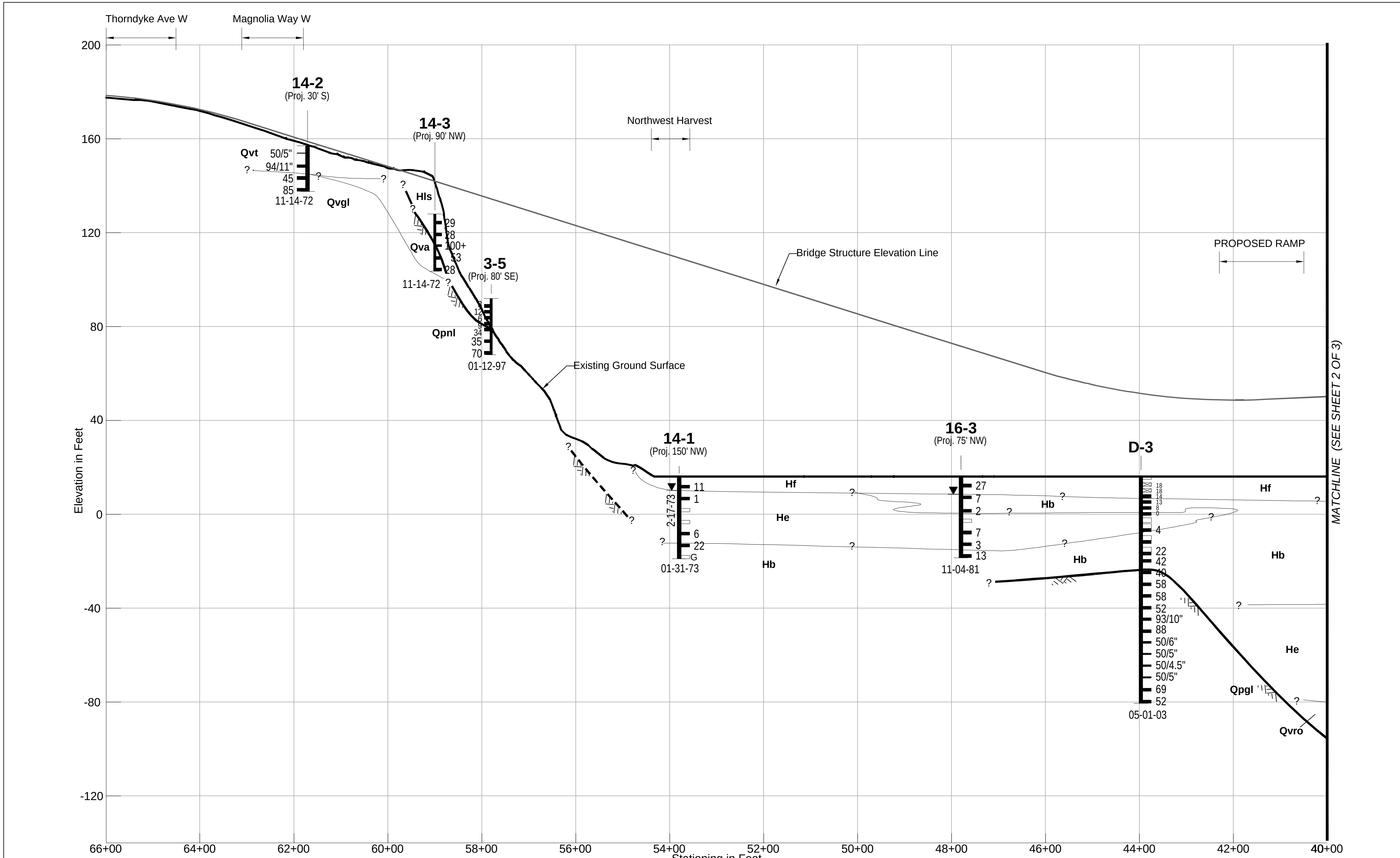


Figure 21, Sheet 1 of 3 - Generalized Subsurface Profile - Alternative D

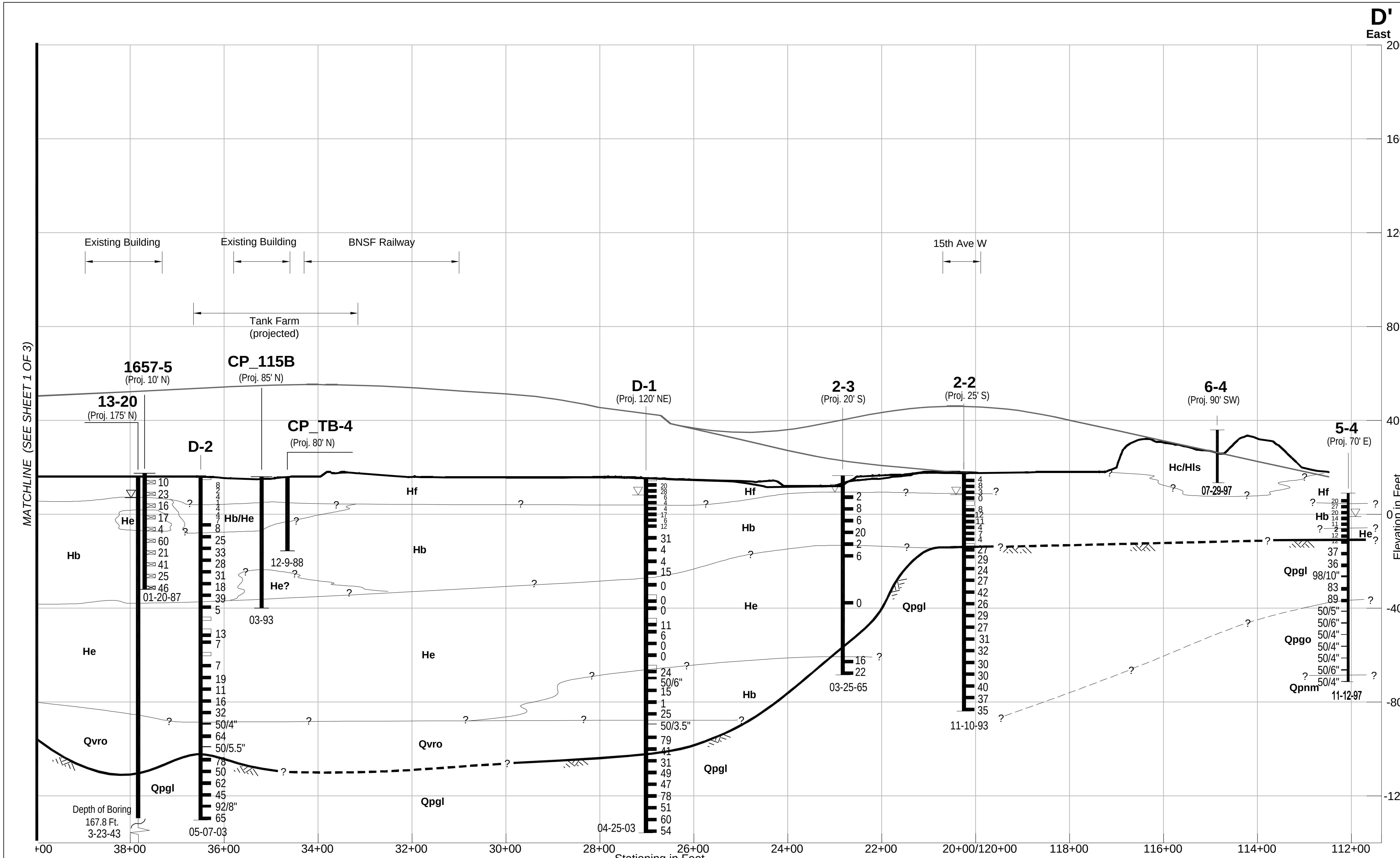
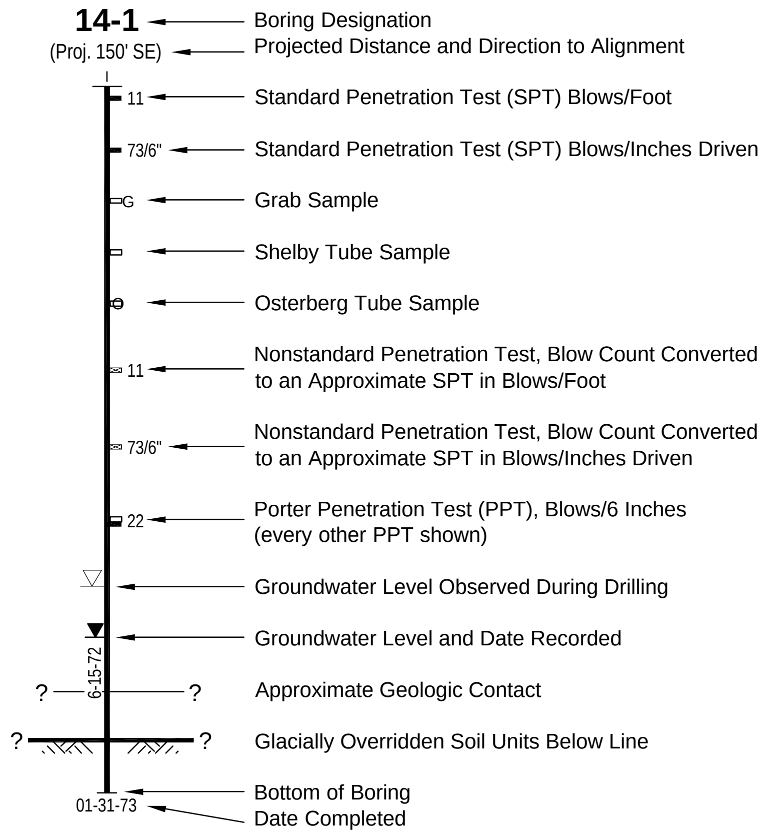
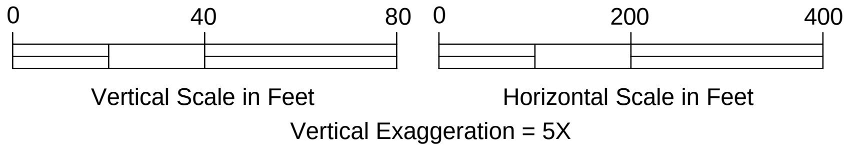


Figure 21, Sheet 2 of 3 - Generalized Subsurface Profile - Alternative D

LEGEND





NOTES

1. This subsurface profile is generalized from materials observed in soil borings. Variations may exist between profile and actual conditions. Contact lines are dashed where questionable.

2. Figure adapted from electronic files provided by HNTB, received 10-31-03.

3. Datum: NAVD 88

Figure 21, Sheet 3 of 3
Generalized Subsurface Profile - Alternative D

File: J:\211\09759-008\G&S Discipline Report (1-05)\21-1-09759-008 fig. 21.dwg Date: 02-07-2005 Author: SAC

Fraser Glaciation), the thickness of ice is estimated to have been about 3,000 feet in the alignment area. The last ice receded from the study area about 13,500 years ago.

The distribution of the sediments in the Puget Lowland is complex, because each glacial advance deposited new sediment and partially eroded older sediments. During interglacial episodes, the complete or partial erosion, or the reworking of some deposits, as well as the local deposition of other sediments further complicated the geologic setting. Pre-Vashon sediments are all of those deposited prior to the Vashon Stade, including both glacial and nonglacial materials.

The soils that were deposited during ice recession (Qvro) and after the disappearance of the Vashon ice in the Puget Lowland have engineering characteristics very different from soils that have been overridden by glacial ice. Of particular note in the project area are fill, beach, estuarine, and reworked glacial deposits that underlie the Interbay area. These Holocene deposits have not been overridden by glacial ice and exhibit densities and consistencies ranging between very loose to dense and very soft to very stiff.

Based on the results from current explorations, understanding of the geology in the area, review of the available subsurface information collected in data searches, and published references, the following geologic units would likely be encountered along the three project alignments.

Holocene (post-glacial) deposits consist of four units:

Hf	Human-placed fill materials
Hls	Landslide deposits
He	Estuarine deposits
Hb	Beach deposits

Vashon recessional deposits consist of one unit:

Qvro	Vashon recessional outwash

Vashon glacial (glacially consolidated) deposits consist of three units:

Qvt	Vashon lodgment till
Qva	Vashon advance outwash
Qvgl	Vashon glaciolacustrine deposits

Pre-Vashon glacial deposits consist of three units:

Qpgo	Glacial outwash deposits
Qpgl	Glaciolacustrine deposits
Qpgt	Pre-Vashon lodgment till

Pre-Vashon nonglacial deposits consist of two units:

Qpnl	Lacustrine deposits
Qpnm	Mudflow deposits

A general soil description for each of the above geologic units is presented on Figure 18.

Hydrogeologic Regime

The hydrogeologic regime in the Puget Sound area is highly variable. Groundwater flow is generally controlled by glacial stratigraphy and groundwater recharge/discharge relationships. Groundwater recharge typically occurs in the upland areas of Seattle. Groundwater movement is then, in principle, primarily downward to the discharge areas, and then eventually to the major surface water bodies such as Elliott Bay, Lake Washington, and Puget Sound.

The complex glacial stratigraphy in the Seattle area has a strong influence on the nature of groundwater flow. The direction of groundwater movement is controlled, in part, by the permeability of the deposits. Groundwater flow in the stratigraphically higher, coarse-grained, high-permeability deposits, such as glacial outwash, likely flows horizontally and vertically under unconfined water table conditions. Groundwater in these units is often perched on top of low-permeability till and lacustrine units. Much of groundwater flows laterally and may discharge at springs or seeps on the hillsides. However, a portion of this groundwater percolates vertically downward through the lower-permeability units or windows/cracks in the impervious layers to underlying deposits. The permeabilities of glacial deposits typically differ by orders of magnitude. Because of this, there is commonly more than one unit that perches groundwater in the stratigraphic sequence; therefore, there are commonly multiple, areawide piezometric surfaces.

The direction of groundwater movement is also governed by hydraulic gradients, which may decrease or increase with depth in the stratigraphic section. Downward hydraulic gradients are typical in upland areas; upward hydraulic gradients are typical in water-bearing units close to the major discharge bodies. Discussions of groundwater are provided in the Water Quality Discipline Report.

Soil Description Overview for Proposed Alternatives

Based on the soils encountered in the recent subsurface explorations and review of the available subsurface information within the project area, three subsurface geologic profiles were developed (Figures 19, 20, and 21). The information contained on these profiles is preliminary. A description of the geologic terms used on these profiles is presented on Figure 18. The locations and elevations of the recent subsurface explorations (borings D-1 through D-3 and H-1 through H-3) were not surveyed, and the existing exploration locations and elevations should be considered approximate. Borings H-1 through H-3 were drilled for the now-deleted Alternative H; therefore, the boring locations are not shown on Figures 19, 20, or 21, but the boring logs are included in Appendix A. Furthermore, while the soils encountered in the most recent explorations provided the basis for the subsurface interpretation, additional subsurface information was used from existing field explorations of variable quality from many different sources over a period of 60 years and should, therefore, be considered approximate as well.

The subsurface conditions at the site were characterized in a multi-step process. Soils encountered in the explorations were first described using soil classification terms and then appropriate geologic unit names were assigned. The geologic units used for this project are based on basic divisions of geologic time and on geologic processes. The grouping of soils in this fashion was used because the geotechnical properties of the soils are largely controlled by (1) grain size and sorting, which are functions of depositional processes, and by (2) consolidation and structural discontinuities, which are functions of the geologic history. Understanding the geologic history and depositional processes also allows for better interpolation of the unit boundaries between borings. The geologic unit designations applied to the soils encountered along the alignments represent an interpretation of the grouping of complex sediments and soil types, and are indicated on the current boring logs. The generalized subsurface profiles (Figures 19, 20, and 21) indicate the approximate contact between glacially overconsolidated soil and normally consolidated soil.

The Alternatives A, C, and D are located in the southern portion of the Interbay embayment and extend alongside (Alternative A), approximately 570 feet north (Alternative D) and about 1,800 feet north (Alternative C) of the existing Magnolia Bridge between 15th Avenue West and West Galer Street (on Magnolia Bluff). In addition to older, existing information, the descriptions presented for the soils encountered along these alignment alternatives are based on recent explorations (borings D-1, D-2, and D-3) performed along Alternative D.

The subsurface conditions encountered along the alternatives are illustrated on the Generalized Subsurface Profiles for Alternatives A, C, and D; Figures 19, 20, and 21, respectively. Refer to the site and exploration plans, Figures 10 through 12, for the generalized subsurface profile locations. As shown, the soil conditions encountered in the vicinity east of 15th Avenue West consist of approximately 10 to 15 feet of Holocene fill (Hf) underlain by Holocene beach (Hb) and estuarine (He) deposits to a depth of approximately 30 feet. The Holocene fill is characterized by heterogeneous soils including silty sand and gravel with debris and shell fragments. The fill densities range from loose to medium dense. In the vicinity of the Galer Flyover, approximately 20 to 30 feet of Holocene colluvium (Hc) and landslide debris (Hls) are encountered in existing borings. Hard, Vashon glaciolacustrine, clayey silt, and silty clay (Qvgl) soils were encountered in the existing borings below the Holocene deposits. Pre-Vashon glacial outwash (Qpgo) and a thin, pre-Vashon mudflow deposit (Qpnm) underlie the glaciolacustrine soils at the east end of Alternative D. Pre-Vashon glacial outwash is comprised of very dense, clean to slightly silty sand, and the pre-Vashon nonglacial mudflow deposit is comprised of hard, gravelly, sandy, clayey silt with scattered ash seams.

West of 15th Avenue West, in the vicinity of borings D-1 and D-2, the Holocene beach and estuarine soils thicken substantially to a maximum observed thickness of 103 to 105 feet thick. Ten to 13 feet of normally consolidated Vashon recessional outwash (Qvro) underlie the Holocene deposits in borings D-1 and D-2. Recessional outwash is comprised of very dense to dense, slightly gravelly, silty sand; fine sandy silt; and slightly clayey silt with scattered till-like pockets. Hard, pre-Vashon glaciolacustrine, silty clay to clayey silt was encountered below the recessional outwash sand at an approximate elevation of –100 feet. Borings D-1 and D-2 were both terminated in the glaciolacustrine soils at depths of 151.5 and 146.5 feet, respectively.

North of boring D-2, in the vicinity of the northern limits of Alternative C, the thickness of the Holocene beach (Hb) and estuarine (He) deposits is unknown due to the lack of explorations in the area. In general, the thickness of the Holocene soil decreases to the north, away from the mouth of the Interbay embayment.

West of boring D-2, the Holocene beach (Hb) and estuarine (He) deposits thin to a thickness of approximately 30 feet, as encountered in boring D-3. Approximately 10 feet of fill (Hf) was sampled at the surface in boring D-3. Hard, Vashon glaciolacustrine, silty clay (Qvgl) underlies the Holocene deposits in boring D-3 at an approximate elevation of –24 feet. West of boring D-3, the thickness of the Holocene soils is not known because none of the existing borings penetrated into glacially over-consolidated deposits.

In the general vicinity of Magnolia Way West and West Galer Street (on Magnolia Bluff), existing boring information reveals Holocene fill overlying 3 to 13 feet of Vashon glacial till (Qvt). Some of the existing boring information in this area is 150 to 210 feet away from the proposed alignments and should be considered approximate. Vashon glacial till is comprised of very dense, silty, gravely sand to silty, sandy gravel. Underlying the till layer along the west limit of Alternatives A, C, and D is Vashon advance outwash (Qva), comprised of very dense, slightly silty to silty, fine sand. Vashon glaciolacustrine silt and clayey silt were encountered at the bottom of the existing borings in the vicinity of West Galer Street.

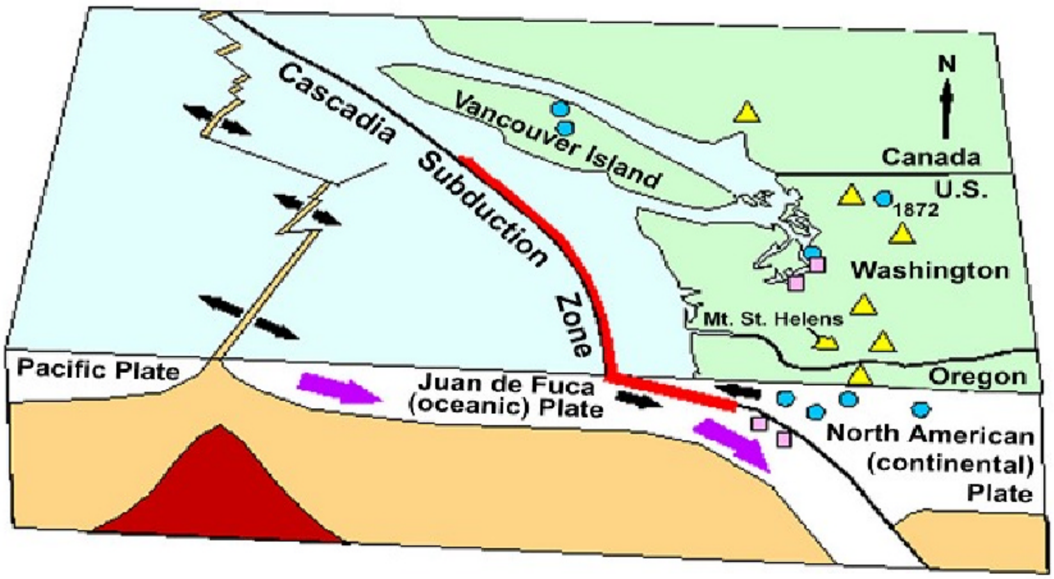
Groundwater

Groundwater levels were obtained during drilling of current borings and from the previous exploration logs. Groundwater was generally observed within 10 feet of the ground surface. However, the groundwater levels are likely to be directly related to the tidal fluctuation of Smith Cove. Therefore, to accurately understand the groundwater level situation along the proposed alternative alignments, tidal variations must be reviewed. The groundwater levels noted on the logs and in the profile represent the level at that particular time, but do not represent the fluctuations that are likely to occur throughout a 24-hour period or the lag time between tidal fluctuation and groundwater level changes. More details on groundwater conditions are provided in the Water Quality Discipline Report.

Tectonics and Seismicity

The study area is located in a moderately active tectonic province that has been subjected to numerous earthquakes of low to moderate strength and occasionally to strong shocks during the brief 170-year written historical record in the Pacific Northwest. 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 relative motion among these plates not only results in east-west compressive strain, but also results 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) above the subduction zone. As in most active convergence zones, the Cascadia Subduction Zone (CSZ) contains a continental fore-arc consisting of accreted sedimentary and volcanic rocks in front of a landward mountainous, active volcanic arc. Unlike most active subduction zones, there is a conspicuous absence of an oceanic trench near the juncture of the two plates.

Within the present understanding of the regional tectonic framework and historical seismicity, three broad seismogenic zones have been identified. These include a shallow crustal source zone, a deep subcrustal (intraslab) source zone in the subducted Juan de Fuca Plate, and an interplate or subduction zone (Figure 22).



Deep earthquakes (40 miles below the Earth's surface) are within the subducting oceanic plate as it bends beneath the continental plate. The largest deep Northwest earthquakes in the 20th century were in 1949 (M 7.1) and 1965 (M 6.5).

Shallow earthquakes (less than 15 miles deep) are caused by faults in the North American Continent. The Seattle fault produced a shallow magnitude 7+ earthquake 1,100 years ago. Other magnitude 7+ earthquakes occurred in 1872, 1918, and 1946.

Subduction earthquakes are huge quakes that result when the boundary between the oceanic and continental plates ruptures. In 1700, the most recent Cascadia Subduction Zone earthquake sent a tsunami as far as Japan.

Mount Saint Helens / Other Cascade Volcanoes

Source: University of Washington Pacific Northwest Seismic Network

Figure 22
Seismogenic Sources

File: J:\21\09759-008\G&S Discipline Report (1-05)\21-1-09759-008 Seismic Figures.dwg Date: 02-08-2005 Author: SAC

Since the 1940s, earthquakes have generally been reported using magnitude scales. Earthquake magnitudes may correspond to several different scales including surface waves (M_s), body waves (m_b), and "Richter" or local magnitude (M_L). The preferred measure is the moment magnitude (M_W), which is a measure of the total energy (seismic moment) released by an earthquake. Unless otherwise noted in this report, use of moment magnitude is implied. All earthquake magnitude scales use Arabic numerals to represent the size of the event.

The largest historic earthquakes to affect the site include 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. All three events were located in the subducted Juan de Fuca slab beneath the Puget Sound Lowland at depths of 53, 63, and 52 kilometers, respectively. The 1949 and 2001 events occurred in the subducted Juan de Fuca slab at nearly the same location. The level of ground shaking that occurred during these three events are likely the maximum vibratory ground motions that would have occurred in project area during the 170 years of historical record. An event similar to these historical intraslab earthquakes but located closer to the site, could cause ground motions at the site with approximate characteristics of the 475-year design ground motion (i.e., ground motions with a 10 percent probability of exceedance in 50 years).

Other large historic earthquakes in the region include the 1872 North Cascades earthquake and two other events in western British Columbia, Canada. The North Cascades earthquake of December 15, 1872, appears to have been one of the largest crustal earthquakes in the Pacific Northwest, with a maximum reported intensity of IX. Although the epicentral location of this event is uncertain, owing to the sparse population of the area at that time, it apparently was a shallow crustal event located about 190 to 230 kilometers (epicentral distance) northeast of Seattle, in the general vicinity of the southeast end of Lake Chelan (near the eastern edge of the North Cascades subprovince). The estimated magnitude for this event ranges from 6.8 (Bakun et al., 2002) to 7.4 (Malone and Bor, 1979). In Canada, major crustal earthquakes occurred on Vancouver Island on June 23, 1946, and in the Queen Charlotte Islands on August 21, 1949 (Coffman and von Hake, 1973). These events had local magnitudes of 7.3 and 8.1, respectively. Because of the large distances of these earthquakes from the Puget Sound area (over 150 kilometers), there were no reports of significant ground shaking or damage in the area.

Until the 1990s, shallow crustal seismicity 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 during the 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 tectonic models (Wells et al., 1998) suggest that the geophysical lineaments/crustal block boundary beneath the Puget Sound Basin are potentially seismogenic and capable of producing shallow crustal events of magnitudes up to about 7.5.

Many of the recent studies regarding the potential for large shallow crustal earthquakes have focused on the Seattle Fault Zone. This zone is characterized as a 60 to 65 kilometers long (east-west) south-dipping reverse or thrust 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 into 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 23. 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 in the center of Puget Sound (Puget Sound Fault) that offsets the east-west trending strands of the Seattle Fault (Figure 23). While there is no paleoseismological evidence of rupture on this structure, based on the observed offset of the Seattle Fault, Johnson et al. (1999) indicate 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 buried at a depth of about 4 kilometers beneath the Seattle Basin. The approximate location of the buried wedge tip is shown on Figure 23. This location is north of the surface deformation zone and about 1 ½ to 2 km south of the site. 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 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).

A third seismogenic zone has been identified where the Juan de Fuca is subducted beneath the North American plate off the coast of the Pacific Northwest. The Cascadia Subduction Zone (CSZ), as it is called, has not been subject to any large earthquakes during historic times (170 years). However, multiple interplate earthquakes have occurred on the CSZ during the Holocene Epoch. Based on historical tsunami records in Japan (Satake et al., 1996), the most recent interplate event on the CSZ was a magnitude 9 event on January 26, 1700. Adams (1990) interpreted the occurrence of turbidites from failures of submarine canyon heads 50 km west of Willapa Bay (Griggs and Kulm, 1970), as the result of rupture on the CSZ. Adams interpreted the ages of the turbidites from the relatively uniform thicknesses of interbedded clay layers. The estimated ages of five distinct events, interpreted to be the result of rupture on the CSZ, were 250 to 360 years, 570 to 830

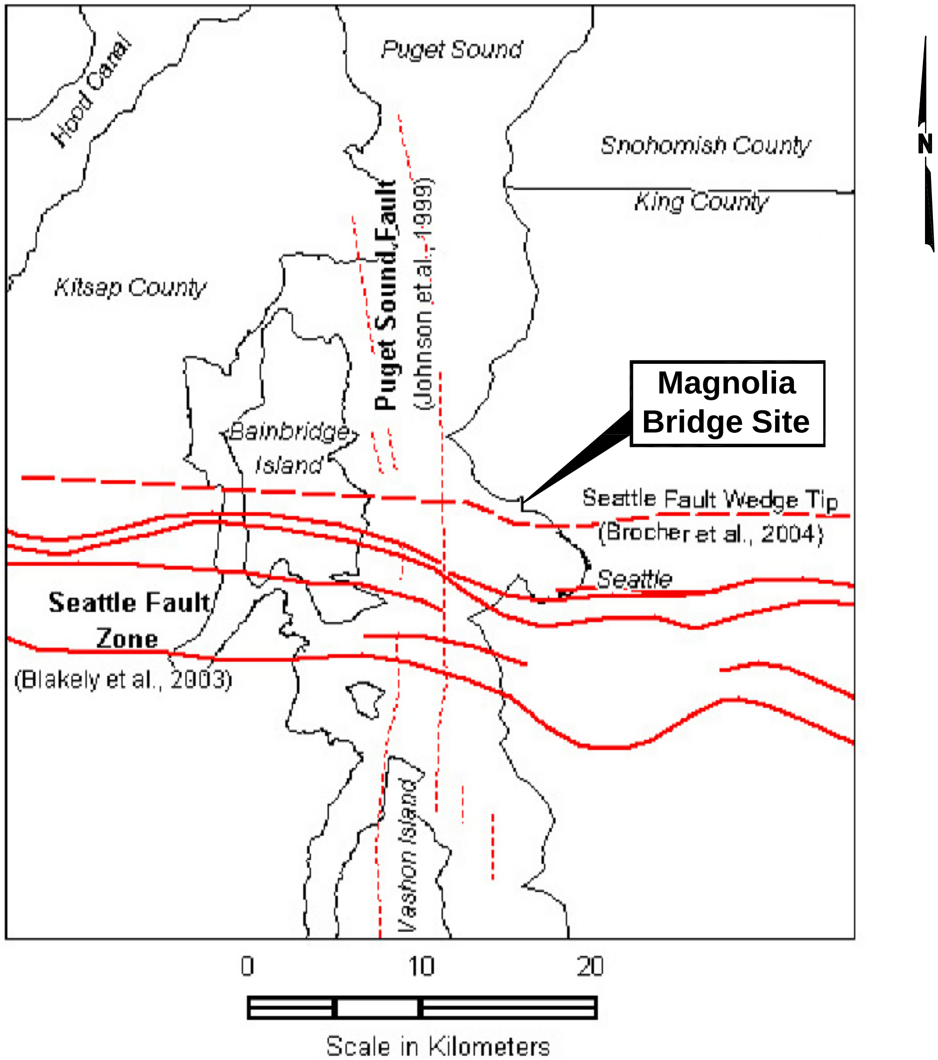


Figure 23
Seattle and Puget Sound Faults

File: J:\21\109759-008\G&S Discipline Report (1-05)\21-1-09759-008 Seismic Figures.dwg Date: 02-07-2005 Author: SAC years, 1,000 to 1,400 years, 1,730 to 2,640 years, and 2,270 to 3,300 years. Atwater and Hemphill-Haley (1997) also reported ranges of age for seven distinct seismic events based on buried soils in Willapa Bay. The estimated ages of these events were 290 to 310 years, 900 to 1,300 years, 1,110 to 1,350 years, 1,500 to 1,700 years, 2,390 to 2,780 years, 2,800 to 3,320 years, and 3,320 to 3,500 years.

While magnitudes, rupture lengths, and recurrence rates have not yet been well defined for subduction zone earthquakes on the CSZ, work to date suggests that earthquake magnitudes may range from 8 to 9. Based on data obtained from Frankel et al. (2002) this seismogenic source does not greatly contribute to the design ground motion in the central Puget Sound region.

Geologic Hazards and Critical Areas

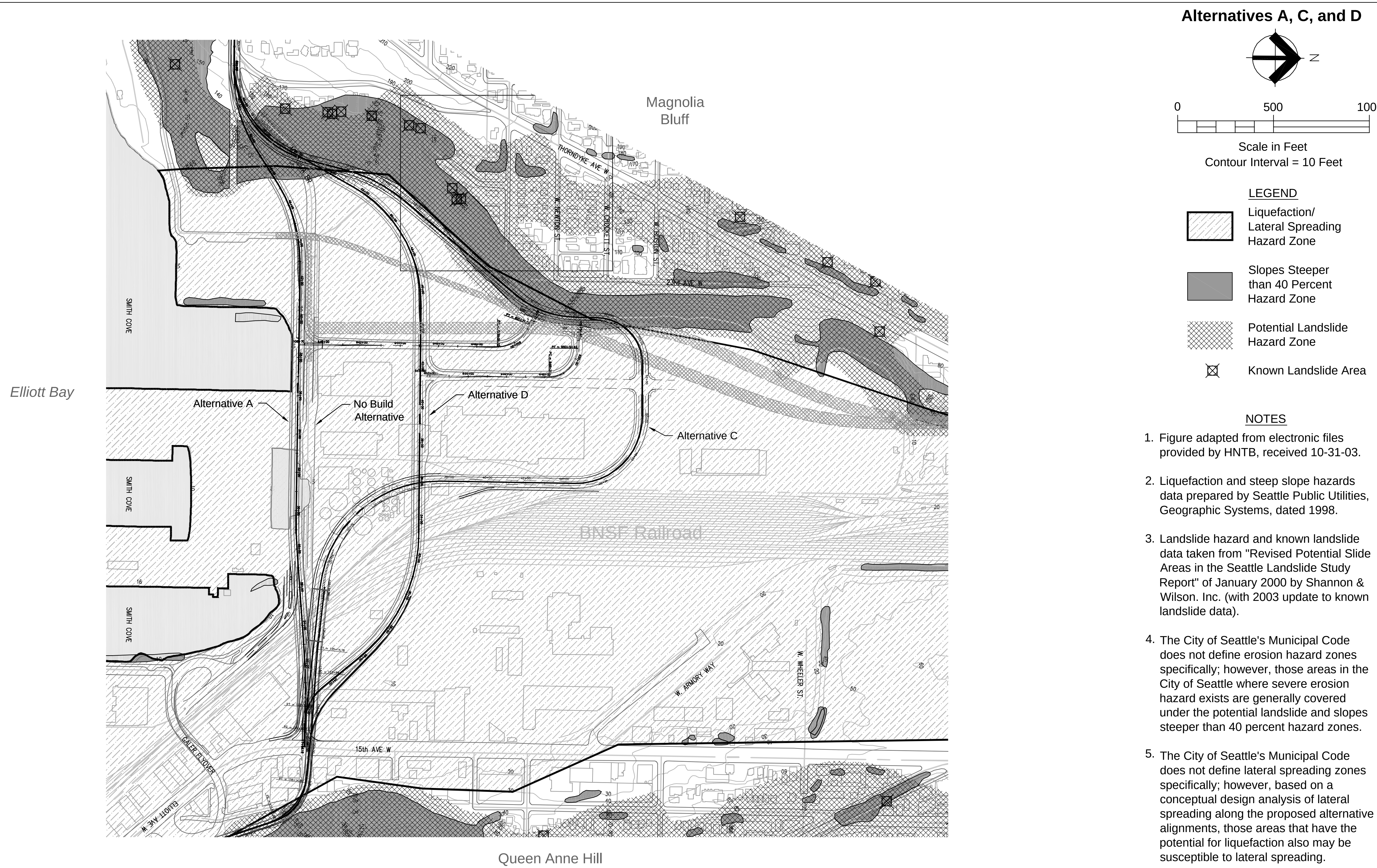
Earthquake-induced geologic hazards include landsliding, fault rupture, soft-soil ground amplification, tsunamis/seiches, and liquefaction and its associated effects (reduction of shear strength, loss of bearing capacity, decrease in lateral support, ground oscillation, slumping, settlement, and lateral spreading). The principal earthquake-induced geologic hazards along the three Magnolia Bridge Replacement alternative alignments include liquefaction and its associated affects, and to a much lesser extent, fault rupture.

In addition, the City of Seattle Critical Areas maps were reviewed. These map folios delineate sensitive areas based on several categories. The categories related to soils and geology include known landslides, potential landslide areas, steep slopes, liquefaction, and flood-prone areas. Based on this reference and experience with similar soils, the slopes of the hills to the east and west of the project area fall within erosion, landslide, and steep slope hazard areas. The hillside map folio information was combined with the field slope reconnaissance data. Based on this reference and the conceptual design analyses, the flat area between the eastern and western slopes fall into both the liquefaction and lateral spreading hazards areas. None of the alternatives fell within a flood-prone area according to the City map folios.

The following provides a brief discussion of the earthquake-induced hazards as well as critical areas. Figure 24 presents the approximate liquefaction, lateral spreading, landslide, and erosion hazards for the project area.

Strong Ground Motion

The earthquake design for the proposed bridge replacement would be in accordance with the Load and Resistance Factor Design (LRFD) Bridge Design Specifications as outlined by the American Association of State Highway and Transportation Officials (AASHTO), including the 2003 interim provisions. AASHTO criteria indicate that bridge design and evaluations should be based on earthquake ground motions with a 10 percent chance of exceedance in 50 years (475-year return period). The U.S. Geological Survey (USGS) National Seismic Hazard Mapping Project has completed regional probabilistic ground motion studies, and posted ground motion maps for the entire country (Frankel et al., 2002). Based on the USGS maps and a recurrence interval of 475 years, the site soft rock peak ground acceleration (PGA) is 0.33g. For the conceptual design phase, the site was classified as AASHTO Soil Profile Type III with a corresponding Site Coefficient (S) of 1.5. AASHTO describes a Soil Profile Type III as a soil profile with 30 feet or more of soft to medium stiff soils with or without intervening layers of sand or other



NOTES

1. Figure adapted from electronic files provided by HNTB, received 10-31-03.
2. Liquefaction and steep slope hazards data prepared by Seattle Public Utilities, Geographic Systems, dated 1998.
3. Landslide hazard and known landslide data taken from "Revised Potential Slide Areas in the Seattle Landslide Study Report" of January 2000 by Shannon & Wilson. Inc. (with 2003 update to known landslide data).
4. The City of Seattle's Municipal Code does not define erosion hazard zones specifically; however, those areas in the City of Seattle where severe erosion hazard exists are generally covered under the potential landslide and slopes steeper than 40 percent hazard zones.
5. The City of Seattle's Municipal Code does not define lateral spreading zones specifically; however, based on a conceptual design analysis of lateral spreading along the proposed alternative alignments, those areas that have the potential for liquefaction also may be susceptible to lateral spreading.

Figure 24 - Liquefaction, Lateral Spreading, Landslide and Erosion Hazards Map

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cohesionless soils. In some areas of the project site, the actual thickness of soft to medium stiff soils may be less than 30 feet, particularly at the east and west ends of the alternative alignments where the depth to very dense or hard soil decreases. However, the site does not correspond to Soil Profile Types I or II. Both of these soil types require that the subsurface soils be stable deposits of sand, gravel, or stiff clays. The relatively high susceptibility of the overlying soils to liquefaction indicates that these soils are not stable and not consistent with Soil Profile Types I or II.

Earthquake-induced Landsliding

Slopes that are susceptible to movement under static (non-earthquake) conditions also present a hazard under earthquake loading conditions. The slopes that present a landslide hazard under static conditions are outlined later in this section.

Fault Rupture

The three alternative alignments are located about 6 kilometers north of the surface deformation zone associated with the Seattle Fault Zone. The surface deformation zone is about 4 to 6 kilometers wide (north-south), consisting of a series of east-west-trending faults. It is postulated that the surface faults coalesce to a master Seattle Fault at depth, which is a south-dipping reverse fault. The sense of movement on secondary or antithetic faults within the fault zone may be opposite (that is, north side up, south side down). Geologic evidence suggests that the most recent earthquake to rupture the ground surface in the fault zone occurred about 1,100 years ago with nearly 22 feet of permanent vertical displacement across the northernmost fault in the zone (Blakely and others, 2002; Johnson and others, 1999; and ten Brinck and others, 2002). Future ground rupture within the zone may or may not occur along the existing mapped faults.

While the site is located relatively near the Seattle Fault Zone, the actual risk posed by ground rupture is relatively small. The return period for large earthquakes on the fault that may rupture the ground surface is on the order of thousands of years, and that is much longer than the 475-year return period ground motions being used in the design of the Magnolia Bridge replacement.

Liquefaction

Soil liquefaction is a phenomenon in which pore pressures in loose, saturated, granular soils increase to a level approximately equal to the effective stress during ground shaking; this results in a reduction of shear strength of the soil (a quicksand-like condition). The effects of liquefaction may include loss of bearing capacity for shallow foundations, reduction in lateral and vertical deep foundation capacities, ground surface settlement, downdrag forces on deep foundations, lateral spreading, and embankment instability or slumping. The three alternative alignments for the Magnolia Bridge Replacement cross recent fill and soft and loose Holocene deposits that are susceptible to liquefaction and its associated affects.

A conceptual design-level liquefaction potential analysis was performed using Seed's simplified method (Youd et al., 2001) and a soil ground motion that corresponds to a 475-year return period ground motion. Based on available subsurface data, potentially liquefiable and soil strength reduced deposits could extend to approximate depths of 95 feet for Alternative A and 100 feet for Alternative D. Due to a lack of subsurface information at the northern portion of

Alternative C, the depth of potential liquefaction is unknown; however, it would likely be similar to or slightly less than Alternatives A and D. The depth of potentially liquefiable soil decreases at the western and eastern ends of the alignments as the depth to dense/hard and/or glacially overridden soil decreases. The results of the conceptual design liquefaction potential analyses were compared to the City of Seattle Critical Areas liquefaction map. Potentially liquefiable areas within the study area, as mapped by the City of Seattle (Seattle Public Utilities, 1998), are shown on Figure 24.

Lateral Spreading

One of the major liquefaction-induced types of ground failure is lateral spreading. Lateral spreading movement of gently sloping ground occurs as a result of pore-pressure build-up or liquefaction in the underlying soil deposit. A lateral spread often contains a liquefied layer overlain by a non-liquefied layer at the ground surface that rides along the top of the liquefied soil during ground movement. The non-liquefied crust either is often present because it lies above the groundwater table or because the layer is too fine-grained to liquefy. Large forces could be generated as this non-liquefied layer is carried along on the lateral spreading ground and driven against fixed foundations. Lateral spreading would not occur if the free-face were stabilized by a suitable structure designed to resist lateral loads induced by the liquefiable soils or appropriate ground improvement measures are performed to increase the density of the soils.

Permanent lateral ground displacements along the alignment were estimated using the empirical procedure by Youd, Hansen, and Bartlett (2002). The magnitude and distance assumed for the preliminary analysis was consistent with the design ground motion. These analyses were completed for the conceptual design and assumed that there was no existing suitable seawall along Piers 90 and 91 to resist the lateral spread. Lateral spreading displacement for the design level earthquake is estimated to be on the order of about 10 feet for any portions of the alignments not protected by a suitable seawall structure. Based on the available subsurface data and the analytical tool used to approximate lateral spreading, the lateral spreading is estimated to be roughly the same for Alternatives A, D, and a majority of C. To date, case histories document lateral spreading occurring to a distance of about 1,200 feet from the free-face. Portions of Alternative C are greater than 1,200 feet from the free face. Displacements along these portions would be less. The lateral spread displacements would generally be in a southerly direction (towards Smith Cove). The estimated lateral spreading displacement for Alternative C is an extrapolation of the case histories used to develop the Youd, Hansen, and Bartlett (2002) equations. The City of Seattle does not specifically map areas of potential lateral spreading. However, based on the conceptual design analysis, the areas mapped as being potentially liquefiable are also areas of potential lateral spreading. These areas are shown on Figure 24.

Soft-Soil Ground Motion Amplification

The type of near-surface soils could affect the level of earthquake ground shaking felt in an area. Amplification of the ground motion at various frequencies may occur for areas underlain by thick (for example, 30 feet or more) deposits of relatively soft, cohesive soils. The Holocene geologic units encountered along the proposed alternative alignments are thick enough to result in ground motion amplification. Consequently, some soft-soil ground motion amplification is expected in the project area. The effects of soft-soil ground motion amplification are an intrinsic property in the analysis of liquefaction and lateral spreading; therefore, the associated effects are described under liquefaction and lateral spreading.

Tsunamis/Seiches

Tsunamis and seiches are earthquake-generated waves developed in a body of water. A tsunami wave could be generated by permanent ground displacements in a basin that contains a water body. Seiches are standing or oscillating waves developed in a closed body of water as a result of earthquake shaking and could be generated by distant earthquakes; Smith Cove is not a closed body of water and would therefore not experience a seiche.

Depending on the height of the tsunami wave produced and the elevation of the subject site, these water waves could pose a significant hazard. However, based on the Magnolia Bridge design-level earthquake return period (475 years) as compared to the recurrence rate for large earthquakes on the Seattle Fault (thousands of years) or the Cascadia Subduction Zone, the hazard posed by tsunamis in the study area is low.

Landsliding and Erosion

The City of Seattle presently regulates public and private development in environmentally critical areas by requiring special standards for design and construction in potential slide, known slide, and steep slope areas. Potential and known slide areas are defined by historical landslides and by a zone encircling many of the hills and ridges based on the sand/clay contact as shown in Tubbs' *Landslides in Seattle*, 1974. Steep slopes are defined as slopes steeper than 40 percent, with a rise exceeding 10 vertical feet. Other restricted slope areas defined by the City of Seattle include:

- All Class 3 zones of Tubbs' (1974) report, areas steeper than 15 percent slope gradient and underlain by the Vashon glaciolacustrine or pre-Vashon sediments.
- Areas with springs or groundwater seepage; however, this criterion is not shown on maps.

As a part of the Seattle Landslide Study (Shannon & Wilson, Inc., 2000), the Potential Slide Areas were re-mapped by consulting geologists using additional criteria to better define those areas in the City with the potential for impacts from slope instability. The refined criteria for the revised Potential Slide Areas include:

- The presence of historic landslide activity
- Runout zones at the toes of hillsides
- Instability not related to the Vashon glaciolacustrine clay or pre-Vashon sediments
- Geologic conditions unknown at the time of Tubbs' (1974) work

The revised Potential Slide Areas are shown on the Liquefaction, Lateral Spreading, Landslide and Erosion Hazards Map, Figure 24. Figure 24 also presents steep slope areas and known landslide areas.

Seattle's Municipal Code does not define erosion hazards or erosion hazard zones. Nor does the Soil Survey of King County (Snyder and others, 1973) include the City of Seattle on its maps. Therefore, in order to approximate the areas that may be susceptible to erosion when disturbed by construction, the geologic units in this study area were matched with the approximate soil units from the King County Soil Survey. This was accomplished based on the descriptions provided in the Soil Survey and local experience and knowledge of the geologic units. Note 4 on Figure 24 addresses erosion hazard zones.

Soil units are considered to be erosion hazards if they are considered to be "severe" or "very severe" in Table 6 (Woodland Groups, Wood Crops and Factors in Management) of the Soil Survey. Table 2 below presents the soil units, their geologic unit equivalents, and the level of erosion hazard.

Table 2
Erosion Hazard Units

Soil Type	Geologic Unit	Erosion Hazard
Alderwood on Slopes >15%	Qvt	Severe to very severe
Everett on slopes >15%	Qva	Moderate to severe
Kitsap on slopes >15%	Qvgl, Qpgl, Qpnl	Severe

For alternative specific locations of these units, refer to the preliminary geologic maps, Figures 15, 16, and 17 for Alternatives A, C, and D, respectively. Fill materials (Hf), colluvium, and landslide debris (Hls), by their nature of being widely variable, mostly containing large percentages of fine-grained soil particles, and being poorly compacted, should be considered severe to very severe erosion hazards on slopes exceeding 15 percent.

Alderwood soil (Qvt) is not a severe erosion hazard in its native, undisturbed condition due to its very compact condition, but is susceptible when it is unvegetated and/or disturbed.

In addition to the potential landslide and known landslide features mapped on Figure 24, a field slope reconnaissance was performed along the relatively steep slope adjacent to the west approach for the Alternatives A, C, and D, near the location of the existing Magnolia Bridge west approach. Mr. William D. Nashem, a geologist with Shannon & Wilson, Inc., performed the field reconnaissance in March 2003 and July 2004. Limited soil exposures, evidence of past landslides, locations of springs, and vegetational clues to geologic conditions were noted during the reconnaissance and are shown on Figures 15, 16, and 17. The slope descriptions were based on field observations and currently available subsurface information presented in Appendix A. The discussions and conclusions below should be considered conceptual.

Alternatives A, C, and D are located in the southern portion of the Interbay embayment and generally extend alongside (Alternative A), approximately 570 feet north (Alternative D), and about 1,800 feet north (Alternative C) of the existing Magnolia Bridge between 15th Avenue West and West Galer Street (on Magnolia Bluff).

West Approach Reconnaissance

At the west abutments, the alternatives generally coincide with the existing bridge alignment. A geologic reconnaissance was performed along the relatively steep, east-facing slope between the West Galer Street right-of-way (ROW) and the West Boston Street ROW for the west approaches to the alternatives (Figures 15 through 17). Several signs of slope instability were observed along the approximately 150-foot-high slope. Although limited soil exposures exist along the slope, the upper portions of the slope are characterized by silty, gravelly sand to silty, sandy gravel (Vashon Till). Based on previous work and existing explorations, the basal contact between the Vashon Till and the underlying glaciolacustrine silt, clay and fine sand exists at approximate elevation 150 feet (NAVD88). The top of the slope in this area is at about elevation 170 feet.

Most of the instability and the seepage observed during the reconnaissance existed at, or below the Vashon Till-glaciolacustrine contact. Below approximate elevation 90 feet, significant thicknesses of colluvium cover the slope and generally thicken toward the slope toe. Springs were observed in several landslide scars located midslope. Based on existing subsurface explorations and previous work, the interbedded fine sand, silt and clay underlying the slope in this area may provide thin, discrete seepage paths within the glaciolacustrine soils. A 2-foot-high ecology block toe wall exists at the toe of the slope, behind the Northwest Harvest facility. While abundant hydrophitic vegetation grows along the slope toe throughout the subject area, limited seepage was observed during the site visit.

Along the northern portions of the east-facing slope (Alternative C), several older landslide scars were observed during field reconnaissance. Several concrete crib walls exist along the toe of the slope – some with colluvial/landslide debris accumulations over the top of the walls. In the vicinity of W. Newton Street, a 6-inch-diameter and a 12-inch-diameter steel pipe were observed in a small eroded ravine, which appears to be caused by erosion of the utility trench backfill.

Along the southern portion of the slope, nearest to the existing Magnolia Bridge, the slope is characterized by the substantial slope modifications made in 1997 in conjunction with the Magnolia Bridge slide repair project (please refer to the report entitled, “Geotechnical Report, Magnolia Bridge Slide Repair, Seattle, Washington,” by Shannon & Wilson, dated October 1997). While the location of the 1997 landslide headscarp is shown on Figures 15 through 17, no evidence of current instability was observed in this area and an existing 250-foot-long, tieback, soldier pile and concrete lagging wall retains the slope east of house numbers 1500 through 1534 on Magnolia Way West.

East Approach Reconnaissance

The slope east of the east approaches of Alternatives A, C, and D was not evaluated. This area, on the western slope of Queen Anne Hill, has a history of landsliding; however, the City of Seattle has completed major repairs in recent years such as the West Garfield Street Slide Repair. Because the alignments of Alternatives A, C, and D generally follow the existing east approach, the hillside in this area has recently been regraded and stabilized, and the area does not appear to have a landslide hazard, a field slope reconnaissance was not completed in that area.

Studies and Coordination

Studies

Geologic data were obtained for the three proposed build alternatives by collecting and reviewing existing data, performing a geologic slope reconnaissance, and drilling six soil borings. The geologic evaluation of the build alternatives was performed based on this data. Preliminary evaluations were made related to foundation axial capacities, liquefaction, lateral spreading, slope stability, and other geologic issues. The evaluations were made based on experience with similar projects and similar soil conditions, and preliminary engineering analyses. Mitigation measures were developed from work with similar project/soil conditions.

Data Sources

Existing Foundations

As a part of this study, available information was collected regarding existing building foundations along the alternatives. Structures within approximately 200 feet of the proposed alignments were included in the data collection. Table 1 presents the available existing building foundation information. The building identification numbers on the table correspond to the identification numbers on Figures 11, 12, and 13 for Alternatives A, C, and D, respectively. The table also includes the site or business name and type of structure, address, parcel number, additional pertinent information, and the source of the data. Data were collected from the City of Seattle Department of Planning and Development (DPD) files and parcel records, King County website and Port of Seattle archive records, and tax assessor records.

The 1929 construction drawings for the existing bridge, which was originally called the West Garfield Street Viaduct, were reviewed. The 1929 plans show pile foundations supporting most of the bridge, but do not clearly indicate the pile type, size, or length. For the purposes of this study, it is assumed, based on the age of the bridge and the soil conditions, that the pile foundations are timber piles. Based on pile driving records, the pile lengths range from about 15 to 55 feet, with the majority between 30 and 55 feet. Bridge piers at the western end on the slope are founded on footings. The 1929 construction plans were copied from the City of Seattle files; the pile driving records were provided by HNTB.

Existing Subsurface Data

Project files and archives from several sources were reviewed to obtain existing geotechnical subsurface information along the three proposed build alternatives. These efforts were concentrated on sources where large amounts of information were already stored and easily accessed. Data, primarily consisting of borings logs but also including probes and hand borings, were collected from the following sources:

- Shannon & Wilson, Inc.
- City of Seattle

- City of Seattle DPD
- Seattle-area Geologic Mapping project office
- Port of Seattle

The stored files from each source listed above were reviewed, and selected exploration logs were copied. At some of these locations, the data reviewed were of poor quality and therefore were not used in the geological studies. Only data that contained sufficient information to locate the explorations and to evaluate the subsurface geology were selected. The approximate locations of the existing explorations are shown on Figures 11, 12, and 13 for Alternatives A, C, and D, respectively. The locations of the previous explorations were estimated from available plans and should be considered approximate. The approximate elevations of the previous explorations were determined in three ways: (1) the elevations were estimated based on the current site topography and their approximate location, (2) the elevations given on the logs were assumed to be in terms of the 1988 North American Vertical Datum (NAVD88) based on their date and their correlation with the current topography, and (3) the elevations given on the logs were given in terms of other data and were then converted to the NAVD88 datum. The exploration logs and additional information regarding each exploration are included in Appendix A.

Field Explorations

An initial field exploration program was performed for the conceptual design phase of the project to supplement the existing subsurface information and to obtain more specific data in the locations of the proposed bridge structures. The field exploration program included drilling six borings, three each along Alternative Alignments D and H (now deleted). Monitoring wells were not installed. Existing subsurface information was used to evaluate Alternatives A and C; some of the Alternative D borings were applicable to both of those alignments.

In general, the explorations were located in areas where bridge structures are proposed and where geologic conditions were not documented. The locations of the recent field explorations are shown on Figures 12 and 13 for Alternatives C and D, respectively. The boring locations were not surveyed, but were measured from existing features and plotted on the site topographic map provided by HNTB. After plotting the approximate locations of the borings, the boring elevations, in terms of NAVD88, were estimated. The boring logs are presented in Appendix A.

Geologic Literature Review

In addition to the field geologic reconnaissance of the western approach slopes of Alternatives A, C, and D, available published geologic literature was reviewed for the proposed alternatives. These data included the following:

- Geographic Systems electronic map layers for liquefaction and slopes greater than 40 percent provided by Seattle Public Utilities (1998)
- Geographic Systems electronic map layer for potential landslide areas included as a part of the Revised Landslide Study Report (Shannon & Wilson, Inc., 2000)
- National Resource Conservation Service Soil Survey for King County (Snyder and others, 1973)

- Department of Ecology's Coastal Zone Atlas of Washington for King County (1979)
- United States Geological Survey Shilshole Bay 7.5-minute Quadrangle topographic map (USGS, 1949)
- Preliminary Geologic Map of Seattle and Vicinity (Waldron and others, 1962)
- Topographic and Bathymetric Map of Seattle Harbor (1879)
- Magnolia Park Map (Don Sherwood Parks History Collection, 1907)
- Topographic Maps of Seattle, South (Engineering Department, 1958)
- Geotechnical report for the Magnolia Bridge Slide Repair (Shannon & Wilson, Inc., 1997)
- Geologic Map of Seattle NE 7.5-minute Quadrangle (Troost and Booth, in preparation)
- Landslides in Seattle (Tubbs, 1974)
- Causes, Mechanisms and Prediction of Landsliding in Seattle (Tubbs, 1975)

Geologic Reconnaissance

A field geologic reconnaissance of the western approach slopes of Alternatives A, C, and D was performed in March 2003 and July 2004. Geologic features such as soil exposures, cut and fill slopes, evidence of past landslides, locations of springs, and vegetational clues to geologic conditions were noted by the Shannon & Wilson representative walking the slopes. Field reconnaissance information, pertinent geologic features observed, and preliminary geology based on subsurface data and the geologic literature review are shown on Figures 15, 16, and 17 for Alternatives A, C, and D, respectively.

Major Assumptions

This Geology and Soils Discipline Report is based on the assumption that the subsurface and surficial soil conditions encountered in recent and previous soil explorations, observed during the 2003 geologic slope reconnaissance, and presented in the geologic literature listed above, represent the actual conditions at and near the proposed alternative alignments. In the conceptual design-level analysis of potential lateral spreading, it was assumed that there was no existing suitable seawall along Piers 90 and 91 to resist a lateral spread. For the No Build Alternative, it is assumed, based on available pile driving records and the subsurface conditions, that the existing pile foundations were driven approximately 15 to 55 feet below ground surface.

Impacts

Impacts created by soil and geology issues would be related to the effect of new structures on the existing features in the study area. Three types of structures are anticipated in the proposed build alternatives: elevated structures, fill embankments, and cut walls (limited cuts). In addition, at-grade roadways would be constructed. In general, the impacts of Alternatives A, C, and D are comparable because their proposed design and layout are similar and they are located relatively close together within the study area. Alternatives A, C, and D have similar subsurface soil, groundwater, and geologic conditions. Groundwater impacts are discussed in the Water Quality Discipline Report.

The proposed build alternatives would be designed based on the available subsurface information, additional field explorations completed for final design, existing site conditions, and design and construction procedures and criteria approved for this project. If subsurface conditions at the site are different from those disclosed during the previously completed conceptual design field explorations, or site conditions change during the design and construction period of the project, future impacts to the site could occur.

Many of the impacts described in the following sections could be addressed by following established AASHTO criteria for proper design and/or standard construction practice. The following paragraphs state if the impact that is described could be addressed by proper design and/or standard construction practice. Only those impacts that would use nonstandard construction procedures are included in the subsequent mitigation section of this report.

No Build Alternative

Cuts Into Existing Slopes

No cuts into existing slopes are proposed under the No Build Alternative; however, slope instabilities may occur upslope of the eastern end of the No Build Alternative. The uphill slope instabilities may cause damage to the bridge or deposit debris onto the roadway.

Fills

No fills are proposed under the No Build Alternative.

Seismic Considerations

During the design life of the No Build Alternative, design-level earthquakes could occur. If the design ground motion or some threshold ground motion were to occur, there would be a potential for liquefaction, lateral spreading, and slope instability. Liquefaction alone could cause excessive settlement (due to downdrag around the pile perimeters and loss of bearing capacity at the pile tips). Should significant lateral spreading occur, the lateral deflection of the existing bridge foundations would likely cause bridge collapse. In addition, if the Magnolia Bluff and Queen Anne Hill slopes experience earthquake-induced instability, the slope movements could cause damage to the existing bridge foundations and deposit debris onto the existing access ramp.

Elevated Structure Foundations

The existing bridge is supported by deep foundations. The foundations have not been designed to current AASHTO seismic design criteria or to account for potential liquefaction and potential lateral spreading under current design earthquake ground shaking. These seismic considerations are described above.

Relationship Between Topography and Alignment Design

No cuts or fills are proposed for the No Build Alternative.

Settlement Potential

Seismically induced settlements may occur as described above. Significant settlements are not anticipated under existing loading conditions.

Alternative A

Cuts Into Existing Slopes

When material is removed from the toe of a slope, the overall stability of a slope generally decreases. Any unretained cuts into existing slopes may experience erosion and surface sloughing over the lifetime of the project. The degree of erosion would depend on near-surface soils, weather conditions, establishment of vegetation, surface drainage, and other causes. In addition, in areas where retaining walls are proposed, the slope stability of the existing hillside may be adversely affected if the walls are not properly constructed. Surface slumps or landslides occurring in the future may result in the deposit of material onto the surface streets and ramps and may damage the proposed bridge structure. Evidence of previous instabilities has been observed along the Magnolia Bluff and Queen Anne Hill slopes. For Alternative A, cuts are anticipated to be less than 3 feet high for roadways. Design and standard construction procedures could address impacts from cuts into existing slopes.

The design approach for the proposed cuts into existing slopes should include performing proper design of the walls or slopes, defining the location and extent of unstable soils, and using proper construction procedures. To address slope instability in cut areas, retaining wall design could retain the soils in the cut and any potential landslide forces. Based on the soil types present at the site, if roadway walls are used, they would likely consist of gravity retaining walls or concrete walls. The base of the wall would extend a sufficient depth into undisturbed soils so that adequate passive resistance in front of the wall is generated to resist the lateral earth pressures behind the wall.

In areas where slope instability has been observed, the extent of the landslide deposits would be determined so the proposed retaining walls could be designed to retain the unstable soil. For debris flow and debris avalanche material that may come from above the walls (and whose source may be outside of the proposed action area), catchment walls could be constructed. These catchment walls would extend above the top of the retaining walls and serve as temporary retention measures for soil and debris (such as shrubs and trees) that may slide down the slopes from landslides occurring above and outside of the proposed action area. If a slide occurs, the soils that are retained by the catchment walls should be removed after the event.

Fills

Fill approaches are proposed at the Alternative A intersection on Port of Seattle property. Mechanically-stabilized earth walls (MSE walls) are being considered to retain the approach fills. These walls would be a maximum of about 13 feet tall. The upper Holocene deposits of fill, estuarine, and beach materials are of varying densities and consistencies. The looser and softer materials could experience significant settlement due to the proposed fill approaches. Settlements on the order of 5 to 10 inches could be anticipated where the proposed fill approach is about 13 feet high. One inch of settlement may occur roughly 20 feet from the MSE wall toe, and ½ inch of settlement may occur roughly 40 feet away from the wall toe. For shorter fill heights, less settlement would be anticipated. This settlement would occur primarily in the first three to six months after approach fill construction.

Existing utilities that are located within proposed fill areas would be subjected to loading and settlement due to the overlying fill. Settlement and some lateral loading may also extend out from the toe of the new MSE walls, resulting in potential settlement or lateral loading of adjacent facilities such as existing roadways, railways, buildings, ramps, and utilities. Excessive lateral or vertical loading and movement could then result in damage to those facilities.

Where fills are near the proposed bridge structure, the settlement could cause downdrag and lateral loading on buried, deep foundations. Downdrag occurs when the soil moves downward along the buried perimeter of a deep foundation member or other buried foundation, and, through friction along the sides, increases the compressive load. The proposed foundations could be designed to overcome these impacts. This would be a concern for existing facilities with pile foundations. As a part of this study, information regarding existing structure foundations along the alternatives was collected. Structures within approximately 200 feet of the proposed alignments were included in the data collection; Table 1 presents the available existing building foundation information. The building identification numbers on the table correspond to the identification numbers on Figures 11, 12, and 13.

The presence of soft soils beneath the proposed fill approaches would also result in lateral movement as the subsurface soil compresses under the weight of the new fill. Lateral movement near the toe of the proposed fill could be as much as one half of the estimated settlement. Existing adjacent utilities or structures could be subjected to lateral loading due to this movement.

In some areas, the existing, soft, subgrade soils may not have sufficient strength to allow for a stable fill approach, especially during the short-term construction period. Rotational and bearing capacity failures through the surficial soils and the approach fill could occur. Over time, the stability of the approach fill would improve as the soils beneath the embankment consolidate and gain strength. Proper design and standard construction procedures would address this impact. This impact is discussed further in the Construction Impacts section, because stability during construction would likely be the most critical case.

Instability during earthquake loading may also result in fill approach failure. This type of failure would cause potential damage to structures or pavements located on or near the approaches.

The design for fill approaches must consider the estimated settlements, lateral movements and stability issues related to the presence of soft/loose, near-surface soils at the site. Because settlements may be on the order of several inches near the highest portions of the proposed fill approaches, the fills would be designed and constructed to consider this settlement and related impacts. Design and construction measures that address settlement include the following:

- Preload the site in areas where site availability and time schedules allow.
- For retained fills, use walls that could accommodate large settlements such as mechanically-stabilized earth (MSE) walls.
- Sequence construction so that impacted settlement-sensitive structures are installed after most of the fill settlement has occurred.
- Perform ground improvement where existing structures need to be protected from settlement.
- Relocate existing utilities that are beneath or nearby proposed fills if the proposed loads and settlements would cause damage to the utilities.
- Use lightweight fill materials where settlements must be minimized and alternative measures are not feasible.
- Utilize geosynthetics (such as geogrids or geotextiles) below and within the fill to help stabilize and reinforce the approaches.

Design approaches for lateral movement due to fill approach placement are the same as those presented above for settlement. As settlement is reduced, lateral movement would be reduced correspondingly.

Existing piles and proposed deep foundations or other buried structures would be evaluated for potential downdrag loads caused by settlement of adjacent new fill approaches. The new deep bridge and ramp foundations would be designed to accommodate the additional compressive loads caused by downdrag. Alternatively, construction sequencing could be performed so that the foundations are installed after most of the settlement due to the fill approaches has occurred. Another potential approach would consist of using permanent casing around the proposed deep foundations in the upper soils to reduce the negative skin friction on the foundation.

For existing bridge and ramp deep foundations, if estimated downdrag loads cannot be accommodated, lightweight fill could be used to reduce the settlement and corresponding downdrag. Alternatively, ground improvement could be performed. If the downdrag loads cannot be accommodated by these other methods, additional foundation elements could be installed to support the increased compression loads.

Generally, short-term (during construction) stability is the most critical for new fills over soft soil. Staged construction could be considered to improve the stability of the embankments during construction. This is discussed further in the Construction Impacts section. In general, the stability of the fill approach would improve with time as the soils beneath the fill embankments consolidate and gain strength. Preloading of the site could be considered to obtain this strength prior to construction of embankments. If additional slope stability is necessary (such as stability under earthquake loading), ground improvement could be performed to improve the soils beneath and adjacent to the embankments. Alternatively, geotextiles could be used within the fill materials to provide additional strength and resistance to failures.

Seismic Considerations

Alternative A crosses recent fill and Holocene deposits that are susceptible to liquefaction and its associated effects. The effects of liquefaction may include loss of bearing capacity for existing shallow foundations, reduction in lateral and vertical capacities of deep foundations, ground surface settlements, lateral spreading, slope instability or slumping, and fill approach instability. During lateral spreading, the proposed deep foundations would likely be subjected to large passive forces applied by the approximate 5- to 10-foot-thick layer of non-liquefied crust riding on top of the liquefied soil. Case histories have shown that these passive forces could cause excessive permanent deformation and rotation of the piles/shafts (or pile/shaft cap) by a relatively shallow non-liquefied soil layer (e.g., Berrill and Yasuda, 2002; Berrill et al., 2001; Hamada, 1992). In addition, cut slopes may experience surface sloughing or raveling that could deposit material onto the ramps and surface streets. Design could address seismic impacts. The Mitigation Measures section describes some of the construction procedures that could be used.

Pavements

Poor subgrade preparation for proposed pavements could lead to settlement, potholes, cracks, and other roadway distress. In addition, if the design pavement section is inadequate, these types of distress could also occur. Frost heave may occur in some areas as well, depending on the weather over the life of the project. Design and standard construction procedures could overcome these impacts.

Pavement design would include proper subgrade preparation and pavement cross sections. The design should be completed in accordance with City of Seattle Standard Specifications for Road, Bridge, and Municipal Construction (2003). All pavement areas should be proof-rolled with a heavy vibratory roller prior to placement of the pavement section. Soft areas would be identified by this process and should be removed and replaced with structural fill. Alternatively, the subgrade could be reinforced with geosynthetics prior to placing pavement subbase materials. In fill areas, mitigation measures as previously discussed for fill approaches should be performed. The upper part of the fill approaches should be well compacted to provide good bearing for the pavement. The pavement section should also be designed to prevent frost heave by providing an appropriate thickness for the climate conditions anticipated along the proposed alignment. The pavement section could also be designed to accommodate poor subgrade soils.

Elevated Structure Foundations

Because of the depth of loose, soft, and potentially liquefiable soil as well as the anticipated bridge loads, the elevated structures would be supported by deep foundations bearing in underlying competent soil. The deep foundation design would take into account the current AASHTO seismic design criteria and the potential for liquefaction and lateral spreading. It would also account for downdrag and lateral loading due to fill approach settlement. Therefore, because the design would account for site subsurface conditions, no soils- or geology-related direct impacts are anticipated for the proposed bridge foundations.

Relationship Between Topography and Alignment Design

The cut and fill discussions above address impacts from cuts and fills. Cut volumes are anticipated to be minimal for Alternative A, while approach and ramp fill volumes may be on the order of 40,000 cubic yards.

Settlement Potential

The cut, fill, and pavement discussions above address settlement potential as an operational impact.

Alternative C

Cuts into Existing Slopes

See the discussion under Alternative A; Alternative C cuts are also anticipated to be minimal.

Fills

See discussion under Alternative A. Alternative C fill embankments are anticipated to be a maximum of about 20 feet high. Settlements on the order of 10 to 15 inches could be anticipated where the fill height reaches 20 feet. One inch and ½ inch of settlement may occur roughly 40 and 80 feet away from the MSE wall toe, respectively.

Seismic Considerations

See the discussion under Alternative A.

Pavements

See the discussion under Alternative A.

Elevated Structure Foundations

See the discussion under Alternative A.

Relationship between Topography and Alignment Design

The cut-and-fill discussions above and under Alternative A address impacts from cuts and fills. Cut volumes are anticipated to be minimal for Alternative C, while approach and ramp fill volumes may be on the order of 25,000 cubic yards.

Settlement Potential

See the discussion under Alternative A.

Alternative D

Cuts Into Existing Slopes

See the discussion under Alternative A; Alternative D cuts are also anticipated to be minimal.

Fills

See the discussion under Alternative A. Alternative D fill embankments are anticipated to be a maximum of about 26 feet high. Settlements on the order of 15 to 20 inches could be anticipated where the fill height reaches 26 feet. One inch and ½ inch of settlement may occur roughly 50 and 90 feet away from the MSE wall toe, respectively.

Seismic Considerations

See the discussion under Alternative A.

Pavements

See the discussion under Alternative A.

Elevated Structure Foundations

See the discussion under Alternative A.

Relationship Between Topography and Alignment Design

The cut-and-fill discussions above and under Alternative A address impacts from cuts and fills. Cut volumes are anticipated to be minimal for Alternative D, while approach and ramp fill volumes may be on the order of 40,000 cubic yards.

Settlement Potential

See the discussion under Alternative A.

All impacts presented previously could be mitigated, as presented in the following sections. Adequate geotechnical exploration and design studies could be used to plan and design appropriate mitigation of many of the impacts discussed in the previous section. Soil borings and test pits should be performed at appropriate intervals along the proposed alignment in accordance with accepted engineering practices to provide adequate subsurface information for design studies. In addition, explorations should be performed in the following areas:

- Cuts or fills higher than 5 feet
- Fills over soft soils
- Each bridge pier location
- Cuts and fills in areas where slope stability may be an issue

The soil and geology-related impacts listed previously would be evaluated by an experienced geotechnical engineer who would then provide design recommendations considering the subsurface conditions encountered in the field explorations. These design recommendations would take into account the proposed features included in the project and would provide for adequate mitigation for these impacts unless otherwise directed by the City of Seattle. An evaluation of the seismicity of the site should be performed, and the affects of the design seismic event on the proposed cuts, fills, and structures should be considered.

Only those impacts that would use nonstandard construction procedures to mitigate are included in this section. Although nonstandard, these construction procedures are not uncommon given current seismic design criteria and earthquake engineering technology. Impacts that could be addressed by design and standard construction procedures are described in the Impacts section. Alternatives A, C, and D have similar subsurface soil, groundwater, and geologic conditions; therefore, the mitigation measures for these three alternatives are similar.

No Build Alternative

Because the No Build Alternative is offered as a base for comparison, no mitigation measures would be considered.

Alternative A

Cuts Into Existing Slopes

With proper design and construction procedures, no additional mitigation measures would be required.

Fills

With proper design and construction procedures, no additional mitigation measures would be required.

Seismic Considerations

The project features should be designed considering the seismicity of the site and the project seismic design criteria. The seismic design criteria would be used to determine depths of liquefaction at various locations along the proposed alignment. Estimates of lateral spreading would also be developed. Liquefaction (and its associated effects such as lateral spreading and foundation damage) could be mitigated using ground improvements such as Earthquake Drains™, compaction grouting, cement deep soil mixing, and vibro-replacement (stone columns). Catchment areas or small catchment walls could be constructed at the base of slopes or behind retaining walls to minimize sediment deposit from debris flows and debris avalanches onto the roadways.

Groundwater mitigation measures due to ground improvements, etc., are discussed in the Water Quality Discipline Report.

Pavements

With proper design and construction procedures, no additional mitigation measures would be required.

Elevated Structure Foundations

No impacts were determined for the elevated structure foundations.

Relationship Between Topography and Alignment Design

With proper design and construction procedures, no additional mitigation measures would be required.

Settlement Potential

With proper design and construction procedures, no additional mitigation measures would be required.

Alternative C

Cuts Into Existing Slopes

With proper design and construction procedures, no additional mitigation measures would be required.

Fills

With proper design and construction procedures, no additional mitigation measures would be required.

Seismic Considerations

See discussion under Alternative A.

Pavements

With proper design and construction procedures, no additional mitigation measures would be required.

Elevated Structure Foundations

With proper design and construction procedures, no additional mitigation measures would be required.

Relationship Between Topography and Alignment Design

With proper design and construction procedures, no additional mitigation measures would be required.

Settlement Potential

With proper design and construction procedures, no additional mitigation measures would be required.

Alternative D

Cuts Into Existing Slopes

With proper design and construction procedures, no additional mitigation measures would be required.

Fills

With proper design and construction procedures, no additional mitigation measures would be required.

Seismic Considerations

See discussion under Alternative A.

Pavements

With proper design and construction procedures, no additional mitigation measures would be required.

Elevated Structure Foundations

With proper design and construction procedures, no additional mitigation measures would be required.

Relationship Between Topography and Alignment Design

With proper design and construction procedures, no additional mitigation measures would be required.

Settlement Potential

With proper design and construction procedures, no additional mitigation measures would be required.

Construction Impacts

Construction activity impacts differ from the impacts previously discussed in that the duration of the impact takes place during construction or within a short period of time after construction. Construction impacts do not exist in the long term.

Mitigation for the construction impacts discussed below are based on the site information as presented in the Studies and Coordination and Affected Environment sections of this report as well as on standard construction procedures in use at the time of this report. All construction impacts presented for Alternatives A, C, and D could be addressed by design and standard construction procedures, as presented in the following sections. Alternatives A, C, and D have similar subsurface soil, groundwater, and geologic conditions; therefore, the construction impacts and mitigation measures for these three alternatives are similar.

Groundwater impacts due to temporary dewatering, etc., are discussed in the Water Quality Discipline Report.

No Build Alternative

Impacts

Because the No Build Alternative is offered as a base for comparison, no construction impacts would occur.

Mitigation Measures

Because the No Build Alternative is offered as a base for comparison, no mitigation measures would be considered.

Alternative A

Impacts

Settlement

As stated previously, the proposed fill approaches would be constructed over some surficial, loose and soft, soil conditions. If the subgrade has soft soil to a sufficient depth, the proposed height of the fill approaches may not be stable on the existing ground. Failures could occur as the fill is placed and the shear strength of the soil resisting failure is exceeded. This could result in rotational failure through the fill and/or a bearing capacity failure of the entire fill, depending on the subsurface conditions and the fill configuration. In areas where the soft subgrade soils are cohesive, consolidation and strength gain would occur over time as the fill is placed. Therefore, slope failures under the proposed fill embankments are primarily a short-term, construction impact. Design and standard construction procedures could address the settlement impact.

Vibrations, Noise, and Excavation Stability Due to Foundation Construction and Ground Improvement Installation

Because of the depth of loose, soft, and potentially liquefiable soil as well as the anticipated bridge loads, deep foundations would be required to support the proposed bridge. Deep foundations could consist of driven piles or drilled shafts. If foundations deeper than about 100 to 120 feet are required, driven piles would be used; drilled shafts become uneconomical at depths greater than about 100 to 120 feet. Impacts associated with these foundation types are discussed below.

Pile driving would result in noise and vibration impacts to the site. The vibration caused by driving piles through the site soils could impact nearby facilities. These impacts could consist of settlement, and pavement or structure cracking. Settlements to existing nearby structures founded on shallow footings would likely be more significant than settlements to structures founded on deep foundations; structure settlements would depend on the type and density of the subsurface soil where pile driving is occurring as well as the type and proximity of the existing structure's foundations. In general, facilities and utilities within about 20 to 30 feet of pile driving operations may be significantly impacted. Vibration impacts generally diminish as the distance from pile driving increases. Information regarding existing buildings' foundations within about 200 feet of the alignment is presented in Table 1. Settlements of nearby utilities may also occur. Noise from pile driving may result in structure and/or glass cracking; however, it would more likely be an annoyance to humans nearby. Noise impacts are discussed in the Noise Discipline Report.

Drilled shafts could be installed with equipment that does not cause significant vibrations. Because of the depth of loose/soft soil and the high groundwater table at the site, open hole excavation methods would be difficult. Caving or sloughing soil within the open hole excavation could impact adjacent structures and buried utilities. Bottom heave within the drilled shaft excavation could also occur. Typically, drilled shaft installations do not cause excessive noise.

Appropriate ground improvement methods may include Earthquake Drains™, compaction grouting, cement deep soil mixing, and vibro-replacement (stone columns). In general, Earthquake Drains™, compaction grouting, and cement deep soil mixing would not generate much vibration. These methods may generate some noise from equipment operation. However, stone column installation would result in noise and vibration impacts to the site. The vibration impacts caused by stone column installation would be identical to those caused by pile driving. Existing structures, facilities, and utilities within about 30 feet of stone column installation may be significantly impacted.

Standard construction procedures could be used to address foundation construction vibration, noise, and excavation stability impacts.

Potential Soil and Groundwater Contamination

There is a possibility that construction activities would encounter potentially contaminated soil and groundwater. These issues are discussed in the Hazardous Materials Discipline Report.

Erosion and Sediment Transport

Construction of Alternative A features would require some land clearing, grubbing, removal of topsoil, and other site preparation work. Because a significant portion of Alternative A is over areas that have been previously developed and paved, construction would create relatively few erosion impacts. The areas beneath proposed fills and structures and in cut areas would be cleared and grubbed of all vegetation and debris and stripped of all organic topsoil. The debris resulting from clearing and stripping would be removed from the Alternative A area or stockpiled for later re-use in landscaped areas. Topsoil material would not be suitable for reuse as structural fill because of the high organic content.

On slopes greater than 15 percent, the prepared ground surface would have a high erosion potential if exposed during the rainy season or in the presence of surface water, and on slopes less than 15 percent, there would be a low to medium erosion potential. Any areas that are disturbed during construction would be subject to increased erosion if proper control measures are not incorporated into the design. The surface water flow across exposed soil, including any ground improvement spoils, would remove sediment and deposit it in a downslope area. The amount of erosion and sedimentation would depend on the amount of soil exposed and/or disturbed, weather conditions and/or groundwater conditions, and the erosion control measures implemented. The surface soil could erode and flow into stormwater drains, into Smith Cove, and/or onto adjacent properties or streets. Erosion, sedimentation, and stormwater impacts are discussed in the Water Quality Discipline Report.

Within construction areas, the tires and tracks of heavy equipment may sink into soft surface soil if no work pad is present. The construction vehicle tires could also carry soil onto roadways (haul routes) when leaving construction areas.

Standard construction procedures could be used to address construction erosion and sediment transport impacts. Standard long-term erosion control measures would also be implemented including paving, landscaping, and slope revegetation.

Haul Routes

Haul routes are anticipated to be on existing streets. Sediment transport impacts on haul routes were discussed in the previous section.

Sundry Sites

The construction staging area for Alternative A is on level ground east of the Magnolia Bluff slope toe and the Northwest Harvest building and along the proposed alignment. The presence of wetlands and historical/archeological sites within the staging areas are being addressed by the Wildlife/Fisheries/Vegetation and Cultural/Historic/Archeological Resources Discipline Reports, respectively. The proposed size of the staging area is approximately 126,000 to 129,000 square feet. There are existing houses immediately uphill of the proposed staging area that may be impacted by construction noise; noise impacts are addressed in the Noise Discipline Report. Dust impacts are addressed in the Air Quality Discipline Report. Costs will be determined by the Contractor.

Alternative A does not require major excavation. Where possible, cut soils would be re-used as fill; however, cuts are anticipated to be less than 3 feet high for roadways. If additional structural fill is required to construct MSE walls, it could be imported from several different borrow sites within the Puget Sound area and stockpiled on site.

Mitigation Measures

Settlement

The short-term construction stability of the proposed fill approaches could be improved (if necessary) by using staged construction and/or geotextiles. These methods would improve the short-term stability of the embankments as the underlying cohesive soil consolidates and gains strength over time.

Staged construction consists of building the fill approaches in stages, depending on the amount of load the subsurface soil could accommodate at its existing strength. As the strength increases over time due to consolidation, additional fill could be placed on the strengthened subgrade while maintaining a similar factor-of-safety against failure. Monitoring of the settlement and pore pressure buildup and dissipation would be performed using instrumentation to determine the appropriate staging.

Geotextiles could be used to reinforce potential failure zones within the fill. For example, several layers of geotextile could be placed at the base of the proposed fill approaches. A higher staged fill approach could be constructed on the reinforced base than a fill approach without geotextiles. Although staged construction may still be necessary to construct the entire fill approach, using geotextile reinforcements could reduce the number of stages required or could allow for single-stage construction.

Lightweight fill material could be used to construct the approaches in areas where staged construction is not feasible. Because of the lighter weight of the fill material, the subgrade soil could support a higher fill approach than if standard fill were used. Lightweight fills that could be considered include expanded polystyrol (EPS), foamed cement, and other lightweight materials that would be stable over the life of the proposed action.

Vibrations, Noise, and Excavation Stability Due to Foundation Construction and Ground Improvement Installation

Driven piles may be used to support elevated structures, especially where existing soil/groundwater contamination is present and/or where the depth to competent soil is deeper than about 100 to 120 feet (too deep for drilled shaft installation). To mitigate noise and vibration during driven pile installation, low vibration/noise pile driving equipment could be selected. Alternatively, the piles could be driven open-ended or could be driven into a near-surface predrilled hole, which would result in lower vibrations. Preconstruction surveys of existing structures and vibration monitoring during pile driving may be required to monitor and mitigate potential damage to adjacent sensitive structures. Mitigation for noise due to pile driving is discussed further in the Noise Discipline Report.

Drilled shafts also may be used to support elevated structures. To mitigate vibrations, low vibration equipment (such as an oscillator system) could be selected. To mitigate potential caving of the soil in the excavated holes, casing would be used in the upper soft/loose soil. Water or slurry inside the casing could mitigate potential bottom heave that could be caused by the high groundwater table.

Immediately following drilled shaft installation, the casing would be removed. Alternatively, the casing could be left in place; however, the frictional capacity of the drilled shaft would have to be re-evaluated.

Stone columns are one of the most cost-effective ground improvement techniques and methods. This method can be used in any open areas greater than 30 to 50 feet away from existing structures, facilities and utilities. In order to mitigate the impacts of vibration, compaction grouting or cement deep soil mixing may be used for ground improvement to mitigate liquefaction and lateral spreading. These two ground improvement methods would not generate significant vibrations.

Erosion and Sediment Transport

Construction best management practices (BMPs), such as construction staging barrier berms, filter fabric fences, temporary sediment detention basins, and use of slope coverings to contain sediment on site, would be effective in protecting water resources and reducing erosion from areas with cuts, fills, excavations, and any ground improvement installation disturbance. Erosion control measures suitable to the site conditions would be included as part of the proposed action design. Temporary erosion and sediment control plans would be prepared for approval in accordance with BMPs included in the current City of Seattle specifications (City of Seattle Standard Specifications, 2003). Erosion control measures would include vegetative and structural controls. Other controls that could be implemented include restricting slope work activities to the dry season and limiting access to the site.

Vegetative methods would include covering cleared or graded areas and excavation or fill approach slopes with jute or other netting as well as mulching or hydroseeding, as appropriate to minimize erosion and encourage revegetation. Vegetation buffers would be maintained between construction areas and Smith Cove to filter out sediments.

Structural controls consist of artificial means of preventing sediment from leaving the construction area. Parking and staging areas for vehicles and equipment could be covered with a gravel work pad where appropriate to prevent the disturbance and erosion of the underlying soil. Silt fences would be placed around disturbed areas to filter sediment from unconcentrated surface water runoff. Straw bales would be placed in paths of concentrated runoff to filter sediment. Temporary ditches, berms, and sedimentation ponds would be constructed to collect drainage. Cleaning tires and tracks on heavy equipment before they leave the site would also assist in retaining sediment on site. In addition, truck loads should be covered to mitigate sediment deposit onto roadways.

Proposed mitigation measures would comply with stormwater design and treatment procedures based on the current City of Seattle requirements. The erosion and sediment control measures would be in place before any clearing, grading, or construction. The Water Quality Discipline Report discusses stormwater mitigation.

Haul Routes

Haul routes are anticipated to be on existing streets. Sediment transport control is discussed above.

Sundry Sites

Mitigation of wetlands, historical/archeological sites, noise and dust within and due to the staging areas are being addressed in other discipline reports. These other discipline reports include Wildlife/Fisheries/Vegetation, Cultural/Historic/Archeological Resources, Noise, and Air Quality.

Alternative C

Impacts

Settlement

See discussion under Alternative A.

Vibrations, Noise, and Excavation Stability Due to Foundation Construction and Ground Improvement Installation

See discussion under Alternative A.

Potential Soil and Groundwater Contamination

See discussion under Alternative A.

Erosion and Sediment Transport

See discussion under Alternative A.

Haul Routes

See discussion under Alternative A.

Sundry Sites

See discussion under Alternative A. The proposed location of the staging area is on level ground east of the Magnolia Bluff slope toe and along the proposed alignment. The proposed size of the staging area is about 116,000 square feet.

Mitigation Measures

Settlement

See discussion under Alternative A.

Vibrations, Noise, and Excavation Stability Due to Foundation Construction and Ground Improvement Installation

See discussion under Alternative A.

Erosion and Sediment Transport

See discussion under Alternative A.

Haul Routes

See discussion under Alternative A.

Sundry Sites

See discussion under Alternative A.

Alternative D

Impacts

Settlement

See discussion under Alternative A.

Vibrations, Noise, and Excavation Stability Due to Foundation Construction and Ground Improvement Installation

See discussion under Alternative A.

Potential Soil and Groundwater Contamination

See discussion under Alternative A.

Erosion and Sediment Transport

See discussion under Alternative A.

Haul Routes

See discussion under Alternative A.

Sundry Sites

See the discussion under Alternative A. The proposed location of the staging area is in the vicinity of the existing Northwest Harvest building just east of the Magnolia Bluff slope toe on developed Port of Seattle property. The proposed size of the staging area is approximately 108,000 square feet.

Mitigation Measures

Settlement

See the discussion under Alternative A.

Vibrations, Noise, and Excavation Stability Due to Foundation Construction and Ground Improvement Installation

See the discussion under Alternative A.

Erosion and Sediment Transport

See the discussion under Alternative A.

Haul Routes

See discussion under Alternative A.

Sundry Sites

See discussion under Alternative A.

Summary of Findings

This geology and soils discipline report describes the geologic conditions present along the three proposed build alignments (designated Alternatives A, C, and D) and the existing alignment (the No Build Alternative), and discusses the geotechnical-related operational and construction impacts and recommended mitigations for the Magnolia Bridge Replacement Project. Subsurface data used to assess these issues are presented in Appendix A.

Affected Environment

Geologic Setting

The proposed alternatives extend across a north-trending topographic trough called Interbay. The trough is bounded on both sides by glacial uplands: Magnolia Bluff on the west and Queen Anne Hill on the east. While the uplands are comprised of very dense and hard glacial soils laid down during the advance and retreat of several glaciations, the intervening topographic swale/trough of Interbay is comprised of much weaker glacial, beach, and estuary deposits laid down since the last retreat of glacial ice approximately 13,000 years ago. Since the late nineteenth century, the Interbay area (specifically Smith Cove) has been filled by humans with various materials. These weak soils in Interbay are underlain by more competent, glacial soils at depth. The depth to these more competent soils varies considerably along and in the vicinity of the existing bridge and three proposed alternatives.

Groundwater

Groundwater levels within the project corridor are generally within 10 feet of the ground surface; however, the groundwater is likely directly related to the tidal fluctuation of Smith Cove. Additional details are provided in the Water Quality Discipline Report.

Geologic Hazards

The project area is located in a moderately active tectonic province that has been subjected to numerous earthquakes of low to moderate magnitude and occasionally to strong shocks during the brief 170-year written, historical record in the Pacific Northwest. Earthquake-induced geologic hazards that may affect any given alternative include strong ground motion, liquefaction (and its related effects including lateral spreading), and landsliding. Other non-earthquake-related hazards, such as landsliding and erosion, could also occur.

Impacts and Mitigation Measures

Soil- and geology-related operational and construction impacts and recommended mitigation measures were developed based on the project area geology, known subsurface conditions, and the No Build and build alternative alignments. Alternatives A, C, and D have similar subsurface soil, groundwater, and geologic conditions; therefore, the impacts and mitigation measure costs for these three alternatives would be similar. A summary matrix of these impacts and mitigation measures is presented on Table 3. Nearly all of the impacts could be addressed by proper design and standard construction procedures and therefore no additional mitigation measures would be required. Liquefaction and, in particular, lateral spreading would require nonstandard construction procedures; however, these procedures, although nonstandard, are not uncommon given current seismic design criteria and earthquake engineering technology.

Should a design-level ground motion or other threshold ground motion occur that resulted in liquefaction, the No Build Alternative foundations would likely experience excessive settlement. The proposed foundation design and construction would address the effects of potential liquefaction for the build alternatives.

Should significant lateral spreading occur, the lateral deflection of the existing bridge foundations (No Build Alternative) would likely cause bridge collapse. The northern portion of Alternative C may experience less lateral spreading than Alternatives A and D because of the distance to the free-face slope at the edge of Smith Cove. Proper design and construction would address the effects of lateral spreading for the build alternatives.

The western slope (near the end of the bridge structure) of the No Build and build alternatives already have a stabilizing retaining wall. Landslides could occur uphill of Alternative C as it extends along the Magnolia Bluff slope toe; these potential landslides may impact Alternative C. Landslides could also occur uphill of the eastern end of the No Build Alternative and Alternatives A, C, and D; these potential landslides may impact these alternatives. Proper design and construction would address the effects of slope stability on the build alternatives.

Table 3
Summary Matrix – Geology and Soils

Alternative	Impacts	Mitigation Measures
No Build	*Operation:*	
	Current design-level earthquakes could occur during the life of the existing structure, causing liquefaction, lateral spreading and slope instability. The effects of liquefaction may include loss of bearing capacity for existing shallow foundations, reduction in lateral and vertical capacities of existing bridge foundations, ground surface settlement, lateral spreading, lateral deflection of existing bridge foundations and utilities, and slope instability or slumping. Liquefaction alone could cause excessive settlement. Should significant lateral spreading occur, the lateral deflection of the existing bridge foundations would likely cause bridge collapse. Slope instability could cause damage to existing bridge foundations and deposit debris onto existing roadways and ramps.	No seismic mitigation would be performed for the No Build Alternative.
	Future landsliding could occur near the east and west ends of the existing bridge, which may impact the bridge's operation. The landsliding would most likely not occur where engineered retaining walls and slopes have already been installed unless unforeseen conditions arise.	No landsliding mitigation would be performed for the No Build Alternative.
A, C, and D	*Operation:*	
	Cuts into existing slopes could result in slope instability. Retaining walls would be used to support the cuts and the soil slopes behind the cuts. The walls would be designed by experienced structural and geotechnical engineers whose design would be based on subsurface information and standard design procedures.	Appropriate design and construction procedures would address impacts.
	Cut walls used to retain slopes could lack soil resistance in areas where existing landslide deposits are present. Subsurface explorations would be performed to evaluate the vertical and lateral extent of the existing landslide deposits. The walls would be designed so that the base of the walls extends into undisturbed deposits.	Appropriate design and construction procedures would address impacts.
	Settlement of fill approaches could impact underlying and adjacent structures or utilities as well as walls or structures constructed on the fill. Settlement impacts could be mitigated by several methods, including preloading, use of mechanically-stabilized earth (MSE) walls, construction sequencing, ground improvement, or use of lightweight fill. Affected utilities may be relocated, or the use of lightweight fill could be considered.	Appropriate design and construction procedures would address impacts.
	Downdrag caused by ground settlement could result in additional loads and potential damage to existing buried foundations and new deep foundations. New deep foundations could be designed to accommodate the downdrag loads, or construction sequencing could be used so that the foundations are installed after most of the	Appropriate design and construction procedures would address impacts.

Alternative	Impacts	Mitigation Measures
	settlement has occurred. Existing foundations should be evaluated for the settlement-induced downdrag loads. Mitigation measures such as use of lightweight fill, ground improvement, and/or additional foundation members may be considered.	
	Fill placement over soft soil could cause slope instability. Fill approach stability would be primarily of concern during the short-term (construction) period. Over the long-term (static loading conditions), the soft soil beneath the fill approaches would consolidate and gain strength, thereby improving the stability. Preloading, staged construction, ground improvement, or use of geotextile reinforcements or lightweight fills could improve stability in the short term.	Appropriate design and construction procedures would address impacts.
	Future landsliding could occur above cut or fill walls. Catchment walls could be constructed above the retaining walls to temporarily retain future debris flow and debris avalanche material and reduce sediment deposit onto roadways and ramps.	Appropriate design and construction procedures would address impacts.
	A design-level earthquake could occur during the life of the proposed structure causing liquefaction, lateral spreading, and slope instability. The effects of liquefaction may include loss of bearing capacity for existing shallow foundations, reduction in lateral and vertical capacities of new deep foundations, ground surface settlement, lateral spreading, slope instability or slumping, and fill approach instability. In addition, slopes may experience surface sloughing or raveling that could deposit material onto the ramps and surface streets.	Additional borings and engineering studies could be conducted to evaluate the bridge foundations relative to the site's seismicity and seismic design criteria. Estimates of liquefaction and lateral spreading potential would then be developed. Liquefaction and lateral spreading could be mitigated using ground improvement measures such as Earthquake Drains™, compaction grouting, cement deep soil mixing, and vibro-replacement (stone columns). Catchment areas or small catchment walls could be constructed at the base of slopes or behind walls to minimize sediment deposit onto roadways and ramps from debris flow/debris avalanches and to reduce potential damage to bridge foundations.
	Poor subgrade preparation and/or design for proposed pavements could lead to settlement, potholes, cracks, and other roadway distress. Proof-rolling of the subgrade, removal of soft subgrade materials, proper fill compaction, and a pavement design that accounts for frost heave and poor subgrade soils could mitigate pavement issues.	Appropriate design and construction procedures would address impacts.
	Erosion could cause increased sediment transport onto other areas of the project, into stormwater drains, and into Smith Cove. Standard erosion control measures would be implemented including paving, landscaping, and slope revegetation.	Appropriate design and construction procedures would address impacts.
A, C, and D	*Construction:*	
	Fill placement over soft soil could cause slope instability. Short-term (construction) stability could be improved by using staged construction and/or geotextiles. Monitoring of the settlement and pore pressure dissipation beneath the fill could be	Standard construction procedures would address impacts.

Alternative	Impacts	Mitigation Measures
	performed to optimize the staging and construction. Lightweight fill could be used in areas where staged construction is not feasible.	
	Driven pile foundation installation and stone column installation could cause noise and vibrations that would impact adjacent facilities. Casing installation for drilled shaft foundations could cause vibrations that would impact adjacent facilities. Appropriate pile driving equipment could be selected to reduce noise and vibration levels to the specified limits. The Noise Discipline Report addresses construction noise impacts and mitigations. Driving open-ended piles or predrilling a near-surface hole prior to pile driving could also reduce vibration levels. Appropriate drilled shaft equipment (such as an oscillator) could be selected to reduce vibration levels. As an alternative to stone columns, compaction grouting or cement deep soil mixing may be used for ground improvement.	Standard construction procedures would address impacts.
	Drilled shaft excavation could experience bottom heave or caving. Temporary casing could be used in the upper soft/loose soil to mitigate caving. Maintaining a proper level of water or slurry inside the casing could be used to mitigate potential bottom heave.	Standard construction procedures would address impacts.
	Erosion from areas with cuts, fills, excavations, and any ground improvement installation disturbance could cause increased sediment transport onto other areas of the project, into stormwater drains, and into Smith Cove. Construction would be performed according to the City of Seattle Best Management Practices (BMPs). Standard erosion control measures would be implemented including both vegetative controls and structural controls. In sensitive areas, construction could be limited to the dry weather season. Stormwater treatment would be performed in accordance with the City of Seattle requirements.	Standard construction procedures would address impacts.

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Appendix A

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Appendix A

Introduction

The current subsurface exploration program consisted of drilling six borings designated D-1 through D-3 and H-1 through H-3. The approximate locations of the Alternative D explorations are shown on the Site and Exploration Plans, presented as Figures 12 and 13 in the main text of this discipline report. Borings H-1 through H-3 are not shown on the site and exploration plans; they were drilled for the now deleted Alternative H. The exploration locations were approximated from existing site features and ground surface elevations in terms of the 1988 North American Vertical Datum (NAVD88) and were estimated using project topography. In addition, several previous explorations from other studies were used. These explorations are also shown on Figures 11, 12, and 13 in the main text of the report; their locations were approximated using log descriptions and report exploration plans. The elevations given on several of the previous logs may not be accurate. We determined the approximate elevations of the previous explorations in three ways: (1) the elevations were estimated based on the current site topography and their approximate location, (2) the elevations given on the logs were assumed to be in terms of NAVD88 based on their date and their correlation with the current topography, and (3) the elevations given on the logs were given in terms of other data and were then converted to the NAVD88 datum.

Current Soil Borings

Soil Classification

An engineering technician from Shannon & Wilson, Inc. was present throughout the drilling and sampling operations for the current borings. Our representative retrieved representative soil samples and prepared a descriptive field log of the explorations. Classification of the boring samples was based on American Society for Testing and Materials (ASTM) Designation: D 2487-98, Standard Test Method for Classification of Soil for Engineering Purposes, and ASTM Designation: D 2488-93, Standard Recommended Practice for Description of Soils (Visual-Manual Procedure). The Unified Soil Classification System (USCS), as described on Figure A-1 of this appendix, was used to classify the soils encountered in the soil borings. For quality assurance purposes, an engineering geologist also went through the samples and classified the soil in our laboratory. The boring logs in this report (Figures A-3 through A-8) represent our interpretation of the contents of the field logs. Figure A-2 presents our Geologic Unit Explanation; geologic units are noted on the current boring logs.

Drilling Procedures

The subsurface conditions along the proposed Alternative D and Alternative H (now deleted) alignments were explored with three soil borings each. The borings were drilled to depths of 45.9 to 151.5 feet and were accomplished between April 25 and May 7, 2003.

Geo-Tech Explorations of Kent, Washington, drilled the soil borings under subcontract to Shannon & Wilson, Inc. They employed a truck-mounted, drill rig; the borings were drilled using a combination of hollow-stem auger and open-hole mud-rotary methods. Hollow-stem auger drilling was performed to a depth of 20 feet below ground surface (bgs) or 5 feet below the groundwater, whichever came first. Soil samples were collected every 2.5 feet to 20 feet or to groundwater in each boring, for field screening, geologic classification, and environmental sampling purposes. Field screening was performed using a photoionization detector (PID), which provides a qualitative measurement of the volatile organics in soil. PID measurements associated with Alignment H were non-detect in all three borings, while PID measurements ranged from non-detect to 355 parts per million (ppm) in the borings along Alignment D. The PID measurements are recorded on the boring logs.

Once drilling had advanced to 20 feet bgs or 5 feet below groundwater, the borings were advanced to depth using mud rotary drilling techniques. Soil samples were collected every 5 feet for geologic classification and geotechnical testing purposes. During the mud rotary drilling, the auger flights were left in the borehole as a temporary casing.

The hollow-stem auger borehole depth segments were drilled using a $6^{5}/_{8}$-inch inside-diameter (I.D.), 9-inch outside-diameter (O.D.) continuous flight auger. Samples were taken from the bottom of the hollow stem. The mud-rotary portions were advanced by circulating thick drilling mud from the rig down through rods to a $4^{7}/_{8}$-inch-diameter tri-cone bit at the bottom of the borehole. The drilling mud is a mixture of bentonite powder and water. Cuttings are transported from the bottom of the borehole to the surface by drilling mud flowing between the drilling rods and the sides of the borehole. The cuttings are deposited in a settling tank at the ground surface and the mud is recirculated.

Prior to moving to a new borehole location and between each environmental sample, personnel decontaminated drilling and non-disposable sampling equipment using a solution of Alconox and water, with a final tap water rinse. Decontamination fluids were drummed separately from soil cuttings and drilling mud, and were labeled and temporarily stored below the existing Magnolia Bridge at a location determined by Port of Seattle personnel. New decontamination water was used for each boring. Two drums of decontamination water were generated during this field investigation. No samples of the decontamination water were collected for laboratory analysis. Disposal was determined based upon the laboratory results for the environmental soil samples. Environmental analytical results are presented in the Hazardous Materials Discipline Report.

After completion of drilling and sampling, all boreholes were sealed with bentonite chips. No observation wells were installed. All cuttings and drilling mud were transferred into drums, labeled, and stored below the existing Magnolia Bridge while environmental testing was completed. Nine drums of soil cuttings, 20 drums of drilling mud, and 8 drums of mud cuttings were generated during this field investigation.

Upon receipt of the soil sample results, on June 20, 2003, Emerald Services, our disposal subcontractor, picked up the drums and disposed of them.

Geotechnical Soil Sampling

During drilling, three types of soil samplers were used: thin-walled tubes, standard 2-inch O.D. split-spoons, and non-standard 3.25-inch O.D. split spoons. Symbols used on the boring logs indicate which sampler was used at each depth interval. The sampler types are discussed in the following sections.

Thin-Walled Tube Samples

Relatively undisturbed samples of cohesive soils were obtained using thin-walled (Shelby) tubes in general accordance with ASTM Designation: D 1587, Standard Practice for Thin-Walled Tube Geotechnical Sampling of Soils. This sampling method employs a 3-inch O.D. thin-walled, steel tube connected to a sampling head that is attached to the drill rods. The tube is slowly pushed by the hydraulic rams of the drill rig into the soil below the bottom of the drilled hole and then retracted to obtain a sample. The samples were classified in the field and recorded on the logs by our field representative. The samples were carefully sealed and transported to our laboratory for testing.

Standard Penetration Test Samples

Relatively disturbed soil samples were obtained from borings using Standard Penetration Tests (SPTs) in general accordance with the ASTM Designation: D 1586, Test Method for Penetration Test and Split-Barrel Sampling of Soils (ASTM, 2001). In the SPT, a 2-inch O.D., 1.375-inch I.D., split-spoon sampler is driven with a 140-pound hammer falling 30 inches. The number of blows required to achieve each of three 6-inch increments of sampler penetration is recorded. The number of blows required to cause the last 12 inches of penetration is termed the Standard Penetration Resistance (N-value), or blow count, N. When penetration resistances exceed 50 blows for 6 inches or less of penetration, the test is terminated and the number of blows and inches driven are recorded. The samples were sealed in jars and returned to our laboratory for testing.

The SPTs were recorded by our field representative and are plotted on the boring logs. The N-values are designated with an upright triangle. These values are empirical parameters that provide a means of evaluating the relative density or compactness of cohesionless (granular) soils and the relative consistency (stiffness) of cohesive soils. The terminology used to describe the relative density or consistency of the soil is presented on Figure A-1.

Non-standard Split Spoon Samples

Where a larger amount of recovered sample was desired in order to obtain a geotechnical and possibly an environmental sample, a 3.25-inch O.D. split-spoon sampler was used. This sampler was driven with a 140-pound hammer free falling 30 inches. The energy ratio for this type of sampling is not equivalent to an SPT; therefore, we have converted the field blow counts to approximate N-values using the method described by Fang (1991). These converted blow counts are designated on the logs by an upside-down triangle.

Groundwater Observations

Groundwater was noted during drilling and is shown on the boring logs. These measurements may not be representative of the highest groundwater level at the boring locations; please refer to the tidal fluctuation discussion in the main discipline report text.

Environmental Soil Sampling

During drilling, representative soil samples were obtained for geotechnical classification at 2.5-foot intervals to a depth of 20 feet or 5 feet below groundwater and at 5-foot intervals thereafter. Select samples were also obtained for environmental testing by OnSite Environmental Laboratory of Redmond, Washington.

Environmental Soil and Water Sample Results

The environmental analytical results are presented in the Hazardous Materials Discipline Report.

Boring Logs

The current boring logs along the proposed alignments are presented in this appendix. A boring log is a written record of the subsurface conditions encountered. It graphically illustrates the geologic units (layers) encountered in the boring and the USCS symbol of each geologic layer. It also includes the natural water content and blow count. Other information shown on the boring logs includes the groundwater level observations made during drilling, approximated ground surface elevation, types and depths of sampling, and Atterberg Limits (where tested).

Previous Field Explorations

Numerous previous field explorations by Shannon & Wilson as well as many other firms, the City of Seattle, and the Port of Seattle are also included on the site and exploration plans in the main text of the report. The previous exploration logs are presented as Figures A-9 to A-71. Several of the explorations had groundwater level readings during drilling and some had readings from observation wells. These readings are included on the generalized subsurface profiles in the main text of the report. Table A-1 lists the sources for each of the previous field exploration logs included in this data report.

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Table A-1
Sources of Previous Explorations

Shannon & Wilson Geology and Soils Discipline Report Figure Number	Shannon & Wilson Geology and Soils Discipline Report Exploration Designation	Original Exploration Designation	Type of Exploration	Exploration Company	Date Exploration Completed	Source of Exploration Log
A-9	2-2	B-2	boring	S&W	11/10/1993	S&W
A-10	2-3	Boring 12	boring	City of Seattle?	3/25/1965	S&W
A-11	2-4	B-1	boring	S&W	12/7/1993	S&W
A-12	2-5	B-2	boring	S&W	12/9/1993	S&W
A-13	3-3	B-3	boring	S&W	1/12/1997	S&W
A-14	3-5	PB-2	boring	S&W	1/12/1997	S&W
A-15	5-4	B-4	boring	S&W	11/12/1997	S&W
A-16	6-4	HB-1	hand boring	S&W	7/29/1997	S&W
A-17	13-2	P-2	CPT	HCA	6/25/1981	Port of Seattle
A-18	13-3	P-3	CPT	HCA	7/3/1981	Port of Seattle
A-19	13-6	B-1	boring	HCA	6/26/1981	Port of Seattle
A-20	13-7	B-2	boring	HCA	7/14/1981	Port of Seattle
A-21	13-18	B-101	boring	HCA	7/20/1981	Port of Seattle
A-22	13-19	B-102	boring	HCA	7/20/1981	Port of Seattle
A-23	13-20	Well No. 1	deep well	Unknown	March-43	Port of Seattle
A-24	14-1	Boring 5	boring	SED Materials Laboratory	1/31/1973	City of Seattle
A-25	14-2	Boring 1	boring	SED Materials Laboratory	11/14/1972	City of Seattle
A-26	14-3	Boring 2	boring	SED Materials Laboratory	11/14/1972	City of Seattle
A-27	15-1	Boring 8	boring	SED Materials Laboratory	6/7/1972	City of Seattle
A-28	16-3	Boring 3	boring	SED Materials Laboratory	11/4/1981	City of Seattle
A-29	22-1	Boring 2	boring	SED Materials Laboratory	7/21/1988	City of Seattle
A-30	23-1	Boring 1	boring	RZA	February-78	City of Seattle
A-31	23-2	Boring 2	boring	RZA	February-78	City of Seattle
A-32	23-3	Boring 3	boring	RZA	February-78	City of Seattle
A-33	23-4	Boring 4	boring	RZA	February-78	City of Seattle
A-34	23-5	Boring 5	boring	RZA	February-78	City of Seattle
A-35	23-6	Boring 6	boring	RZA	February-78	City of Seattle
A-36	282-1	Boring 1	Boring	ECI	9/6/1989	SAGMP
A-38	710-1	Boring 1	boring	GCI	10/13/1995	SAGMP
A-39	1647-2	B-1	boring	GEI/CEO	6/8/1994	SAGMP
A-40	1650-1	HC-1	boring	HCA	8/27/1994	SAGMP
A-41	1657-1	Boring 1	boring	GEI	1/19/1987	SAGMP

Table A-1 (cont.)
Sources of Previous Explorations

Shannon & Wilson Geology and Soils Discipline Report Figure Number	Shannon & Wilson Geology and Soils Discipline Report Exploration Designation	Original Exploration Designation	Type of Exploration	Exploration Company	Date Exploration Completed	Source of Exploration Log
A-42	1657-2	Boring 2	boring	GEI	1/20/1987	SAGMP
A-43	1657-3	Boring 3	boring	GEI	1/19/1987	SAGMP
A-44	1657-4	Boring 4	boring	GEI	1/20/1987	SAGMP
A-45	1657-5	Boring 5	boring	GEI	1/20/1987	SAGMP
A-46	3352-1	B-1	boring	GEI	1/24/1990	SAGMP
A-47	3352-2	B-2	boring	GEI	1/25/1990	SAGMP
A-48	3352-3	B-3	boring	GEI	1/26/1990	SAGMP
A-49	CP_103B	CP_103B	boring	SEAI	12/2/1987	Port of Seattle
A-50	CP_108B	CP_108B	boring	SEAI	1/20/1989	Port of Seattle
A-51	CP_111	CP-111	boring	BE	10/10/1992	Port of Seattle
A-52	CP_115B	CP-115B	boring	BE	March-93	Port of Seattle
A-53	CP_TB-4	TB-4	boring	SEAI	12/9/1988	Port of Seattle
A-54	11-1	HB-1	hand boring	S&W	7/31/1989	S&W
A-55	11-6	HB-6	hand boring	S&W	8/24/1989	S&W
A-56	11-7	HB-7	hand boring	S&W	8/23/1989	S&W
A-57	11-8	HB-8	hand boring	S&W	8/23/1989	S&W
A-58	711-1	Boring No. 1	boring	TA	11/17/1986	SAGMP
A-59	711-2	Boring No. 2	boring	TA	11/17/1986	SAGMP
A-60	711-3	Boring No. 3	boring	TA	11/17/1986	SAGMP
A-61	711-4	Boring No. 4	boring	TA	11/17/1986	SAGMP
A-62	2216-1	Boring 1	boring	GCI	7/31/1996	SAGMP
A-63	2216-2	Boring 2	boring	GCI	7/31/1996	SAGMP
A-64	2669-1	B-1	boring	GGN	2/12/1997	SAGMP
A-65	2669-2	B-2	boring	GGN	2/12/1997	SAGMP
A-66	3440-1	B-1	boring	ZZA	3/21/2000	SAGMP
A-67	14-5	Hole Number 4	boring	SED	11/14/1972	SED
A-68	16-1	Hole Number 1	boring	SED	10/30/1981	SED
A-60	16-2	Hole Number 2	boring	SED	10/30/1981	SED
A-70	CP_205B	CP_205B	boring	PNG	11/21/1995	Port of Seattle
A-71	CP_109	CP_109	boring	SEAI	12/13/1988	Port of Seattle

Notes:
BE = Burlington Environmental
CPT = Cone Penetration Test
ECI = Earth Consultants, Inc.
GCI = Geotech Consultants, Inc.
GEI = GeoEngineers Incorporated/GeoEngineers
GEI/CEO = GeoEngineers, Inc. and Creative Engineering Options, Inc.
GGN = GeoGroup Northwest, Inc.
HCA = Hart-Crowser & Associates, Inc./Hart-Crowser
PNG = Pacific Northern GeoScience
RZA = Rittenhouse-Zeman & Associates
SAGMP = Seattle Area Geologic Mapping Project
SEAI = Sweet, Edwards & Associates, Inc.
SED = Seattle Engineering Department
S&W = Shannon & Wilson, Inc.
TA = Terra Associates
ZZA = Zipper Zeman Associates, Inc.

Shannon & Wilson, Inc. (S&W), uses a soil classification system modified from the Unified Soil Classification System (USCS). Elements of the USCS and other definitions are provided on this and the following page. Soil descriptions are based on visual-manual procedures (ASTM D 2488-93) unless otherwise noted.

S&W CLASSIFICATION OF SOIL CONSTITUENTS

- MAJOR constituents compose more than 50 percent, by weight, of the soil. Major consituents are capitalized (i.e., SAND).
- Minor constituents compose 12 to 50 percent of the soil and precede the major constituents (i.e., silty SAND). Minor constituents preceded by "slightly" compose 5 to 12 percent of the soil (i.e., slightly silty SAND).
- Trace constituents compose 0 to 5 percent of the soil (i.e., slightly silty SAND, trace of gravel).

MOISTURE CONTENT DEFINITIONS

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, from below water table

ABBREVIATIONS

ATD	At Time of Drilling
Elev.	Elevation
ft	feet
FeO	Iron Oxide
MgO	Magnesium Oxide
HSA	Hollow Stem Auger
ID	Inside Diameter
in	inches
lbs	pounds
Mon.	Monument cover
N	Blows for last two 6-inch increments
NA	Not applicable or not available
NP	Non plastic
OD	Outside diameter
OVA	Organic vapor analyzer
PID	Photo-ionization detector
ppm	parts per million
PVC	Polyvinyl Chloride
SS	Split spoon sampler
SPT	Standard penetration test
USC	Unified soil classification
WLI	Water level indicator

GRAIN SIZE DEFINITION

DESCRIPTION	SIEVE NUMBER AND/OR SIZE
FINES	< #200 (0.8 mm)
SAND* - Fine	#200 to #40 (0.8 to 0.4 mm)
- Medium	#40 to #10 (0.4 to 2 mm)
- Coarse	#10 to #4 (2 to 5 mm)
GRAVEL* - Fine	#4 to 3/4 inch (5 to 19 mm)
- Coarse	3/4 to 3 inches (19 to 76 mm)
COBBLES	3 to 12 inches (76 to 305 mm)
BOULDERS	> 12 inches (305 mm)

* Unless otherwise noted, sand and gravel, when present, range from fine to coarse in grain size.

RELATIVE DENSITY / CONSISTENCY

COARSE-GRAINED SOILS		FINE-GRAINED SOILS	
N, SPT, BLOWS/FT.	RELATIVE DENSITY	N, SPT, BLOWS/FT.	RELATIVE CONSISTENCY
0 - 4	Very loose	Under 2	Very soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium dense	4 - 8	Medium stiff
30 - 50	Dense	8 - 15	Stiff
Over 50	Very dense	15 - 30	Very stiff
		Over 30	Hard

WELL AND OTHER SYMBOLS

Bent. Cement Grout	Surface Cement Seal
Bentonite Grout	Asphalt or Cap
Bentonite Chips	Slough
Silica Sand	Bedrock
PVC Screen	
Vibrating Wire	

Magnolia Bridge Replacement
Seattle, Washington

SOIL CLASSIFICATION AND LOG KEY

August 2004 21-1-09759-008

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-1
Sheet 1 of 2

BORING_CLASS1 21-09759.GPJ SWNEW.GDT 8/13/03

Figure A-1, Sheet 1 of 2
Soil Classification and Log Key

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)
(From ASTM D 2487-98 & 2488-93)

MAJOR DIVISIONS			GROUP/GRAPHIC SYMBOL	TYPICAL DESCRIPTION
COARSE-GRAINED SOILS *(more than 50% retained on No. 200 sieve)*	Gravels *(more than 50% of coarse fraction retained on No. 4 sieve)*	Clean Gravels *(less than 5% fines)*	GW	Well-graded gravels, gravels, gravel/sand mixtures, little or no fines
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
		Gravels with Fines *(more than 12% fines)*	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands *(50% or more of coarse fraction passes the No. 4 sieve)*	Clean Sands *(less than 5% fines)*	SW	Well-graded sands, gravelly sands, little or no fines
			SP	Poorly graded sand, gravelly sands, little or no fines
		Sands with Fines *(more than 12% fines)*	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS *(50% or more passes the No. 200 sieve)*	Silts and Clays *(liquid limit less than 50)*	Inorganic	ML	Inorganic silts of low to medium plasticity, rock flour, sandy silts, gravelly silts, or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		Organic	OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays *(liquid limit 50 or more)*	Inorganic	MH	Inorganic silts, micaceous or diatomaceous fine sands or silty soils, elastic silt
			CH	Inorganic clays or medium to high plasticity, sandy fat clay, or gravelly fat clay
		Organic	OH	Organic clays of medium to high plasticity, organic silts
HIGHLY-ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		PT	Peat, humus, swamp soils with high organic content (see ASTM D 4427)

NOTES

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.

Magnolia Bridge Replacement
Seattle, Washington

SOIL CLASSIFICATION AND LOG KEY

August 2004 | 21-1-09759-008

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FIG. A-1
Sheet 2 of 2

BORING_CLASS2 21-09759.GPJ SWNEW.GDT 8/13/03

Figure A-1, Sheet 2 of 2
Soil Classification and Log Key

GEOLOGIC UNITS

HOLOCENE DEPOSITS

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.

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.

He ESTUARINE DEPOSITS: Estuary deposits of intertidal zones associated with rivers and streams located along the present and former Puget Sound shoreline.
Clayey Silt, silty Clay, Silt, and fine Sand; very soft to very 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 and shell debris; loose to dense.

QUATERNARY VASHON DEPOSITS

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.

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.

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: Fine-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.

QUATERNARY PRE-VASHON DEPOSITS

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.

Qpnm MUDFLOW DEPOSITS: Distal deposits of mass movements such as landslides or lahars.
Stratified or irregular bodies of a heterogeneous mixture of Gravel, Sand, Silt, and Clay; pumice, obsidian and ash common; rare organics (charcoal); 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.

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.

NOMENCLATURE

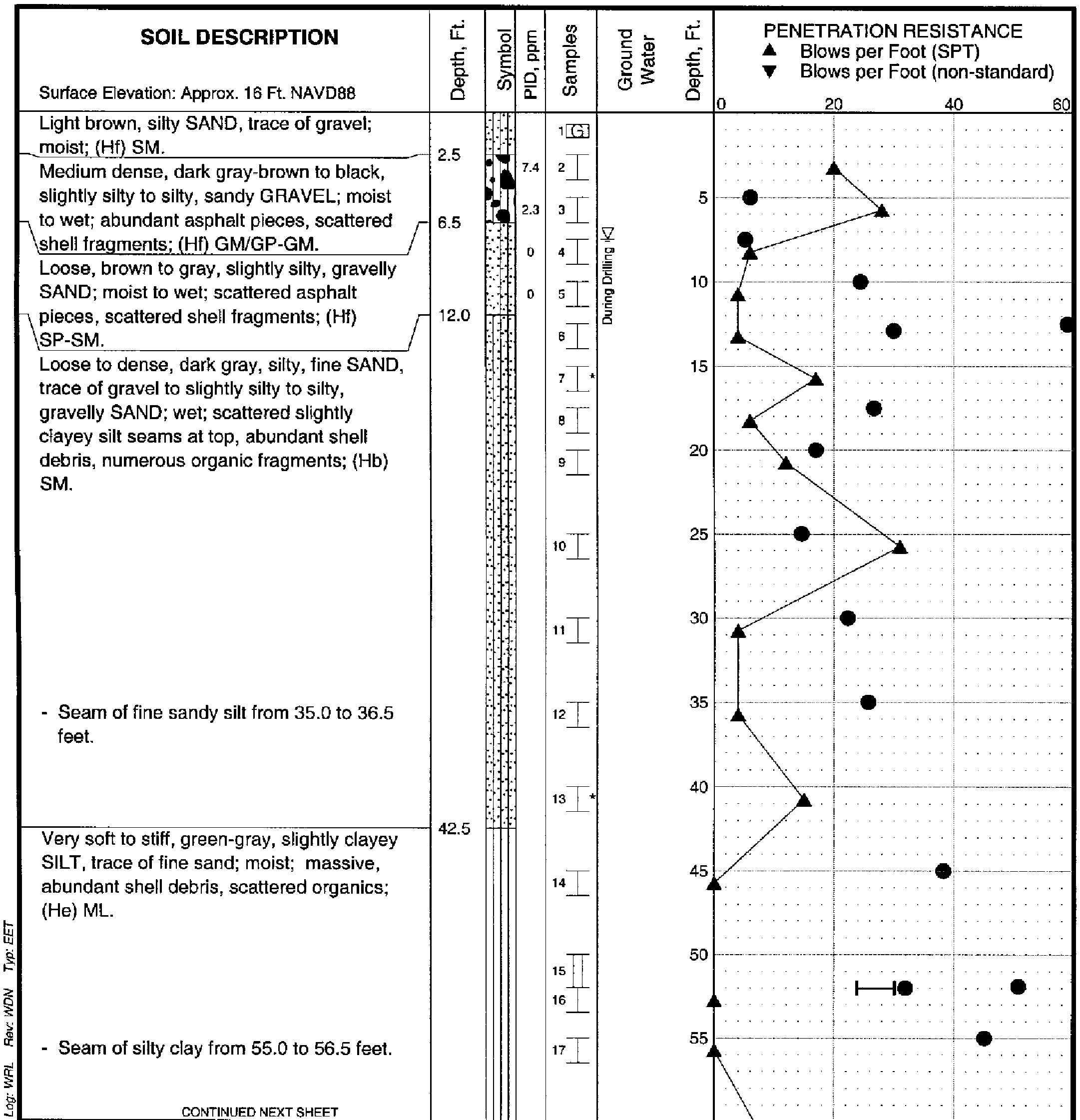
<table>
<tr><th colspan="2">GEOLOGIC AGE DESIGNATION</th><th colspan="2">DEPOSITIONAL ENVIRONMENT, GEOLOGIC PROCESS, OR LITHOLOGY</th><th></th></tr>
<tr><td colspan="2">H = Holocene</td><td>f = fill
c = colluvium
ls = landslide</td><td>e = estuarine
b = beach</td><td>Present</td></tr>
<tr><td rowspan="4">Q = Quaternary</td><td rowspan="2">v = Vashon</td><td>r = recessional</td><td>o = outwash
at = ablation till</td><td>13,500 yrs Be</td></tr>
<tr><td colspan="2">t = till (lodgment)
a = advance outwash
gl = glaciolacustrine</td><td></td></tr>
<tr><td rowspan="2">p = Pre-Vashon
6 or more glacial and interglacial episodes</td><td>n = nonglacial (interglacial)</td><td>l = lacustrine
nm = mudline</td><td>15,000 yrs Be</td></tr>
<tr><td>g = glacial</td><td>l = lacustrine
o = outwash
m = marine
t = till (lodgment)</td><td></td></tr>
</table>

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.

NOTE

The description of each geologic unit includes only general information regarding the environment of deposition and basic soil characteristics.

Figure A-2 - Geologic Unit Explanation



LEGEND

- * Sample Not Recovered
- G Grab Sample
- Standard Penetration Test
- Thin Wall Sample
- ∇ Ground Water Level ATD

NOTES

1. The boring was performed using HSA and Rotary Combined drilling methods.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
5. Refer to KEY for explanation of symbols, codes and definitions.
6. USCS designation is based on visual-manual classification and selected lab testing.

Magnolia Bridge Replacement
Seattle, Washington

LOG OF BORING D-1

November 2003 21-1-09759-002

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FIG. A-3
Sheet 1 of 3

MASTER_LOG2 21-09759.GPJ SHAN_WIL.GDT 11/12/03 Log: WPL Rev: WDN Typ: EET

Figure A-3, Sheet 1 of 3
Log of Boring D-1

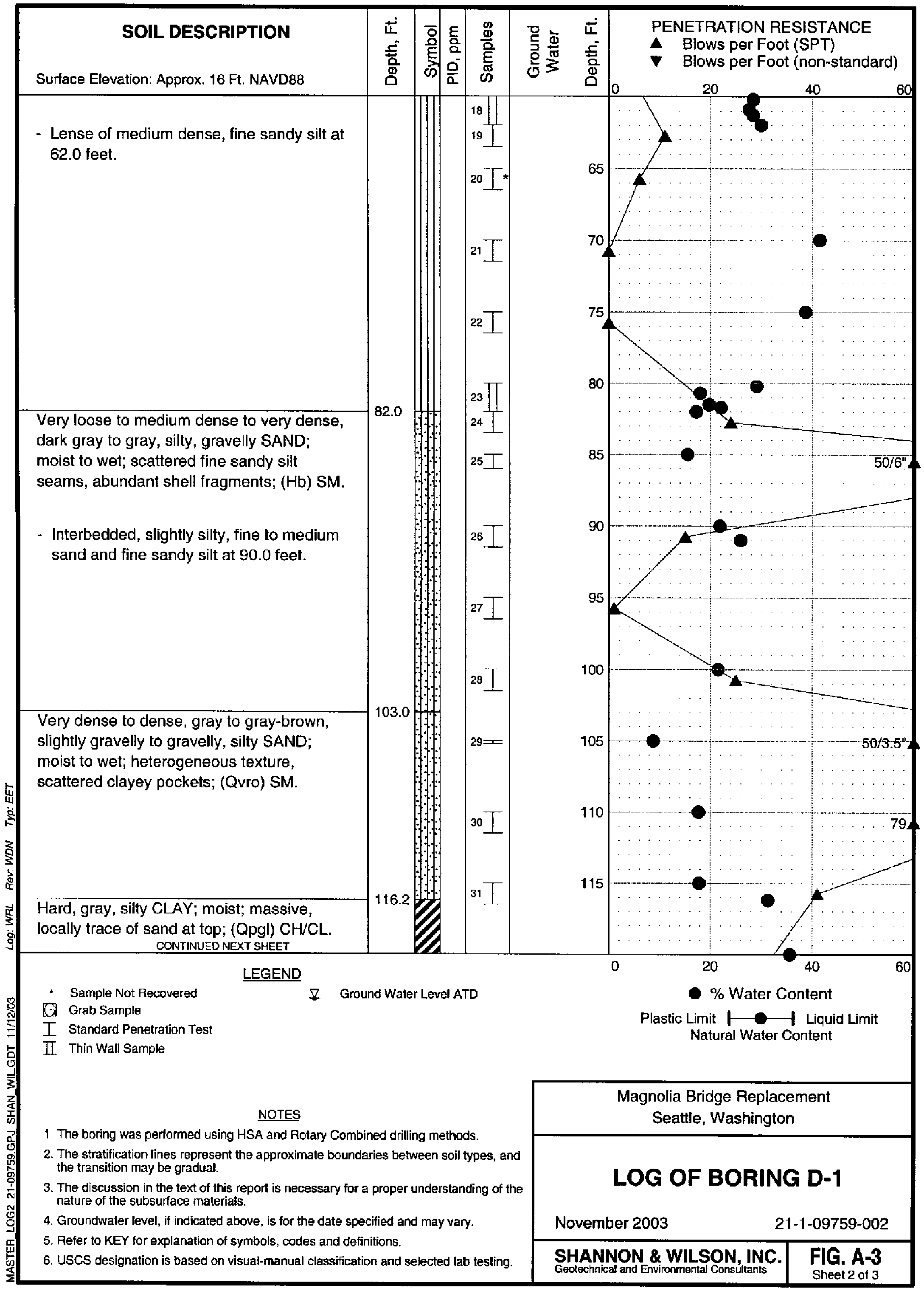


Figure A-3, Sheet 2 of 3
Log of Boring D-1

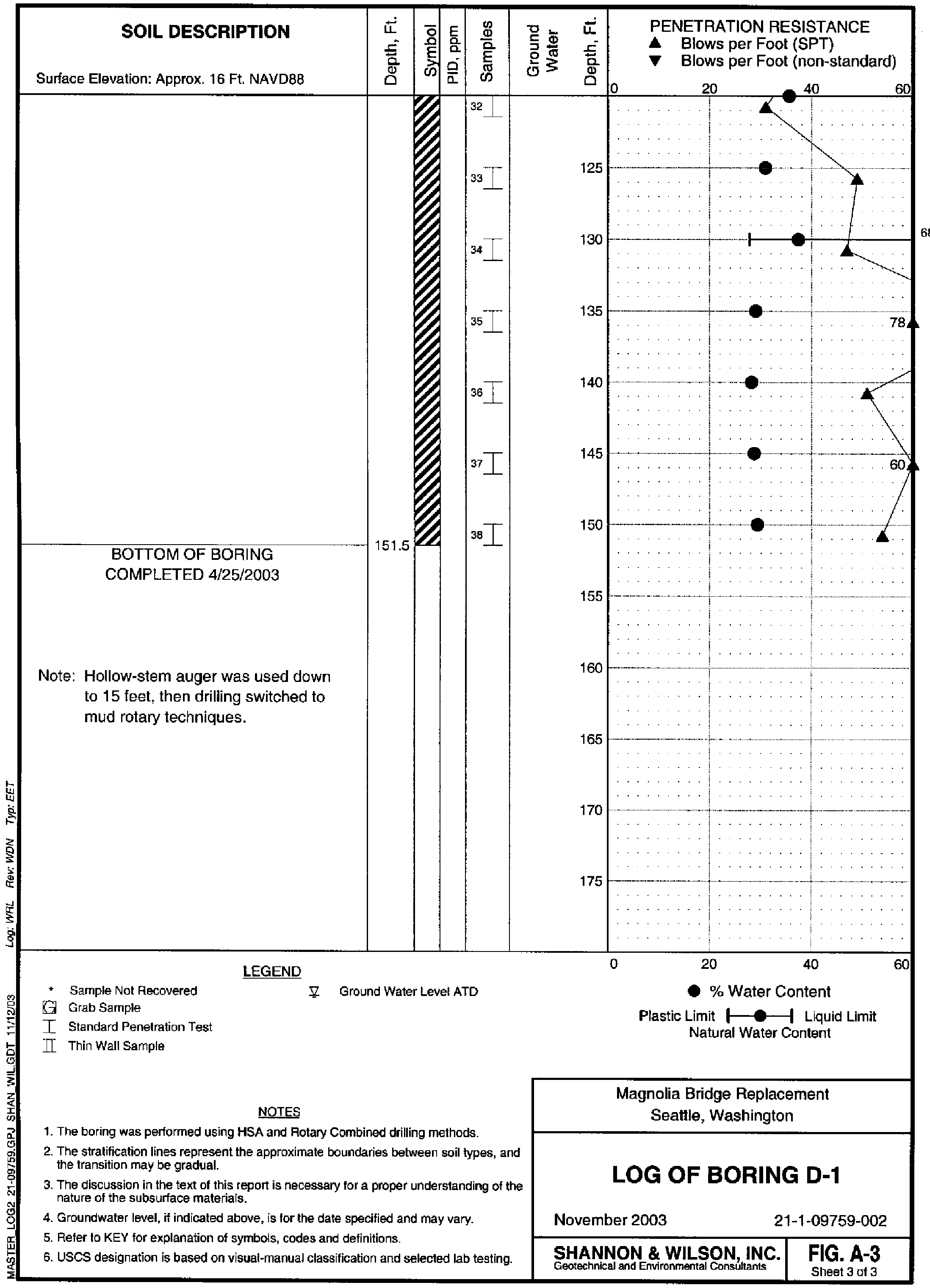


Figure A-3, Sheet 3 of 3
Log of Boring D-1

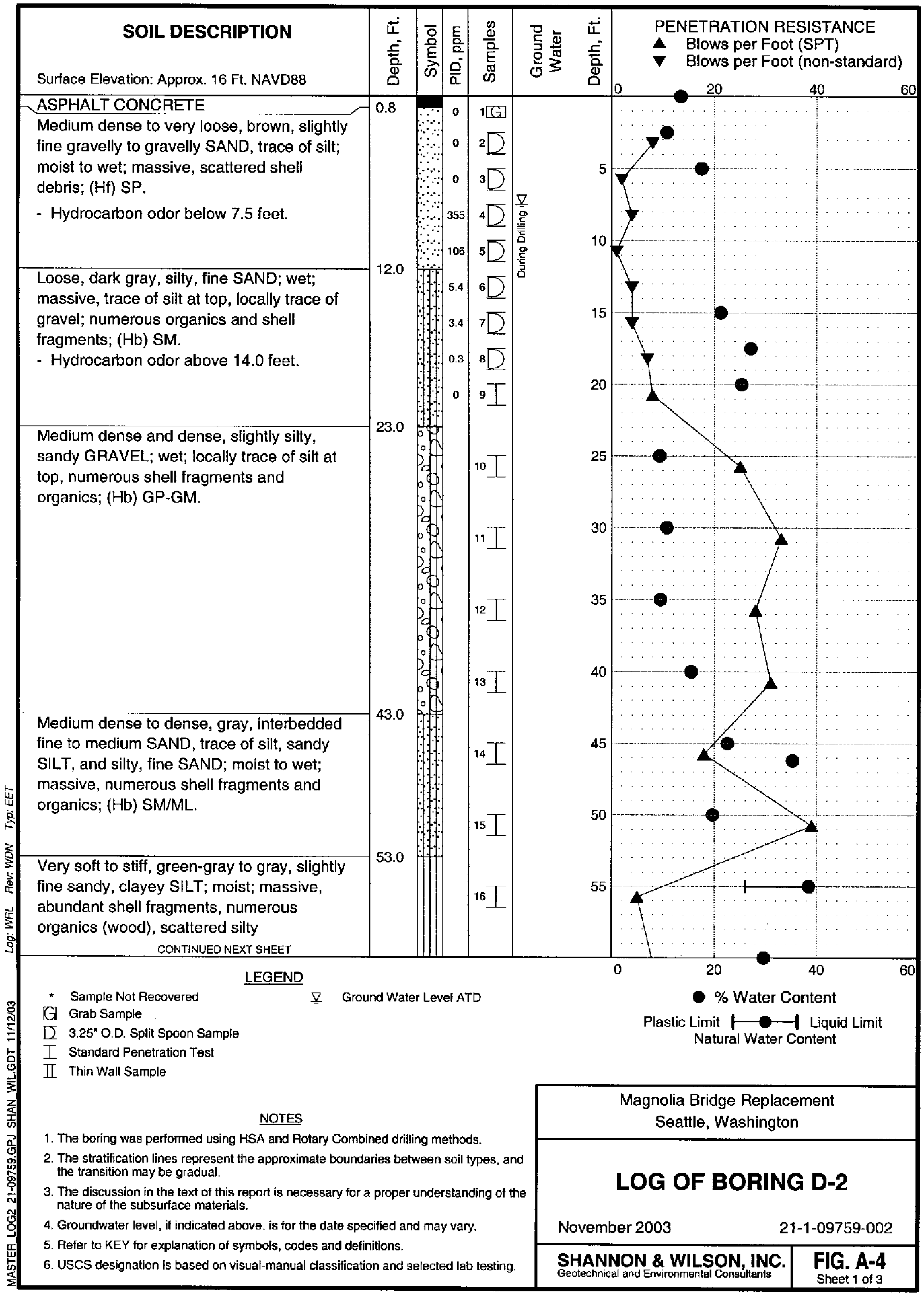


Figure A-4, Sheet 1 of 3
Log of Boring D-2

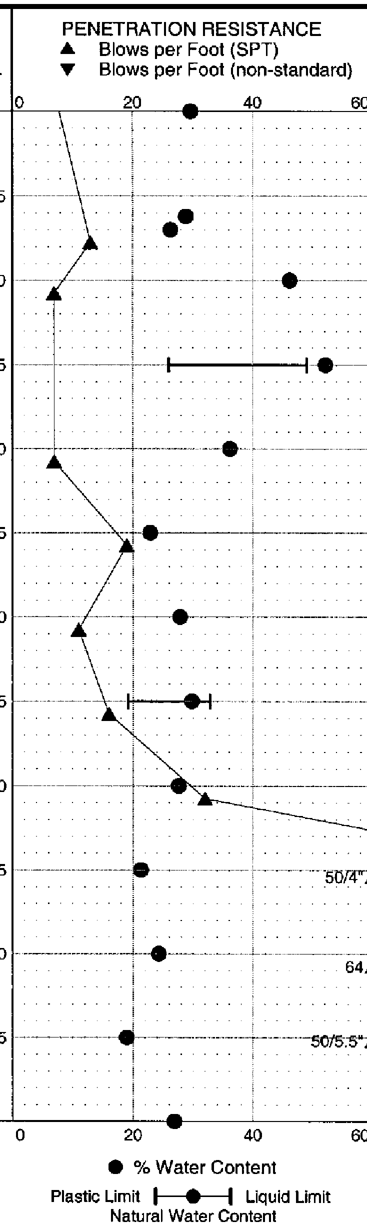
SOIL DESCRIPTION

Surface Elevation: Approx. 16 Ft. NAVD88

Soil Description	Depth, Ft.	Samples
sand and silty clay seams; (He) ML/CL.		17
- Very soft material in sample S-18.		18, 19
- Medium dense, silty, fine sand from 67.0 to 68.5 feet.		20, 21, 22
- Medium dense, silty, fine to medium sand below 85.0 feet.		23
Stiff to very stiff, olive-green, clayey, fine sandy SILT to dense, interbedded, slightly silty, fine SAND and hard, silty CLAY; moist; scattered iron-oxide staining at top; (He) ML/SP-SM.	88.0	24, 25, 26
Very dense, brown to gray-brown, interbedded, slightly silty and silty, fine SAND, fine sandy SILT, and slightly clayey SILT; moist to wet; scattered gravelly pockets, scattered till-like seams; (Qvro) SM/ML.	105.0	27, 28, 29
Hard, gray, silty CLAY; moist; massive to	117.0	

CONTINUED NEXT SHEET

Symbol | PID, ppm | Ground Water | Depth, Ft.



LEGEND

- * Sample Not Recovered
- Grab Sample
- 3.25" O.D. Split Spoon Sample
- Standard Penetration Test
- Thin Wall Sample
- Ground Water Level ATD

NOTES

1. The boring was performed using HSA and Rotary Combined drilling methods.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
5. Refer to KEY for explanation of symbols, codes and definitions.
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Magnolia Bridge Replacement
Seattle, Washington

LOG OF BORING D-2

November 2003 21-1-09759-002

SHANNON & WILSON, INC.
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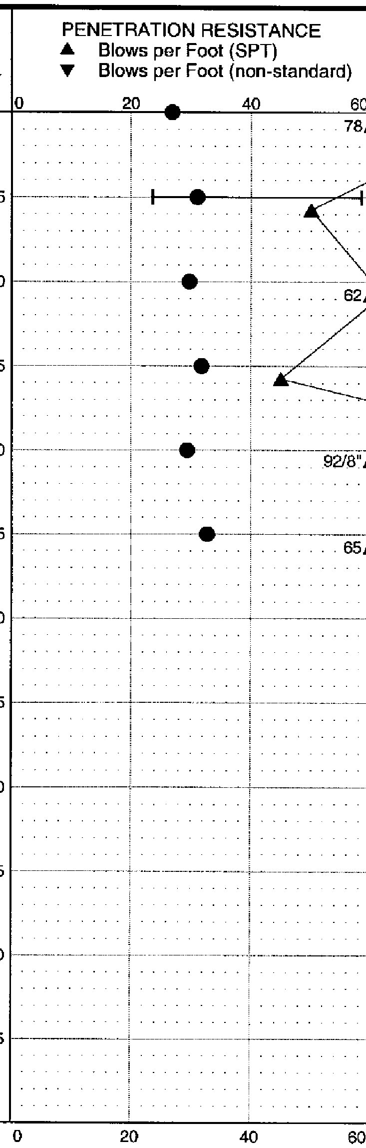
FIG. A-4
Sheet 2 of 3

MASTER LOG2 21-09759.GPJ SHAN WIL.GDT 11/12/03 Log: WRL Rev: WDN Typ: EET

Figure A-4, Sheet 2 of 3
Log of Boring D-2

SOIL DESCRIPTION Surface Elevation: Approx. 16 Ft. NAVD88	Depth, Ft.	Symbol	PID, ppm	Samples	Ground Water	Depth, Ft.
laminated, scattered dropstones, scattered sandy pockets, scattered sheared seams with high-plasticity clay; (Qpgl) CH/CL.				30, 31, 32, 33, 34, 35		125, 130, 135, 140, 145
BOTTOM OF BORING COMPLETED 5/7/2003	146.5					150, 155, 160, 165, 170, 175

Notes:
1. Hollow-stem auger was used down to 20 feet, then drilling switched to mud rotary techniques.
2. Non-standard blow counts have been converted to approximate SPTs.



LEGEND

- * Sample Not Recovered
- Grab Sample
- 3.25" O.D. Split Spoon Sample
- Standard Penetration Test
- Thin Wall Sample
- Ground Water Level ATD

NOTES

1. The boring was performed using HSA and Rotary Combined drilling methods.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
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Magnolia Bridge Replacement
Seattle, Washington

LOG OF BORING D-2

November 2003 21-1-09759-002

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FIG. A-4
Sheet 3 of 3

MASTER_LOG2 21-09759.GPJ SHAN_WIL.GDT 11/12/03 Log: WRL Rev: WDN Typ: EET

Figure A-4, Sheet 3 of 3
Log of Boring D-2

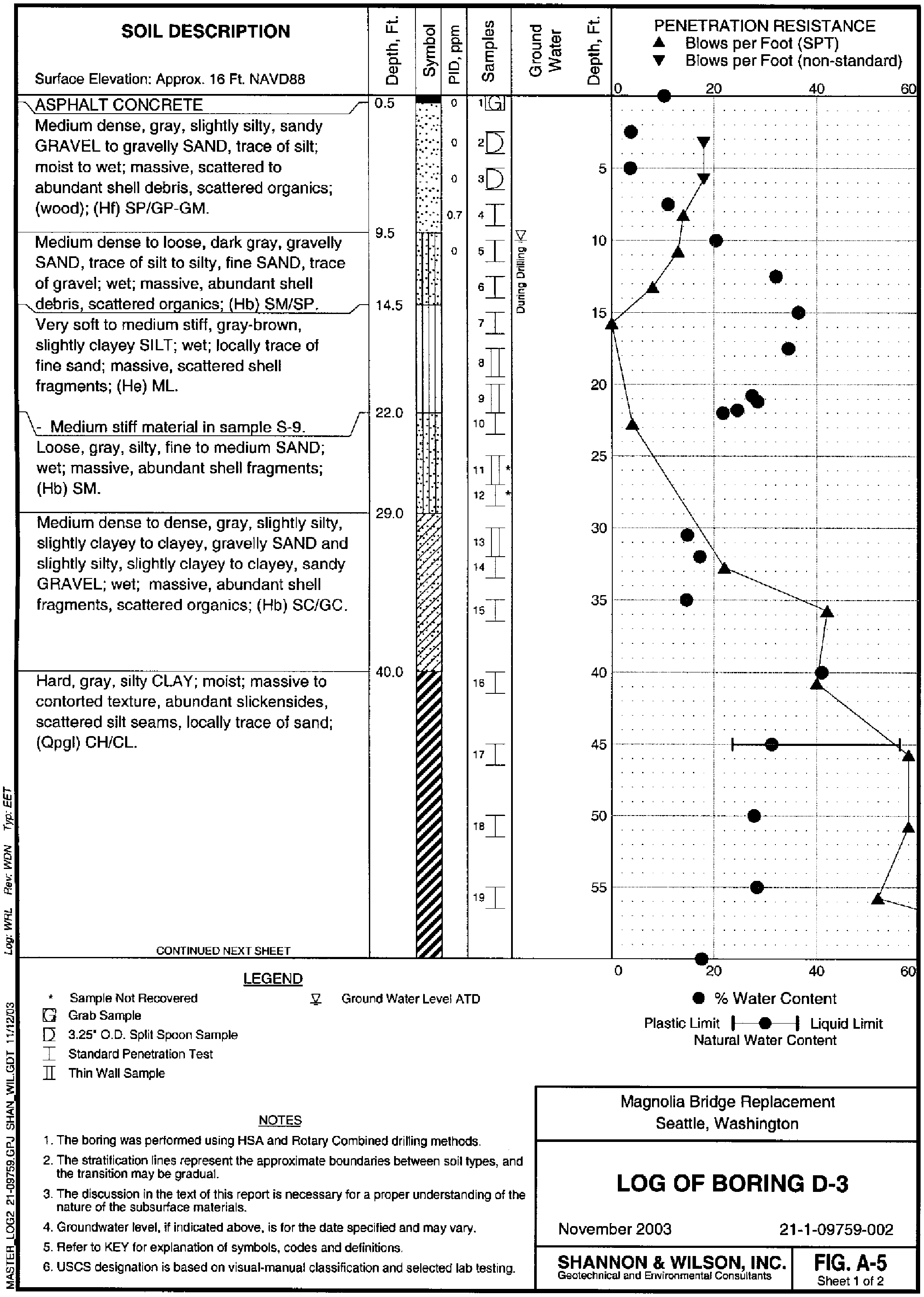


Figure A-5, Sheet 1 of 2
Log of Boring D-3

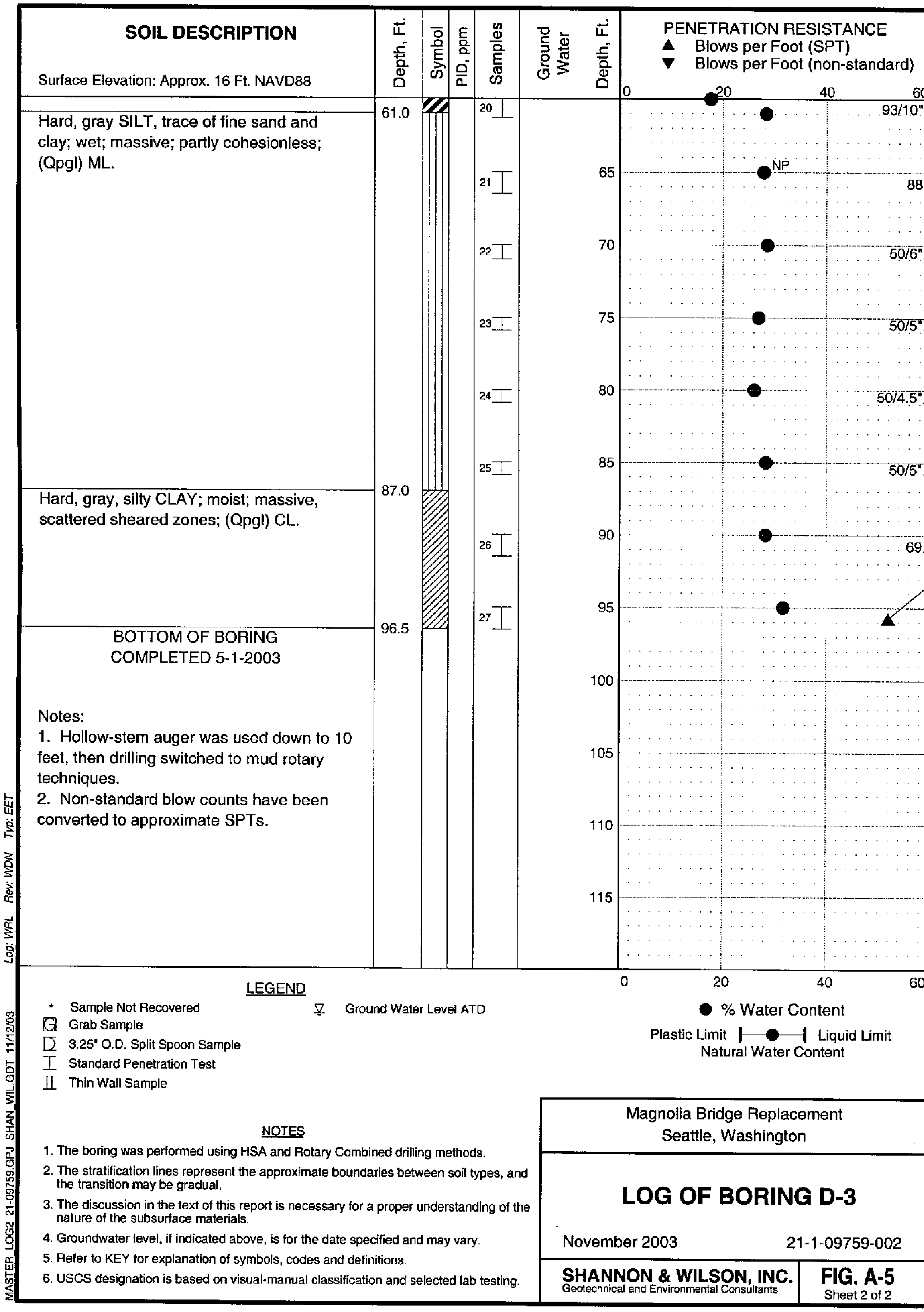


Figure A-5, Sheet 2 of 2
Log of Boring D-3

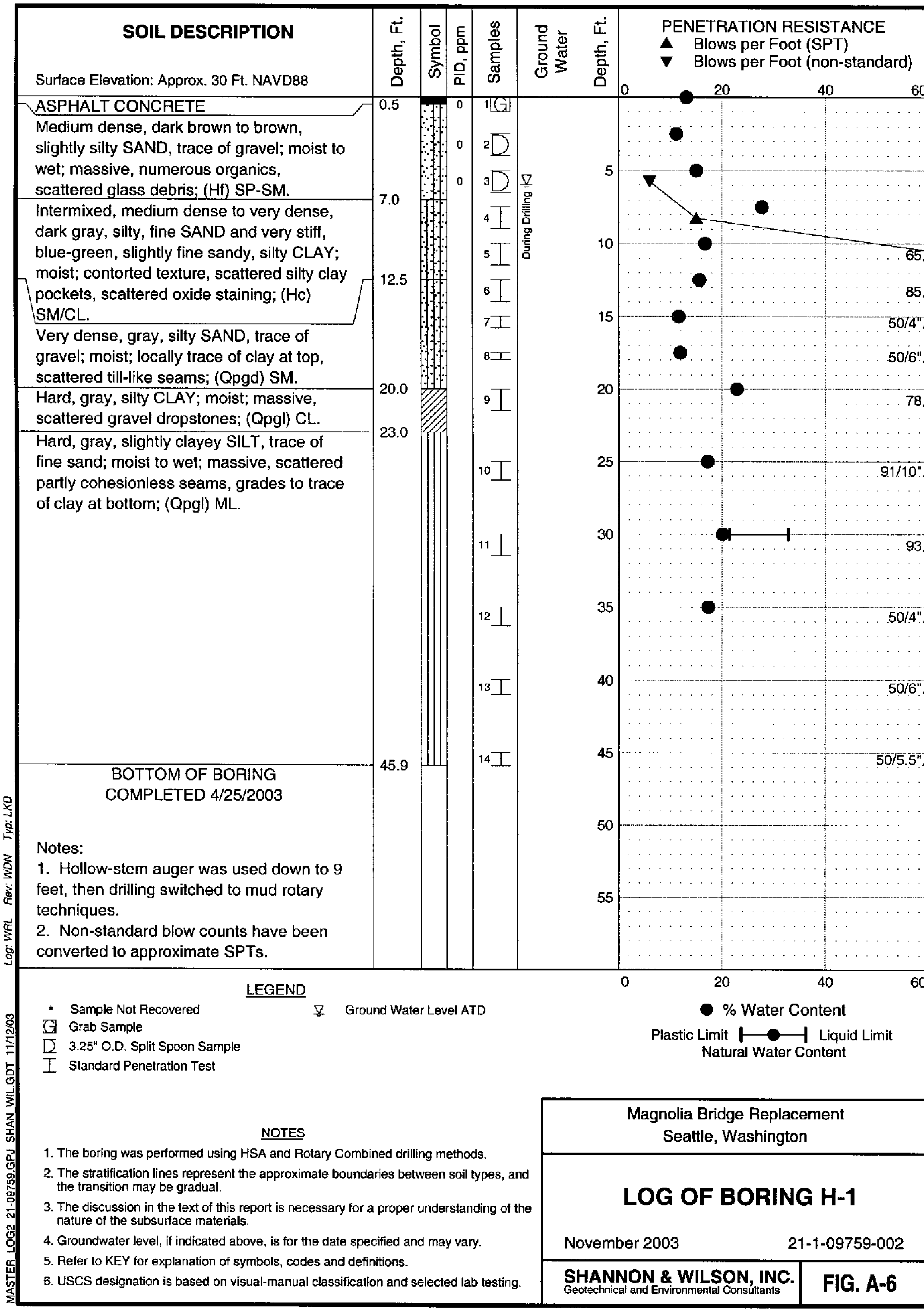


Figure A-6
Log of Boring H-1

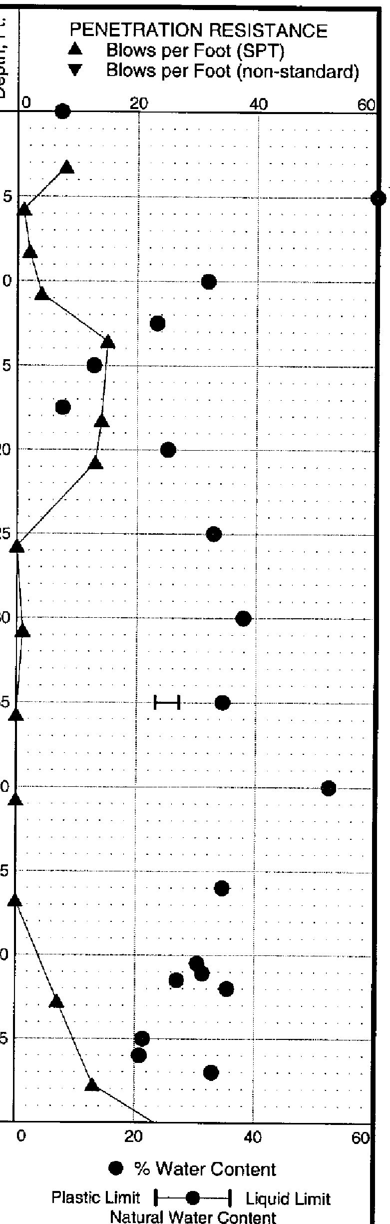
SOIL DESCRIPTION Surface Elevation: Approx. 18 Ft. NAVD88	Depth, Ft.
Loose, dark brown to brown, silty, sandy GRAVEL to silty, fine to medium SAND; moist to wet; massive, abundant organics; (Hf) GM/SM.	4.0
Very loose to medium dense, dark brown to dark gray, slightly gravelly to gravelly, silty SAND; wet; abundant metal and glass debris; (Hf - Landfill debris) SM.	14.5
Medium dense, dark gray, sandy GRAVEL, trace of silt; wet; massive, scattered organics; (Hb) GW/GP-GM.	19.5
Stiff to very soft, gray SILT, trace of fine sand and clay; wet; layer of medium dense, fine sandy silt at top; abundant shell fragments and organics; (He) ML.	48.5
Loose, gray-brown to green-gray, silty SAND; moist; locally trace of clay, bedded, abundant organic-rich seams; (Hb) SM.	57.0
Medium stiff to very stiff, green-gray to blue-green, silty CONTINUED NEXT SHEET	

Symbol | PID, ppm | Samples | Ground Water | Depth, Ft.

PID, ppm: 0, 0, 0, 0, 0

Samples: 1 G, 2, 3, 4, 5, 6, 7 D, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19

Ground Water: During Drilling ∇



LEGEND

- * Sample Not Recovered
- G Grab Sample
- Standard Penetration Test
- D 3.25" O.D. Split Spoon Sample
- Thin Wall Sample
- ∇ Ground Water Level ATD

NOTES

1. The boring was performed using HSA and Rotary Combined drilling methods.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
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Magnolia Bridge Replacement
Seattle, Washington

LOG OF BORING H-2

November 2003 21-1-09759-002

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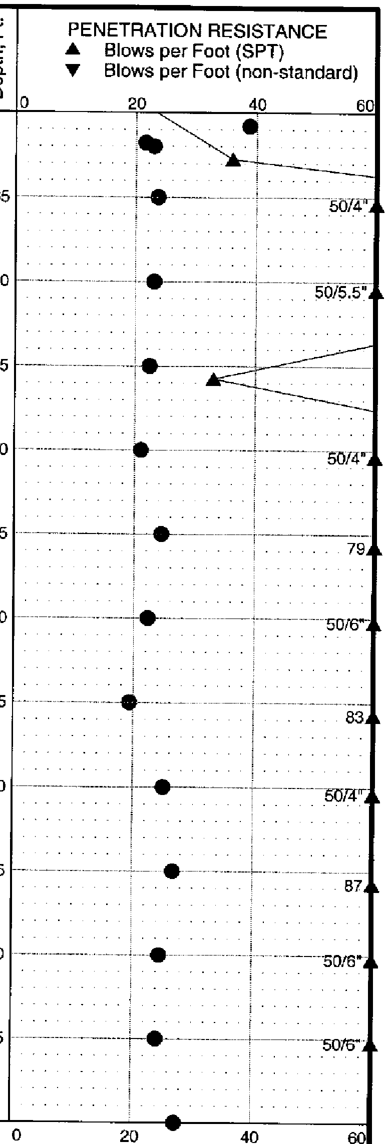
FIG. A-7
Sheet 1 of 3

MASTER_LOG2 21-09759.GPJ SHAN_WIL.GDT 11/12/03 Log: WRL Rev: WDN Typ: EET

Figure A-7, Sheet 1 of 3
Log of Boring H-2

SOIL DESCRIPTION Surface Elevation: Approx. 18 Ft. NAVD88	Depth, Ft.	Samples
CLAY grading to dense, slightly fine sandy SILT; moist; massive, scattered organics at bottom; (He) CH/ML.	62.2	20, 21
Very dense to dense, gray, slightly silty, fine SAND; moist to wet; massive, scattered to abundant organic fragments, scattered seams of soft silty clay; (Hrw) SP-SM/SP.		22, 23, 24
Very dense, gray, slightly silty to silty, fine SAND; moist to wet; massive, scattered very fine organic fragments; (Qpnf) SM/SP-SM.	78.0	25, 26, 27, 28
Very dense, gray SILT, trace of clay and fine sand; wet; seam of hard, slightly clayey silt at top; abundant very fine organic fragments; partly cohesionless, scattered shell fragments at bottom; (Qpnl) ML.	98.0	29, 30, 31, 32

CONTINUED NEXT SHEET



LEGEND

- * Sample Not Recovered
- Grab Sample
- Standard Penetration Test
- 3.25" O.D. Split Spoon Sample
- Thin Wall Sample
- Ground Water Level ATD

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit
Natural Water Content

NOTES

1. The boring was performed using HSA and Rotary Combined drilling methods.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
5. Refer to KEY for explanation of symbols, codes and definitions.
6. USCS designation is based on visual-manual classification and selected lab testing.

MASTER_LOG2 21-09759.GPJ SHAN_WIL.GDT 11/12/03 Log: WRL Rev: WDN Typ: EET

Magnolia Bridge Replacement
Seattle, Washington

LOG OF BORING H-2

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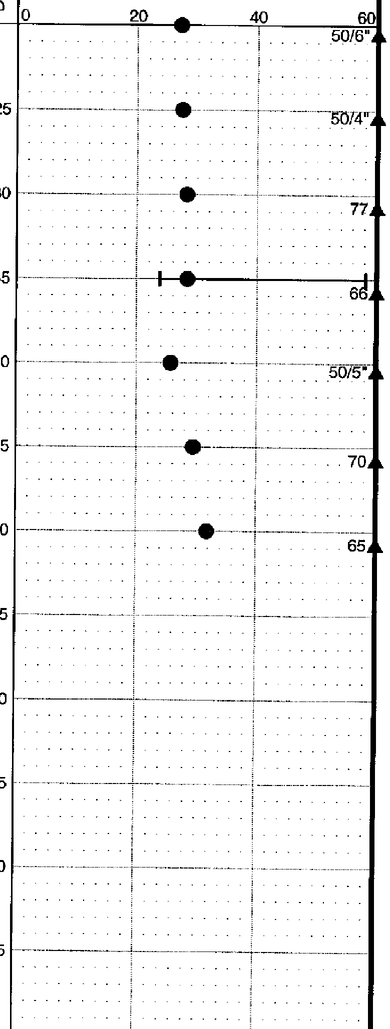
FIG. A-7
Sheet 2 of 3

Figure A-7, Sheet 2 of 3
Log of Boring H-2

SOIL DESCRIPTION Surface Elevation: Approx. 18 Ft. NAVD88	Depth, Ft.	Symbol	PID, ppm	Samples	Ground Water	Depth, Ft.
				33		
				34		125
Hard, gray, silty CLAY; moist; contorted bedding; (Qpgl) CH/CL.	128.0			35		130
				36		135
Hard, gray, slightly clayey SILT; moist; massive; (Qpgl) ML.	138.0			37		140
Hard, gray, silty CLAY; moist; contorted texture, scattered slickensides; (Qpgl) CL/CH.	143.0			38		145
				39		150
BOTTOM OF BORING COMPLETED 4/29/2003 Note: Hollow-stem auger was used down to 20 feet, then drilling switched to mud rotary techniques.	151.5					155
						160
						165
						170
						175

PENETRATION RESISTANCE
▲ Blows per Foot (SPT)
▼ Blows per Foot (non-standard)



LEGEND

- * Sample Not Recovered
- Grab Sample
- Standard Penetration Test
- 3.25" O.D. Split Spoon Sample
- Thin Wall Sample
- Ground Water Level ATD

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit
Natural Water Content

NOTES

1. The boring was performed using HSA and Rotary Combined drilling methods.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
5. Refer to KEY for explanation of symbols, codes and definitions.
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Magnolia Bridge Replacement
Seattle, Washington

LOG OF BORING H-2

November 2003 21-1-09759-002

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FIG. A-7
Sheet 3 of 3

MASTER_LOG2 21-09759.GPJ SHAN_WIL.GDT 11/12/03 Log: WRL Rev: WDN Typ: EET

Figure A-7, Sheet 3 of 3
Log of Boring H-2

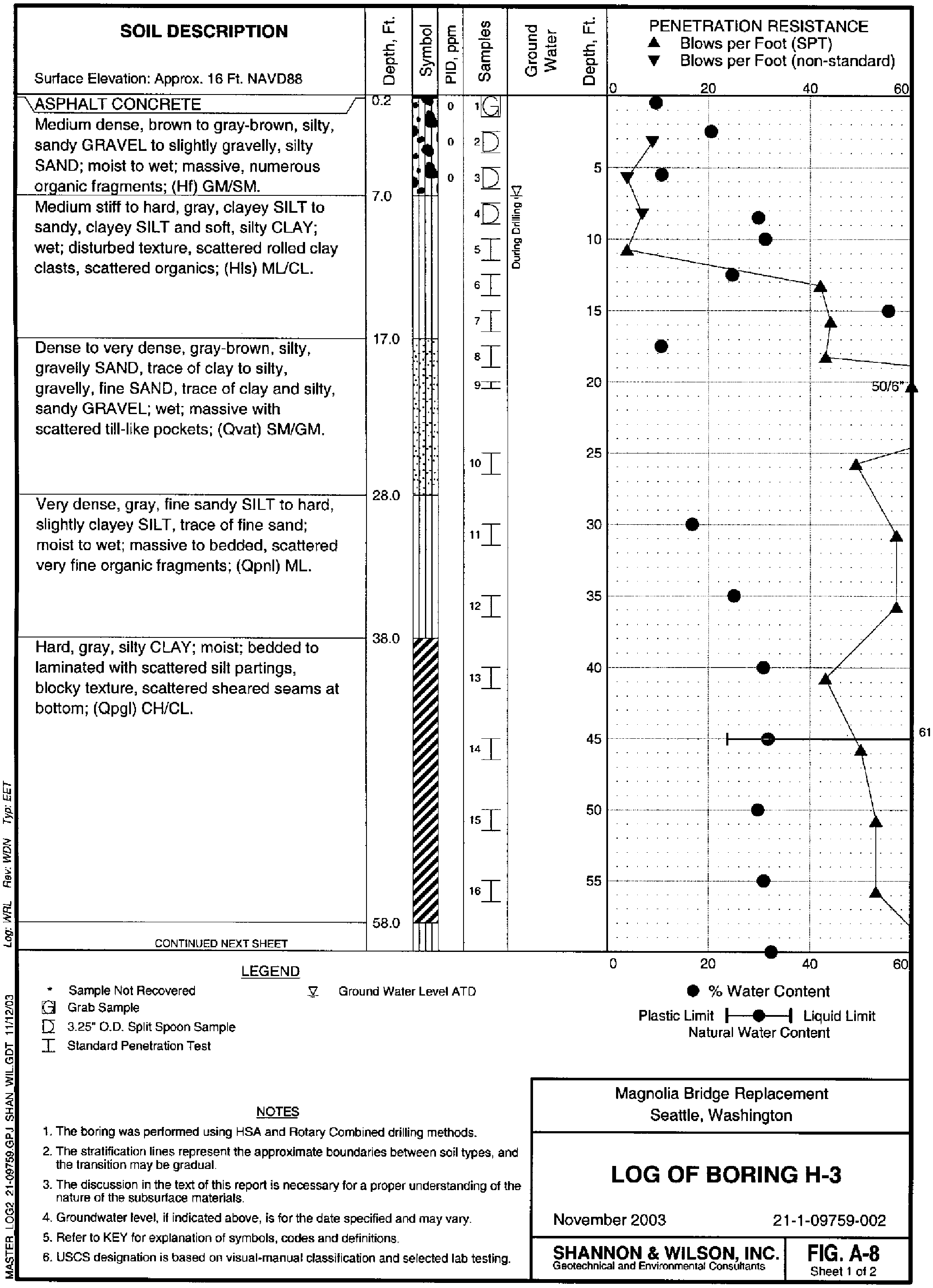
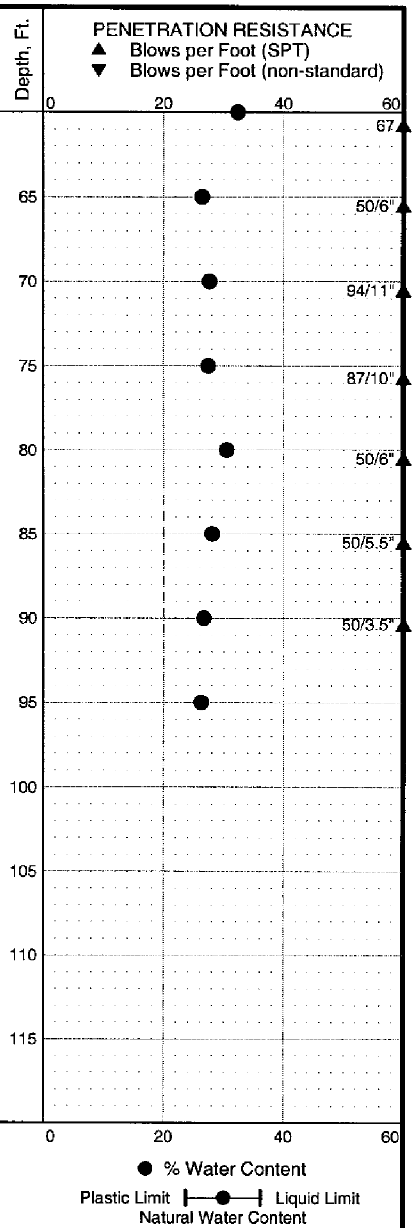


Figure A-8, Sheet 1 of 2
Log of Boring H-3

SOIL DESCRIPTION Surface Elevation: Approx. 16 Ft. NAVD88	Depth, Ft.	Symbol	PID, ppm	Samples	Ground Water
Very dense, gray SILT, trace of fine sand and clay; wet; massive to bedded, scattered to numerous very fine organic fragments, becomes slightly sandy at bottom; (Qpnl) ML. - Lense of gray, silty clay from 75.0 to 75.8 feet.				17, 18, 19, 20, 21, 22, 23	
BOTTOM OF BORING COMPLETED 5/5/2003	90.7				

Notes:
1. Hollow-stem auger was used down to 10 feet, then drilling switched to mud rotary techniques.
2. Non-standard blow counts have been converted to approximate SPTs.



LEGEND

- * Sample Not Recovered
- Grab Sample
- 3.25" O.D. Split Spoon Sample
- Standard Penetration Test
- Ground Water Level ATD

NOTES

1. The boring was performed using HSA and Rotary Combined drilling methods.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
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Magnolia Bridge Replacement
Seattle, Washington

LOG OF BORING H-3

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FIG. A-8
Sheet 2 of 2

Log: WRL Rev: WDN Typ: EET

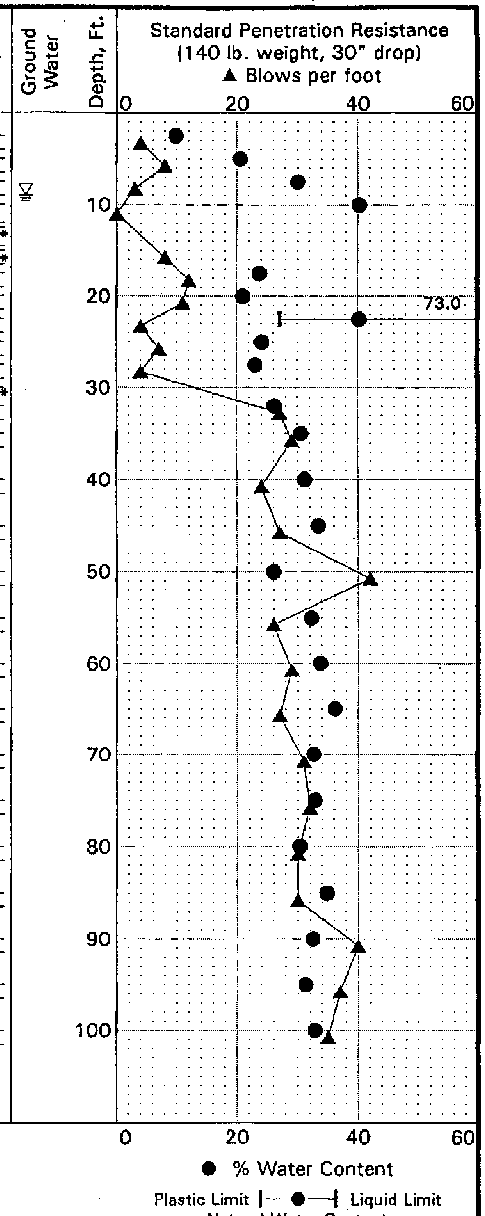
MASTER_LOG2 21-09759.GPJ SHAN_WIL.GDT 11/12/03

Figure A-8, Sheet 2 of 2
Log of Boring H-3

MASTERLG 12/1/94

MATERIAL DESCRIPTION Surface Elev: Approx. 3 ft (City of Seattle Datum)	Depth, Ft.
Asphalt paving.	0.8
Concrete paving.	1.2
Very loose to loose, brown, slightly fine, gravelly, silty, fine SAND; moist; trace of debris; some iron-oxide staining; (Fill).	6.5
Soft, gray CLAY; moist; trace of fine sand; trace of iron-oxide staining; occasional pieces of wood; (Fill?).	8.0
Very loose to medium dense, gray, slightly silty to silty, fine SAND; wet; occasional fine gravel; slightly fine sandy SILT from 10 to 12 feet; trace organics from 20 feet; (Fill?).	21.5
Soft, gray and brown mottled, trace to slightly sandy CLAY; moist to wet; trace organics; CH.	24.0
Very loose to loose, gray, slightly gravelly, sandy SILT; moist to wet; organics; scattered shell fragments; ML.	32.0
Very stiff to hard, silty CLAY and clayey SILT; moist; occasional fine sand partings; 1/2-inch-thick disturbed zone between 45 and 46.5 feet; CL/ML.	
BOTTOM OF BORING COMPLETED 11/10/93	101.5

Samples: 1–27



LEGEND

- * Sample Not Recovered
- 2" O.D. Split Spoon Sample
- 3" O.D. Shelby Tube Sample
- Surface Seal
- Annular Sealant
- Piezometer Screen
- Grout
- Water Level

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.
4. Refer to KEY for explanation of 'Symbols' and definitions.
5. USC letter symbol based on visual classification.

SED Seismic Retrofit Study
Magnolia Extension
Seattle, Washington

LOG OF BORING B-2

November 1994 W-6470-03

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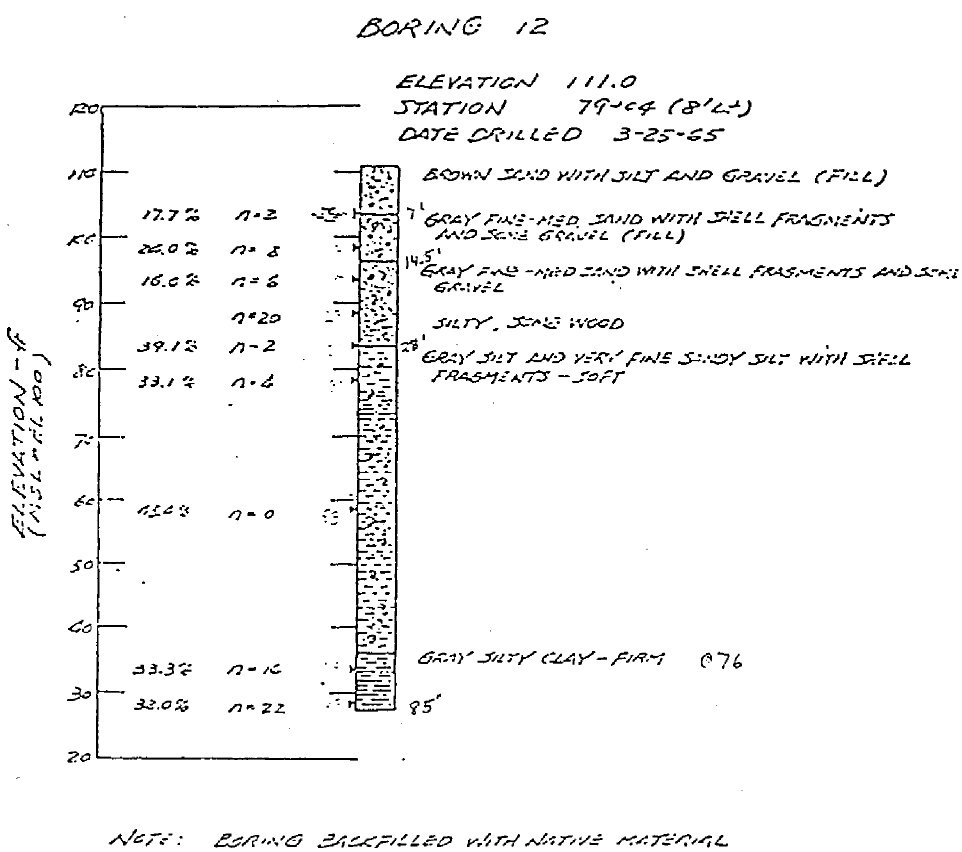
FIG. A-3

Old City of Seattle datum elevation
~3 Ft. + 9.7 Ft. = ~13 Ft. NAVD88.

Figure A-9
Log of Boring 2-2



SED Seismic Retrofit Study
Magnolia Extension
Seattle, Washington

LOG OF PREVIOUS BORING 12

November 1994 W-6470-03

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FIG. A-6

Assume ground surface elevation of ~17 Ft. NAVD88.

Figure A-10
Log of Boring 2-3

MASTERLG 11/29/94

MATERIAL DESCRIPTION Surface Elev: Approx. -5 ft (City of Seattle Datum)	Depth, Ft.	Samples
Asphalt paving.	1.5	
Loose to medium dense, brown-gray, sandy, slightly silty GRAVEL; moist; scattered clam shell fragments; (Fill?).	6.5	1, 2, 3
Loose to medium dense, gray, slightly silty to silty, gravelly SAND; wet; numerous organics and iron-oxide staining from 7.5 feet to 9 feet; numerous clam shell fragments from 10 feet to 14 feet; grades to silty, gravelly SAND and sandy GRAVEL at 10 feet; (Fill?) SP-SM.	14.0	4, 5, 6, 7, 8, 9, 10
Loose to medium dense, gray, silty, fine SAND and ORGANICS (wood debris) to 20 feet, silty, fine SAND with scattered organics and clam shell fragments below 20 feet; wet; occasional thin layer of silty clay with organics; SM.	29.0	11*, 12, 13, 14
Very soft to soft, gray, slightly sandy, slightly gravelly, clayey SILT and silty CLAY; wet; scattered organics and clam shell fragments; decaying organics odor; ML/CL.	41.5	15
Loose, gray, silty, slightly gravelly, fine to medium SAND; wet; scattered clam shell fragments and organics; SM.	46.5	16, 17, 18
Dense to very dense, gray, silty, sandy GRAVEL and gravelly SAND; wet; scattered clam shell fragments from 50 feet to 51.5 feet; GM/SM.		19, 20
Soft to stiff, gray and brown, silty CLAY and slightly clayey to clayey SILT; moist; numerous to scattered clam shell fragments and organics; locally trace to slightly sandy; CH/MH. - Layer of loose, gray, silty SAND and sandy SILT with numerous clam shell fragments and scattered organics from 90 feet to 91.5 feet.	70.5	21, 22, 23, 24, 25
	98.5	

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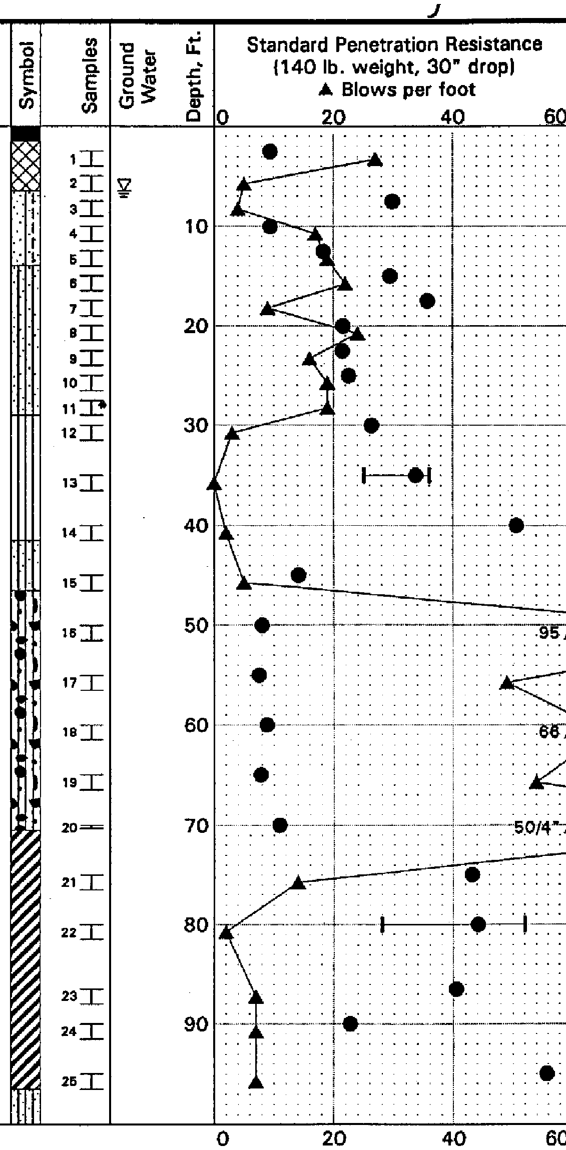

LEGEND

- • Sample Not Recovered
- 2" O.D. Split Spoon Sample
- 3" O.D. Shelby Tube Sample
- Surface Seal
- Annular Sealant
- Piezometer Screen
- Grout
- Water Level

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
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4. Refer to KEY for explanation of 'Symbols' and definitions.
5. USC letter symbol based on visual classification.

SED Seismic Retrofit Study
Magnolia Viaduct
Seattle, Washington

LOG OF BORING B-1

November 1994 W-6470-08

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FIG. A-4
Sheet 1 of 2

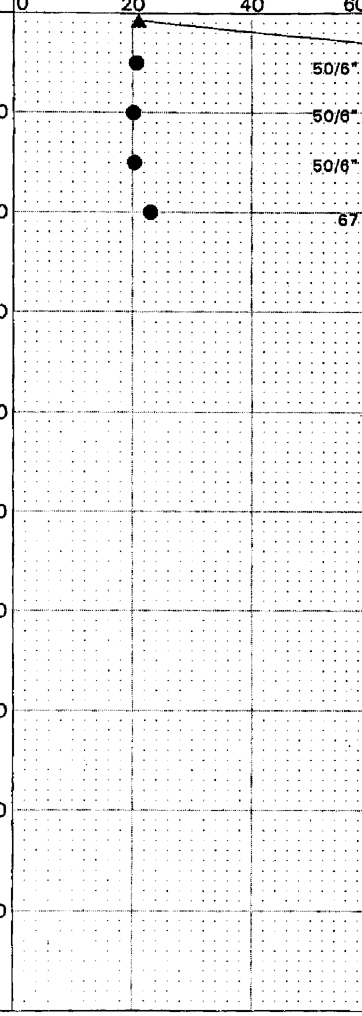
Assume ground surface elevation of ~15 Ft. NAVD88.

Figure A-11, Sheet 1 of 2
Log of Boring 2-4

MASTERLG 11/29/94

MATERIAL DESCRIPTION Surface Elev: Approx. -5 ft (City of Seattle Datum)	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.
Medium dense to very dense, gray, silty, fine SAND; moist to wet; locally interlayered with SILT and sandy SILT; SM.			26 *		
			27		
			28		110
			29		
	121.5		30		120
BOTTOM OF BORING COMPLETED 12/7/93					
					130
					140
					150
					160
					170
					180
					190

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



● % Water Content
Plastic Limit ├──●──┤ Liquid Limit
Natural Water Content

LEGEND

- * Sample Not Recovered
- 2" O.D. Split Spoon Sample
- 3" O.D. Shelby Tube Sample
- Surface Seal
- Annular Sealant
- Piezometer Screen
- Grout
- Water Level

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.
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5. USC letter symbol based on visual classification.

SED Seismic Retrofit Study
Magnolia Viaduct
Seattle, Washington

LOG OF BORING B-1

November 1994 W-6470-08

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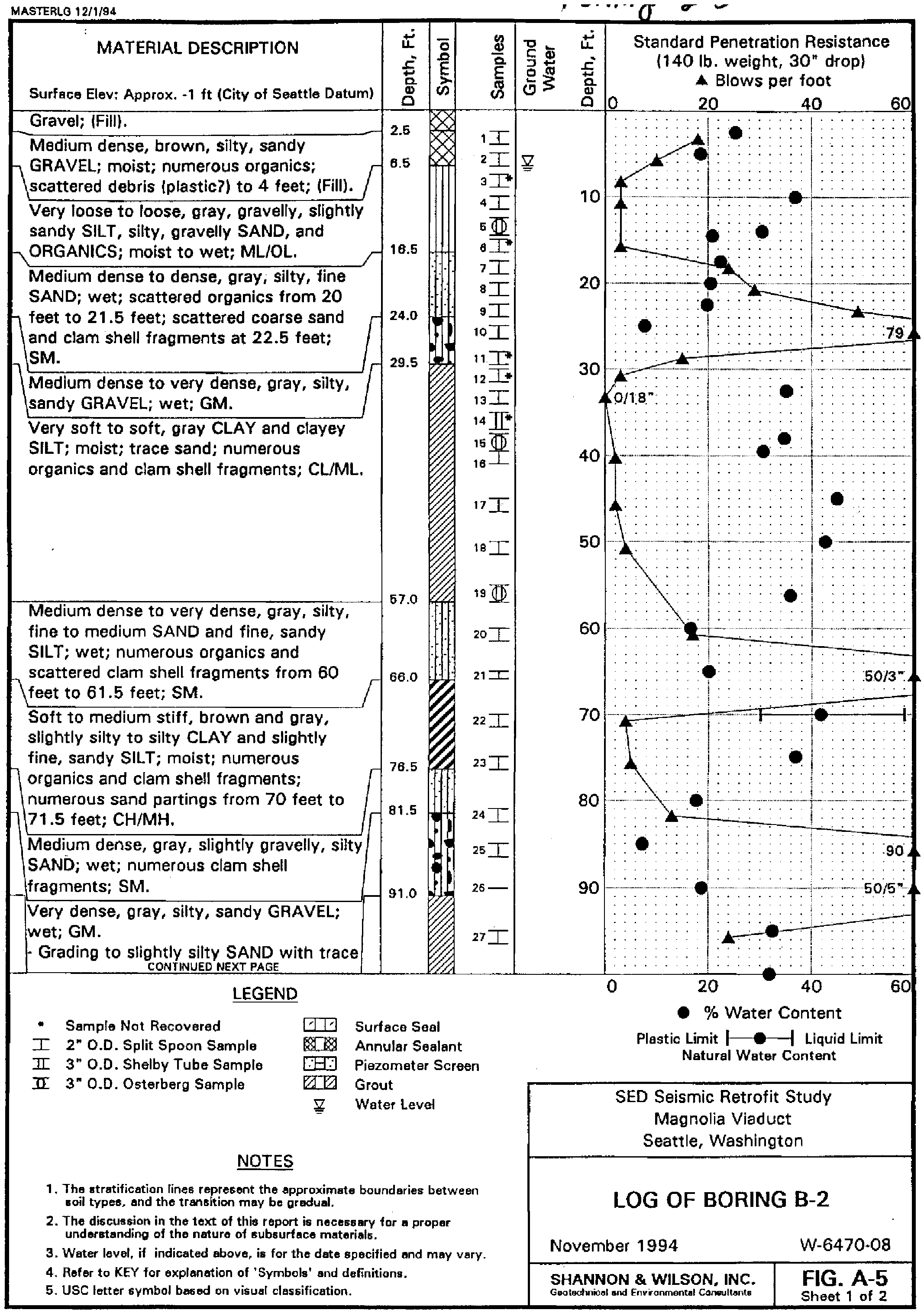
Geotechnical and Environmental Consultants

FIG. A-4
Sheet 2 of 2

Assume ground surface elevation of ~15 Ft. NAVD88.

Figure A-11, Sheet 2 of 2
Log of Boring 2-4



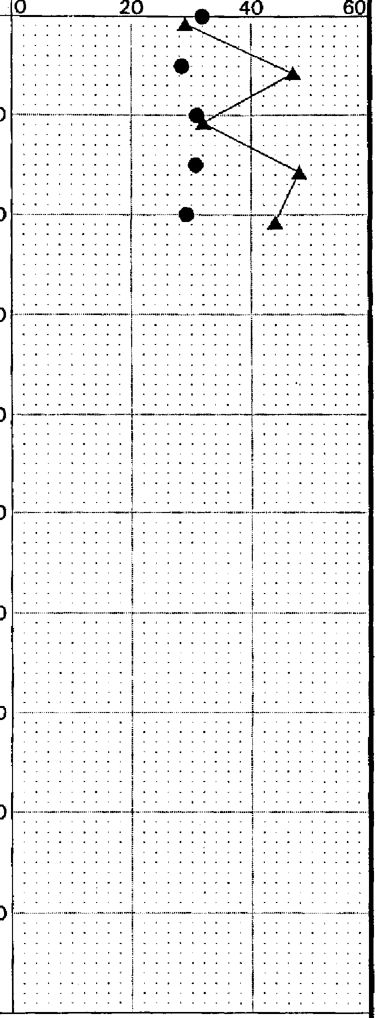
Assume ground surface elevation of ~16 Ft. NAVD88.

Figure A-12, Sheet 1 of 2
Log of Boring 2-5

MASTERLG 12/1/94

MATERIAL DESCRIPTION Surface Elev: Approx. -1 ft (City of Seattle Datum)	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.
gravel at 90 feet.			28		
Hard, gray, silty CLAY and clayey SILT; CL/ML.			29		
			30		110
			31		
	121.5		32		120
BOTTOM OF BORING COMPLETED 12/9/93					
					130
					140
					150
					160
					170
					180
					190

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



● % Water Content

Plastic Limit |—●—| Liquid Limit
Natural Water Content

LEGEND

- * Sample Not Recovered
- 2" O.D. Split Spoon Sample
- 3" O.D. Shelby Tube Sample
- 3" O.D. Osterberg Sample
- Surface Seal
- Annular Sealant
- Piezometer Screen
- Grout
- Water Level

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.
4. Refer to KEY for explanation of 'Symbols' and definitions.
5. USC letter symbol based on visual classification.

SED Seismic Retrofit Study
Magnolia Viaduct
Seattle, Washington

LOG OF BORING B-2

November 1994 W-6470-08

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-5
Sheet 2 of 2

Assume ground surface elevation of ~16 Ft. NAVD88.

Figure A-12, Sheet 2 of 2
Log of Boring 2-5

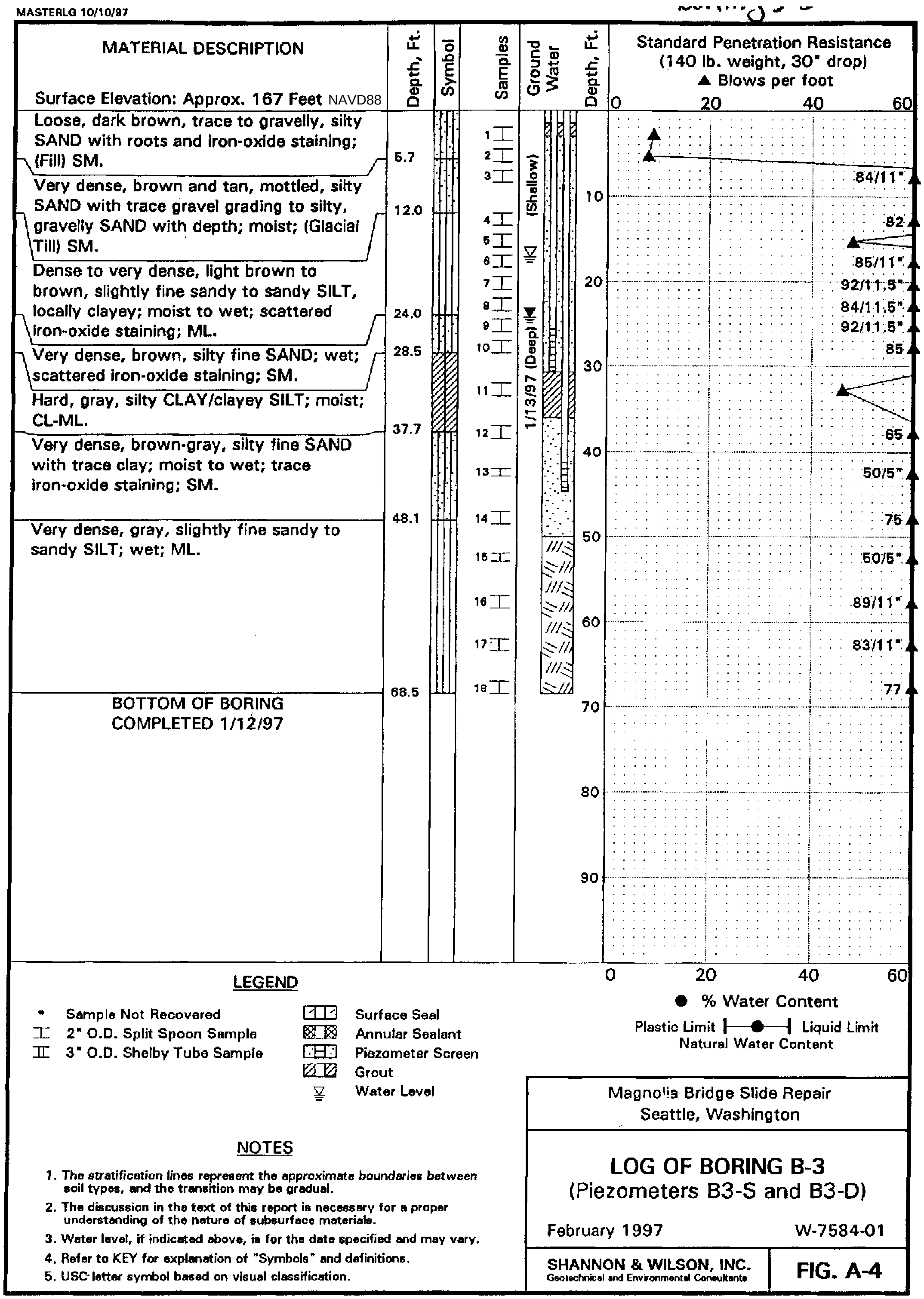


Figure A-13
Log of Boring 3-3

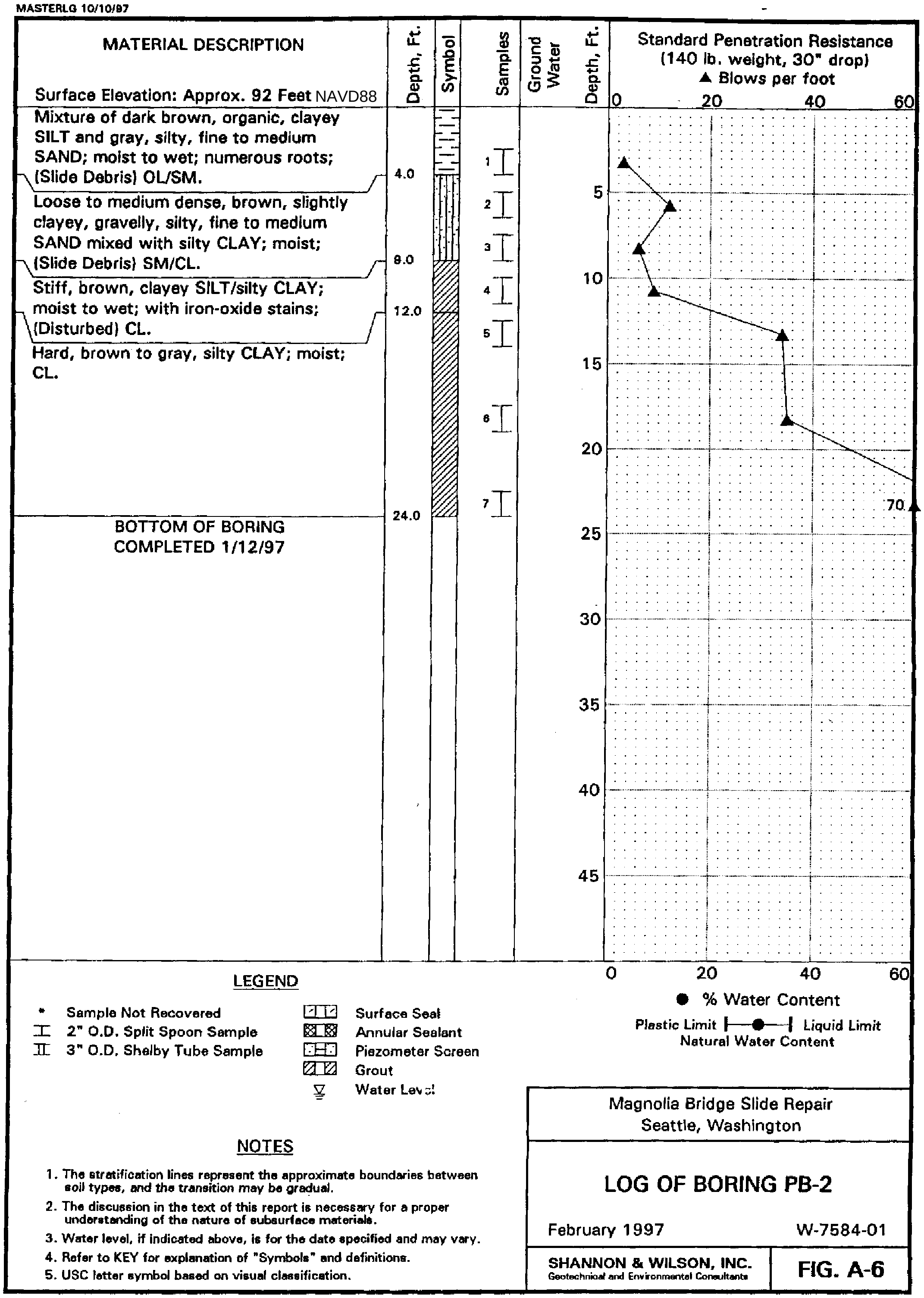


Figure A-14
Log of Boring 3-5

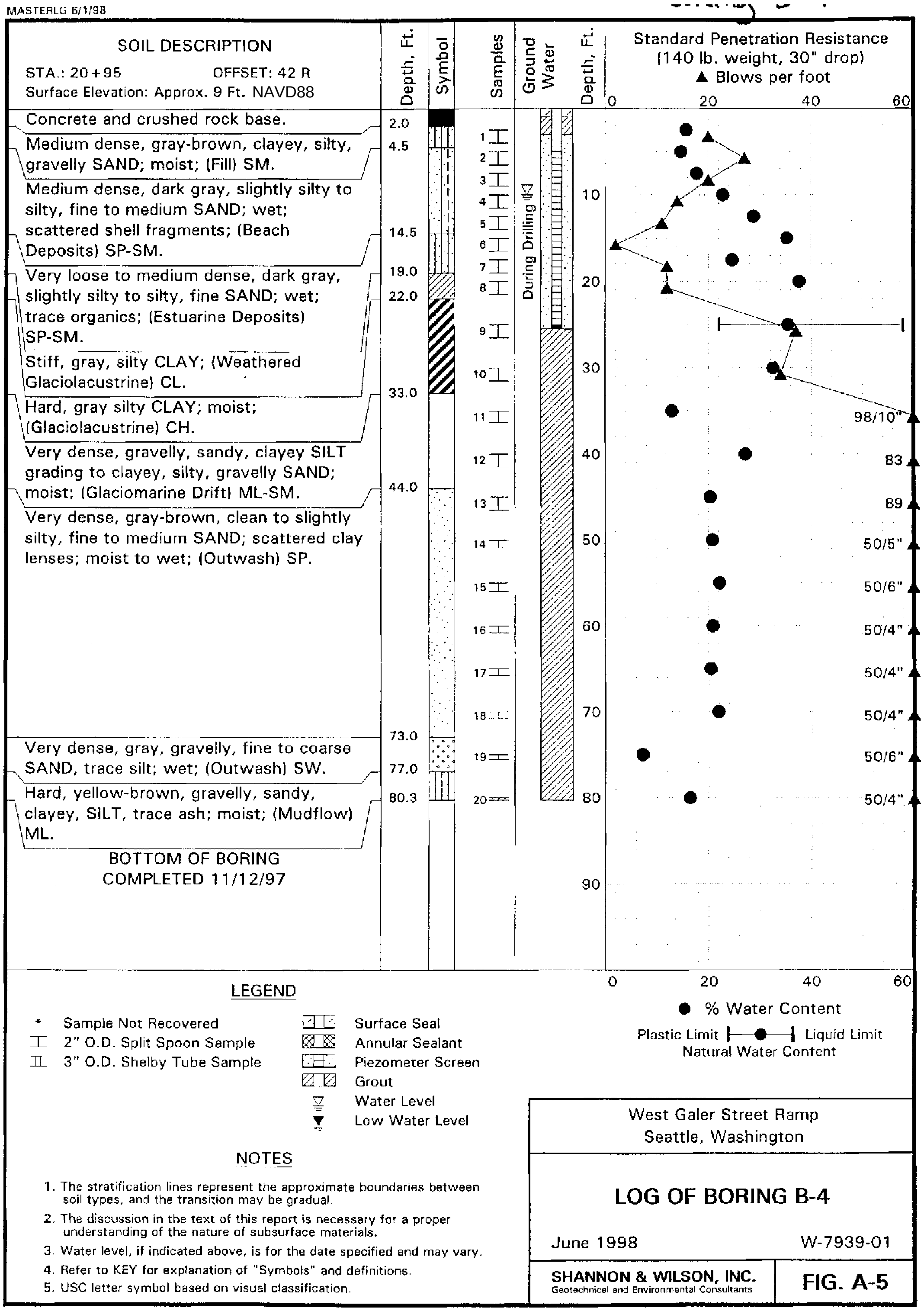


Figure A-15
Log of Boring 5-4

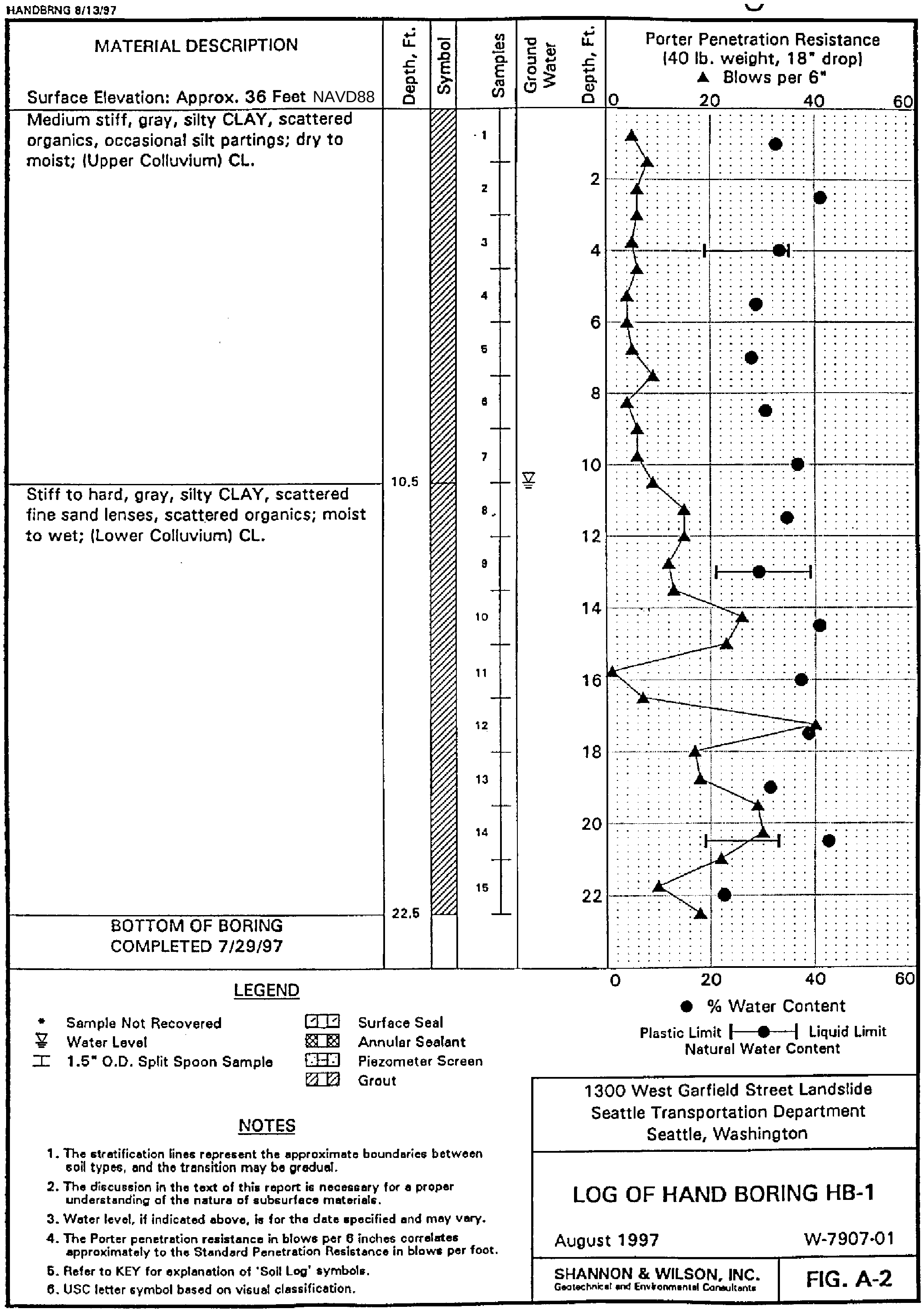
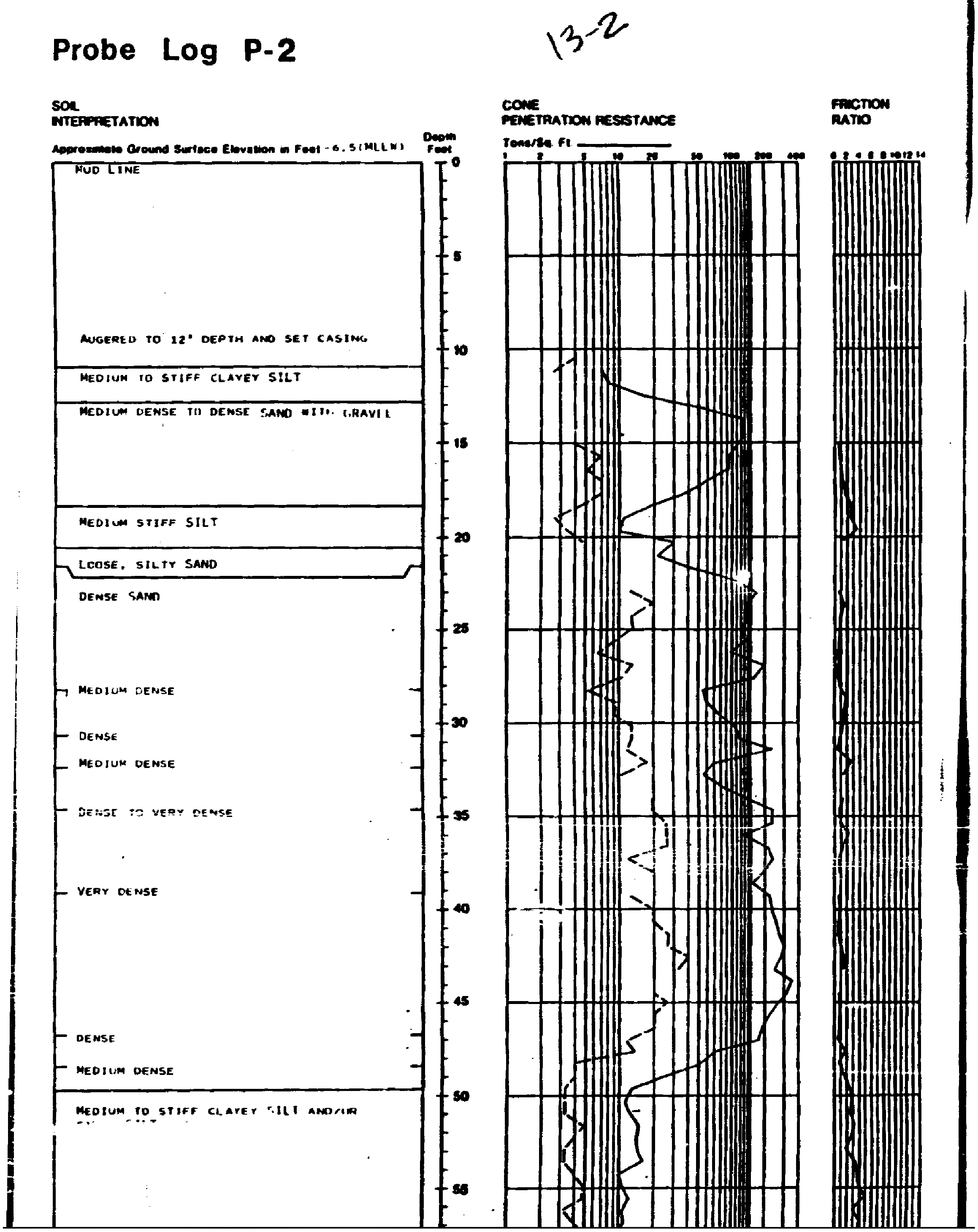


Figure A-16
Log of Boring 6-4

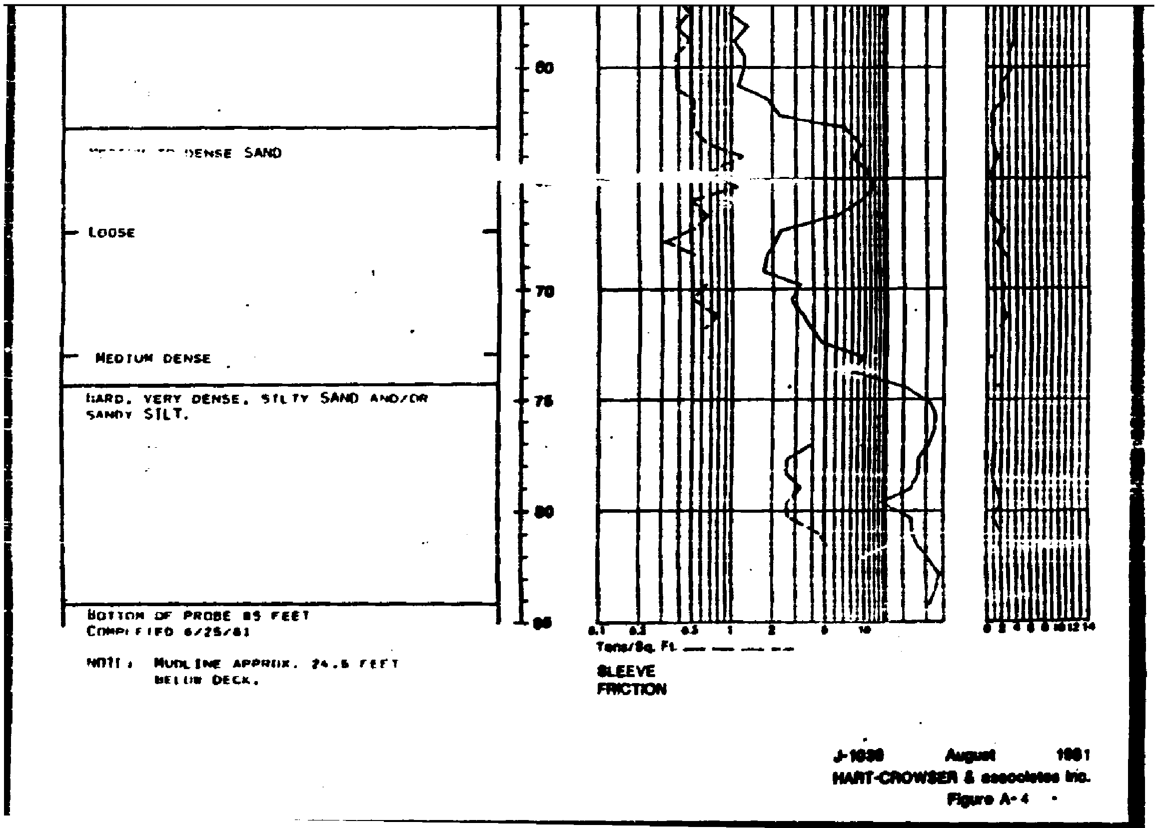


MATCHLINE (SEE SHEET 2 of 2)

MLLW datum elevation of -6.5 Ft. - 2.8 Ft. = -9.3 Ft. NAVD88.

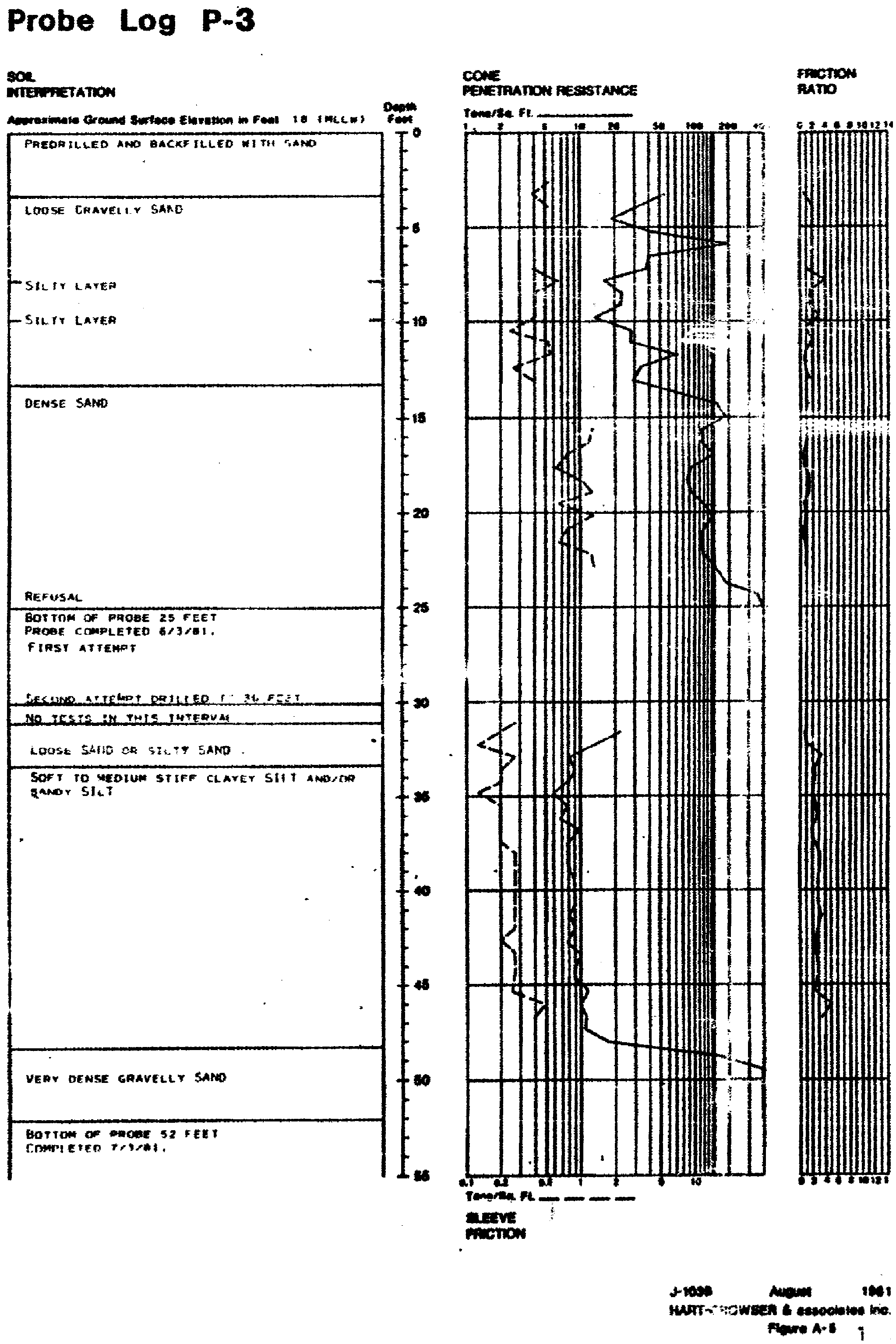
Figure A-17, Sheet 1 of 2
Log of Boring 13-2

MATCHLINE (SEE SHEET 1 of 2)



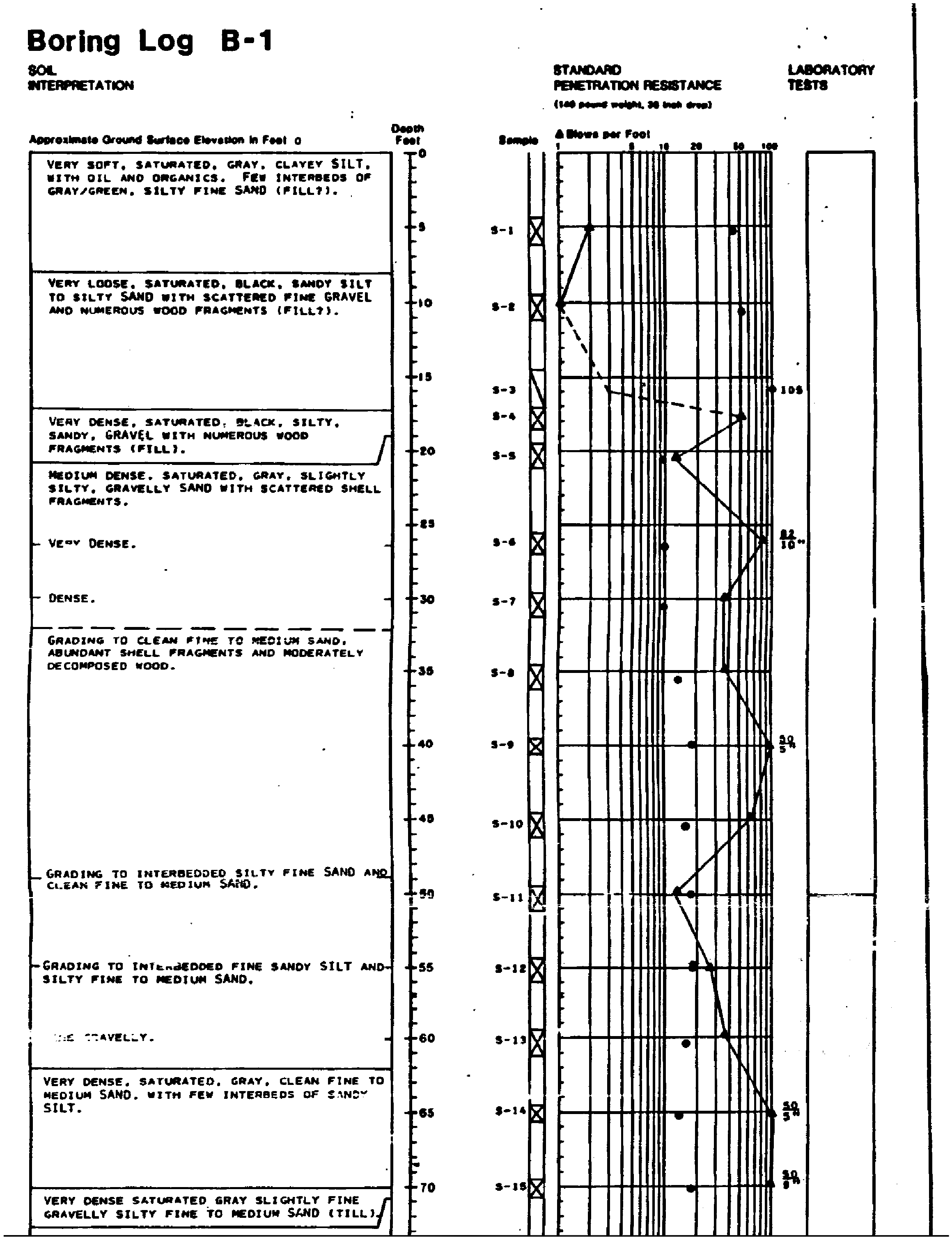
MLLW datum elevation of -6.5 Ft. - 2.8 Ft. = -9.3 Ft. NAVD88.

Figure A-17, Sheet 2 of 2
Log of Boring 13-2



MLLW datum elevation of 18 Ft - 2.8 Ft. = 15.2 Ft. NAVD88.

Figure A-18
Log of Boring 13-3

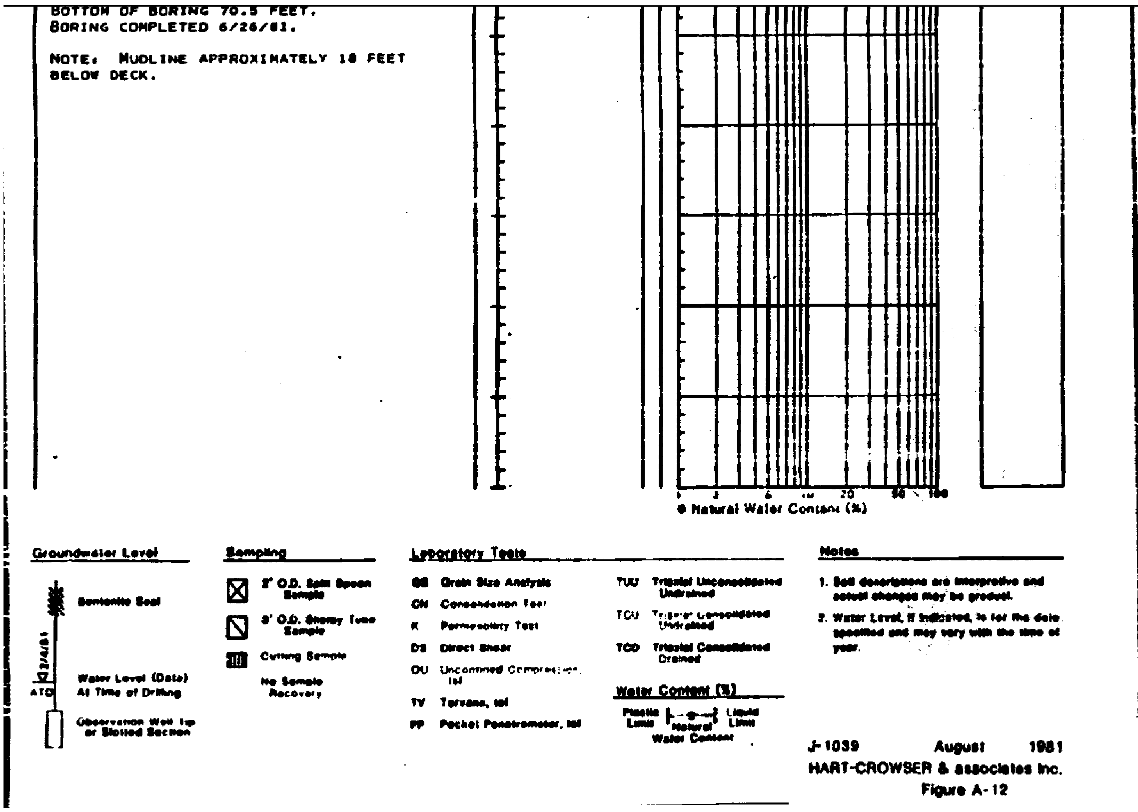


MATCHLINE (SEE SHEET 2 of 2)

Assume ground surface elevation of ~10 Ft. NAVD88.

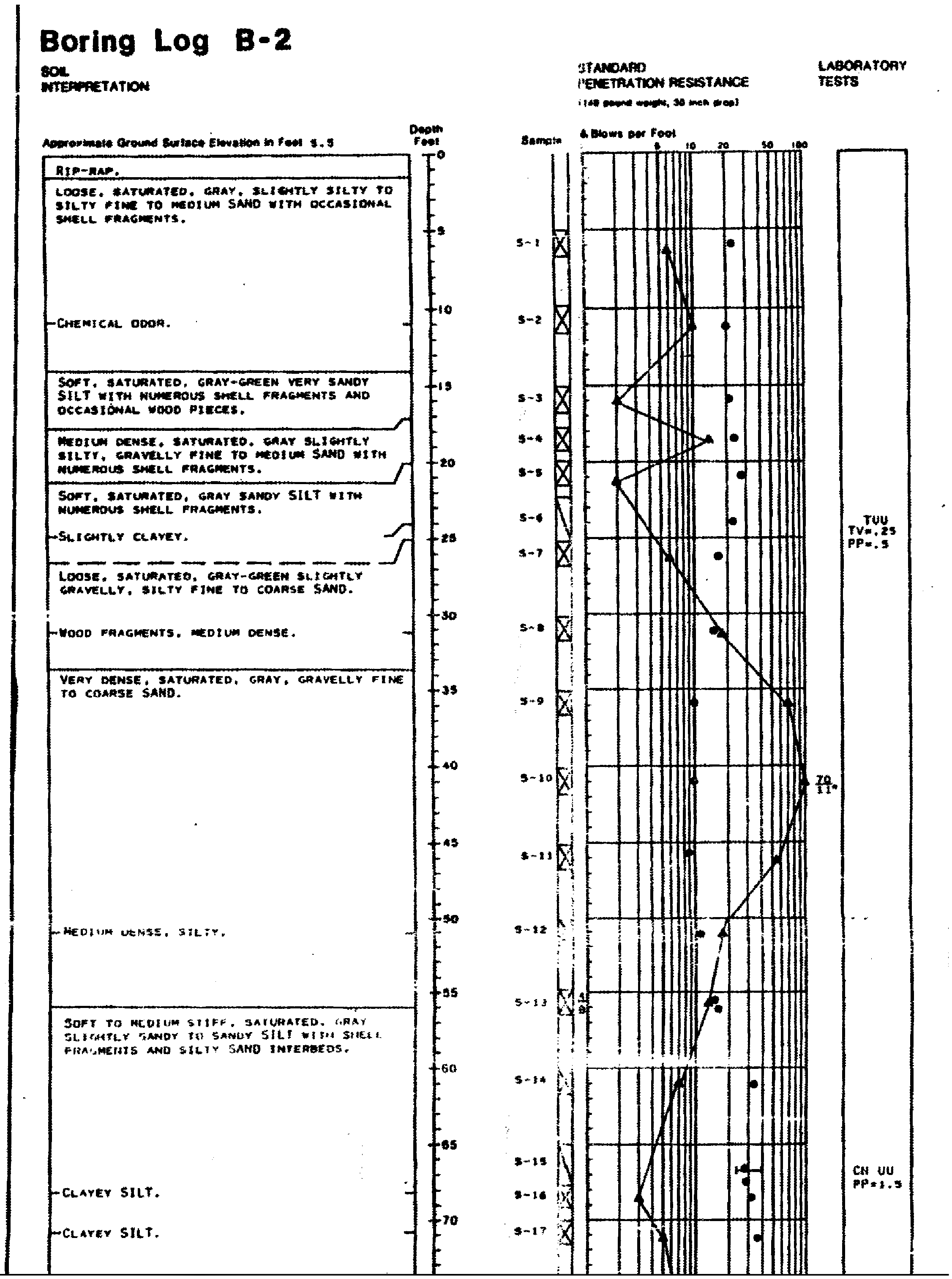
Figure A-19, Sheet 1 of 2
Log of Boring 13-6

MATCHLINE (SEE SHEET 1 of 2)



Assume ground surface elevation of ~10 Ft. NAVD88.

Figure A-19, Sheet 2 of 2
Log of Boring 13-6

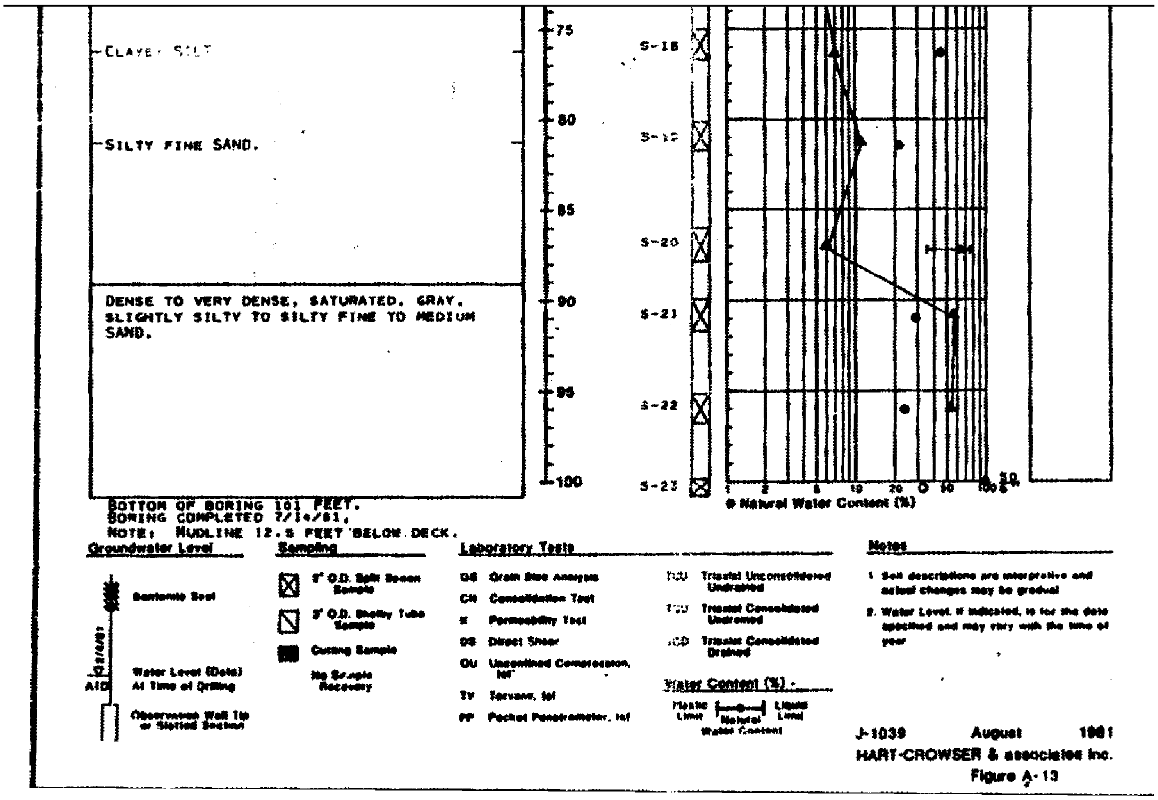


MATCHLINE (SEE SHEET 2 of 2)

Assume ground surface elevation of ~16 Ft. NAVD88.

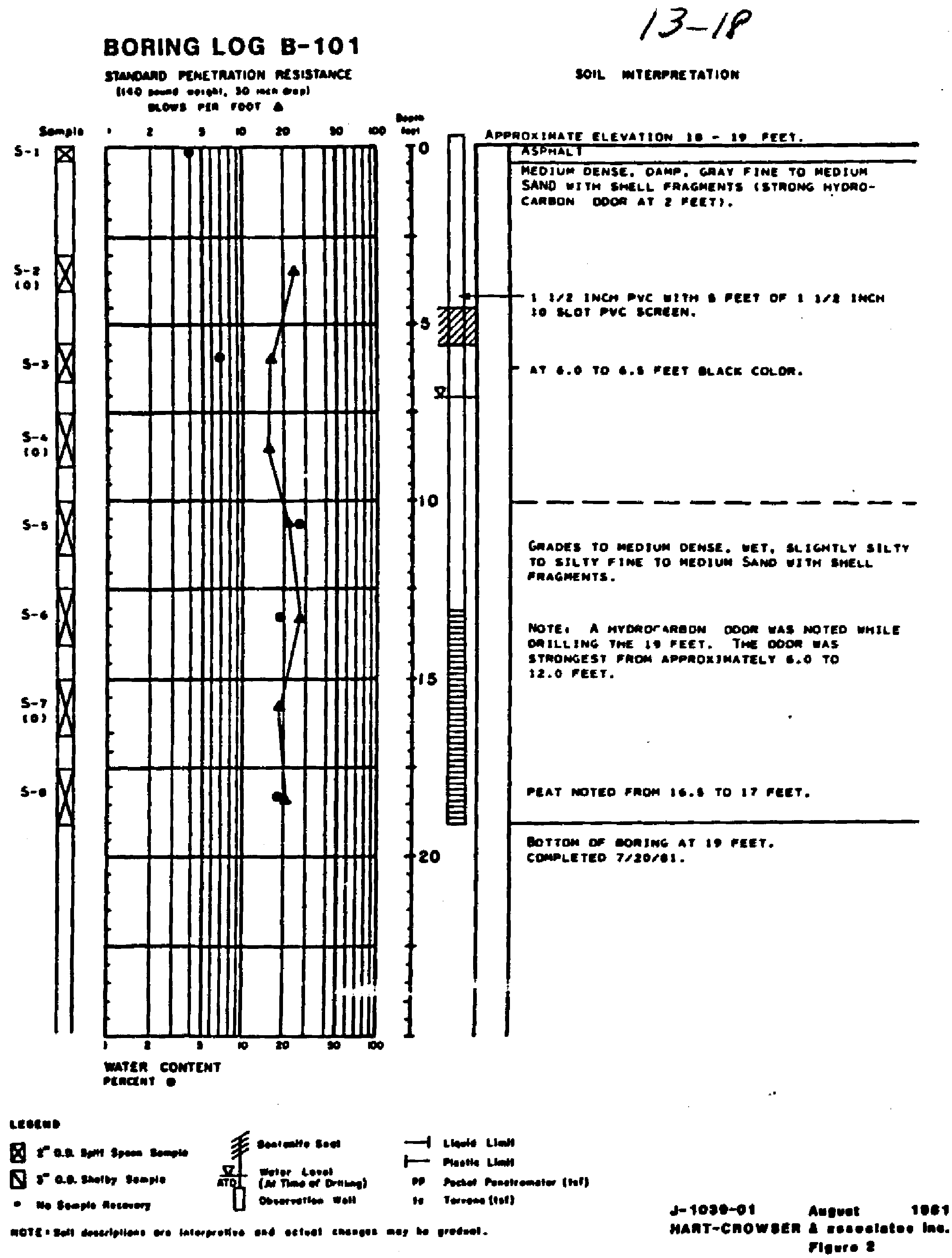
Figure A-20, Sheet 1 of 2
Log of Boring 13-7

MATCHLINE (SEE SHEET 1 of 2)



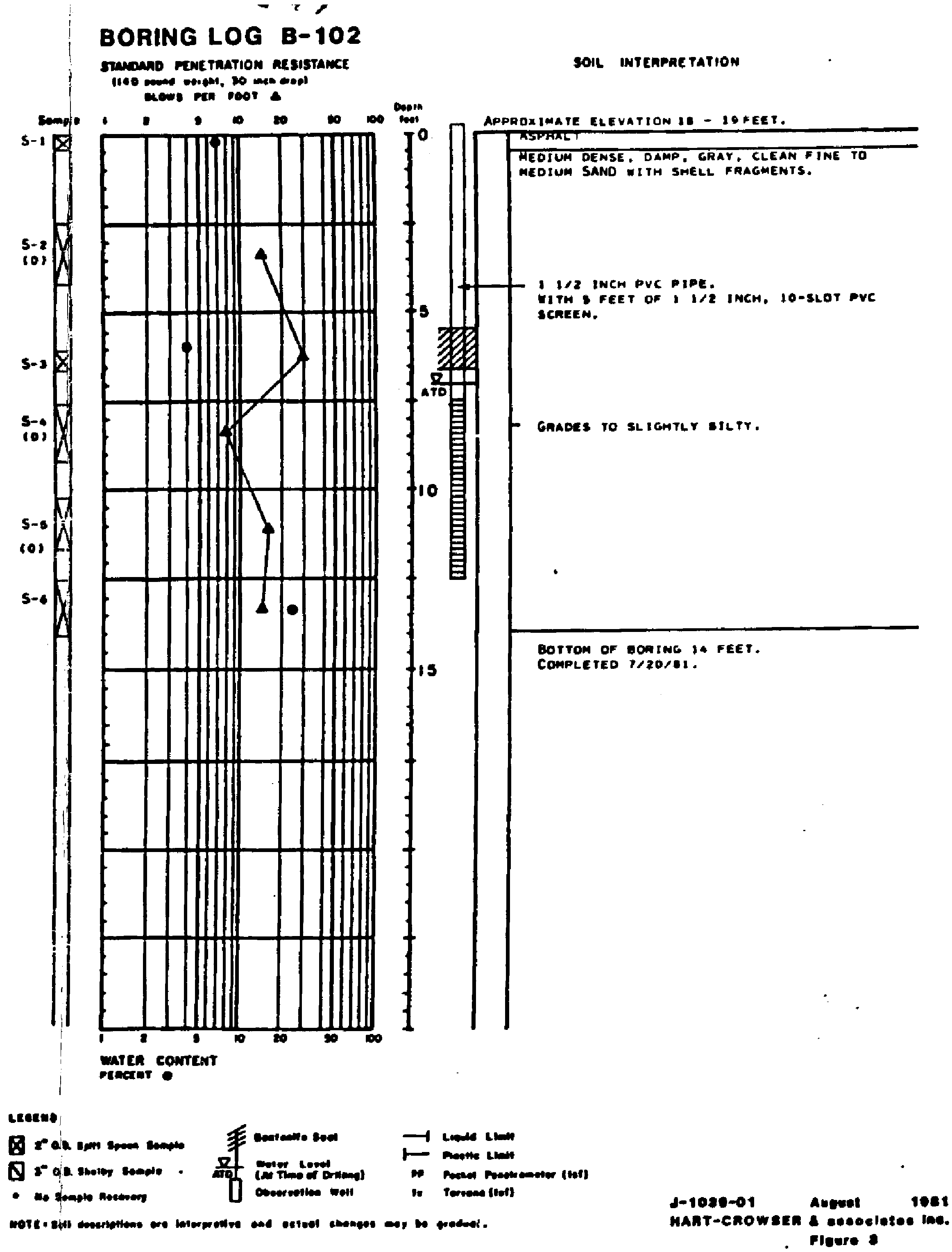
Assume ground surface elevation of ~16 Ft. NAVD88.

Figure A-20, Sheet 2 of 2
Log of Boring 13-7



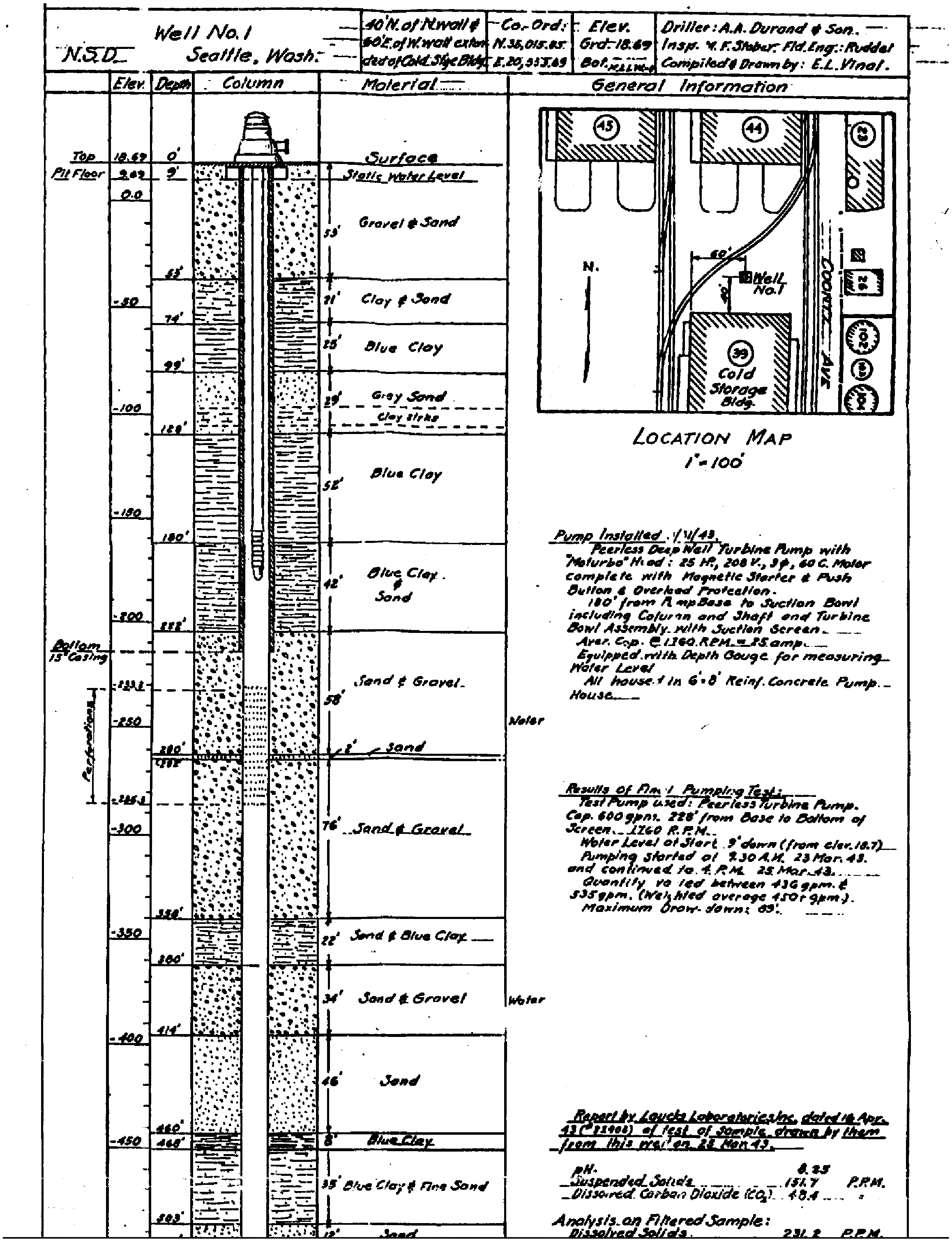
Assume ground surface elevation of ~15 Ft. NAVD88.

Figure A-21
Log of Boring 13-18



Assume ground surface elevation of ~16 Ft. NAVD88.

Figure A-22
Log of Boring 13-19

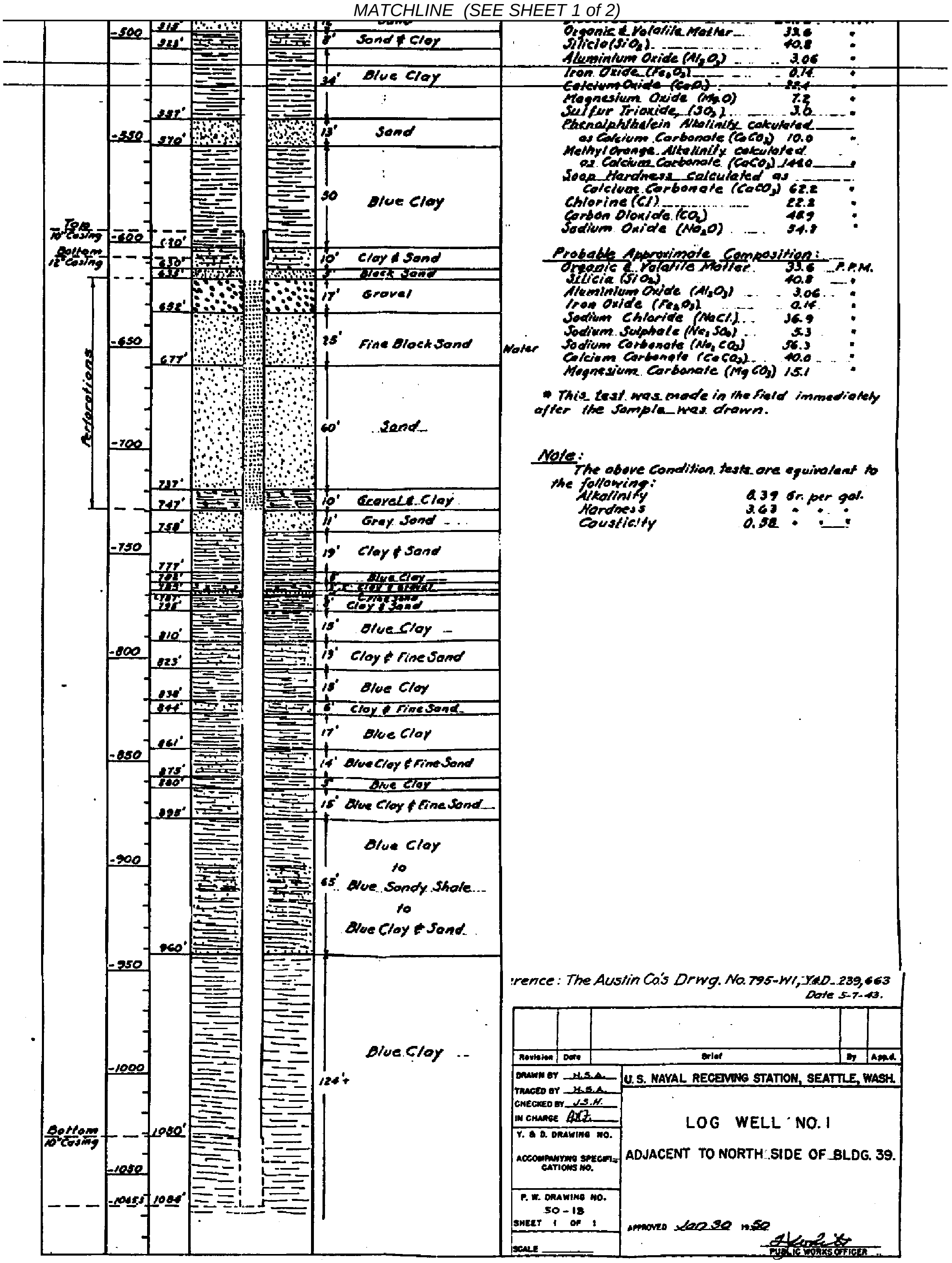


MATCHLINE (SEE SHEET 2 of 2)

MLLW datum elevation of
18.69 Ft. - 2.8 Ft. = 15.89 Ft. NAVD88.

Figure A-23, Sheet 1 of 2
Log of Boring 13-20

MATCHLINE (SEE SHEET 1 of 2)



MLLW datum elevation of
18.69 Ft. - 2.8 Ft. = 15.89 Ft. NAVD88.

Figure A-23, Sheet 2 of 2
Log of Boring 13-20

Boring 14-1
1 of 2

CB 7.241

SEATTLE ENGINEERING DEPARTMENT
MATERIALS LABORATORY

LOG OF TEST BORING

DATE 1-31-73 HOLE NO. 5
PROJECT DARTMOUTH SAN. SEWER GRD. ELEV.
LOCATION 10' N. OF COL. # 61

STRATA	DEPTH	SAMPLE NO.	BLOW COUNT			STD. PEN.	COMPOSITION	CONSISTENCY	MOISTURE	COLOR	WATER LEVEL
FIRM - SAND FINE - COARSE w/CLAYEY SILT LAYERS							ASPHALT & CR. ROCK				
							CLAYEY SILTY SAND		MOIST	GRAY BRN	
							LAYER OF CLAY SILT		MOIST	GRAY BRN	
	5	A	2	6	5	11	& SAND FINE-COARSE	FIRM	MOIST	BRN	2-13-73
LOOSE - SAND w/SHELLS											2-1-73
											1-31-73
				12"			6" SAND & SHELLS		SAT	WHITE GRAY	
V. SOFT CLAYEY SILT w/SHELL & WOOD ON BOTTOM	10	B	1	1		1	6" V.FINE SAND	LOOSE	SAT	BLK GRAY	
							6" CLAYEY SILT	V. SOFT	WET	GRAY	
	15	C	PUSH				CLAYEY SILT	V.SOFT	WET	GRAY	1-31-73
							(SHELL & WOOD)				
	20	D	PUSH				CLAYEY SILT w/ORGANIC	V.SOFT	WET	GRAY	
MED. CLAYEY SILT w/SAND & GRAVEL							& TRACE GRAVEL				
							& SAND POCKETS				
	25	E	1	3	3	6	CLAYEY SILT w/SAND	MED	WET	GRAY	
							& GRAVEL				
FIRM - SAND FINE - COARSE w/SILT & SHELLS & GRAVEL	30	F	4	9	13	22	SAND FINE - COARSE	FIRM	SAT	GREY	PIEZOMETER INSTALLED
							w/ SHELLS & SILT				
							& GRAVEL				

INSPECTOR H W KOLITA

Assume ground surface elevation of ~16 Ft. NAVD88.

Figure A-24, Sheet 1 of 2
Log of Boring 14-1

SEATTLE ENGINEERING DEPARTMENT
MATERIALS LABORATORY

CS 7.241

LOG OF TEST BORING

2 of 2

DATE 1-31-73

HOLE NO. 5

PROJECT DARTMOUTH

GRD. ELEV.

LOCATION 10' N. of Cow. # 61

STRATA	DEPTH	SAMPLE NO.	BLOW COUNT	STD. PEN.	COMPOSITION	CONSISTENCY	MOISTURE	COLOR	WATER LEVEL
					HEADED 3º				
					WATER LEVEL				
	35	9			ROSE IMMEDIATELY				
					TO 19½ FT				

INSPECTOR Hiw Kent

Assume ground surface elevation of ~16 Ft. NAVD88.

Figure A-24, Sheet 2 of 2
Log of Boring 14-1

CB 7.241

SEATTLE ENGINEERING DEPARTMENT BORING 14-2
MATERIALS LABORATORY

LOG OF TEST BORING

DATE 11-14-72 HOLE NO. 1

PROJECT DARTMOUTH AVE. W. SAN. SEWER GRD. ELEV.

LOCATION MAGNOLIA WY & W. GALER 70' E. & 17' S. OF ℄

STRATA	DEPTH	SAMPLE NO.	BLOW COUNT			STD. PEN.	COMPOSITION	CONSISTENCY	MOISTURE	COLOR	WATER LEVEL
v. compact silty SAND w/ gravel							BRN SILTY SAND w/GRAVEL				
			5"							BRN LAYERS	
		A	50	X	X	100+	SILTY SAND w/GRAVEL (FRACTURED)	V. COMP	MOIST	GRAY	
	5										
					5"						
		B	12	44	50	94+	SILTY SAND w/GRAVEL (FRACTURED)	V. COMP	MOIST	BRN	
	10										
v. stiff SILT		C	10	19	26	45	SILT	COMP	MOIST	BRN	
	15										
		D	15	40	45	85	SILT	V. COMP	MOIST	BRN	
	20										SAND
	25										

No water 11-16-72

PIEZOMETER INSTALLED 11-14-72 NO WATER

INSPECTOR H W KOKITA

Assume ground surface elevation of ~157 Ft. NAVD88.

Figure A-25
Log of Boring 14-2

Boring 14-3

SEATTLE ENGINEERING DEPARTMENT
MATERIALS LABORATORY

CE 7.241

LOG OF TEST BORING

DATE 11-14-72 HOLE NO. 2

PROJECT Dartmouth Ave. W. San. Sewer GRD. ELEV.

LOCATION Magnolia Wy & W. Galer St 385' E. & 16' N. of ℄

STRATA	DEPTH	SAMPLE NO.	BLOW COUNT			STD. PEN.	COMPOSITION	CONSISTENCY	MOISTURE	COLOR	WATER LEVEL
Loose Fill							1" Cr. Rock				
							Brn silty sand w/gravel				
Silty compact SAND		A	2	7	22	29	Silty sand w/gravel	Firm	Moist	Brn	
	5						& trace organic (root)				
		B	2	9	19	28	Silty sand w/trace gravel	Firm	Wet / Moist	Brn	
	10						(1" decomposed rock)			Blk	
		C	22	50 (6")	x	100+	Silty fine sand	V.comp	Moist	Brn	
	15						w/gravel				
		D	10	25	28	53	Silty sand	V.comp	Moist	Brn	
	20						w/sand seams		Wet	Brn	
											11-16-72
CLAY-SILT		E	4	11	17	28	Clay-silt w/	Firm	Moist	Brn Gray	
	25						thin silt layers				Sand

Piezometer installed 11-14-72 No water

INSPECTOR H W Kocita

Assume ground surface elevation of ~128 Ft. NAVD88.

Figure A-26
Log of Boring 14-3

CB 7.241

SEATTLE ENGINEERING DEPARTMENT
MATERIALS LABORATORY

15-1 ½

LOG OF TEST BORING

DATE 6-7-72 HOLE NO. 8

PROJECT Queen Anne W Unit 1 GRD. ELEV.

LOCATION 6'E & 7'S of SE Corner Pump Sta. building 201

STRATA	DEPTH	SAMPLE NO.	BLOW COUNT			STD. PEN.	COMPOSITION	CONSISTENCY	MOISTURE	COLOR	WATER LEVEL
Loose silty fine SAND w/ gravel (FILL)							brn silty fine SAND w/gravel				
	5	A	1	2	2	4	silty fine SAND w/gravel	Loose	moist	brn	
Loose fine SAND w/ occasional layers SILT & silty fine SAND										gray & brn	6/15/72 300 pm
	10	B	1	1	2	3	fine SAND w/gravel pocket organics & some silty fine sand	Loose	wet		
	15	C	X	2	4	6	fine SAND w/1" layer organic SILT	Loose	sat	gray	
	20	D	1	2	7	9	fine SAND w/ pocket wood	Loose	sat	gray	
Firm SAND & GRAVEL	25	E	5	5	9	14	Top fine SAND Bott SAND & GRAVEL w/pockets wood	firm	sat	gray	6/7/72
	30	F	X	—	1	1	No Recovery	very soft			

INSPECTOR A C Ring

Assume ground surface elevation of ~16 Ft. NAVD88.

Figure A-27, Sheet 1 of 2
Log of Boring 15-1

GB 7.241

SEATTLE ENGINEERING DEPARTMENT
MATERIALS LABORATORY

15-1 2/2

LOG OF TEST BORING

DATE 6/7/72 HOLE NO. 8

PROJECT Queen Anne West Unit 1 GRD. ELEV.

LOCATION

STRATA	DEPTH	SAMPLE NO.	BLOW COUNT	STD. PEN.	COMPOSITION	CONSISTENCY	MOISTURE	COLOR	WATER LEVEL
Very Soft SILT w/shells	35'	G	PUSH		SILT w/shells	Very soft	sat	gray	Piezometer Installed 6/7/72
	40	H	PUSH		SILT w/shells	Very soft	sat	gray	

INSPECTOR A C Rice

Assume ground surface elevation of ~16 Ft. NAVD88.

Figure A-27, Sheet 2 of 2
Log of Boring 15-1

9531

SEATTLE ENGINEERING DEPARTMENT
MATERIALS LABORATORY

LOG OF TEST BORING

DATE 11.4.81 HOLE NO. 3

PROJECT 23RD AVE W. SANIT. SEWER REPLACE. GRD. ELEV.

LOCATION 150'± N. ℄ W. HAYES ST (EXTENDED) & 120'E ℄ 23RD W. (EXTENDED)

STRATA	DEPTH	SAMPLE NO.	BLOW COUNT			STD. PEN.	COMPOSITION	CONSISTENCY	MOISTURE	COLOR	WATER LEVEL
		A		13	14	27	SAND & GRAVEL	FIRM	MOIST	GRAY	
	5										
											11-13-81
		B	1	3	4	7	SAND W/ GRAVEL SOME SHELL	LOOSE	SAT	GRAY	
	10										
		C	1	1	1	2	SAND W/ SOME SHELL	LOOSE	SAT	GRAY	
	15										
		D	PUSH				SILT W/ TRACE SAND WOOD & SHELL	V. SOFT	MOIST	DK GRAY	
	20										
		E	2	2	5	7	F. SAND W/ SILT TRACE SHELL	LOOSE	WET	DK. GRAY	
	25										
		F	1	1	2	3	F. SAND & SILT W/ SHELL	LOOSE	WET	DK GRAY	
	30										

INSPECTOR [signature]

Assume ground surface elevation of ~16 Ft. NAVD88.

Figure A-28, Sheet 1 of 2
Log of Boring 16-3

SEATTLE ENGINEERING DEPARTMENT

MATERIALS LABORATORY

CB 7.241

LOG OF TEST BORING

DATE 11-4-81

HOLE NO 2

PROJECT 23 AVE W. SANIT SEWER REPLA

GRD ELEV

LOCATION

STRATA	DEPTH	SAMPLE NO.	BLOW COUNT			STD PEN.	DESCRIPTION OF MATERIAL: COMPOSITION	CONSISTENCY	MOISTURE	COLOR	WATER LEVEL
		6	3	6	7	13	GRAVEL w/ SAND, SILTY CLAY	FIRM	WET	DK GRAY	PIEZO INSTALLED 11-4-81
BOTTOM 11-4-81	35										
	40										

INSPECTOR M. Mar[illegible]

Assume ground surface elevation of ~16 Ft. NAVD88.

Figure A-28, Sheet 2 of 2
Log of Boring 16-3

Boring 22-1

SEATTLE ENGINEERING DEPARTMENT
MATERIALS LABORATORY

CB 7.241

LOG OF TEST BORING

1/2

DATE 7-21-88 HOLE NO. 2

PROJECT 15TH AVE. W. C.S.O. - S 1560 A GRD. ELEV. 12'±

LOCATION 15'± S/O NORTH R.O.W. (15' W/O ℄ WHEELER) & 10' W/O SLOPE BOTTOM (~250'± W/O ℄ 15TH AVE W.)

STRATA	DEPTH	SAMPLE NO.	BLOW COUNT			STD. PEN.	COMPOSITION	CONSISTENCY	MOISTURE	COLOR	WATER LEVEL
FLUVIAL DEPOSITION						—	GROUNDWATER AT 1' —				BENTONITE SEAL
											7-25-88
		A	2	2	2	4	6" SILTY GRAV. SAND W/WOOD, ORGANICS, GARBAGE (SANDY SILT IN TIP)	V. LOOSE	WET	GRAY	7-27-88 8-16-88
	5	B	2	2	2	4	GRAV. SANDY SILT W/ORGANICS (3" RECOVERY)	SOFT		BLACK	
		C	—	1	2	3	LENS - SILT - SILTY SAND - SILTY SAND W/ORGANICS	SOFT		GRAY BLACK	
	10	D	1	1	2	3	LENS - SILTY SAND W/ORGANICS - FINE SAND - GRAV. SILTY F-M SAND	V. LOOSE		BLACK GRAY	
		E_1					NOTE - BLOCKS (NOT LENS) - FINE SAND			GRAY AND BROWN	
		E_2	3	6	9	15	- FINE SANDY SILT - FINE SANDY SILT W/GRAVEL - FINE SANDY SILT W/ORGANICS	MEDIUM			
	15										
							TOP 8" - GRAV. ORGANIC SILTY SAND			BLACK	
		F_1					2" - COARSE SAND		WET	BROWN	
		F_2	7	17	38	55	4" - SILTY M-F SAND	V. COMPACT	MOIST	BLOCKY BROWN AND GRAY	
	20						BOT 4" - SILT		MOIST		
GLACIAL DEPOSITION		G_1					TOP 8" - C-SANDY GRAVEL		WET	GRAY	
		G_2	16	50/6		50+	BOT 10" - F-M SAND	V. COMPACT			
	25										
							TOP 4" - SLIGHTLY GRAV. C-SAND				
VERY HARD DRILLING		H	20	50/5½		50+	BOT 14" - SILTY F-SAND	V. COMPACT			
	30										

INSPECTOR JON M.

Assume ground surface elevation of ~18 Ft. NAVD88.

Figure A-29, Sheet 1 of 2
Log of Boring 22-1

11225

SEATTLE ENGINEERING DEPARTMENT
MATERIALS LABORATORY

CB 7.241

LOG OF TEST BORING

2/2

DATE 7-21-88 HOLE NO. 2

PROJECT 15TH AVE. W. C.S.O. - S1560 A GRD. ELEV. 12'±

LOCATION 15'± S/O NORTH R.O.W. (15TH W/O ℄ WHEELER) & 10' W/O SLOPE BOTTOM (~250'± W/O ℄ 15TH AVE W)

STRATA	DEPTH	SAMPLE NO.	BLOW COUNT	STD. PEN.	COMPOSITION	CONSISTENCY	MOISTURE	COLOR	WATER LEVEL
GLACIAL DEPOSITION					TOP 2" - F-M SAND		WET		
		I	50/6	50+	BOT 6" GRAY, F-M SANDY SILT	V. HARD	MOIST	GRAY	
					GRAY, SILT				
BOH	35				[GRAY, SILT IN TIP]				PIEZO. TIP

INSPECTOR JON M.

Assume ground surface elevation of ~18 Ft. NAVD88.

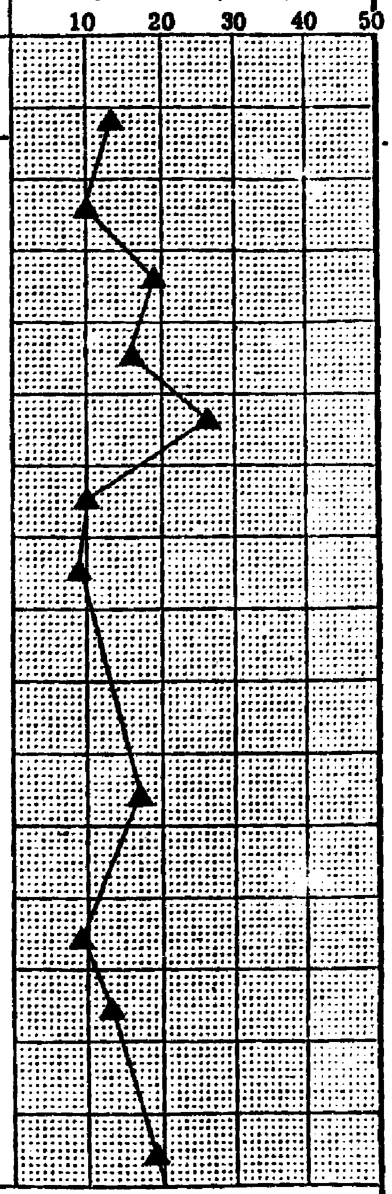
Figure A-29, Sheet 2 of 2
Log of Boring 22-1

DEPTH, FEET	SOIL DESCRIPTION Surface Elevation:	SAMPLES	GROUNDWATER
	Asphalt and crushed rock base		
	Medium dense, moist, gray, fine to medium sand (Fill)	I	∇
5	Loose to medium dense, wet, gray, silty, fine to coarse sand with gravel	I	
		I	
10		I	
		I	
15	Loose, wet, gray, silty, fine sand with wood debris and shell fragments	I	
		I	
20			
		II	
25	Medium dense, wet gray, fine to medium layered sand.	I	
		II	
30	Loose to medium dense, wet, gray, silty fine to medium sand with wood debris and shell fragments	I	
		I	
35			
		I	
40			

STANDARD PENETRATION RESISTANCE

▲ Blows per foot

(140 lb. hammer, 30" drop)



● % Water Content

LEGEND

I 2.0" O.D. split spoon sampler
II 3.0" O.D. undisturbed sampler
P Sampler pushed
• Sample not recovered
Piezometer tip
∇ Water level
Atterberg limits: Liquid Limit, Natural water content, Plastic Limit

NWCR
W-2501
LOG OF BORING NO. 1

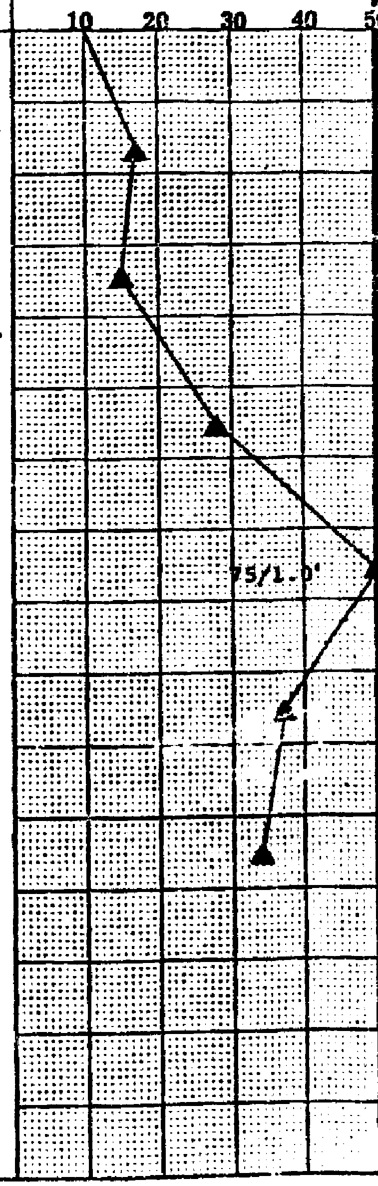
RITTENHOUSE-ZEMAN & ASSOC.
SOILS ENGINEERING AND GEOLOGY

Figure A-30, Sheet 1 of 2
Log of Boring 23-1

BORING 23-1

DEPTH, FEET	SOIL DESCRIPTION Surface Elevation:	SAMPLES	GROUNDWATER
	Medium dense, wet, gray, medium to coarse sand with gravel and silt	I	∇
45			
	Medium dense, wet, gray, fine to medium sand	I	
50	Medium dense, wet, brown to light brown, silty fine to medium sand with gravel	I	
55	Very dense, wet, gray, silty fine to medium layered sand with gravel	I	
60			
	Dense, wet, gray, silty fine to medium layered sand	I	
65		I	
70	Boring Completed - 69.5 feet		
75			
80			

STANDARD PENETRATION RESISTANCE
▲ Blows per foot
(140 lb. hammer, 30" drop)



LEGEND
I 2.0" O.D. split spoon sampler
II 3.0" O.D. undisturbed sampler
P Sampler pushed
Atterberg limits: Liquid limit, Natural water content, Plastic Limit
• Sample not recovered
Piezometer tip
∇ Water level
● % Water Content

09997

NWCR
W-2501

LOG OF BORING NO.1 (cont.)

RITTENHOUSE-ZEMAN & ASSOC.
SOILS ENGINEERING AND GEOLOGY

Figure A-30, Sheet 2 of 2
Log of Boring 23-1

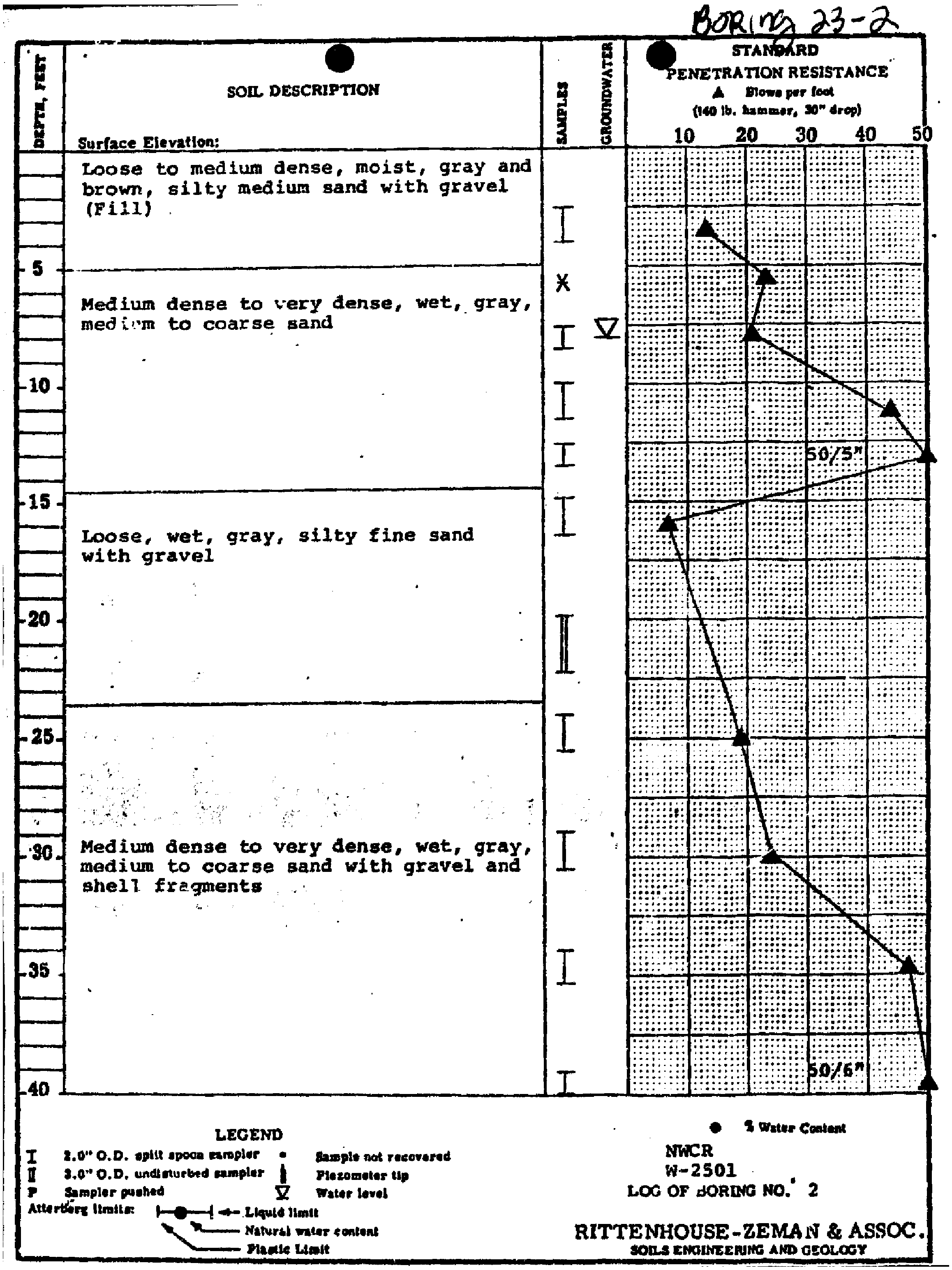
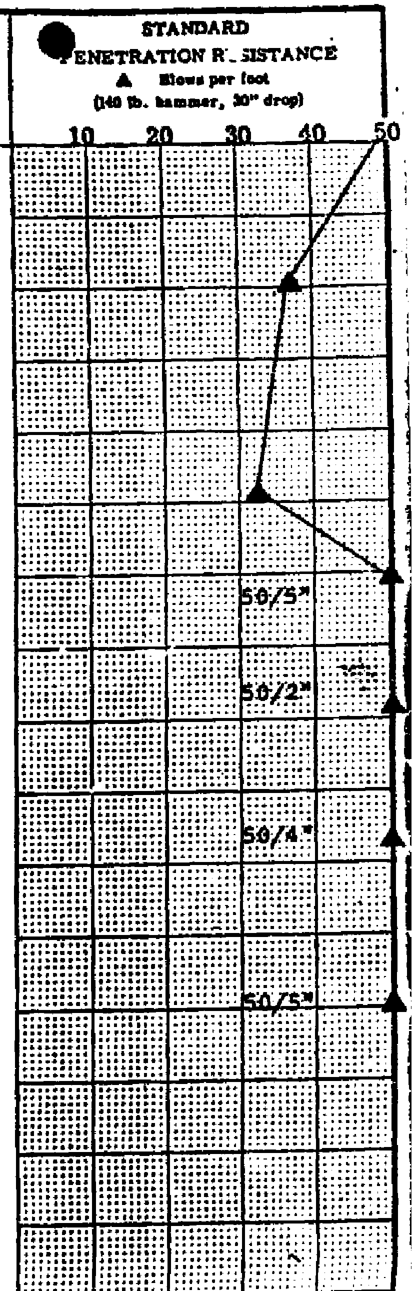


Figure A-31, Sheet 1 of 2
Log of Boring 23-2

DEPTH, FEET	SOIL DESCRIPTION Surface Elevation:	SAMPLES	GROUNDWATER
45	Dense to very dense, wet, gray, fine to medium sand with silt and gravel layers.	I	
50		II I	
55		I	
60	Very dense, moist, gray, silt grading to silty clay	I	
65		I	
70		I	
	Boring Completed - 70.5 feet		
80			



LEGEND

I 2.0" O.D. split spoon sampler
II 3.0" O.D. undisturbed sampler
P Sampler pushed
Atterberg limits: Liquid limit, Natural water content, Plastic Limit
* Sample not recovered
Piezometer tip
Water level

09998

● % Water Content

NWCR
W-2501
LOG OF BORING NO. 2 (cont.)

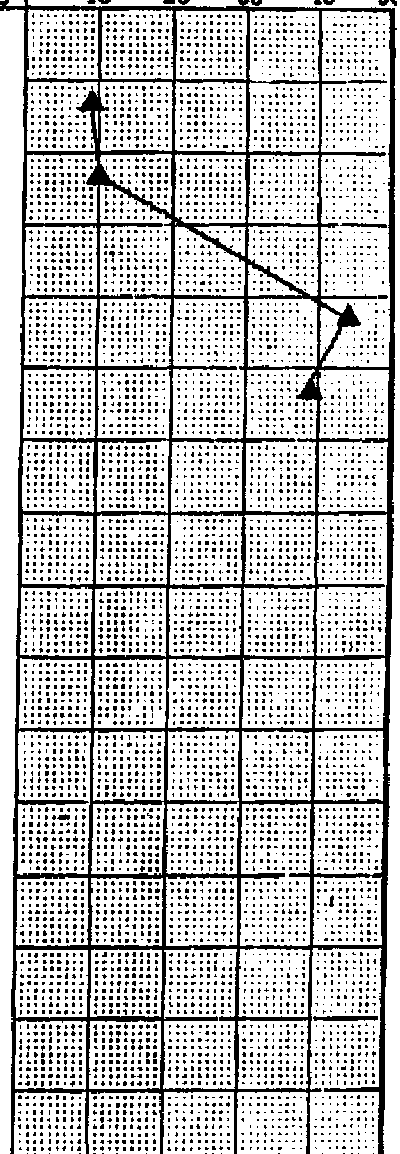
RITTENHOUSE-ZEMAN & ASSOC.
SOILS ENGINEERING AND GEOLOGY

Figure A-31, Sheet 2 of 2
Log of Boring 23-2

DEPTH, FEET	SOIL DESCRIPTION Surface Elevation:	SAMPLES	GROUNDWATER
0–5.5	Loose, moist, brown, silty gravelly fine to medium sand (Fill)	I I	
5.5–7.5	Medium dense, wet, gray, fine sand	II	
7.5–14	Dense, wet, gray, gravelly medium to coarse sand	I I	
15	Boring Completed - 14.0 feet		

Depth scale: 5, 10, 15, 20, 25, 30, 35, 40

STANDARD PENETRATION RESISTANCE
▲ Blows per foot
(140 lb. hammer, 30" drop)
10 20 30 40 50



● % Water Content

LEGEND

- I 2.0" O.D. split spoon sampler
- II 3.0" O.D. undisturbed sampler
- P Sampler pushed
- * Sample not recovered
- Piezometer tip
- Water level
- Atterberg limits: Liquid limit, Natural water content, Plastic Limit

NWCR
W-2501

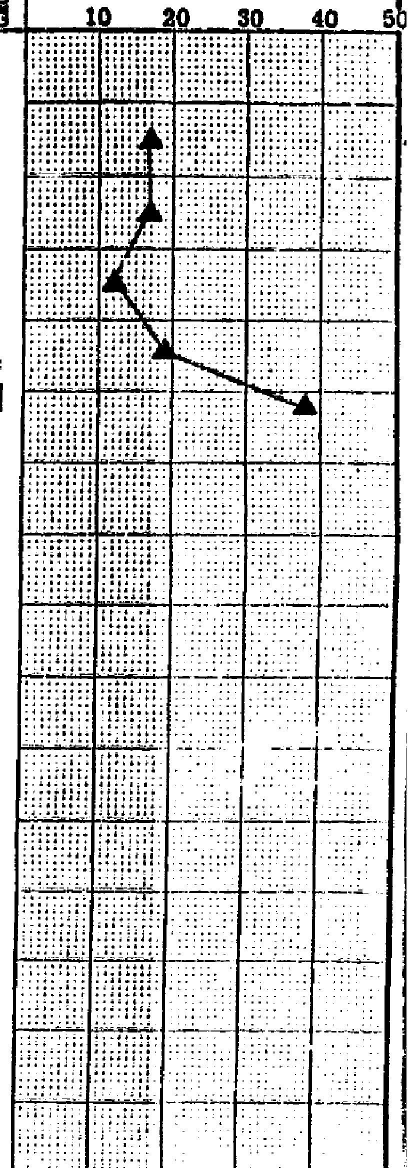
LOG OF BORING NO. 3

RITTENHOUSE-ZEMAN & ASSOC.
SOILS ENGINEERING AND GEOLOGY

Figure A-32
Log of Boring 23-3

DEPTH, FEET	SOIL DESCRIPTION	SAMPLES	GROUNDWATER
	Surface Elevation:		
	Asphalt and crushed rock base		
	Loose to medium dense, moist, brown and gray, fine to medium sand (FILL)	I	
5		I	
	Medium dense to dense, wet, gray, gravelly, medium to coarse sand	I	
10		I	
		I	
	Boring Completed - 13.5 feet		
15			
20			
25			
30			
35			
40			

STANDARD PENETRATION RESISTANCE
▲ Blows per foot
(140 lb. hammer, 30" drop)



● % Water Content

LEGEND

I 2.0" O.D. split spoon sampler
[] 3.0" O.D. undisturbed sampler
P Sampler pushed
• Sample not recovered
Piezometer tip
▽ Water level
Atterberg limits: Liquid limit; Natural water content; Plastic Limit

NWCR
W-2501

00999

LOG OF BORING NO. 4

RITTENHOUSE-ZEMAN & ASSOC.
SOILS ENGINEERING AND GEOLOGY

Figure A-33
Log of Boring 23-4

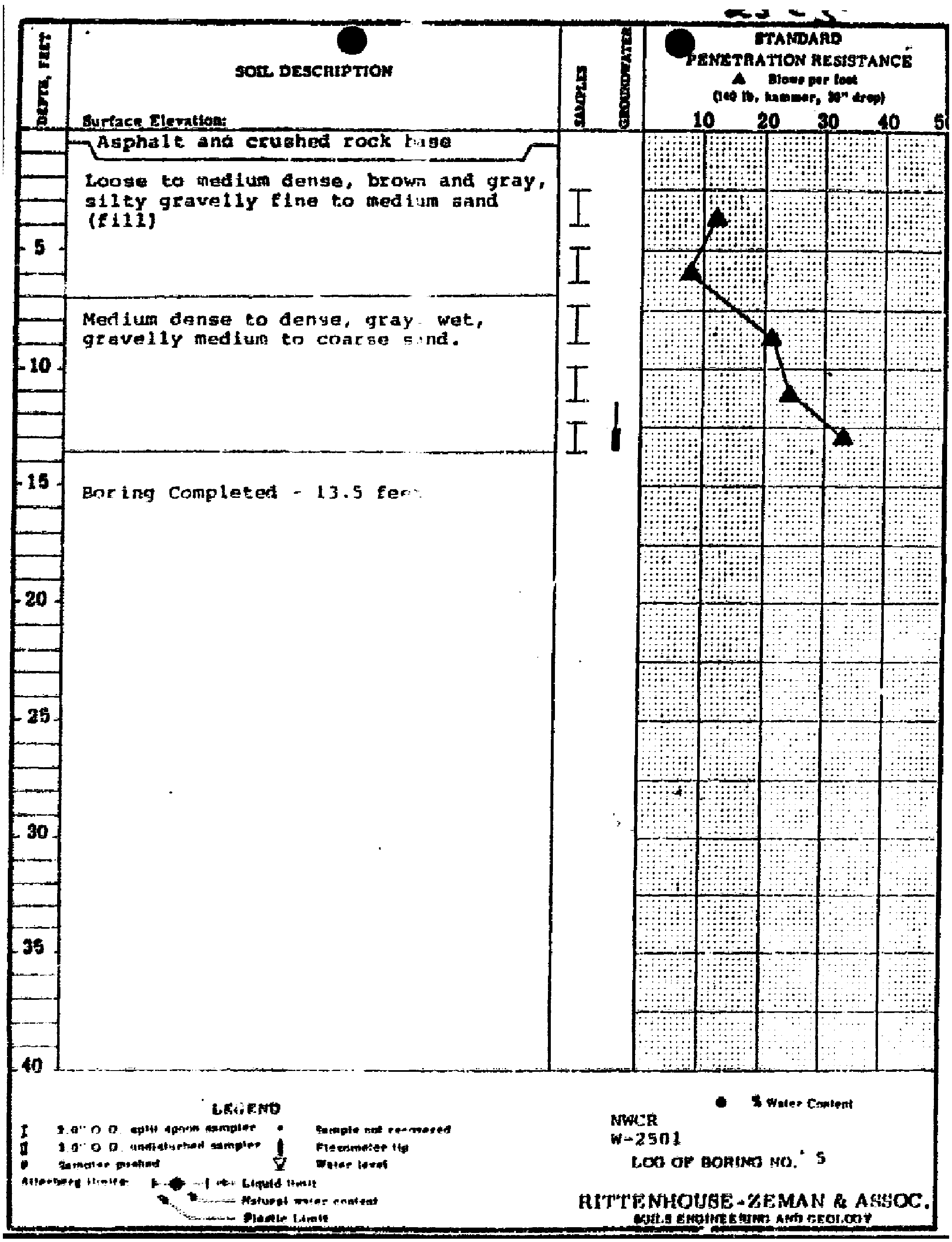
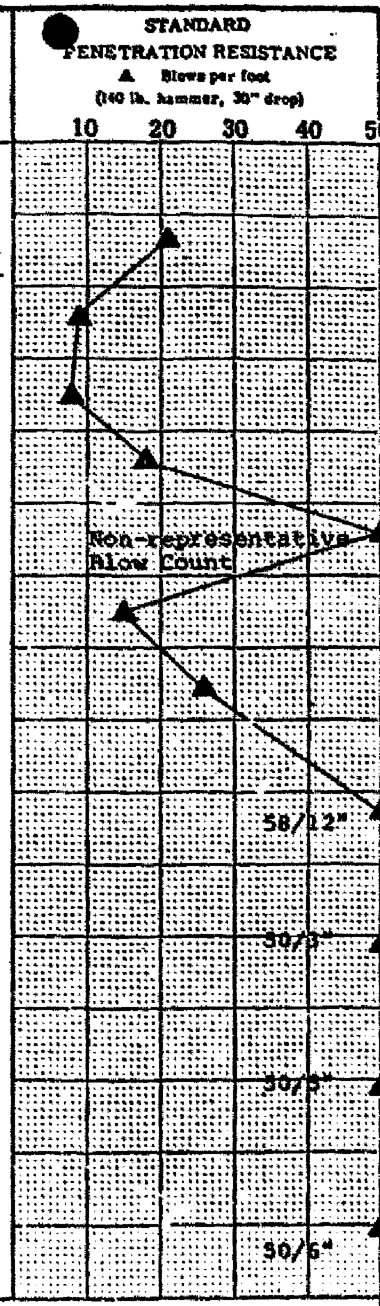


Figure A-34
Log of Boring 23-5

DEPTH, FEET	SOIL DESCRIPTION Surface Elevation:	SAMPLES	GROUNDWATER
	Sod and topsoil		
	Medium dense, moist, gray, fine to medium sand with silt and clay (Fill)	I	
5	Loose, wet, gray fine to medium sand.	I	∇
	Very loose, moist, black, peaty silt	I	
10	Loose to medium dense, wet, gray, gravelly medium to coarse sand	I	
		I	
15		I	
		I	
20	Very dense, moist, gray, gravelly, silty, fine to medium sand (Glacial till)	I	
25		I	
30		I	
35		I	
	Boring Completed - 38.0 feet		
40			



10000

LEGEND
I 2.0" O.D. split spoon sampler
I 3.0" O.D. undisturbed sampler
P Sampler pushed
Atterberg limits: Liquid limit, Natural water content, Plastic Limit
* Sample not recovered
Piezometer tip
∇ Water level
● % Water Content

NWCR
W-2501
LOG OF BORING NO. 6

RITTENHOUSE-ZEMAN & ASSOC.
SOILS ENGINEERING AND GEOLOGY

Figure A-35
Log of Boring 23-6

Boring 282-1

BORING NO. 1

Logged By ND

Date 9-6-89

Elev. 100±

Graph	USCS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)
	sm	Brown silty SAND with gravel, moist, medium dense				
	sm	Gray/brown silty SAND with some gravel, moist, loose	5	I	7	11.8
		Dark gray silty SAND with gravel, moist, medium dense	10	I	12	7.2
	sm	Black silty SAND with some wood chips and some organics, moist, medium dense	15	I	12	22.7
		Black silty SAND, moist, medium dense	20	I	21	14.5
		-water at 21-22' -no sample recovered	25	I	18	
	sm	Gray silty SAND with some gravel, moist, very dense	30	I	59	9.9
		Gray silty SAND, moist, dense	35	I	40	11.6
		-hard drilling Gray silty SAND with gravel, moist, very dense		I	50/5"	11.2

Boring terminated at 39 feet below existing grade.
Groundwater encountered at 21 feet during drilling.

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis, and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

Earth Consultants Inc.
Geotechnical Engineers, Geologists & Environmental Scientists

BORING LOG
AMIMAL HOSPITAL
SEATTLE, WASHINGTON

Proj. No. 4605	Drwn. GLS	Oct'89	Checked ND	Date 10-2-89	Plate 4

Figure A-36
Log of Boring 282-1

282-2

BORING NO. 2

Logged By ND

Date 9-6-89

Elev. 100±

Graph	USCS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)
	sm	"Fill" brown silty SAND with gravel, moist, medium dense Brown/gray silty SAND, some gravel, moist, medium dense	5	I	15	11.5
	sm	Gray silty SAND, moist, loose to medium dense -with some wood chips and organics	10	I	10	10.5
		Gray silty SAND with some gravel, wet, medium dense	15	I	23	12.1
	sm-sp	Gray SAND, some silt, wet, very dense	20	I	50/5"	22.7
	sp	Gray coarse grained SAND and gravel, moist, very dense	25	I	50/6"	9.6
	sm	Gray silty SAND, moist,very dense -very hard drilling	30	I	50/6"	10.2

Boring terminated at 31 feet below existing grade.
Groundwater encountered at 18 feet during drilling.

Elevation determined by eye-level assuming that street level is at EL-100 feet.

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis, and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

Earth Consultants Inc.
Geotechnical Engineers, Geologists & Environmental Scientists

BORING LOG

ANIMAL HOSPITAL
SEATTLE, WASHINGTON

Proj. No. 4605	Drwn. GLS	Oct'89	Checked ND	Date 10-2-89	Plate 5

Assume ground surface elevation of ~37 Ft. NAVD88.

Figure A-37
Log of Boring 282-2

BORING 1

Moisture Content (%)	Sample	Blows per Foot	USCS	Description
	1	7	FILL	Brown/mottled, silty SAND with gravel and some wood debris, moist, loose
30.1	2	19	ML	Tan, clayey SILT with some sand lenses, medium-plastic, moist, very stiff
36.9	3	31		-becomes more clayey, wet
31.8	4	31		-lens of saturated SAND -becomes blue-gray
	5	27	SP	Gray SAND, fine-grained, very moist, medium-dense to dense

Depth scale (ft): 5, 10, 15, 20, 25, 30, 35, 40

Test boring terminated at 24 feet below grade on 10-13-95.
No groundwater seepage was encountered during drilling.

GEOTECH
CONSULTANTS, INC.

TEST BORING LOG
2328 - 34 THORNDYKE AVENUE W
SEATTLE, WA

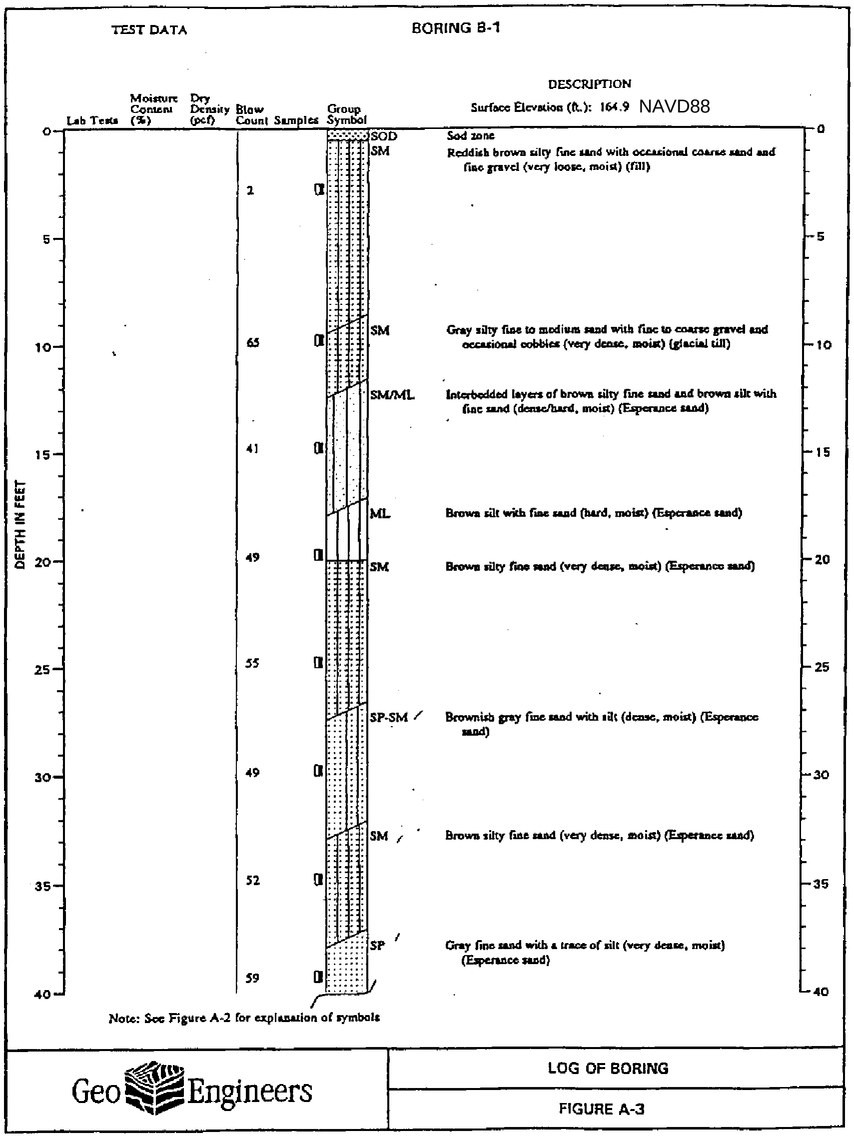
Job No:	Date:	Logged by:	Plate:
95361	OCT 1995	DRW	3

Assume ground surface elevation of ~111 Ft. NAVD88.

Figure A-38
Log of Boring 710-1

Boring 1647-2

Boring B-1



Reference: Supplemental Design Recommendations, Retaining Wall Design, 2625 West Galer Street, Seattle, Washington, by GeoEngineers, dated September 1, 1994.

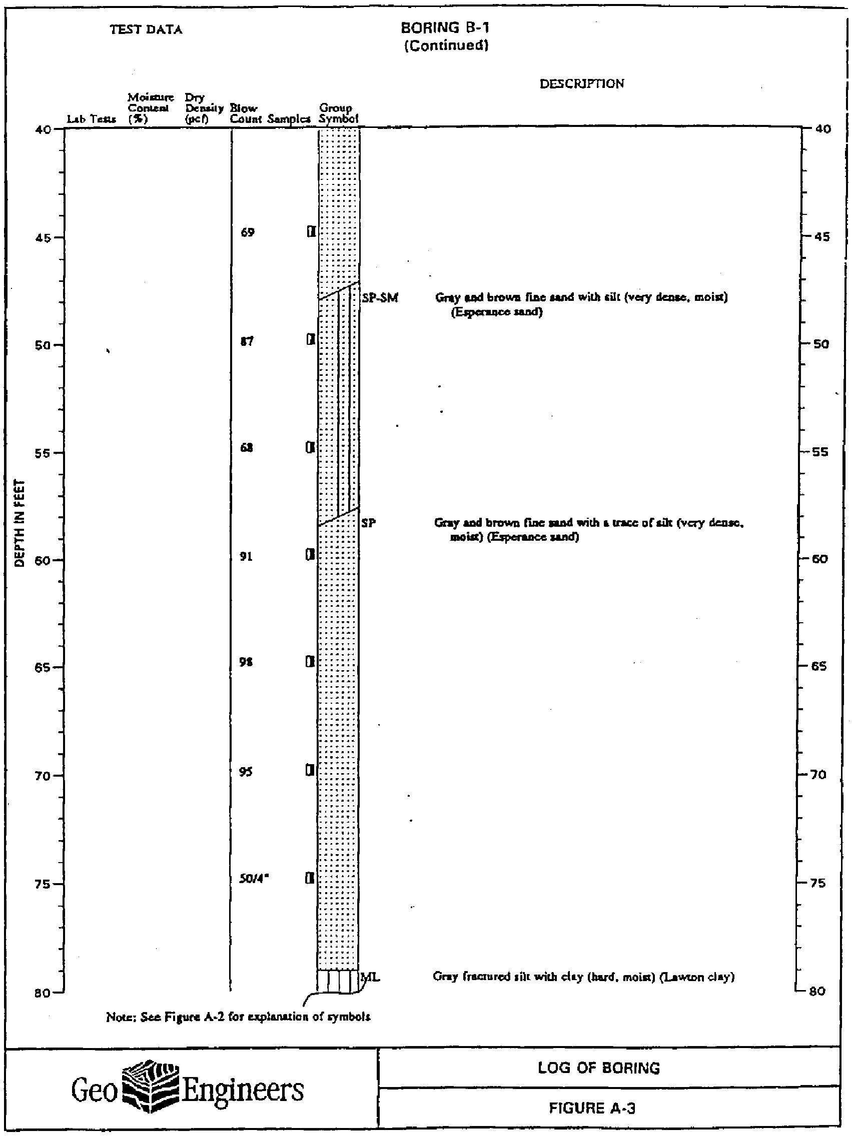
Note: This boring log has been included for informational purposes only. CEO, Inc., makes no representations whatsoever regarding the accuracy of the data or for its interpretation by others.

CEO Creative Engineering Options INC.	BORING LOG	Plate 15

Figure A-39, Sheet 1 of 3
Log of Boring 1647-2

Boring 1647-2

Boring B-1
(continued)



Reference: Supplemental Design Recommendations, Retaining Wall Design, 2625 West Galer Street, Seattle, Washington, by GeoEngineers, dated September 1, 1994.

Note: This boring log has been included for informational purposes only. CEO, Inc., makes no representations whatsoever regarding the accuracy of the data or for its interpretation by others.

Creative Engineering Options INC. CEO	BORING LOG	Plate 15

Figure A-39, Sheet 2 of 3
Log of Boring 1647-2

Boring B-1
(continued)

TEST DATA

BORING B-1
(Continued)

DESCRIPTION

Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Samples	Group Symbol	Description
						Boring completed at 80.5 feet on 06/08/94 No ground water observed during drilling

DEPTH IN FEET

Note: See Figure A-2 for explanation of symbols

GeoEngineers

LOG OF BORING

FIGURE A-3

Reference: Supplemental Design Recommendations, Retaining Wall Design, 2625 West Galer Street, Seattle, Washington, by GeoEngineers, dated September 1, 1994.

Note: This boring log has been included for informational purposes only. CEO, Inc., makes no representations whatsoever regarding the accuracy of the data or for its interpretation by others.

CEO Creative Engineering Options INC.
A Firm Practicing in the Geosciences

BORING LOG

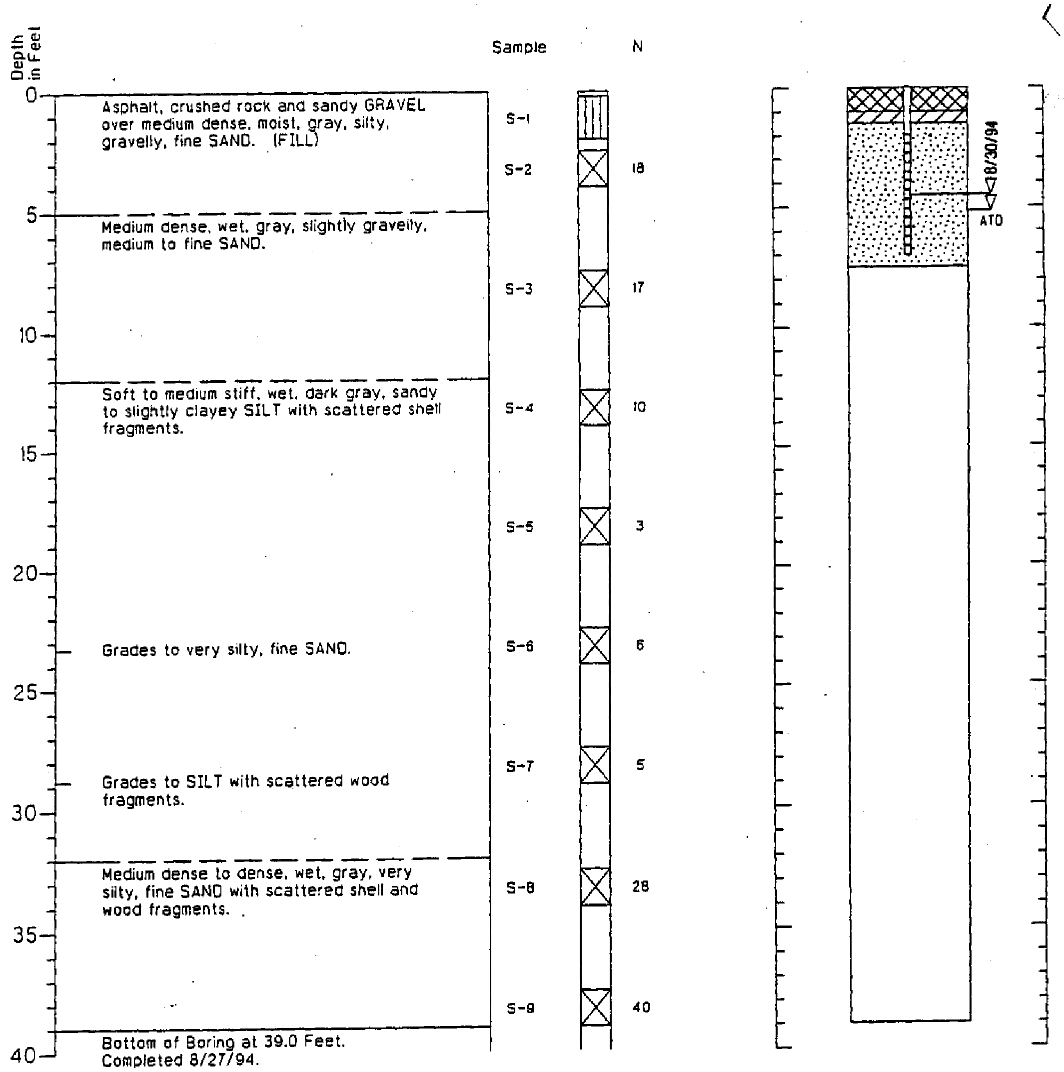
Plate 15

Figure A-39, Sheet 3 of 3
Log of Boring 1647-2

Boring Log HC-1 and Construction Data for Vapor Probe P-1

Geologic Log

Vapor Probe Design



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Ground water level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER
J-4168 8/94
Figure A-2

Assume ground surface elevation of ~15 Ft. NAVD88.

Figure A-40
Log of Boring 1650-1

1657-1

BORING NO. 1

DEPTH IN FEET	TEST DATA: Lab Tests	Moisture Content	Dry Density	Blow-Count	Samples	Group Symbol	DESCRIPTION Surface Elevation: 17.5 NAVD88
0						GW	4" ASPHALT PAVEMENT GRAVEL BASE COURSE
						SP	DARK BROWNISH-GRAY FINE TO MEDIUM SAND WITH OCCASIONAL GRAVEL (LOOSE, DRY TO MOIST) (FILL)
	DS	9.1%	107	8	■		
5							
	MD	76.9%	73	5	■	ML	MOTTLED GRAY AND BLACK SILT WITH WOODY ORGANIC MATTER AND OCCASIONAL GRAVEL (SOFT TO MEDIUM STIFF, WET)
10							
						GW	GRAY FINE TO COARSE SANDY GRAVEL WITH SHELL FRAGMENTS (LOOSE TO MEDIUM DENSE, WET)
				6	■	ML	GRAY SILT WITH FINE SAND AND ABUNDANT WOOD FRAGMENTS (SOFT, WET)
						SM	GRAY SILTY FINE SAND WITH OCCASIONAL GRAVEL AND SHELL FRAGMENTS (LOOSE TO MEDIUM DENSE, WET)
15							
	MD	18.3%	113	8	■		
20							
						SW/GW	GRAY SAND AND GRAVEL WITH A TRACE OF SILT AND OCCASIONAL SHELL FRAGMENTS (LOOSE TO MEDIUM DENSE, WET)
				9	■		
25							
				14	■	SP	GRAY FINE TO MEDIUM SAND WITH A TRACE OF SILT AND OCCASIONAL SHELL FRAGMENTS (MEDIUM DENSE, WET)
30							
	DS	17.3%	116	27	■		OCCASIONAL WOOD FRAGMENTS
35							
				29	⊠	SW/SM	GRAY FINE TO MEDIUM SAND WITH SILTY FINE TO COARSE SAND AND OCCASIONAL SHELL FRAGMENTS (MEDIUM DENSE TO DENSE, WET)
40							

Note: See Figure A-2 for Explanation of Symbols

GeoEngineers Incorporated

LOG OF BORING

FIGURE A-3

Figure A-41, Sheet 1 of 2
Log of Boring 1657-1

1657-1

BORING NO. 1 (Continued)

Depth (ft)	Lab Tests	Moisture Content	Dry Density	Blow-Count	Samples	Group Symbol	Description
40–46				28	■	SP	GRAY FINE TO MEDIUM SAND WITH TRACE SILT, OCCASIONAL WOOD AND SHELL FRAGMENTS (MEDIUM DENSE TO DENSE, WET)
46–51	MD	25.3%	100	7	■	SM	GRAY SILTY FINE TO MEDIUM SAND WITH ABUNDANT WOOD AND SHELL FRAGMENTS (LOOSE, WET)
51–56				74	■	SP	GRAY FINE TO MEDIUM SAND WITH ABUNDANT SHELL FRAGMENTS, OCCASIONAL WOOD AND COARSE SAND (VERY DENSE, WET)
56–60.5	MD	42.2%	78	4	■	ML	GRAY SILT WITH FINE SAND AND OCCASIONAL WOOD AND SHELL FRAGMENTS (SOFT TO MEDIUM STIFF, WET)
60.5–69.5				3	■	SM/ML	GRAY SILTY FINE SAND WITH SANDY SILT AND OCCASIONAL SHELL AND WOOD FRAGMENTS (VERY LOOSE TO SOFT, WET)
				4	■		

BORING COMPLETED AT 69.5 FEET ON 1/19/87

OBSERVATION WELL INSTALLED TO 69.5 FEET ON 1/19/87

WATER LEVEL MEASURED AT 5.2 FEET ON 2/6/87

DEPTH IN FEET: 40, 45, 50, 55, 60, 65, 70, 75, 80

Note: See Figure A-2 for Explanation of Symbols

GeoEngineers Incorporated

LOG OF BORING

FIGURE A-4

Figure A-41, Sheet 2 of 2
Log of Boring 1657-1

1657-2

BORING NO. 2

TEST DATA

Lab Tests	Moisture Content	Dry Density	Blow-Count	Samples
DS	2.5	130	21	■
			10	⊠
			9	■
MD	20.8	108	13	■
MD	71.7	63	4	■
			31	□
			20	■
DS	11.4%	127	32	■

DESCRIPTION

Surface Elevation: 17.7

Group Symbol	Description
	3½" ASPHALT PAVEMENT
GW	GRAVEL BASE COURSE
SP	BROWN FINE TO MEDIUM SAND WITH GRAVEL AND OCCASIONAL SHELL FRAGMENTS (MEDIUM DENSE, DRY TO MOIST) (FILL)
	STRONG HYDROCARBON ODOR - SHEEN ON SAMPLE
SP/ML	MOTTLED BLACK AND GRAY FINE TO MEDIUM SAND AND SILT (LOOSE, SOFT TO WET) HYDROCARBON ODOR
SW	DARK GRAY FINE TO COARSE SAND WITH GRAVEL, OCCASIONAL LARGE WOOD FRAGMENTS AND SHELL FRAGMENTS (LOOSE, WET) HYDROCARBON ODOR OCCASIONAL LENSES OF FINE SAND
SP-SM	GRAY FINE SAND WITH SILT AND OCCASIONAL GRAVEL AND SHELL FRAGEMENTS (MEDIUM DENSE, WET)
SM/ML	GRAY SILTY FINE SAND AND FINE SANDY SILT WITH OCCASIONAL GRAVEL, WOOD AND SHELL FRAGMENTS (VERY LOOSE TO SOFT, WET)
SW	GRAY FINE TO COARSE SAND WITH GRAVEL AND SILT (MEDIUM DENSE, WET)
SP/SW	GRAY FINE TO MEDIUM SAND WITH ABUNDANT SHELL AND WOOD FRAGMENTS AND GRAY FINE TO COARSE SAND WITH GRAVEL (MEDIUM DENSE, WET)

DEPTH IN FEET: 0, 5, 10, 15, 20, 25, 30, 35, 40

Note: See Figure A-2 for Explanation of Symbols

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LOG OF BORING

FIGURE A-5

Figure A-42, Sheet 1 of 2
Log of Boring 1657-2

1657-2

BORING NO. 2 (Continued)

Depth in Feet	Lab Tests	Moisture Content	Dry Density	Blow-Count	Samples	Group Symbol	Description
40							
				48	▪		
45							
						SM/SP	GRAY SILTY FINE SAND AND FINE SAND WITH OCCASIONAL SHELL FRAGMENTS (MEDIUM DENSE, WET)
	DS	20.3%	107	15	▪		
50							
						SM	GRAY SILTY FINE TO MEDIUM SAND WITH SHELL FRAGMENTS (LOOSE, WET)
				4	▪		
55							
						ML	GRAY FINE SANDY SILT WITH OCCASIONAL SHELL AND WOOD FRAGMENTS (SOFT TO MEDIUM STIFF, WET)
	CS	41.5%	83	4	▪		
60							BORING COMPLETED AT 59.5 FEET ON 1/20/87
							OBSERVATION WELL INSTALLED TO 59.5 FEET ON 1/20/87
							WATER LEVEL MEASURED AT 4.4 FEET ON 2/6/87
65							
70							
75							
80							

Note: See Figure A-2 for Explanation of Symbols

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LOG OF BORING

FIGURE A-6

Figure A-42, Sheet 2 of 2
Log of Boring 1657-2

1657-3

BORING NO. 3

Lab Tests	Moisture Content	Dry Density	Blow-Count	Group Symbol	DESCRIPTION Surface Elevation: 18.1 NAVD88
				GW	4" ASPHALT PAVEMENT GRAVEL BASE COURSE
				SP	GRAYISH-BROWN FINE TO MEDIUM SAND WITH OCCASIONAL GRAVEL AND SHELL FRAGMENTS (MEDIUM DENSE, DRY TO MOIST) (FILL?)
MD	4.9%	109	31		
DS	27.2%	102	13		
				SP/ SM	GRAY FINE TO MEDIUM SAND WITH SILT, OCCASIONAL GRAVEL AND SHELL FRAGMENTS (LOOSE, WET)
			5		
			5		OCCASIONAL FINE TO COARSE SAND
			3		
				SW	GRAY FINE TO COARSE SAND WITH OCCASIONAL GRAVEL, SHELL FRAGMENTS AND LARGE WOOD FRAGMENTS (DENSE, WET)
			33		
			25		
			23		

DEPTH IN FEET: 0, 5, 10, 15, 20, 25, 30, 35, 40

Note: See Figure A-2 for Explanation of Symbols

GeoEngineers Incorporated

LOG OF BORING

FIGURE A-7

Figure A-43, Sheet 1 of 2
Log of Boring 1657-3

1657-3

BORING NO. 3 (Continued)

Depth in Feet	Lab Tests	Moisture Content	Dry Density	Blow-Count	Samples	Group Symbol	Description
40						SP	GRAY FINE TO MEDIUM SAND WITH OCCASIONAL COARSE SAND AND GRAVEL AND ABUNDANT SHELL FRAGMENTS (MEDIUM DENSE, WET)
	DS	15.1%	116	23	■		
45							
				14	■	SW	GRAY FINE TO COARSE SAND WITH GRAVEL AND ABUNDANT SHELL FRAGMENTS (MEDIUM DENSE, WET)
50							
				9	■		
55						ML	GRAY FINE SANDY SILT WITH OCCASIONAL SHELL AND WOOD FRAGMENTS (SOFT TO MEDIUM STIFF, WET)
	MD	37.3%	83	3	■		
60							
				5	■	SM	GRAY SILTY FINE SAND WITH OCCASIONAL SHELL FRAGMENTS (LOOSE, WET)
65							
	MD	35.4%	85	6	■	ML	GRAY FINE SANDY SILT WITH OCCASIONAL SHELL AND WOOD FRAGMENTS (MEDIUM STIFF, WET)
70							BORING COMPLETED AT 69.5 FEET ON 1/19/87
							WATER LEVEL MEASURED AT 4.45 FEET ON 2/6/87
75							
80							

Note: See Figure A-2 for Explanation of Symbols

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LOG OF BORING

FIGURE A-8

1074-01 KSK:KGH:EL 1/29/87

Figure A-43, Sheet 2 of 2
Log of Boring 1657-3

1657-4

BORING NO. 4

TEST DATA

Depth (ft)	Lab Tests	Moisture Content	Dry Density	Blow-Count	Samples	Group Symbol	DESCRIPTION Surface Elevation: 17.6
0							4" ASPHALT PAVEMENT
						GM	GRAVEL BASE COURSE
						SP	BROWN FINE TO MEDIUM SAND WITH OCCASIONAL GRAVEL AND A TRACE OF SHELL FRAGMENTS (LOOSE, DRY TO MOIST) (FILL)
	MD	9.8%	114	9	■		
5						ML	MOTTLED GRAY AND BLACK SILT WITH OCCASIONAL WOODY ORGANIC MATTER AND PODS OF BROWN AND GRAY FINE SAND (SOFT AND MEDIUM STIFF, WET) (FILL?)
	MD	49.7%	43	4	■		
10						SP/SM	GRAY FINE TO MEDIUM SAND WITH SILT AND OCCASIONAL GRAVEL (LOOSE, WET)
				3	□		
15				4	⊠		
20						SM	GRAY SILTY FINE TO MEDIUM SAND WITH GRAVEL AND SHELL FRAGMENTS (VERY LOOSE, WET)
	DS	23.6%	103	2	■		
25				18	⊠		
30						SW/SM	GRAY FINE TO COARSE SAND WITH SILT, OCCASIONAL GRAVEL AND ABUNDANT SHELL FRAGMENTS (LOOSE TO MEDIUM DENSE, WET)
	MD	14.5%	125	10	■		
35						SW	GRAY FINE TO COARSE SAND WITH GRAVEL AND ABUNDANT SHELL FRAGMENTS (DENSE, WET)
				35	■		
40							

DEPTH IN FEET

Note: See Figure A-2 for Explanation of Symbols

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LOG OF BORING

FIGURE A-9

Figure A-44, Sheet 1 of 2
Log of Boring 1657-4

1657-4

BORING NO. 4 (Continued)

Depth (ft)	Lab Tests	Moisture Content	Dry Density	Blow-Count	Samples	Group Symbol	Description
40						SP-SM	GRAY FINE TO MEDIUM SAND WITH SILT AND OCCASIONAL WOOD FRAGMENTS (MEDIUM DENSE, WET)
				14	■		
45						SP	GRAY FINE TO MEDIUM SAND WITH A TRACE OF SILT AND OCCASIONAL SHELL FRAGMENTS (DENSE, WET)
	DS	15.1%	119	37	■		
50						SM	GRAY SILTY FINE SAND WITH OCCASIONAL SHELL FRAGMENTS (LOOSE, WET)
				4	■		
55						ML	GRAY FINE SANDY SILT WITH OCCASIONAL SHELL FRAGMENTS AND LENSES OF SILTY FINE SAND (SOFT TO MEDIUM STIFF, WET)
	CS	28.5%	92	4	■		
60							BORING COMPLETED AT 59.5 FEET ON 1/20/87

DEPTH IN FEET: 40, 45, 50, 55, 60, 65, 70, 75, 80

Note: See Figure A-2 for Explanation of Symbols

GeoEngineers Incorporated

LOG OF BORING

FIGURE A-10

Figure A-44, Sheet 2 of 2
Log of Boring 1657-4

1657-5

BORING NO. 5

Depth (ft)	Lab Tests	Moisture Content	Dry Density	Blow-Count	Samples	Group Symbol	DESCRIPTION Surface Elevation: 17.9
0							4" ASPHALT PAVEMENT
						GW	GRAVEL BASE COURSE
						SP	GRAYISH-BROWN FINE TO MEDIUM SAND WITH OCCASIONAL GRAVEL (LOOSE, DRY TO MOIST) (FILL?)
	MD	3.7%	103	7	■		
5							
						ML	MOTTLED GRAY AND BLACK FINE SANDY SILT WITH OCCASIONAL WOODY ORGANIC MATTER (MEDIUM STIFF, (WET)
				17	■		
						SM	DARK GRAY SILTY FINE TO MEDIUM SAND WITH GRAVEL AND OCCASIONAL SHELL FRAGMENTS (MEDIUM DENSE, WET)
10							
						SW	GRAY FINE TO COARSE SAND WITH GRAVEL (LOOSE TO MEDIUM DENSE, WET)
				10	⊠		
15							
						SP-SM	GRAY FINE SAND WITH SILT AND OCCASIONAL WOOD FRAGMENTS (LOOSE TO MEDIUM DENSE, WET)
	MD	24.5%	101	11	■		
20							
						SM	GRAY SILTY FINE SAND WITH OCCASIONAL WOOD FRAGMENTS (VERY LOOSE, WET)
	DS	30.2%	88	3	■		
25							
						SW	GRAY GRAVELLY FINE TO COARSE SAND WITH OCCASIONAL SHELL FRAGMENTS (DENSE, WET)
				41	■		
30							
				14	□		
35							
	MD	13.1%	125	30	■		
40							

DEPTH IN FEET

Note: See Figure A-2 for Explanation of Symbols

GeoEngineers Incorporated

LOG OF BORING

FIGURE A-11

Figure A-45, Sheet 1 of 2
Log of Boring 1657-5

1657-5

BORING NO. 5 (Continued)

TEST DATA

Depth in Feet	Lab Tests	Moisture Content	Dry Density	Blow-Count	Samples	Group Symbol	Description
40						SP	GRAY FINE TO MEDIUM SAND WITH A TRACE OF SILT, OCCASIONAL GRAVEL AND ABUNDANT SHELL FRAGMENTS (MEDIUM DENSE, WET)
				19	■		
	DS	18.0%	113	32	■		
50							BORING COMPLETED AT 49.5 FEET ON 1/20/87

Depth scale: 40, 45, 50, 55, 60, 65, 70, 75, 80

Note: See Figure A-2 for Explanation of Symbols

GeoEngineers Incorporated

LOG OF BORING

FIGURE A-12

Figure A-45, Sheet 2 of 2
Log of Boring 1657-5

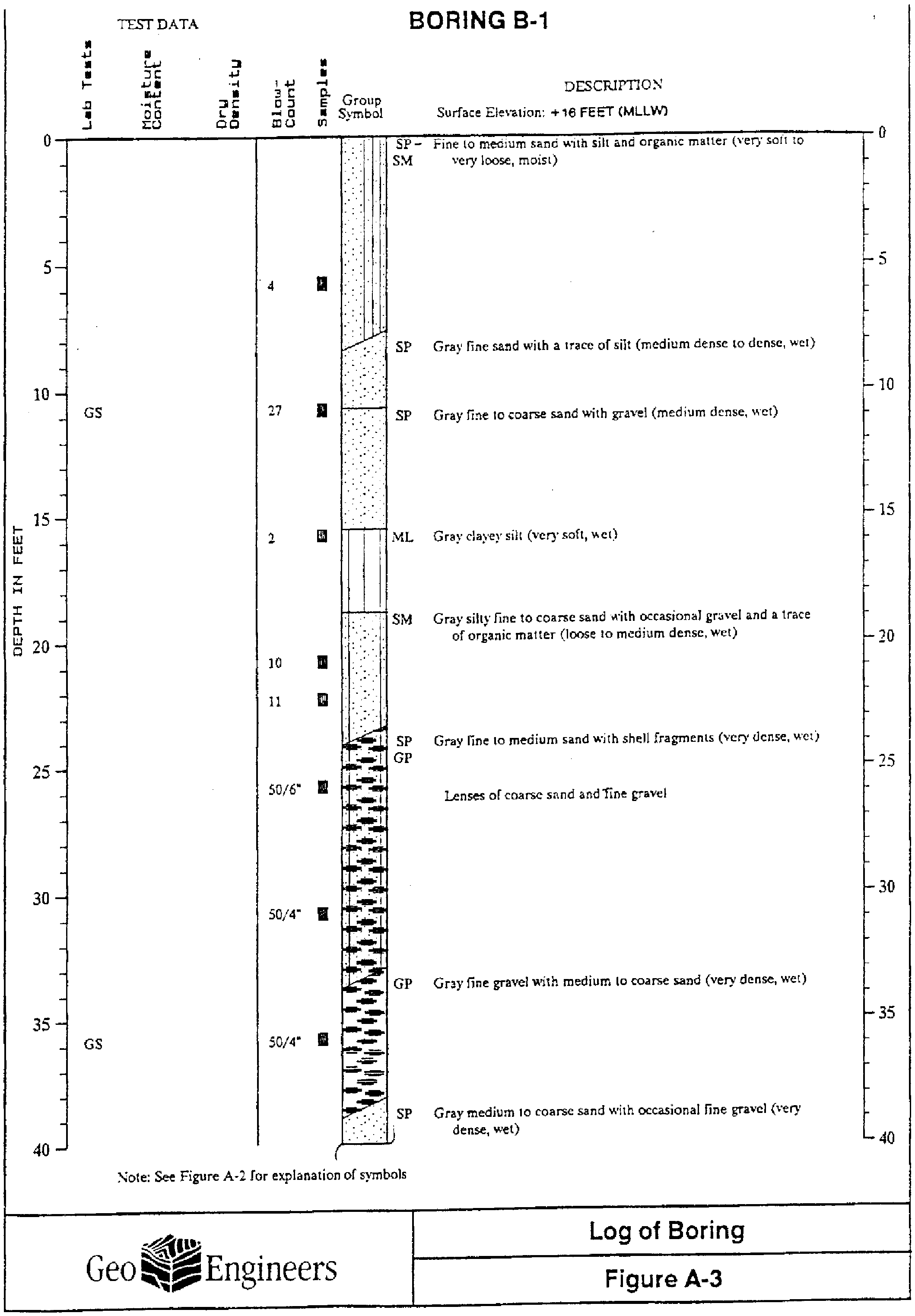


Figure A-46, Sheet 1 of 2
Log of Boring 3352-1

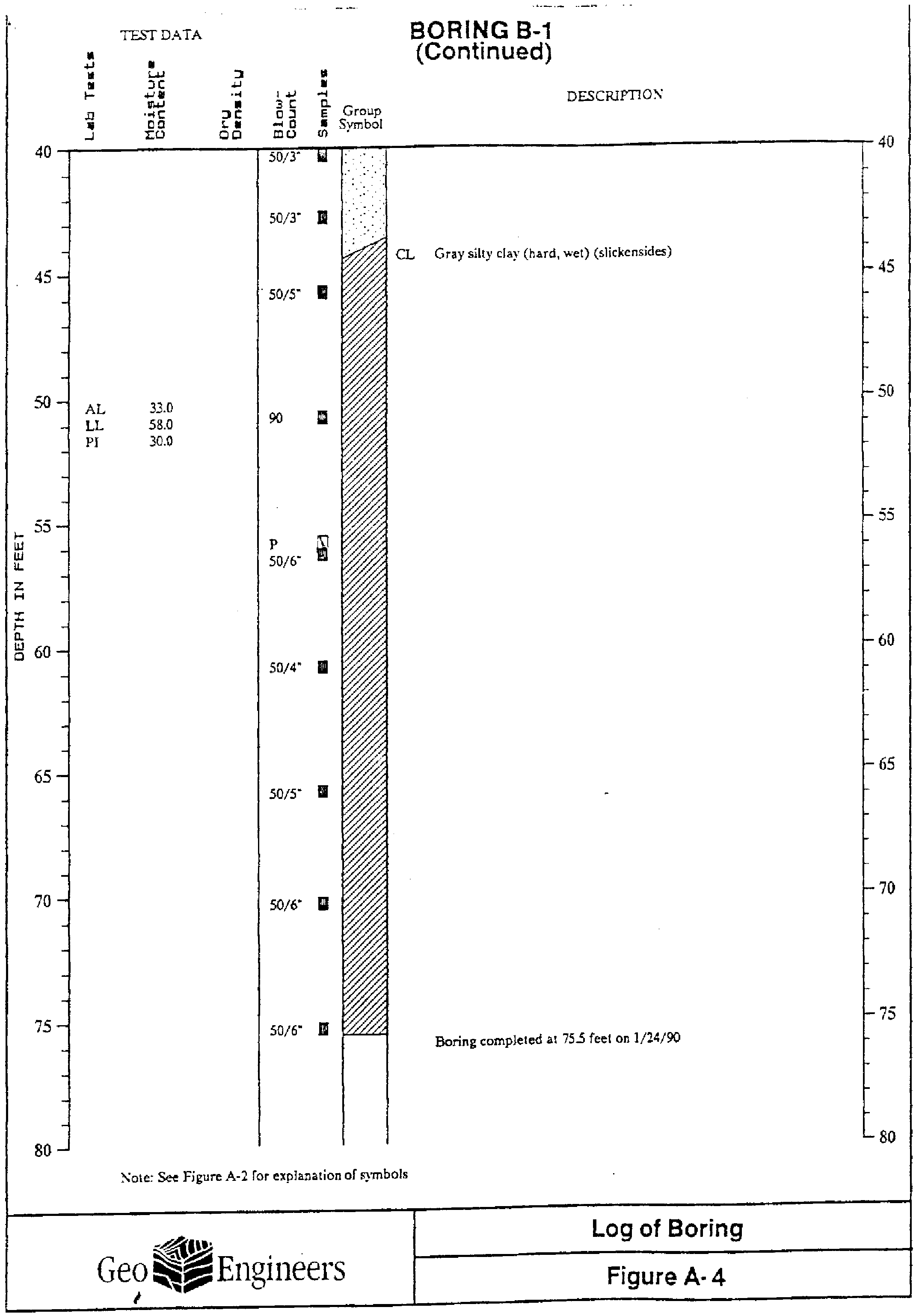


Figure A-46, Sheet 2 of 2
Log of Boring 3352-1

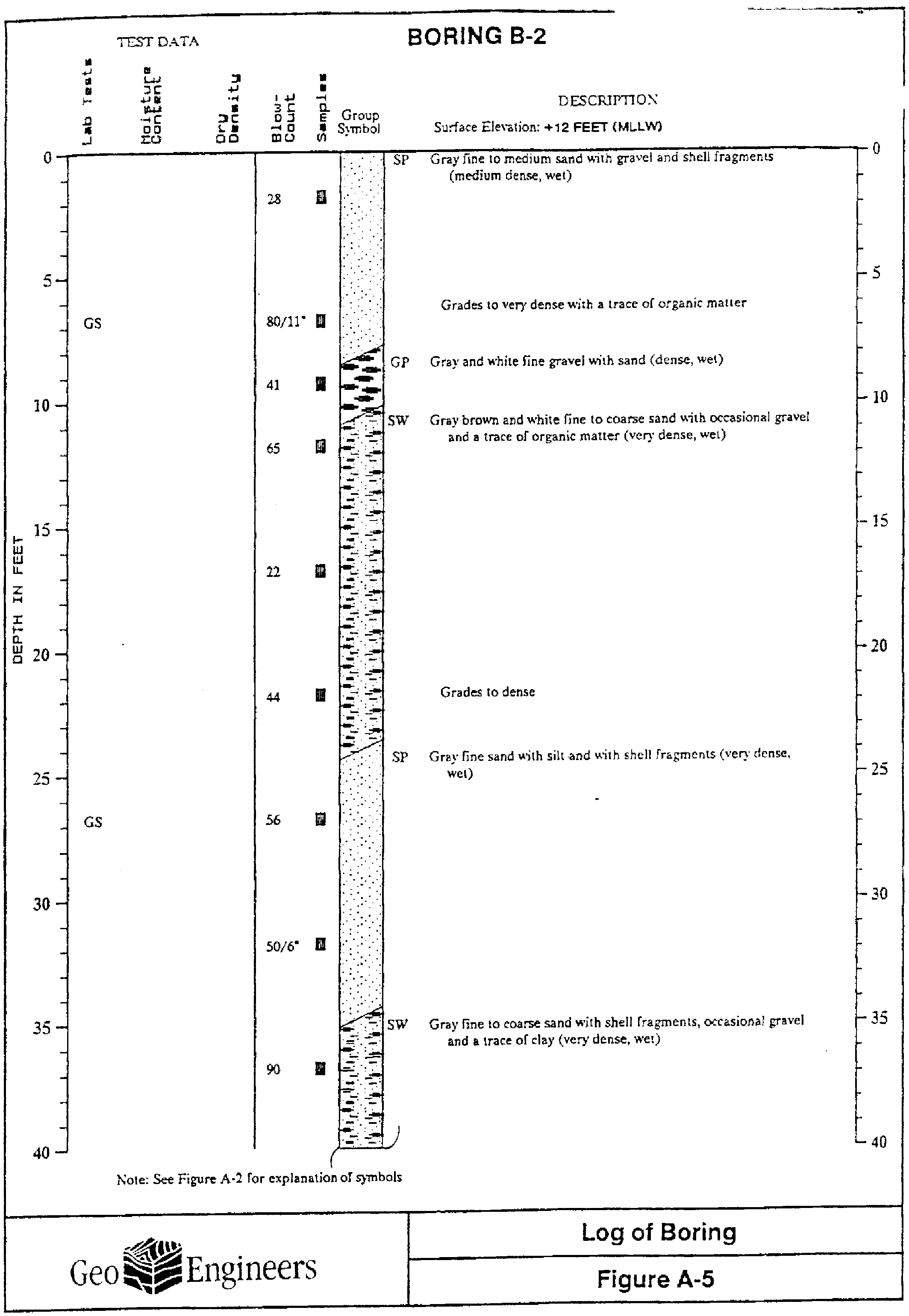


Figure A-47, Sheet 1 of 2
Log of Boring 3352-2

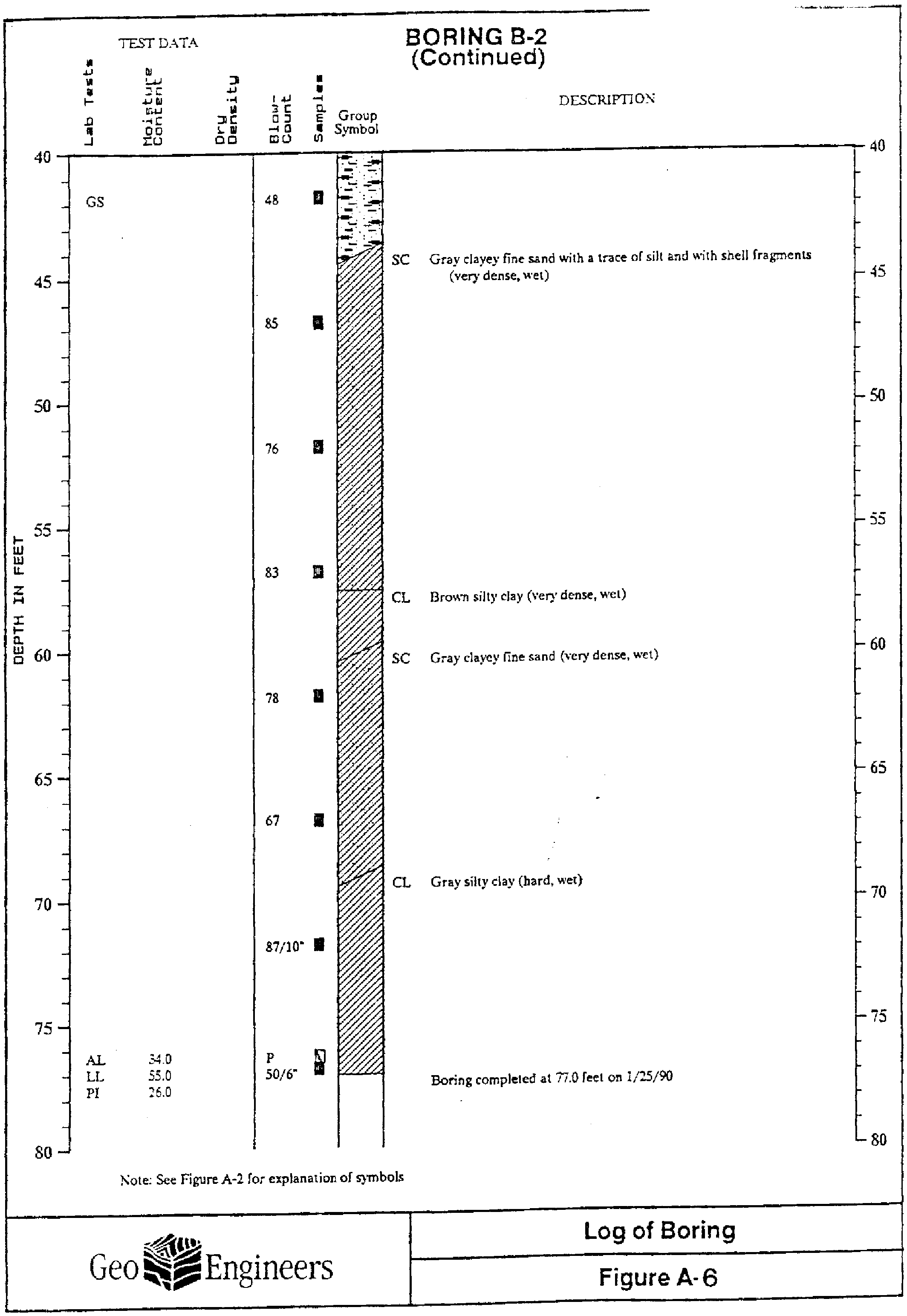


Figure A-47, Sheet 2 of 2
Log of Boring 3352-2

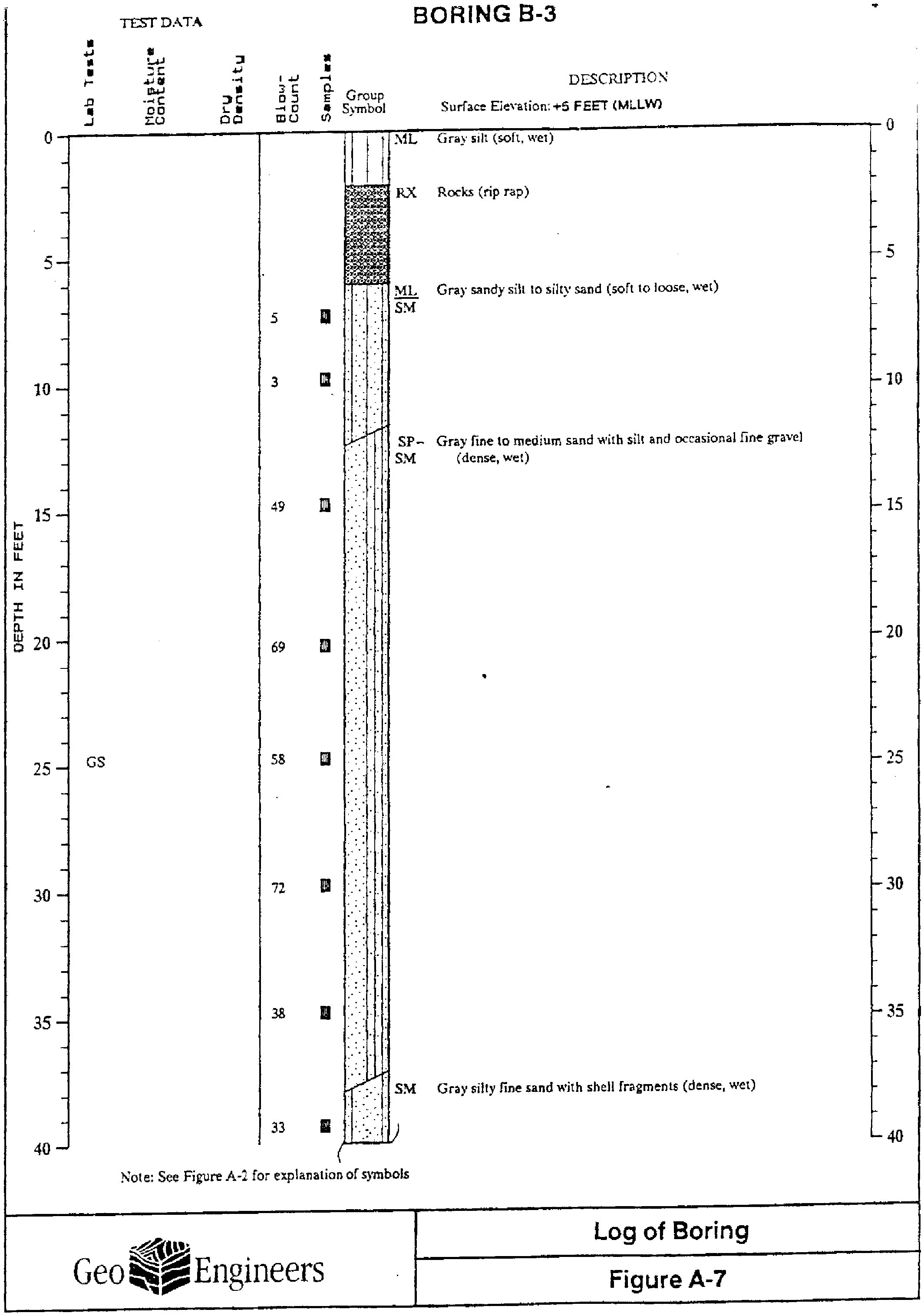


Figure A-48, Sheet 1 of 2
Log of Boring 3353-3

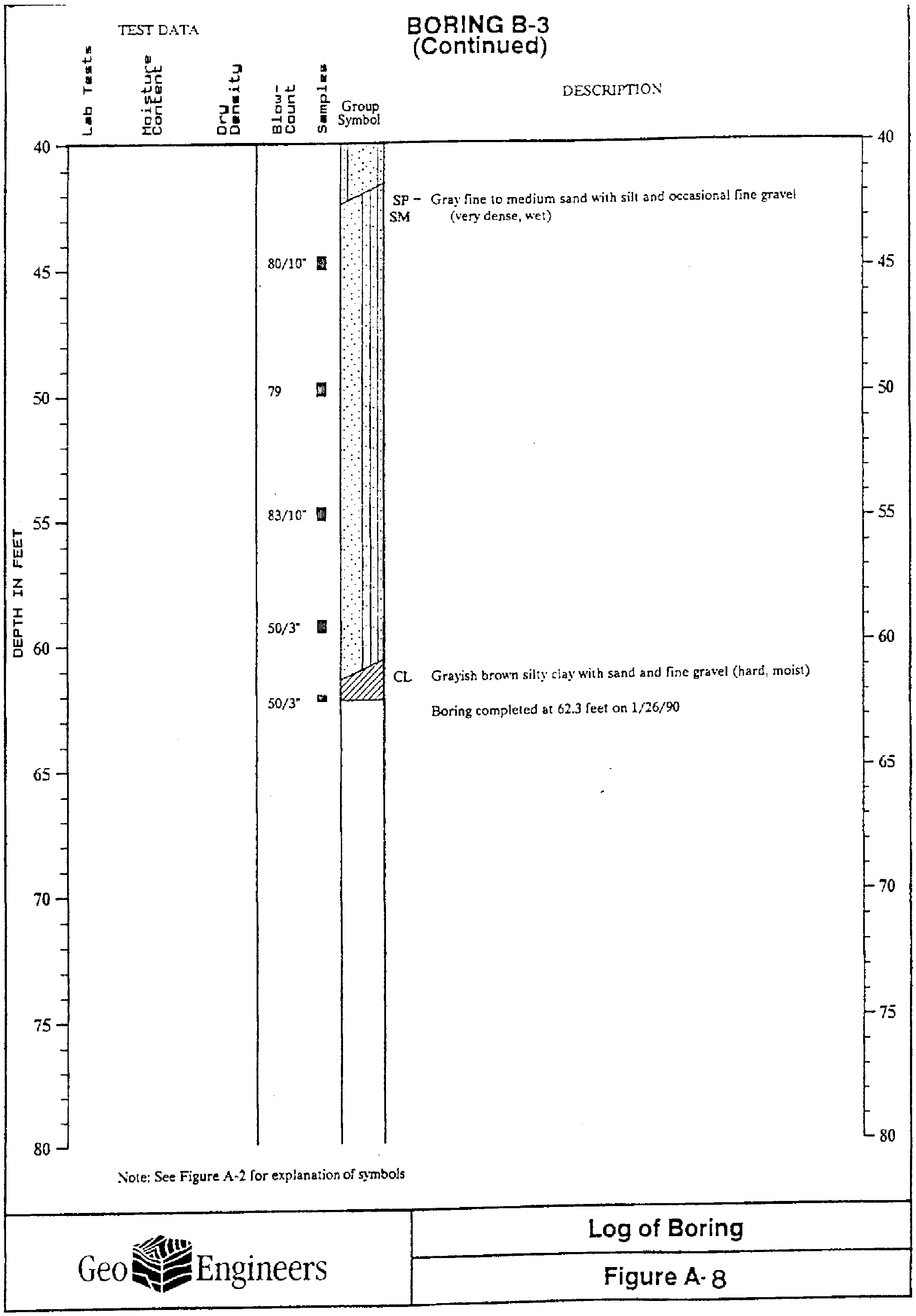


Figure A-48, Sheet 2 of 2
Log of Boring 3353-3

Sweet, Edwards & Associates, Inc.

BORING LOG

PROJECT Chemoro, Pier 91 — Page 1 of 2

Location: See Figure 2.1 — Boring No. CP-103-B

Surface Elevation: — Drilling Method: Cable Tool Rig with 6" Bit

Total Depth: 69.5' — Drilled By: Holt Drilling

Date Completed: 12/2/87 — Logged By: S. R. Henshaw

WELL DETAILS	PENETRATION TIME/RATE	DEPTH (FEET)	SAMPLE NO.	SAMPLE TYPE	PERMEABILITY TESTING	SYMBOL	LITHOLOGIC DESCRIPTION	WATER QUALITY
Flush Mount Security Casing w/Locking Cap; Concrete; Schedule 40 PVC Casing 2-inch		0–15				GW	0-15' GRAVELLY SAND, gray, medium to coarse grained, 20-30% gravel (basalt, quartzite) up to 4" in diameter, petroleum observed at 10 saturated at 10'.	
		10						
		20	103-A	SPT		SM	15-28' SILTY SAND, gray, medium grained, 15-25% silt, 5-10% sub-rounded gravel (basalt) & cobbles up to 4" diam. less than 5% shell fragm. petroleum odor, sat.	
		30	103-B	SPT		SP	28-60' SAND, gray, medium grained, clean, less than 5% silt, poorly stratified, slight petroleum odor, saturated.	
		40	103-C	SPT				
2-inch PVC Screen Schedule 40 w/0.010" Slots; Colorado Silica Sand 8-12		50					50-51.5' strong H_2S odor, saturated.	
End Cap; Slough		60	103-D	SPT		SM	60-66.5' SILTY SAND TO SANDY SILT description on following page	
		70						

SEA-300-02a

Assume ground surface elevation is the same as nearby 13-19 at ~16 Ft. NAVD88.

Figure A-49, Sheet 1 of 2
Log of Boring CP_103B

Sweet, Edwards & Associates, Inc.

BORING LOG

PROJECT Chempro, Pier 91

Boring No. CP-103-B

WELL DETAILS	PENE-TRATION TIME/RATE	DEPTH (FEET)	SAMPLE NO.	SAMPLE TYPE	PERME-ABILITY TESTING	SYMBOL	LITHOLOGIC DESCRIPTION	WATER QUALITY
		70					Cont. gray, fine grained, alternating beds of silt and sand observed in drill cuttings. 15% shell fragements (some whole shells), 5% wood debris (peat), strong H_2S odor, saturated.	
		80						
		90					Terminated boring at 69.5' 12/2/87	

Assume ground surface elevation is the same as nearby 13-19 at ~16 Ft. NAVD88.

Figure A-49, Sheet 2 of 2
Log of Boring CP_103B

LOG OF EXPLORATORY BORING

PROJECT NAME	Chemical Processors	BORING NO.	CP-108B
LOCATION	Pier 91	PAGE	1 OF 4
DRILLED BY	Tacoma Pump & Drill	REFERENCE ELEV.	4.84'
DRILL METHOD	H.S.Auger	TOTAL DEPTH	62.00'
LOGGED BY	S. Nelson	DATE COMPLETED	1/20/89

SAMPLE NUMBER	SAMPLE TYPE	BLOW COUNT (per six inches)	GROUND WATER LEVELS	DEPTH IN FT.	SAMPLES	LITHOLOGIC COLUMN	WELL DETAILS	LITHOLOGIC DESCRIPTION
								0 - 0.25 foot: ASPHALT. (AS)
								0.25 - 2.5 feet: GRAVELLY SAND; brown, fine to medium, 15% subround gravel to 1 inch in diameter. Trace to 5% shell fragments, 0-5% silt, compact, dry. (SW) (FILL)
				5				2.5 - 15.8 feet: SAND; light olive brown to olive, fine to medium, 5-10% subround gravel to 1 inch in diameter, 0-5% shell debris, some banding. Saturated, petroleum odor below 5.5 feet. (SP)
								— @ 8.0-9.0 feet: coarse sand layer with strong petroleum odor.
			▽	10				
				15				
								15.8 - 45.0 feet: SILTY SAND; olive, very fine to medium, 5-40% silt, 0-10% wood debris, organic decay - H2S odor. Saturated. Silt decreasing to 5% at 30.0 feet, wood and shell debris increase to 10%. Gravel increases to 20% at 35.0 feet, silt to 15% at 40.0 feet. (SM)
				20				

REMARKS
1) Specific Location: Garfield. 2) H.S.Auger = Hollow Stem Auger. 3) SS = Split Spo sample. 4) Water measurement at 10.0 feet BGS, at 14:00 on 1/26/89. See ADDITI REMARKS at end of Description column.

SWEET-EDWARDS/EMCON S94-07.03 CH

Assume datum is old City of Seattle datum:
4.84 Ft. + 9.7 Ft. = 14.54 Ft. NAVD88.

Figure A-50, Sheet 1 of 4
Log of Boring CP_108B

LOG OF EXPLORATORY BORING

PROJECT NAME	Chemical Processors	BORING NO.	CP-108B
LOCATION	Pier 91	PAGE	2 OF 4
DRILLED BY	Tacoma Pump & Drill	REFERENCE ELEV.	4.84'
DRILL METHOD	H.S.Auger	TOTAL DEPTH	62.00'
LOGGED BY	S. Nelson	DATE COMPLETED	1/20/89

SAMPLE NUMBER	SAMPLE TYPE	BLOW COUNT (per six inches)	GROUND WATER LEVELS	DEPTH IN FT.	SAMPLES	LITHO-LOGIC COLUMN	WELL DETAILS	LITHOLOGIC DESCRIPTION
								15.8 - 45.0 feet: SILTY SAND; see previous page for Description.
1	3" SS	5- 5- 7		25				
2	3" SS	3-17-16		30				
3	3" SS	5- 6- 8		35				
				41				

REMARKS
1) Specific Location. -field. 2) H.S.Au . = Hollow Stem Auger. 3) SS = Split Spoon sample. 4) Water me. -ment at 10.0 fe BGS, at 14:00 on 1/26/89. See ADDITIONAL REMARKS at end of L -iption column.

SWEET-EDWARDS/EMCON S94-07.03 CHEM2 SMW 04/12/89

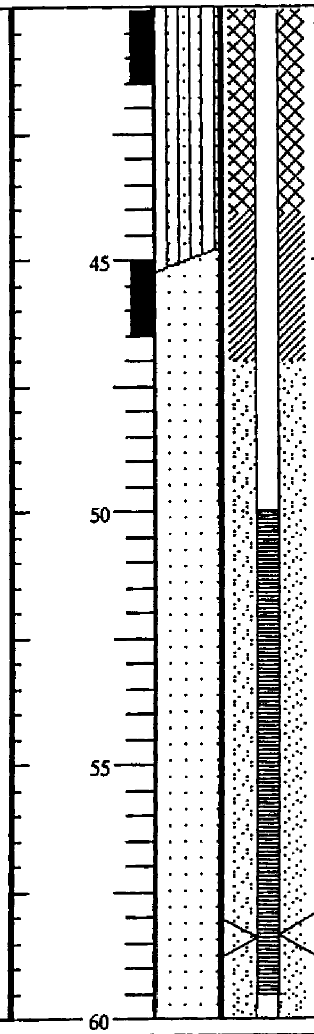
Assume datum is old City of Seattle datum:
4.84 Ft. + 9.7 Ft. = 14.54 Ft. NAVD88.

Figure A-50, Sheet 2 of 4
Log of Boring CP_108B

LOG OF EXPLORATORY BORING

PROJECT NAME	Chemical Processors	BORING NO.	CP-108B
LOCATION	Pier 91	PAGE	3 OF 4
DRILLED BY	Tacoma Pump & Drill	REFERENCE ELEV.	4.84'
DRILL METHOD	H.S.Auger	TOTAL DEPTH	62.00'
LOGGED BY	S. Nelson	DATE COMPLETED	1/20/89

SAMPLE NUMBER	SAMPLE TYPE	BLOW COUNT (per six inches)	GROUND WATER LEVELS	DEPTH IN FT.	SAMPLES	LITHO-LOGIC COLUMN	WELL DETAILS	LITHOLOGIC DESCRIPTION
4	3" SS	3- 3- 4						
5	3" SS	7- 7- 9		45				45.0 - 60.0 feet: SAND; olive, medium, 5-25% subround gravel to 1 1/2 inch in diameter, 3-10% shell debris, gravel increases in size and quantity with depth. (SP)
				50				
				55				
				60				



REMARKS
1) Specific Location: Garfield. 2) H.S.Auger = Hollow Stem Auger. 3) SS = Split Spoon sample. 4) Water measurement at 10.0 feet BGS, at 14:00 on 1/26/89. See ADDITIONAL REMARKS at end of Description column.

SWEET-EDWARDS/EMCON S94-07.03.CHEM2..SMW 04/12/89

Assume datum is old City of Seattle datum:
4.84 Ft. + 9.7 Ft. = 14.54 Ft. NAVD88.

Figure A-50, Sheet 3 of 4
Log of Boring CP_108B

LOG OF EXPLORATORY BORING

PROJECT NAME	Chemical Processors	BORING NO.	CP-108B
LOCATION	Pier 91	PAGE	4 OF 4
DRILLED BY	Tacoma Pump & Drill	REFERENCE ELEV.	4.84'
DRILL METHOD	H.S.Auger	TOTAL DEPTH	62.00'
LOGGED BY	S. Nelson	DATE COMPLETED	1/20/89

SAMPLE NUMBER	SAMPLE TYPE	BLOW COUNT (per six inches)	GROUND WATER LEVELS	DEPTH IN FT.	SAMPLES	LITHOLOGIC COLUMN	WELL DETAILS	LITHOLOGIC DESCRIPTION
								Borehole terminated at 62.0 BGS on 1/20/89.
				65				ADDITIONAL REMARKS: 5) Reference elevation at top of PVC casing, City of Seattle datum. 6) Lithologic description for CP-108-A is the same as CP-108-B to depth of 21.5 feet. Samples were taken with a Dames & Moore sampler and 300 lb. jars.
				70				
				75				
				80				

REMARKS

1) Specific Location: Garfield. 2) H.S.Auger = Hollow Stem Auger. 3) SS = Split Spoon sample. 4) Water measurement at 10.0 feet BGS, at 14:00 on 1/26/89. See ADDITIONAL REMARKS at end of Description column.

SWEET-EDWARDS/EMCON

S94-07.03.CHEM9.SMW 04/12/89

Assume datum is old City of Seattle datum:
4.84 Ft. + 9.7 Ft. = 14.54 Ft. NAVD88.

Figure A-50, Sheet 4 of 4
Log of Boring CP_108B

BURLINGTON ENVIRONMENTAL

Location: Pier 91	
Identification: CP-111	
State Permit # 046927	Permit Date: 09/08/92
Remarks: Specific Location: South of Whse 39 Reference Elevation: Top of Casing	

Consulting Firm: Burlington	Date(s): 10/10/92 – 10/10/92					
Drilling Contractor: Burlington	Elevation: 5.70					
Drilling Method: Hollow Stem Auger	Datum: City of Seattle Datum					
Logged By: James Peale	Approved By:					
Surface Casing: 0.00in N/A	From	0.0	to	0.0	ft	
Well Casing: 2.00in PVC	From	0.0	to	5.0	ft	
Sand Pack: 10-20 Silica	From	4.0	to	15.0	ft	
Slot Size: 0.010in	From	5.0	to	15.0	ft	
Seal Type: No. 1 Grout	From	0.0	to	1.0	ft	
Seal Type: No. 2 Bentonite	From	1.0	to	4.0	ft	

ELEVATION (feet) | DEPTH (feet) | SAMPLE (REC %) | SAMPLE NO. | GRAPHIC LOG | Material Description | Well Construction

MP. EL. 5.33

Elevation marks: 0.70, -9.30, -19.30, -29.30, -39.30, -49.30

Depth marks: 0, 10, 20, 30, 40, 50, 60

Sample No.: 2-4, 4-6, 6-8, 8-10, 10-12, 12-14, 14-16, 16-18, 18-20, 20-22, 22-24

(0-0.5') ASPHALT (AS)

(0.5-4') SAND (SP), dark gray brown, fine- to medium-grained, trace fine gravel, slightly damp, dense.

(4-6') SAND (SP), dark gray brown, fine- to medium-grained, some fine to coarse gravel, trace silt, damp, petroleum odor.

(6-8') SAND (SP), black, wet, strong petroleum odor, otherwise as above.

(8-14') SAND (SP), dark gray, otherwise as above.

(14-20') SAND (SP), dark gray, fine- to coarse-grained, some fine gravel, strong petroleum odor.

(20-21') SAND (SP), dark gray, fine-grained, some fine gravel, some silt, no odor.

(21-24') SILTY SAND (SM), gray, fine-grained, moist, slightly plastic.

TD = 24.00 feet

CD=15.00

CP-111

1 of 1

Assume datum is old City of Seattle datum:
5.7 Ft. + 9.7 Ft. = 15.4 Ft. NAVD88.

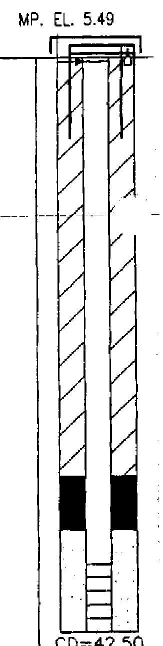
Figure A-51
Log of Boring CP_111

BURLINGTON ENVIRONMENTAL	Consulting Firm: Burlington	Date(s):02/02/93 – 03/0_ _3
	Drilling Contractor: Burlington	Elevation: 5.80
	Drilling Method: HSA/CT	Datum: City of Seattle Datum
Location: Pier 91	Logged By: James Peale	Approved By:

Identification: CP–115B						
State Permit # 046934; Permit Date: 01/19/92	Surface Casing: 8.00in Mild Steel	From	0.0	to	25.0	ft
Remarks: Specific Location: See CP–115; Reference Elevation: Top of Casing	Well Casing: 2.00in PVC	From	0.0	to	37.5	ft
	Sand Pack: 10–20 Silica	From	35.0	to	42.5	ft
	Slot Size: 0.010in	From	37.5	to	42.5	ft
	Seal Type: No. 1 Grout	From	0.0	to	31.0	ft
	Seal Type: No. 2 Bentonite	From	31.0	to	35.0	ft

ELEVATION (feet)	DEPTH (feet)	SAMPLE NO.	Material Description
	0		(0–0.5') CONCRETE (CR)
0.80		4–6	(0.5–10') SAND (SP), gray-brown, fine- to coarse-grained, wet, strong petroleum odor.
		6–8	
		8–10	
	10	10–12	(10–17.5') SAND (SW), dark gray, fine- to coarse-grained, trace fine to coarse gravel.
		12–14	
–9.20		14–16	
		16–18	
		18–20	(17.5–26.5') SILTY SAND (SM), light gray, fine-grained, low plasticity, no odor.
	20	20–22	
		22–24	
		24–26	
–19.20		26–28	(26.5–28') SAND (SP), dark gray, fine-grained, little fine to coarse gravel, trace silt, some shell debris.
		28–30	(28–32') GRAVEL (GM), light gray-brown, fine to coarse, some silt, trace fine sand, slight sulfur odor.
	30	30–32	
		32–34	(32–38.5') SILTY SAND (SM), dark gray, fine- to medium-grained, slight sulfur odor.
		34–36	
–29.20		36–38	
		38–40	(38.5–42.5') SAND (SP), dark gray, fine-grained, some silt, medium plasticity, no odor.
	40	40–42	
		42–44	(42.5–46') SILTY CLAYEY SAND (SC), olive-gray, fine-grained, some clay, stiff, high plasticity, moist.
–39.20		44–46	
		46–48	(46–56') SILTY SAND (SM), olive-gray, fine-grained, trace clay, stiff, high plasticity, damp, trace wood and shell debris.
		48–50	
	50	50–52	
		52–54	
–49.20		54–56	
			TD = 56.00 feet
	60		

Well Construction



CP_115B

Assume datum is old City of Seattle datum:
5.8 Ft. + 9.7 Ft. = 15.5 Ft. NAVD88.

Figure A-52
Log of Boring CP_115B

LOG OF EXPLORATORY BORING

PROJECT NAME	Chemical Processors	BORING NO.	TB-4
LOCATION	Pier 91	PAGE	1 OF 2
DRILLED BY	Tacoma Pump & Drill	REFERENCE ELEV.	5.86'
DRILL METHOD	H.S.Auger	TOTAL DEPTH	31.50'
LOGGED BY	S. Nelson	DATE COMPLETED	12/9/88

SAMPLE NUMBER	SAMPLE TYPE	BLOW COUNT (per six inches)	GROUND WATER LEVELS	DEPTH IN FT.	SAMPLES	LITHOLOGIC COLUMN	WELL DETAILS	LITHOLOGIC DESCRIPTION
								0 - 0.7 foot: CONCRETE. (CON)
								0.7 - 8.5 feet: SAND with gravel; olive brown, fine to medium, 5-15% subround gravel to 1 inch in diameter, 0-5% shells, 0-5% coarse sand, dry, loose, faint odor. (SP-SW) (FILL)
1	3" SS	8-10-12						
2	3" SS	11- 7- 9		5				
3	3" SS		▽					
4	2" SS							
5	2" SS							8.5 - 20.0 feet: SAND; olive grey, fine to medium, 3-5% shell debris, trace coarse sand and gravel to 1/2 inch in diameter, saturated faint petroleum odor. Silt increases to 5-10% at 20.0 feet. (SP) (FILL)
6	3" SS			10				
7	3" SS							
8	3" SS			15				
				20				

REMARKS
1) Specific Location: Office and Warehouse. 2) H.S.Auger = Hollow Stem Auger. 3) SS = Split Spoon sample. 4) Water measurement at 6.5 feet BGS, at 11:30 on 12/09/88. See ADDITIONAL REMARKS in Description column.

CP_TB-4

SWEET-EDWARDS/EMCON

S94-07 03 CHEM2 SMW 04/12/89

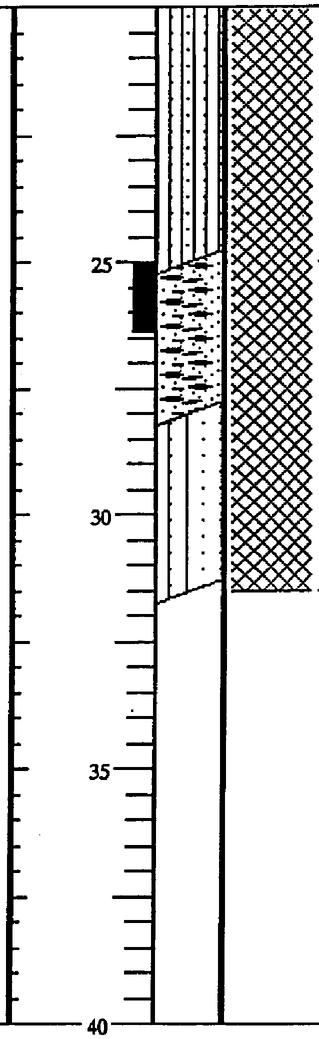
Assume datum is old City of Seattle datum:
5.86 Ft. + 9.7 Ft. = 15.56 Ft. NAVD88.

Figure A-53, Sheet 1 of 2
Log of Boring CP_TB-4

LOG OF EXPLORATORY BORING

PROJECT NAME	Chemical Processors	BORING NO.	TB- 4
LOCATION	Pier 91	PAGE	2 OF 2
DRILLED BY	Tacoma Pump & Drill	REFERENCE ELEV.	5.86'
DRILL METHOD	H.S.Auger	TOTAL DEPTH	31.50'
LOGGED BY	S. Nelson	DATE COMPLETED	12/9/88

SAMPLE NUMBER	SAMPLE TYPE	BLOW COUNT (per six inches)	GROUND WATER LEVELS / DEPTH IN FT. / SAMPLES / LITHOLOGIC COLUMN / WELL DETAILS	LITHOLOGIC DESCRIPTION
				20.0 - 25.0 feet: SILTY SAND; olive, fine, 15-30% silt, trace shells, faint odor, saturated, firm. (SM)
9	3" SS	23-24-30		25.0 - 28.0 feet: SILTY SANDY GRAVEL; olive, 60% round gravel to 2 inches in diameter, 20% fine to coarse sand, 20% silt. Saturated, loose, faint sweet odor. (GM)
				28.0 - 31.5 feet: SAND; olive, fine, 10% medium sand, 10% silt, trace shells and subround gravel to 1 1/2 inch in diameter, saturated, no odor. (SM/SP)
				Borehole terminated at 31.5 feet BGS on 12/09/88. ADDITIONAL REMARKS: 5) Reference elevation at ground surface (Pavement). WELL DETAILS - Boring has been abandoned with bentonite chips and asphalt or concrete.



REMARKS

1) Specific Location: Office and Warehouse. 2) H.S.Auger = Hollow Stem Auger. 3) SS = Split Spoon sample. 4) Water measurement at 6.5 feet BGS, at 11:30 on 12/09/88. See ADDITIONAL REMARKS in Description column.

CP_TB-4

SWEET-EDWARDS/EMCON

S94-07.03 CHEM9 SMW 04/12/89

Assume datum is old City of Seattle datum:
5.86 Ft. + 9.7 Ft. = 15.56 Ft. NAVD88.

Figure A-53, Sheet 2 of 2
Log of Boring CP_TB-4

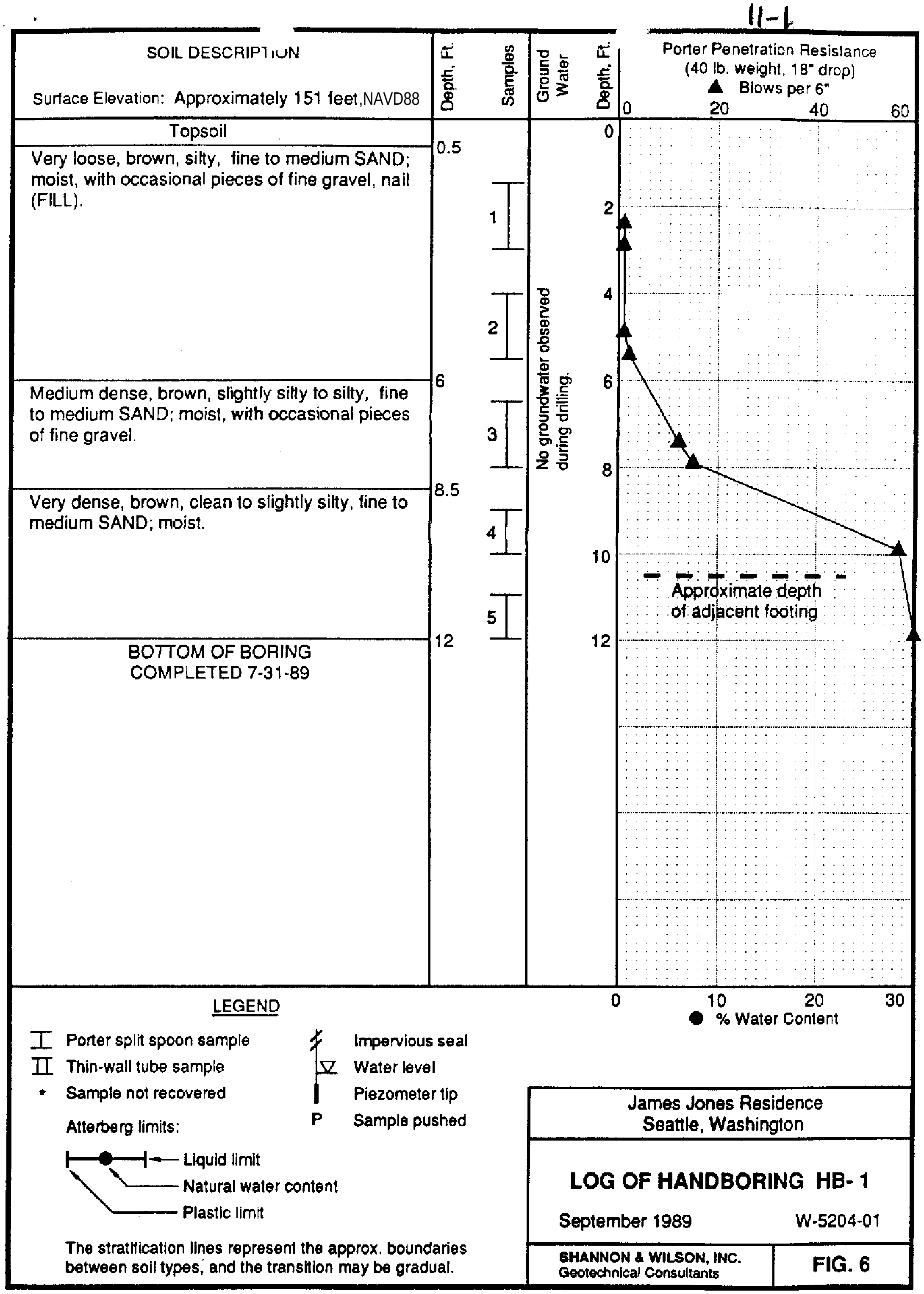
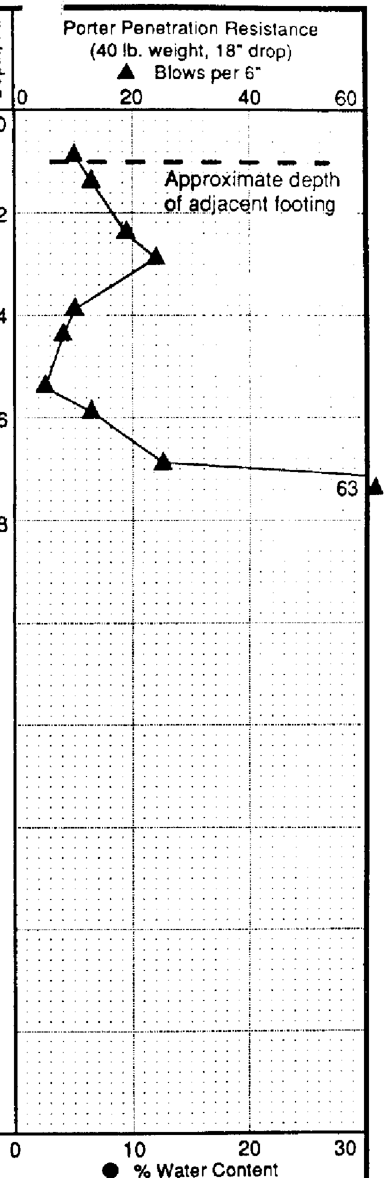


Figure A-54
Log of Hand Boring 11-1

11-6

SOIL DESCRIPTION Surface Elevation: Approximately 136 feet,NAVD88	Depth, Ft.	Samples	Ground Water	Depth, Ft.
Loose, brown, silty, fine to coarse SAND; moist, with occasional pieces of fine gravel.	0	1 2 3 4	No groundwater observed during drilling.	0 2 4
Medium dense to very dense, brown, silty, fine to medium SAND; dry to moist.	6	5		6
BOTTOM OF BORING COMPLETED 8-24-89	7.5			8



LEGEND

- Porter split spoon sample
- Thin-wall tube sample
- • Sample not recovered
- Impervious seal
- Water level
- Piezometer tip
- P Sample pushed

Atterberg limits:

- Liquid limit
- Natural water content
- Plastic limit

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

James Jones Residence
Seattle, Washington

LOG OF HANDBORING HB- 6

September 1989 W-5204-01

SHANNON & WILSON, INC.
Geotechnical Consultants

FIG. 11

Figure A-55
Log of Hand Boring 11-6

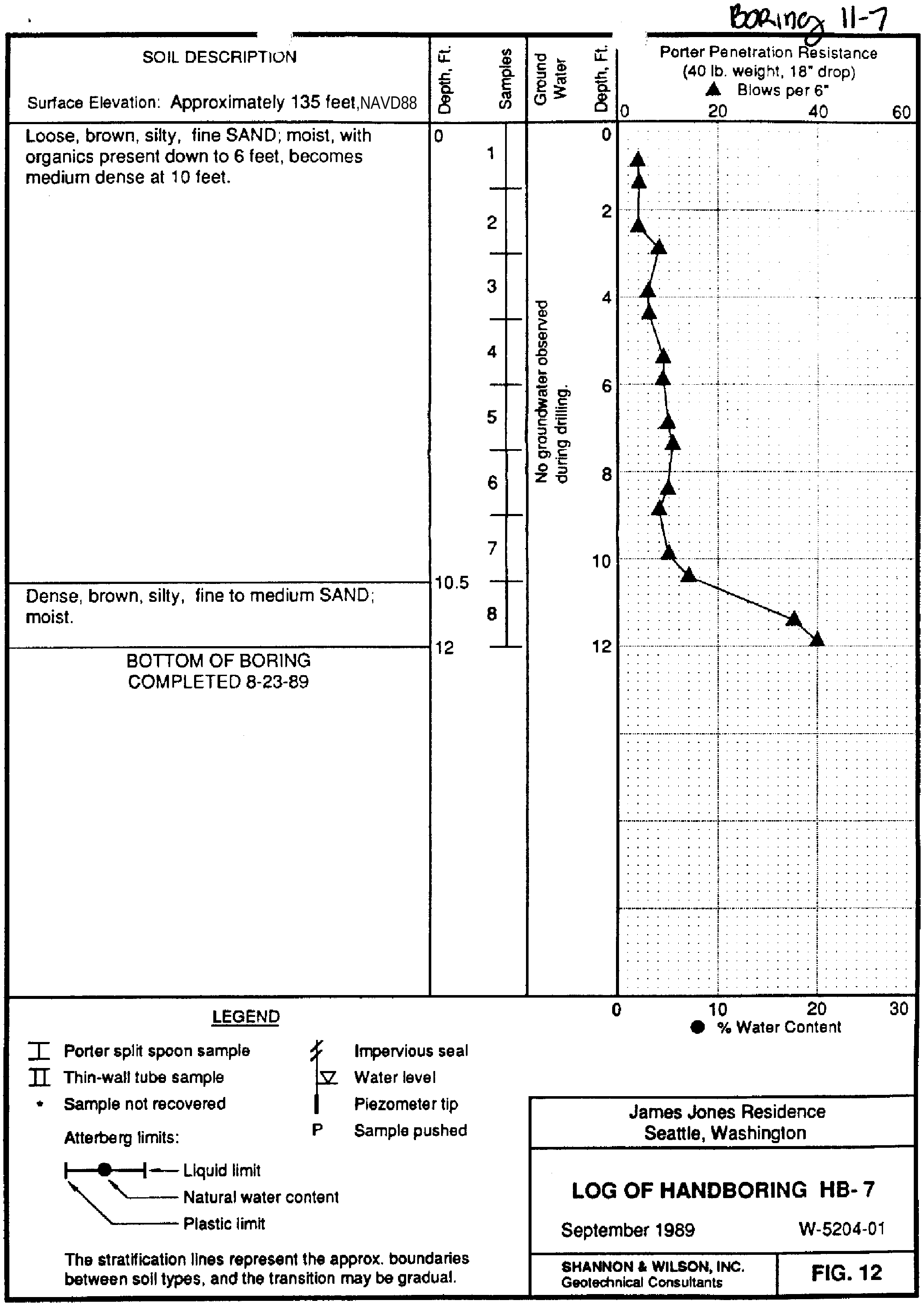


Figure A-56
Log of Hand Boring 11-7

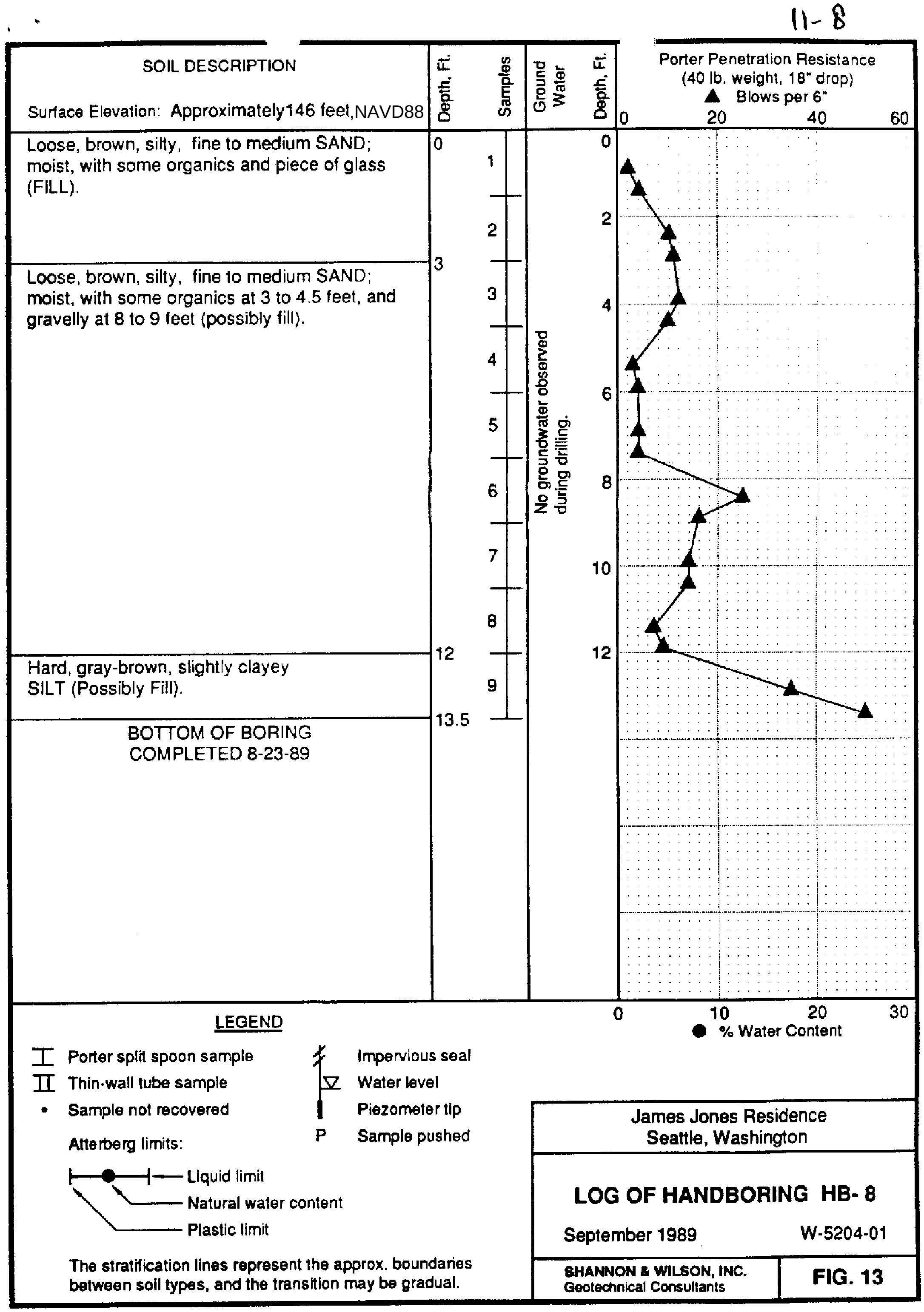


Figure A-57
Log of Hand Boring 11-8

711-1

BORING NO. 1

Logged By GPM
Date 11/17/86

ELEV. ______

Graph	US CS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)
	SM	Tan gravelly silty SAND moist medium dense.		I	18	10
			5			
	ML	Brown sandy SILT moist dense		I	45	30 15
	SM	Brown silty SAND with occasional gravel. Moist, dense becoming very dense below 17.0 feet.	10			
				I	48	14
			15			
				I	77	11
			20			
				I	78	11

Boring terminated at 24.0 feet.

TERRA ASSOCIATES
Geotechnical Consultants

BORING LOG
Proposed Magnolia Apartments
Thorndyke Ave. W. & W. Boston St.
Seattle, Washington

Proj. No. 408 | Date 12/86 | Figure 3

Assume ground surface elevation of ~142 Ft. NAVD88.

Figure A-58
Log of Boring 711-1

711-2

BORING NO. 2

Logged By GPM

Date 11/17/86

ELEV. ________

Graph	US CS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)
	SM	Brown, silty SAND; moist, loose; with wood fragments (Fill)		I	2	10
			5			
				I	4	14
			10			
	SM ML	Tan silty SAND interbedded with sandy SILT; moist loose.		I	8	27
			15			
				I	10	20
			20			
	SM	Brown, gravelly, silty SAND; moist, medium dense with rock at 26.0 feet		I	28	21
			25			
				I	27	22
			30			
	SM	Gray, silty SAND with silt lenses; moist, dense		I	44	13

Boring terminated at 34.0 feet.
Observation Well installed. No groundwater observed.

TERRA ASSOCIATES
Geotechnical Consultants

BORING LOG

Proposed Magnolia Apartments
Thorndyke Ave. W. & West Boston St.
Seattle, Washington

Proj. No. 408	Date 12/86	Figure 4

Assume ground surface elevation of ~140 Ft. NAVD88.

Figure A-59
Log of Boring 711-2

71-3

BORING NO. 3

Logged By GPM
Date 11/17/86

ELEV. ________

Graph	USCS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)
	SM	Gray-tan silty SAND; dry medium dense (Fill)		I	16	12
			5			
		Brown sandy SILT; moist, loose with charcoal bits (Fill?)		I	7	17
			10			
		Gray-black silty SAND; moist soft (Fill?)		I	13	15
			15			
	SM	Gray silty SAND with occasional gravel, Wet		I	17	14
			20			
				I	35	16

Boring terminated at 24.0 feet
Observation Well installed.
No groundwater observed.

TERRA ASSOCIATES
Geotechnical Consultants

BORING LOG
Proposed Magnolia Apartments
Thorndyke Ave. W. & West Boston St.
Seattle, Washington

Proj. No. 408	Date 12/86	Figure 5

Assume ground surface elevation of ~140 Ft. NAVD88.

Figure A-60
Log of Boring 711-3

711-4

BORING NO. 4

Logged By GPM

Date 11/17/86

ELEV. ________

Graph	USCS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)
	SM	Brown silty SAND; wet, soft with wood fragments (Fill)		I	4	20
			5			
	SM	Brown silty SAND with gravel; moist, loose		I	8	18
			10			
				I	10	20
			15			
				I	8	19
			20			
	SM	Black gravelly silty SAND moist medium dense.		I	14	12
			25			
				I	23	13

Boring terminated at 29.0 feet.

TERRA ASSOCIATES
Geotechnical Consultants

BORING LOG
Proposed Magnolia Apartments
Thorndyke Ave. W. & West Boston St.
Seattle, Washington

Proj. No. 408	Date 12/86	Figure 6

Assume ground surface elevation of ~142 Ft. NAVD88.

Figure A-61
Log of Boring 711-4

BORING 1

2216-1

Moisture Content (%)	Sample	Blows per Foot	USCS	Description
	1	7	FILL	Gray/brown SAND with silt, fine-grained, dry to moist, loose
	2	8		
	3	4		
	4	13		
	5	96	SM	Gray/brown, silty, gravelly SAND, moist, very dense (Till)
	6	50	ML	Gray/brown, clayey SILT, moist, hard
	7	47		
	8	49	SP SM	Gray SAND, fine-grained, moist, dense

Depth scale: 5, 10, 15, 20, 25, 30

Test boring 1 continued on the next page.

GEOTECH CONSULTANTS, INC.

TEST BORING LOG
1734 MAGNOLIA WAY W
SEATTLE, WA

Job No:	Date:	Logged by:	Plate:
96259	AUG 1996	DBG	3

Assume ground surface elevation of ~139 Ft. NAVD88.

Figure A-62, Sheet 1 of 2
Log of Boring 2216-1

BORING 1 CONTINUED

2216-1

Depth	Moisture Content (%)	Sample	Blows per Foot	USCS	Description
30					Gray SAND, fine-grained, moist, dense
		9	60		
35					
		10	61	SP SM	
40					
		11	73		
45					
50					

Test boring was terminated at 44 feet below grade on 7-31-96.
No groundwater seepage was encountered during drilling.

GEOTECH
CONSULTANTS, INC.

TEST BORING LOG
1734 MAGNOLIA WAY W
SEATTLE, WA

Job No:	Date:	Logged by:	Plate:
96259	AUG 1996	DBG	4

Assume ground surface elevation of ~139 Ft. NAVD88.

Figure A-62, Sheet 2 of 2
Log of Boring 2216-1

BORING 2

2216-2

Moisture Content (%)	Sample	Blows per Foot	USCS	Description
			FILL	Brown, silty SAND with roots, moist, loose
	1	2		
	2	4		
	3	5		
	4	5		
	5	8		
			SM	Gray, sandy SILT and silty SAND, fine to very fine-grained, moist, medium-dense
	6	27		
	7	42		

Depth scale: 5, 10, 15, 20, 25, 30, 35, 40

Test boring was terminated at 24 feet below grade on 7-31-96.
No groundwater seepage was encountered during drilling.

GEOTECH
CONSULTANTS, INC.

TEST BORING LOG
1734 MAGNOLIA WAY W
SEATTLE, WA

Job No:	Date:	Logged by:	Plate:
96259	AUG 1996	DBG	5

Assume ground surface elevation of ~146 Ft. NAVD88.

Figure A-63
Log of Boring 2216-2

BORING NO. B-1 2669-1

Logged By: TP Date Drilled: 2/12/97 Surface Elev: 185 feet +/-

Depth ft.	USCS	Soil Description	SAMPLE Type	SAMPLE No.	Blows per 6-inches	SPT N Blows per 1-foot	Water Content %	Other Tests & Comment
		5" Asphalt-concrete.						
	SM/ ML	Brown silty fine SAND to SILT with occ. orange mottling, loose to medium dense, non-plastic, moist.	I	S1	3,3,3	6	17.5	
5			I	S2	2,3,10	13	16.6	
	ML	Brown SILT with some sand and orange mottling, occ. thin sand lenses, medium dense, plastic, moist.	I	S3	3,5,12	17	20.4	
10			I	S4	5,8,12	20	18.8	
	SP	Brown very fine SAND with occ. orange mottling, occ. thin silt lenses, medium dense to dense, moist.						
15			I	S5	9,15,23	38	18.9	
20			I	S6	7,18,27	45	11.1	
25		End of Boring at 21.5 feet Drilling Method: 7" OD x 3.25" ID Hollow Stem. Sampling Method: 2-inch Split Spoon Sampler driven by a 140 lb. hammer from a 30 inch drop. No groundwater encountered in boring.						
30								
35								
40								

LEGEND:
- I 2" O.D. Split-Spoon Sampler
- II 3" O.D. Shelby-Tube Sampler
- III 3" O.D. California Sampler

GROUNDWATER OBSERVATION WELL:
- Seal
- Measured Water Level
- Well Tip (Screen)

GEO Group Northwest, Inc.
Geotechnical Engineers, Geologists, & Environmental Scientists

BORING LOG
PROPOSED APARTMENT BUILDING AND ADDITION
2312 - 2318 WEST BOSTON STREET
SEATTLE, WASHINGTON

DATE: 2/20/97 | JOB NO: G-0711 | PLATE 4

Assume ground surface elevation of ~132 Ft. NAVD88.

Figure A-64
Log of Boring 2669-1

2664-2

BORING NO. B-2

Logged By: TP | Date Drilled: 2/12/97 | Surface Elev: 185 feet +/-

Depth ft.	USCS	Soil Description	SAMPLE Type	SAMPLE No.	Blows per 6-inches	SPT N Blows per 1-foot	Water Content %	Other Tests & Comment
		3" Asphalt-concrete.						
	SM	Brown silty SAND, loose, moist.	I	S1	2,3,6	9	28.5	
5	ML	Brown SILT with fine gravel and occ. to heavy orange mottling, occ. thin (2-3 inch) very fine sand lenses, medium dense to very dense, non-plastic, moist.	I	S2	4,9,14	23	25.1	
			I	S3	9,19,30	49	20.9	
10			I	S4	15,24, 50/3"	74/9"	21.0	
15			I	S5	6,14,22	36	18.4	
	ML/ SP	Bluish gray SILT to fine SAND, very dense, dry.						
20			I	S6	11,22,40	62	17.6	
25		End of Boring at 21.5 feet Drilling Method: 7" OD x 3.25" ID Hollow Stem. Sampling Method: 2-inch Split Spoon Sampler driven by a 140 lb. hammer from a 30 inch drop. No groundwater encountered in boring.						
30								
35								
40								

LEGEND:
- I 2" O.D. Split-Spoon Sampler
- II 3" O.D. Shelby-Tube Sampler
- III 3" O.D. California Sampler

GROUNDWATER OBSERVATION WELL: Seal; Measured Water Level; Well Tip (Screen)

GEO Group Northwest, Inc.
Geotechnical Engineers, Geologists, & Environmental Scientists

BORING LOG
PROPOSED APARTMENT BUILDING AND ADDITION
2312 - 2318 WEST BOSTON STREET
SEATTLE, WASHINGTON

DATE: 2/20/97 | JOB NO: G-0711 | PLATE 5

Assume ground surface elevation of ~128 Ft. NAVD88.

Figure A-65
Log of Boring 2669-2

PROJECT: Pocinwong Residence	JOB NO. J-640	BORING B-1	PAGE 1 OF 3
Location: Seattle, Washington	Approximate Elevation: 168 Feet		

Depth (ft)	Soil Description	Sample Number	N-values
	Loose, moist to wet, dark brown, silty fine SAND. (Topsoil/Fill)	S-1	3
		S-2	8
5	Loose to medium dense, wet, brown, silty SAND with minor gravel and brick debris. (Fill)	S-3	10
	Dark brown silty sand in cuttings at 7 to 8 feet.		
	Medium dense, wet, brown to gray, silty fine SAND.	S-4	12
10	Grades to moist at 11 feet.	S-5	13
		S-6	19
15	Trace to minor clay at 15 feet.		
	Hard, moist, gray to brown, clayey SILT with minor to some fine sand.	S-7	25
		S-8	41
20		S-9	48
	Dense, moist, gray to brown, silty fine SAND.		
25			

Penetration Resistance: ▲ Standard, △ Other — Blows per foot (0, 10, 20, 30, 40, 50)

Explanation

- 2-inch O.D. split spoon sample
- 3-inch I.D Shelby tube sample
- No Recovery
- Groundwater level at time of drilling or date of measurement (ATD)

Inclinometer Casing Key

- Clean Sand
- Concrete
- Telescoping Section
- Grout
- 2.75 inch (I.D.) Casing

Moisture Content (0, 10, 20, 30, 40, 50): Plastic Limit — Natural — Liquid Limit

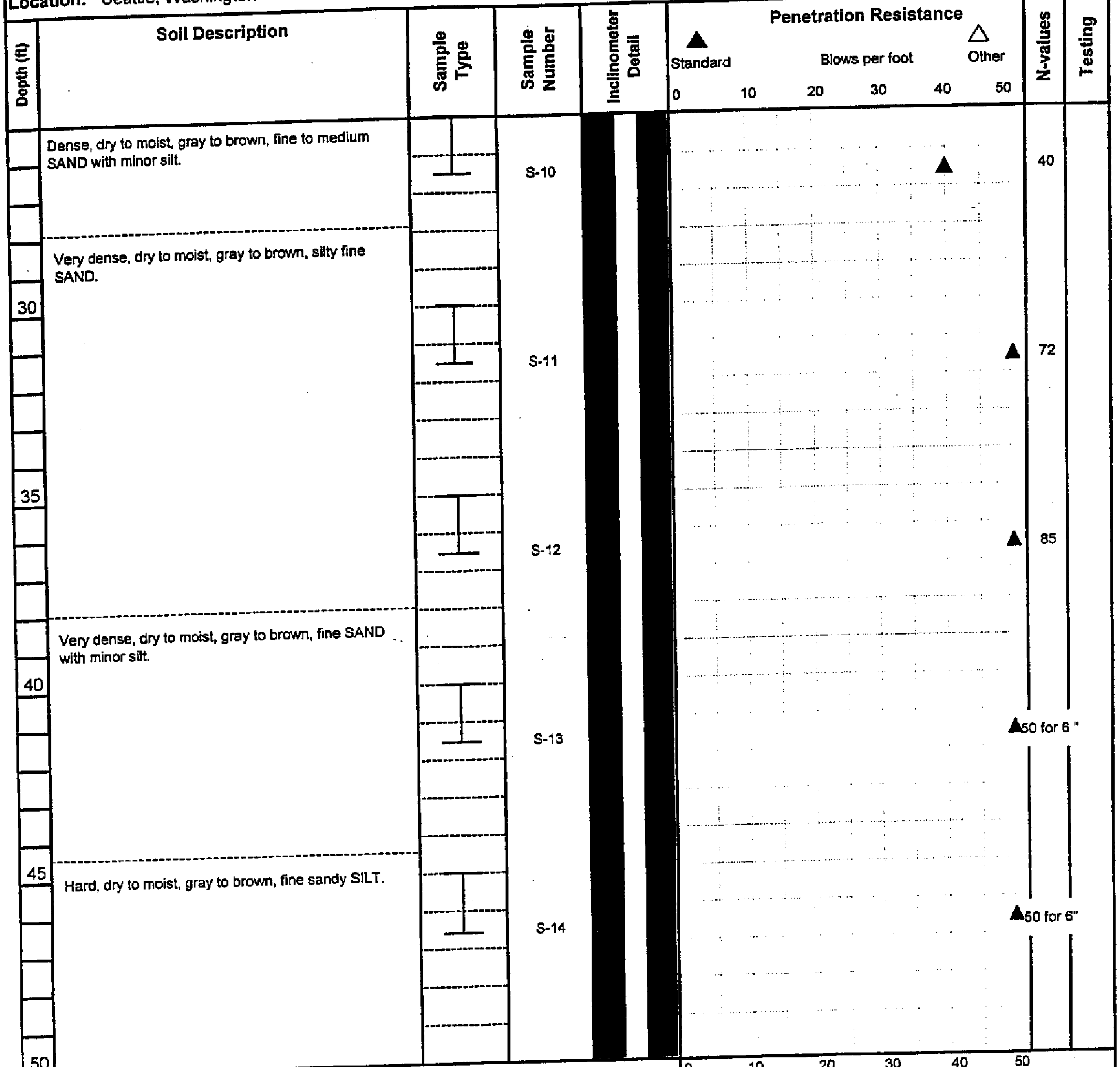
Zipper Zeman Associates, Inc.
Geotechnical & Environmental Consultants

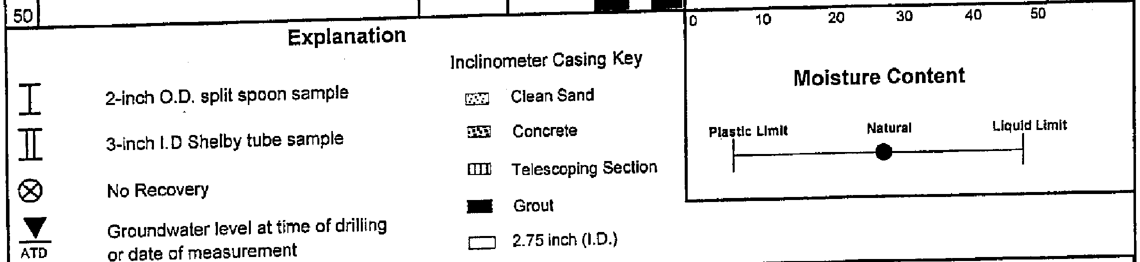
BORING LOG
Figure A-1

Assume ground surface elevation of ~178 Ft. NAVD88.

Figure A-66, Sheet 1 of 3
Log of Boring 3440-1

PROJECT: Pocinwong Residence	JOB NO. J-640	BORING B-1	PAGE 2 OF 3
Location: Seattle, Washington	Approximate Elevation: 168 Feet		





Zipper Zeman Associates, Inc. Geotechnical & Environmental Consultants	BORING LOG Figure A-2

Assume ground surface elevation of ~178 Ft. NAVD88.

Figure A-66, Sheet 2 of 3
Log of Boring 3440-1

3440-1

PROJECT: Pocinwong Residence | **JOB NO.** J-640 | **BORING** B-1 | **PAGE 3 OF 3**

Location: Seattle, Washington | **Approximate Elevation:** 168 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Inclinometer Detail	Penetration Resistance (▲ Standard, △ Other; Blows per foot 0–50)	N-values	Testing
	Very dense, dry to moist, gray, fine SAND with minor silt.	I	S-15		▲	92	
55		I	S-16		▲	50 for 6"	
	Hard, moist, gray, clayey SILT.						
60		I	S-17		▲	54	
65	Very dense, dry to moist, gray to brown, silty fine SAND.	I	S-18		▲	80	
70		I	S-19		▲	50 for 6"	
	Boring completed at 71.5 feet on 3/21/00. No groundwater observed at time of drilling.						
75							

Explanation

- 2-inch O.D. split spoon sample
- 3-inch I.D Shelby tube sample
- No Recovery
- Groundwater level at time of drilling or date of measurement (ATD)

Inclinometer Casing Key

- Clean Sand
- Concrete
- Telescoping Section
- Grout
- 2.75 inch (I.D.)

Moisture Content (0, 10, 20, 30, 40, 50)

Plastic Limit — Natural — Liquid Limit

Zipper Zeman Associates, Inc.
Geotechnical & Environmental Consultants

BORING LOG
Figure A-3

Assume ground surface elevation of ~178 Ft. NAVD88.

Figure A-66, Sheet 3 of 3
Log of Boring 3440-1

07319

SEATTLE ENGINEERING DEPARTMENT
MATERIALS LABORATORY

CB 7.241

LOG OF TEST BORING

14-5

DATE 11-14-72

HOLE NO. 4

PROJECT Dartmouth Ave. W. San. Sewer

GRD. ELEV.

LOCATION Magnolia Wy & W. Blaine St. 12' S. of ℄ #1780

STRATA	DEPTH	SAMPLE NO.	BLOW COUNT			STD. PEN.	COMPOSITION	CONSISTENCY	MOISTURE	COLOR	WATER LEVEL
Laminated SAND and SILT, v. compact Loose silty SAND							1" CL Rock				No water 11-16-72; No water 11-14-72; Installed Piezometer
						BRN	SILT - V. FINE SAND				
		A	3	4	3	7	SILTY FINE SAND	LOOSE	MOIST	BRN	
	5						W/ GRAVEL &				
							SILT LAYERS				
		B	5	13	16	29	FINE SAND W/	FIRM	MOIST	BRN	
	10						SILTY LAYERS				
		C	10	22	32	54	FINE SAND	V. COMP	WET	BRN	
	15						SILT-CLAY LAYERS		MOIST	BRN	
		D	7	17	32	49	LAYERS OF SILT	COMP	MOIST	BRN	
	20						& FINE SAND		WET	BRN	
				5"							
		E	10	50	X	100+	FINE SAND W/ GRAVEL	V. COMP	MOIST TO WET	BRN	SAND
	25						& SILT LAYERS			BRN	
							ROCK IN TIP				

INSPECTOR H.W. Cochran

Assume ground surface elevation of ~192 Ft. NAVD88.

Figure A-67
Log of Boring 14-5

9529
1/2

16-1

SEATTLE ENGINEERING DEPARTMENT
MATERIALS LABORATORY

LOG OF TEST BORING

DATE 10.30.81 HOLE NO. 1

PROJECT 23RD AVE W. SAN. SEWER REPLAC. GRD. ELEV.

LOCATION 15' E ¢ 23RD W & 26' N. ¢ W. NEWTON

STRATA	DEPTH	SAMPLE NO.	BLOW COUNT			STD. PEN.	COMPOSITION	CONSISTENCY	MOISTURE	COLOR	WATER LEVEL
		A	2	3	3	6	SILTY SAND w/ SOME GRAVEL TRACE CHARCOAL & WOOD	LOOSE	MOIST	BRN	
	5										
		B	6	14	22	36	8" SILTY SAND w/ GRAVEL 8" SANDY SILT w/ SOME CLAY	COMP	MOIST	BRN GRAY	
	10										
		C	5	23	16	40	8" SANDY SILT w/ SOME GRAVEL & CLAY 8" SILT-CLAY w/ SOME SAND & GRAVEL	COMP	MOIST	GRAY DK GRAY	
	15										
		D	6	9	15	24	SILT w/ TRACE GRAVEL	STIFF	MOIST	DK GRAY	
	20										
		E	7	9	14	23	SILT w/ SOME CLAY IN LAYERS	STIFF	MOIST	DK GRAY	
	25										
		F	5	9	12	21	SILT w/ SOME CLAY IN LAYERS	STIFF	MOIST	DK GRAY	
	30										

Assume ground surface elevation of ~95 Ft. NAVD88.

Figure A-68, Sheet 1 of 2
Log of Boring 16-1

16-1 9229 2/2

CS 7.241

SEATTLE ENGINEERING DEPARTMENT
MATERIALS LABORATORY

LOG OF TEST BORING

DATE 10.30.81 HOLE NO. 1

PROJECT 23RD AVE W. SAN. SEWER REPLACE. GRD. ELEV

LOCATION

STRATA	DEPTH	SAMPLE NO.	BLOW COUNT			STD. PEN.	COMPOSITION	CONSISTENCY	MOISTURE	COLOR	WATER LEVEL
		G	4	8	12	20	SILT - CLAY	STIFF	MOIST	DK GRAY	
	35										
		H	3	8	14	22	SILT - CLAY	STIFF	MOIST	DK GRAY	
	40										
		I	2	9	11	20	SILT - CLAY	STIFF	MOIST	DK GRAY	
	45										11.2.81
		J	2	8	18	26	SILT - CLAY	HARD	MOIST	DK GRAY	PIEZO INSTALLED SAND IN TIP. 10.30.81
BOTTOM 10.30.81	50										

Assume ground surface elevation of ~95 Ft. NAVD88.

Figure A-68, Sheet 2 of 2
Log of Boring 16-1

9530

SEATTLE ENGINEERING DEPARTMENT
MATERIALS LABORATORY

CS 7.241

16-2

LOG OF TEST BORING

DATE 10-30-81 HOLE NO 2

PROJECT 23rd AVE W. SW SEWER [illegible] GRD. ELEV

LOCATION 60'± N. ¢ W. HOWE (EXTENDED) & 120'± E. ¢ 23rd W. (EXTENDED)

STRATA	DEPTH	SAMPLE NO.	BLOW COUNT			STD. PEN.	COMPOSITION	CONSISTENCY	MOISTURE	COLOR	WATER LEVEL
		A	10	18	20	38	CLEAN SANDY SOME GRAVEL	COMP.	MOIST	GRAY	
	5										11-12-81 11-2-81
		B	3	2	8	10	12" SAND W/ SOME GRAVEL 6" SANDY SILT W/ SOME ORGANIC	LOOSE	WET	DK GRAY	
	10										
		C	—	1	—	1	14" SILT W/ TRACE SHELL 2" CLAY-SILT	V. SOFT	WET MOIST	DK GRAY	
	15										
							GOW STUCK & LOW AIRED WATER DRILLED 1st DEEPER				
	20	D	2	3	6	9	CLAY-SILT W/ SOME CLAY LUMPS	MED	MOIST	DK GRAY	
		E	3	3	4	7	CLAY-SILT	MED	MOIST	DK GRAY	
	25										
		F	5	8	14	22	CLAY-SILT	STIFF	MOIST	DK GRAY	
	30										10-30-81
BOTTOM 32 ± FT		G	7	12	17	29	SILT W/ CLAY IN LAYERS	HARD	MOIST	DK GRAY	PIEZO INSTALLED SAND IN TIP

INSPECTOR [illegible signature]

Assume ground surface elevation of ~16 Ft. NAVD88.

Figure A-69
Log of Boring 16-2

Pacific Northern Geoscience

BORING NUMBER CP205-B | SHEET 1 OF 2
PROJECT PIER 91 WELL REPLACEMENT
LOCATION Seattle, Washington
PROJECT NUMBER 95-33258-01
LOGGED BY WVG

COORDINATES N 235,740.0 E 1,618,603.0
SURFACE ELEVATION 5.53 DATUM Seattle

Depth Feet	Lab Sample	Samp. No.	Blow Counts	Rec. %	PID ppm	DESCRIPTION	BOREHOLE/WELL CONSTRUCTION DETAIL	ELEVATION FEET
						asphalt	Flush Mount Monument/Concrete Surface Seal	5
		1	14 16 17	100		SAND (sp); gray to green, fine- to very fine-grained, shell fragments; loose, damp, no odor	9" ID Mild Steel Casing Grouted in place with 10% Bentonite Cement Grout from 0-16.9' (85 Gallons Total)	
5		2	18 25 27	100		as above; trace gravel, saturated at 5.5'		0
		3	18 25 26	100		GRAVELLY SAND (swg); gray-black, medium- to coarse-grained, gravel to 3/4"; loose, wet, slight petroleum odor		
10		4	15 19 20	100		as above; gravel to 1"; wet, no odor		-5
		5	14 15 16	100		as above; slightly silty, gravel to 1/2"; wet, no odor		
15		6	12 14 16	100		as above; gray to black, fining down to very silty sand with gravel at 16'		-10
						SILTY SAND (sm); gray-black, very silty, trace gravel; firm, wet	Medium Bentonite Chips (11 bags)	
		7	12 14 16	100		SILTY SAND (sm); gray, medium- to very fine-grained, slightly to moderately silty; slightly firm, wet	2" ID Schedule 40 PVC Riser from 0-34.5'	
20		8	10 12 13	100		as above; abundant shell fragments; wet, no odor		-15
						GRAVELLY SAND (swg); gray-black, slightly silty; wet, no odor		
		9	25 30 33	100		SANDY GRAVEL (gw); gray to black, gravel to 3", sand fine to very coarse grained; loose, wet, no odor		
25		10	30 32 35	100		as above		-20
		11	18 20 23	100		SANDY SILT (ml); gray-green, moderately plastic; firm, moist to wet, no odor		
		12	18 30 35	100		SILTY SAND (sm); gray-green, wood fragments; firm, moist to wet		

DRILLING CONTRACTOR Cascade
DRILLING METHOD 10.25" & 4.25"ID HSA
SAMPLING EQUIPMENT Split Spoon
DRILLING STARTED 11/20/95 ENDED 11/21/95

REMARKS Drilling Sequence - Drilled to 26.5' w/ 4 1/4" HSA's. Opened hole to 16.5' w/ 10 1/4" HSA's. Grouted 9" casing at 16.5' and let set for 24 hours. Cleaned out casing and drilled to 44.5' w/ 4 1/4" HSA's

BIGWELL NO 1/15/98

CP_205B

Assume datum is old City of Seattle datum:
5.53 Ft. + 9.7 Ft. = elevation 15.2 Ft. NAVD88.

Figure A-70, Sheet 1 of 2
Log of Boring CP_205B

Pacific Northern Geoscience

COORDINATES N 235,740.0 E 1,618,603.0
SURFACE ELEVATION 5.53 DATUM Seattle

BORING NUMBER CP205-B SHEET 2 OF 2
PROJECT PIER 91 WELL REPLACEMENT
LOCATION Seattle, Washington
PROJECT NUMBER 95-33258-01
LOGGED BY WVG

SAMPLE INFORMATION						STRATA	DESCRIPTION	BOREHOLE/WELL CONSTRUCTION DETAIL	ELEVATION FEET
Depth Feet	Lab Sample	Samp. No.	Blow Counts	Rec. %	PID ppm				
		13	32	100			SILTY SAND (sm); gray, fine- to		-25
			33				medium-grained, abundant wood fragments and		
		14	36	100			shells; loose to slightly firm, wet, H2S odor		
			38					10/20 Colorado Sand (3.5 bags)	
			40				as above; thin silt-rich horizons; H2S odor		
		15	45	100					
			32						
			40				as above; fine- to very-fine grained; H2S odor		
		16	41	100					
35			40						-30
		17	42	100			as above		
			47						
			20					2" ID .010" Slot PVC Screen from	
		18	28	100			as above	34.5-44.5'	
			35						
			22						
		19	27	100			as above		
			33						
40			18						-35
		20	20	100			as above		
			22						
			21				as above; gray-green, fine grained, very silty;		
		21	21	100			firm, wet, H2S odor		
			24						
		22	16	100			SILTY SAND (sm-ml); gray-green, very silty,		
			18				with thin silt horizons to 1/2", wood fragments;		
			24				firm (silt layers moderately plastic), moist to wet	2" ID PVC Tail Pipe from	
							EOB at 44.75 feet.	44.5-44.75'	

BIOWELL 1/15/98

Assume datum is old City of Seattle datum:
5.53 Ft. + 9.7 Ft. = elevation 15.2 Ft. NAVD88.

Figure A-70, Sheet 2 of 2
Log of Boring CP_205B

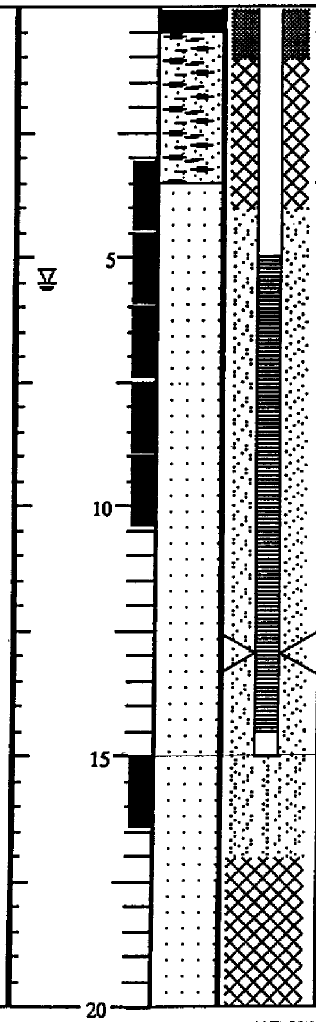
CP 1249

LOG OF EXPLORATORY BORING

PROJECT NAME	Chemical Processors	BORING NO.	CP-109
LOCATION	Pier 91	PAGE	1 OF 2
DRILLED BY	Tacoma Pump & Drill	REFERENCE ELEV.	6.21'
DRILL METHOD	H.S.Auger	TOTAL DEPTH	26.50'
LOGGED BY	S. Nelson	DATE COMPLETED	12/13/88

SAMPLE NUMBER	SAMPLE TYPE	BLOW COUNT (per six inches)
1	3" SS	16-15-17
2	3" SS	5-8-11
3	3" SS	7-9-13
4	2" SS	4-6-9
5	2" SS	2-4-7
6	3" SS	7-9-12

GROUND WATER LEVELS / DEPTH IN FT. / SAMPLES / LITHOLOGIC COLUMN / WELL DETAILS



LITHOLOGIC DESCRIPTION

0 - 0.5 foot: CONCRETE. (CON)

0.5 - 3.5 feet: SANDY GRAVEL; dark olive, fine to medium, subround 20% gravel to 2 inches in diameter, 0-5% shell and wood debris, petroleum saturated at 2.0-2.5 feet, dry, compact. (GW) (FILL)

3.5 - 20.5 feet: SAND; olive, fine to medium, 5-10% subround gravel, trace shell and wood debris, saturated below 5.5 feet with petroleum odor. (SP) (FILL)

--- @ 7.5 feet: thin silty and coarse bands to 0.25 inches thick.

--- @ 16.0 feet: thin layer of silty sand and wood debris, increased shells at 16.0-16.5 feet, petroleum odor decreasing.

REMARKS

1) Specific Location: South Oil Tanks. 2) H.S.Auger = Hollow Stem Auger. 3) SS = Split Spoon sample. 4) Water measurement at 5.5 feet BGS, at 10:00 on 12/13/88. 5)Reference elevation at top of PVC casing, City of Seattle datum.

CP_109

SWEET-EDWARDS/EMCON

S94-07 03 CHEM2 [illegible] 04/12/89

Assume datum is old City of Seattle datum:
6.21 Ft. + 9.7 Ft. = elevation 15.9 Ft. NAVD88.

Figure A-71, Sheet 1 of 2
Log of Boring CP_109

LOG OF EXPLORATORY BORING

PROJECT NAME	Chemical Processors	BORING NO.	CP-109
LOCATION	Pier 91	PAGE	2 OF 2
DRILLED BY	Tacoma Pump & Drill	REFERENCE ELEV.	6.21'
DRILL METHOD	H.S.Auger	TOTAL DEPTH	26.50'
LOGGED BY	S. Nelson	DATE COMPLETED	12/13/88

SAMPLE NUMBER	SAMPLE TYPE	BLOW COUNT (per six inches)	GROUND WATER LEVELS	DEPTH IN FT.	SAMPLES	LITHO-LOGIC COLUMN	WELL DETAILS	LITHOLOGIC DESCRIPTION
7	3" SS	10-11-12						20.5 - 25.0 feet: SILTY SAND; olive, fine, 20-30% silt, 20% wood debris, trace shell debris, organic decay odor (H2S?), saturated. (SM)
8		6-32-70		25				25.0 - 26.5 feet: SILTY SANDY GRAVEL; olive, fine to medium, 30-60% subround gravel to 2 inches in diameter, 5-10% silt, 5% wood debris, no odor, loose, saturated. (GW-GM)
								Borehole terminated at 26.5 feet BGS on 12/13/88.
				30				
				35				
				40				

REMARKS

1) Specific Location: South Oil Tanks. 2) H.S.Auger = Hollow Stem Auger. 3) SS = Split Spoon sample. 4) Water measurement at 5.5 feet BGS, at 10:00 on 12/13/88. 5) Reference elevation at top of PVC casing, City of Seattle datum.

CP_109

SWEET-EDWARDS/EMCON

S94-07.03 CHEM9 SMA 04/12/89

Assume datum is old City of Seattle datum:
6.21 Ft. + 9.7 Ft. = elevation 15.9 Ft. NAVD88.

Figure A-71, Sheet 2 of 2
Log of Boring CP_109

Appendix B

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Attachment to and part of Report 21-1-09759-008

Date: February 9, 2005
To: Mr. Pete Smith
HNTB

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors which were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland