Preliminary Geotechnical and Foundation Considerations

The Waterfront District
Bellingham, Washington

for
Port of Bellingham

June 15, 2011

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File No. 0307-056-02

June 15, 2011

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INTRODUCTION

This report presents preliminary geotechnical considerations related to future development of the Downtown Waterfront Area, the Log Pond Area, and the Shipping Terminal Area, which are essentially "sub-areas" of the Bellingham Waterfront District. The Waterfront District has been identified as an important future area of development for the City of Bellingham (COB). The Port of Bellingham (POB) and COB have joined together to create a path forward to transform this largely vacant brownfield site to a thriving mixed-use urban neighborhood with associated infrastructure and amenities.

Draft and Final Environmental Impact Statements (EIS) have been prepared for the entire 220-acre Waterfront District which identifies impacts and mitigation strategies. Additional sub-area specific development assumptions and strategies are outlined in "The Waterfront District, Draft Sub-Area Plan 2010" prepared by the POB in cooperation with the COB. Figure 1 presents sub-areas identified in the Draft Waterfront District Sub Area Plan.

The Waterfront District includes five unique areas as shown in Figure 1: Marine Trades Area, Downtown Waterfront Area, Log Pond Area, Shipping Terminal Area, and Cornwall Beach Area. The three contiguous areas located along the south side of Whatcom Waterway/Bellingham Bay are the subject of this report: the Downtown Waterfront Area, the Log Pond Area, and the Shipping Terminal Area. This portion of the Waterfront District was formerly the Georgia Pacific paper mill site. The POB has extensive environmental data and has prepared cleanup strategies for the Waterfront District including these areas. Geotechnical data is also available from previous site development activities.

The purpose of this report is to provide preliminary foundation considerations for the type of development envisioned for the Downtown Waterfront Area, the Log Pond Area, and the Shipping Terminal Area. The scope of services included:

- Review of the pertinent sections of the EIS and Draft Sub-Area Plan 2010
- Compilation and review of geotechnical data at the site and preparation of generalized subsurface profiles for each of the three sub-areas for this report
- Preliminary geotechnical analyses related to ground performance under static and seismic loading
- Collaboration with a structural engineer (KingWorks Consulting Engineers) regarding typical foundation loading, foundation alternatives and cost information
- Alternatives analysis of various ground improvement and foundation types for the buildings envisioned for the initial/transitional build-out of each sub-area
- Discussions with specialty contractors
- Presentation of summary data and conclusions regarding foundation support and ground improvement strategies and associated preliminary incremental costs

GeoEngineers' scope of work is presented in our proposal dated April 28, 2011 which was authorized by the POB Standard Agreement for Professional Services dated May 11, 2011. The conclusions presented in this report are presented for preliminary planning purposes only and are not intended for design. The site has complex subsurface conditions, particularly related to depth to foundation bearing layers; seismic ground response, and liquefaction and lateral spreading; and consolidation of the medium stiff clays under foundation loading. Therefore, site specific analyses will be required once specific sites and projects are identified.

THE WATERFRONT DISTRICT DEVELOPMENT

The Waterfront District redevelopment is intended to implement the community vision for the Central Waterfront. It is envisioned to include a mix of uses and infrastructure during different phases of development over a long period of time. The Waterfront District will include parks and open space, residential, educational/institutional, retail, businesses, light industrial and marine-related development. Some areas may have transitional development and different character at different times. The character of the three sub-areas that are subject of this report are shown in Figure 2 from the Draft Waterfront District Sub-Area Plan, and are described in more detail below.

Height limits have been established for the different areas of development and are shown in Figure 3 from the Draft Waterfront District Sub-Area Plan. The height limits can dictate the type of structure and foundation loading, which in turn can drive the foundation and ground improvement strategies considered for each area. We have assumed that the structures will not have more than one-half story below grade because of the high groundwater condition and potential contamination issues associated with the site. A summary of building height assumptions is provided below based on review of the plan and conversations with POB staff, and the character and loading considerations for each area are discussed in the subsequent sections.

- Maximum building heights are limited to 35 feet within 100 feet of the Ordinary High Water Mark (OHWM), and 50 feet within 100 to 200 feet of the OHWM for all areas.
- Any structures within 200 feet of the OHWM within the Downtown Waterfront area would likely be limited to small pedestrian-friendly commercial facilities like coffee shops.
- No structures are planned at this time within 200 feet of the OHWM in the Log Pond Area.
- Structures already exist within 200 feet of the OHWM in the Shipping Terminal area, which is still active. The existing Shipping Terminal pier and bulkheads are the subject of a current study being undertaken by the POB and therefore no new structures are considered at this time.

Downtown Waterfront Area

The Downtown Waterfront Area is approximately 40 acres that is envisioned to be a mix of housing, office and institutional uses. A waterfront promenade will be located along the edge of Whatcom Waterway. A site for a higher education campus is identified along the southern edge of this area, and is within the 35 to 100 foot height limit zone. A high density configuration of housing and office is proposed to be centered around the Commercial Street Green open space and Bloedel Avenue. The buildings in this area could approach 150 to 200 feet tall as indicated in Figure 3. Additional specific assumptions for these two conditions are detailed below.

Institutional Design

- University type project

  - Will have high long-life, Occupancy Category III in accordance with the 2009 International Building Code (IBC)
  - Steel and/or concrete building construction
  - Maximum height of 100 feet, high ceiling ground floor, minimum 2 floors parking, then maximum five floors of 12- to 14-foot floor height (maximum 8 floors)
  - A minimum 30,000 square foot (sf) footprint was assumed to determine reasonable cost distribution over the entire structure

Commercial Mixed Use Design

- Retail/commercial/office in lower floors, some parking, some residential
  - Occupancy Category II or III in accordance with IBC 2009
  - Steel or concrete building construction
  - Maximum height of 200 feet, or 18 stories
  - A minimum 30,000 sf footprint to was used to determine reasonable cost distribution over the entire structure

Log Pond Area

The Log Pond Area is approximately 45 acres and is identified as a transitional mixed area with light industrial uses for several decades with ultimate build out to be more mixed use. Building height limits range from 35 to 100 feet. We have made the following assumptions regarding the typical construction in this area:

- One to two story warehouse buildings
- Typical bay spacing of approximately 40 feet
- Design floor loading of 250 pounds per square foot (psf) and 500 psf
- A minimum 20,000 sf footprint to was used to determine reasonable cost distribution over the entire structure

Shipping Terminal Area

The Shipping Terminal Area is approximately 26 acres. The existing deep water port in this area will be maintained for future shipping, port and institutional related opportunities. The portion of this area closest to the terminal is envisioned to be industrial, with potential other uses in the peripheral portion of the area. Building height limits range from 35 to 100 feet. We have assumed that the interim buildings will be similar to what is currently at the site:

- One to two story warehouse buildings
- Typical bay spacing of approximately 40 feet
- Design floor loading of 250 psf and 500 psf – although floor loading in this area could be higher for businesses supporting terminal/container traffic
- A minimum 20,000 sf footprint to was used to determine reasonable cost distribution over the entire structure

SITE CONDITIONS

The Downtown Waterfront, Log Pond and Shipping Terminal Areas (hereafter referred to as the site) were created by filling tidal areas of Bellingham Bay. Most of the site has been filled to about Elevation 15 feet (MLLW). Original development included wharf supported structures and a railroad trestle across tidal mudflats. The Whatcom Creek Federal Waterway was established in the early 1900s. Filling was accomplished by a variety of methods including silt dredged from the adjacent waterway and conventional land-based filling closer to shore.

Most of the historical development at the site was related to pulp and paper industry. The site along the Whatcom Waterway has a mixture of some earth slopes, and timber and cast-in-place concrete bulkhead retaining structures. Most existing structures were/are supported on pile foundations. The early era structures also typically had/have timber pile foundation support. The piles for the structures close to the original bluff were driven to bedrock; most of the piles in the offshore area gain capacity as friction piles. Some of the more recent structures have longer steel piles driven to bedrock.

GeoEngineers reviewed our own files and geotechnical reports provided by the Port of Bellingham regarding site specific subsurface conditions. Figure 4 shows the approximate locations of explorations identified and reviewed for this report. The explorations were used to create generalized subsurface profiles for the entire site and generalized cross sections for each of the three sub-areas.

Environmental Considerations

As a result of the historic industrial uses, portions of the site are affected by soil, groundwater and/or sediment contamination. The Washington State Department of Ecology (Ecology), the POB and COB are working cooperatively to integrate site cleanup, habitat restoration, and redevelopment activities. This report does not address contamination issues. However, the potential presence of contamination was considered in the selection of ground improvement and foundation support alternatives.

Geology and Generalized Subsurface Conditions

We reviewed a U.S. Geologic Survey (USGS) map, "Geologic Map of Western Whatcom County, Washington" by Easterbrook (1976), Washington Department of Geology and Earth Resources map “Geologic Map of the Bellingham 1:100,000 Quadrangle, Washington,” by Lapen (2000) and “Coastal Zone Atlas for Whatcom County, Washington by Ecology (1978). The geologic deposits in the project vicinity are the result of both glacial and non-glacial processes that have occurred during the last 12,000 years, and recent modified land by human activity. The most recent glacial events include the Vashon and Sumas Stades of the Fraser Glaciation and the intervening Everson Interstade. The Vashon and Sumas Stades were periods of glacial advancement, and the Everson Interstade was a period of glacial retreat. Sea level fluctuated significantly in response to the glacial advance and retreat (melting), relative to the land surface and present day sea level.

The geologic maps indicate that the project site is in an area mapped as artificial fill, which generally occurred in the early 1900s as previously discussed. The site is a previous beach and intertidal area and therefore the upper native soils are a combination of beach deposits and Nooksack deposits. The other known geologic units in increasing age include, Outwash Sand and

Gravel, Bellingham Drift, Deming Sand, Kulshan Drift and Chuckanut Formation bedrock. The various geologic units and generalized geotechnical properties are discussed below.

Artificial Fill

Fill in the project vicinity is typically on the order of 10 to 20 feet thick and is variable in composition, ranging from clay derived from dredging and variable silt, sand, clay and debris from upland sources. The fill is generally loose/soft, compressible, with low shear strength, and the granular zones are moderately to highly susceptible to soil liquefaction below the water table. The fill has been found to contain higher debris content toward the bluff.

Beach Deposits

Beach, intertidal and Nooksack deposits are all lumped into Beach Deposits for purposes of discussion in this report. These deposits are quite variable in character and range from loose sand and silty sand to soft silt and clay. The beach and intertidal deposits typically are generally about 20 feet thick over most of the site, but range from 10 to 50 feet thick. The beach/intertidal deposits are moderately compressible, with low shear strength, and the granular zones are highly susceptible to liquefaction.

Outwash Sand and Gravel

A dense sand layer of unknown original is located toward the middle of the Log Pond Area. This could be an Outwash Sand and Gravel unit, which has also been identified in the Marine Trades Area. The geotechnical data suggests this unit has moderate shear strength and low compressibility.

Bellingham Drift

The Bellingham Drift is a glaciomarine drift deposit which consists of unsorted, unstratified silt and clay with varying amounts of sand, gravel, cobbles and occasional boulders. Glaciomarine drift is derived from sediment melted out of floating glacial ice that was deposited on the sea floor. Glaciomarine drift was deposited during the Everson Interstade approximately 11,000 to 12,000 years ago while the land surface was depressed 500 to 600 feet from previous glaciations. The upper 5 to 15 feet of this unit in upland areas is typically stiff. However, the glaciomarine drift is typically soft to medium stiff clay in offshore environments and has relatively low shear strength and moderate to high compressibility characteristics.

Deming Sand

The Deming Sand is a stratified, well sorted, medium- to coarse-grained sand with some layers of clay, silt and gravel deposited by glacial outwash streams. The Deming Sand is typically medium dense to dense with moderate shear strength and low compressibility.

Kulshan Drift

The Kulshan Drift is also a glaciomarine drift. It is an unsorted and unstratified mixture of silt, clay, sand, and pebbles similar in nature and characteristics to the Bellingham Drift.

Chuckanut Formation

The Chuckanut Formation is a mixture of sandstone, conglomerate, shale, and coal. The formation was partially eroded and weathered prior to the Pleistocene age (about 3 million years ago), when it was subjected to glacial activity. The Chuckanut Formation dips deeper toward the water; however, it is locally irregular and undulating such that it is quite unpredictable. The depth to bedrock is typically 10 to 30 feet near the original bluff line, and extends toward Elevation -100 and even deeper than Elevation -150 feet toward Whatcom Waterway. The sandstone is relatively hard and strong where unweathered; however, weathering can be quite variable over very short distances (even within pile caps). The Chuckanut Formation had a local coal seam southeast of the project area which was actively mined in the 1800s. More information regarding mining activity is provided in the following sections of this report.

Groundwater

The groundwater conditions are typical of an intertidal fill zone and will vary as a function of location, material type, season, distance and influence from Bellingham Bay, and other factors. Groundwater was typically observed in explorations on the order of 3 to 8 feet below the existing ground surface (bgs). For planning purposes at this level of effort, the groundwater elevation across most of the site can be assumed to be near Elevation 10 feet with several feet of fluctuation.

Generalized Area Specific Conditions

We compiled available geotechnical boring data and prepared a generalized discussion of subsurface conditions in each of the three sub-areas. The locations of the borings are shown in Figure 4. A generalized cross section for each of the areas is also presented.

Downtown Waterfront Area

The subsurface conditions within the Downtown Waterfront Area have been explored along the bluff for private development, near the toe of the bluff for railroad and other projects, and sporadically in the remainder of the area for historic industrial projects. The data is relatively sparse. A generalized cross section, Figure 5, was prepared based on the explorations shown in Figure 4.

- **FILL.** The depth of fill encountered varied from about 10 to 15 feet near the railroad tracks, extending to about Elevation 5. The fill thickens toward the water and extends to Elevation 0 feet at Central Avenue and then to -10 feet at the boring along the dock on the Waterway.
- **BEACH DEPOSITS.** The loose sand/soft to medium stiff clay deposits generally extend to about Elevation -15 to -30 feet, except along the southwest boundary of this area where these deposits are shallow over the bedrock and only extend to about Elevation -5 to -8 feet.
- **DENSE SAND AND GRAVEL.** A dense sand and gravel layer, approximately 10 feet thick, was encountered in the middle of this area where institutional uses could occur. This layer extends from approximately Elevation -15 to -25 feet; this layer thins out along the dock to the Whatcom Waterway to about 3 feet thick at approximately Elevation -30 feet.

- **BELLINGHAM DRIFT.** The Bellingham Drift unit was not encountered near the bluff toward the middle of this area as shown in Figure 5; medium stiff clay was encountered typically to about Elevation -32 to -47 feet in most explorations and extends to approximately Elevation -85 feet near the western limits of this area along the Whatcom Waterway.
- **DEMING SAND.** Dense sand was encountered from about El -85 to -110 feet near the western limits toward the Whatcom Waterway.
- **CHUCKANUT FORMATION.** Bedrock was encountered as shallow as Elevation -5 to -10 feet near the bluff toward the middle of this area as shown in Figure 5; generally encountered between about Elevation -32 to -50 feet within the proposed mixed use area, and deepens toward the water where it was encountered at approximately Elevation -110 feet near the western limits toward the Whatcom Waterway.

Log Pond Area

The subsurface conditions within the Log Pond Area have been explored near the toe of the bluff and toward the center of the area during previous evaluations for historical industrial projects. The data is relatively sparse. A generalized cross section, Figure 6, was prepared based on the explorations shown in Figure 4.

- **FILL.** The depth of fill varied from about 15 feet bgs near the railroad tracks and the toe of the bluff to on the order of 20-25 feet bgs at the borings along the dock along the Whatcom Waterway.
- **BEACH DEPOSITS.** The depth of beach deposits varied from about 15 feet thick (Elevation -3 feet) to greater than 20 feet (deeper than Elevation -15 feet) near the railroad tracks, and on the order of 10 to 20 feet thick (to Elevation -28 feet) along the dock to the Whatcom Waterway.
- **DENSE SAND AND GRAVEL.** A dense sand and gravel layer approximately 20 feet thick was encountered from approximately Elevation -15 to -25 feet; this layer thins out along the dock to the Whatcom Waterway to about 3 feet thick at approximately Elevation -30 feet.
- **BELLINGHAM DRIFT.** The Bellingham Drift unit was not encountered near the bluff; medium stiff clay was encountered between approximately El. -20 to -30 feet and extended to near Elevation -80 to -85 feet in the middle and Whatcom Waterway portions of the area.
- **DEMING SAND.** Dense sand was encountered at El -80 to -85 feet and extended to near Elevation -120 to -110 feet in the middle and Whatcom Waterway portions of the area, 20 to 40 feet thick. The dense sand encountered toward the middle of this area is characterized similar to a till-like material rather than Deming Sand.
- **CHUCKANUT FORMATION.** Bedrock was encountered as shallow as 22 feet bgs (Elevation -7 feet) near the bluff, although it was not encountered to -15 at other locations along the railroad. Bedrock was not encountered in the borings completed in the center of this area. Bedrock was encountered in a deep boring along the dock along the Waterway at Elevation -110. feet

Shipping Terminal Area

The subsurface conditions within the Shipping Terminal Area have been explored near the toe of the bluff, slightly north of Cornwall (POB Maintenance Facility) and then at the end of the Shipping

Terminal during previous evaluations. No data is available in between these points. A generalized cross section, Figure 7, was prepared based on the explorations shown in Figure 4.

- **FILL.** The depth of fill varied from about 10 feet bgs near the railroad tracks, and increases to 20 to 25 feet thick north of Cornwall Avenue to the end of the Shipping Terminal.
- **BEACH DEPOSITS.** The depth of beach deposit was less than 5 feet thick near the railroad tracks, increasing to greater than 30 feet at the end of the Shipping Terminal.
- **DENSE SAND AND GRAVEL.** An approximately 10-foot thick dense sand and gravel layer was encountered between about Elevation -35 to -45 feet north of Cornwall Avenue in the borings for the POB Maintenance Facility.
- **BELLINGHAM DRIFT.** Medium stiff clay was encountered only in the boring at the end of the Shipping Terminal between approximately Elevation -50 to -115 feet.
- **DEMING SAND.** Dense sand was encountered only in the boring at the end of the Shipping Terminal between approximately Elevation -115 feet to approximately Elevation -150 feet.
- **KULSHAN DRIFT.** Medium stiff clay was encountered below the Deming Sand to the depth explored (Elevation -160 feet) in the boring at the end of the Shipping Terminal.
- **CHUCKANUT FORMATION.** Bedrock was encountered as shallow as 12 feet bgs (Elevation 6) near the bluff, as deep as 70 feet bgs at the POB Maintenance Facility, and dips deeper toward the water as shown in Figure 7. Bedrock was not encountered in the boring at the end of the Shipping Terminal.

GEOLOGICALLY HAZARDOUS AREAS AND MITIGATION

Geologically hazardous areas are designated by the City of Bellingham Critical Areas Ordinance (CAO per Bellingham Municipal Code 16.55.410 – 16.55.460). The City has developed a folio of maps that identify these areas in their database. In general, the CAO requires that a qualified professional assess the geologic hazards based on review of available information and field studies, evaluate the specific project proposal with respect to relationship and impact on the hazard area and adjacent sites if appropriate, and provide minimum buffers and setbacks and provide mitigation strategies where appropriate for specific geologic hazards. The geologically hazardous areas include erosion, landslide, seismic and mines.

Erosion Hazard Areas

Erosion hazard areas are defined by the CAO as areas prone to soil erosion including:

1. Areas identified in soil unit maps of the U.S. Department of Agriculture Soil Conservation Services Soil Survey of Whatcom County rated as “Severe” due to “slope, wetness, ponding, flooding, cutbanks cave” or any combination thereof.
2. Upland areas immediately adjacent to Bellingham Bay.
3. Any area where soil type is predominantly sand, clay, silt and/or organic matter and slopes greater than 30 percent.

The site is adjacent to Bellingham Bay and does have soil types and slopes that would be defined as an erosion hazard area by the CAO. Additionally, an erosion hazard would exist if soils are disturbed during the earthwork phase of construction. Potential impacts and mitigation strategies are discussed in the subsequent mitigation section of this report.

The primary erosion hazard at the site is from temporary conditions created during construction. Temporary erosion control measures and Best Management Practices (BMPs) are required during construction under current regulations to mitigate on-site and off-site erosion potential. During construction, the contractor would be subject to Ecology regulations which require performance based testing of turbidity at all discharge points. Proper construction practices and monitoring procedures will manage the risks to the standard of practice and meet mitigation requirements.

Landslide Hazard Areas

Landslide hazard areas are defined by the CAO as those susceptible to landslides and/or subsidence that could include movement of soil, fill materials, rock or other geologic strata. The site is not identified in an area with known landslide hazards. However, the steeper unsupported shoreline areas may have some landslide and lateral spreading potential. For the purposes of this report, the development will be 200 back from the shoreline and not impacted by any shoreline instability. However, this condition may warrant site specific studies once individual projects have been identified.

The bluff above the site is steep and identified as a landslide hazard area. We have performed some evaluations for private developments, bridge projects, and railroad projects along the top and base of the bluff. A railroad ROW is located along the toe of the bluff and the railroad will be relocated to this area in the future. We conclude that the risk of potential impacts to and from the bluff are very low such that no mitigation will likely be required for any project at this site.

Seismic Hazard Areas

Seismic hazard areas are defined by the CAO as those areas subject to severe risk of damage as a result of earthquake induced ground shaking, slope failure, settlement, soil liquefaction, lateral spreading, or surface rupture. Specific areas of very high response to seismic shaking include:

1. All landfills placed waterward of the historic 1850 natural coastline of Bellingham Bay.
2. All alluvial deposits near the mouth (delta) of Whatcom Creek.
3. All marine and stream course bluffs greater than 10 feet in vertical relief and steeper than 100%.
4. All rock outcrops greater than 10 feet in vertical relief.

The site meets the definitions of a seismic hazard area and development would thereby need to be addressed in accordance with the CAO. The site is also identified as a seismic hazard in the Geologic Hazard Areas Map Folio and is listed as ranging from low to very high response to seismic shaking. Potential impacts include ground shaking, liquefaction and lateral spreading. Potential mitigation strategies are discussed in the subsequent sections of this report.

Site Seismicity

The site is located within the Puget Sound region, which is seismically active. Seismicity in this region is attributed primarily to the interaction between the Pacific, Juan de Fuca and North American plates. The Juan de Fuca plate is subducting beneath the North American plate. It is thought that the resulting deformation and breakup of the Juan de Fuca plate might account for the deep focus earthquakes in the region. Hundreds of earthquakes have been recorded in the Puget Sound area. In recent history, four of these earthquakes were large events: (1) in 1946, a Richter magnitude 7.2 earthquake occurred in the Vancouver Island, British Columbia area; (2) in 1949, a Richter magnitude 7.1 earthquake occurred in the Olympia area; (3) in 1965, a Richter magnitude 6.5 earthquake occurred between Seattle and Tacoma; and (4) in 2001, a Richter magnitude 6.8 earthquake occurred near Olympia.

Research has concluded that historical large magnitude subduction-related earthquake activity has occurred along the Washington and Oregon coasts. Evidence suggests several large magnitude earthquakes (Richter magnitude 8 to 9) have occurred in the last 1,500 years, the most recent of which occurred about 300 years ago. No earthquakes of this magnitude have been documented during the recorded history of the Pacific Northwest. Local design practice in Puget Sound and local building codes now include the possible effect of a very large subduction earthquake and local known faults in the design of structures.

There are no known faults located at the site. The closest active faults identified include the Vedder Mountain Fault, Sumas Mountain Fault, and the Boulder Creek Fault complex including the Kendal Fault. The closest fault to the site is approximately 10 miles northwest of Bellingham. Therefore, the site is not at risk of known surface rupture.

Seismic Design

Local and national building codes include seismic design provisions. The current code is the 2009 International Building Code (IBC) and incorporates the most recent seismic provisions that are applicable to this site. Using this code for design will require appropriate mitigation strategies. Some if not most of the sites will be classified as Site Class F because of the liquefaction potential. At this time, structures with fundamental periods of vibration equal to or less than 0.5 seconds may be classified as Site Class E. Site specific seismic evaluations will likely be necessary and appropriate for the larger buildings to determine specific design criteria and mitigation strategies. Typical foundation strategies that meet mitigation requirements have been included in the foundation considerations of this report.

Liquefaction and Lateral Spreading

Liquefaction is a phenomenon where soils experience a rapid loss of internal strength as a consequence of strong ground shaking. Loss of bearing support, ground settlement, lateral spreading and/or sand boils may result from liquefaction. Conditions favorable to liquefaction occur in very loose to medium dense, clean to moderately silty sand that is below the groundwater level. Dense soils or soils that exhibit cohesion are less likely to be susceptible to liquefaction.

The results of our preliminary analyses indicate that portions of the loose to medium dense sandy fill and native beach/intertidal soils encountered in the explorations have a moderate to high potential for liquefaction during a design earthquake event. Our analyses indicate that settlements caused by liquefaction of the saturated loose to medium dense sand layers at this site during a design earthquake range from a few inches up to 1 foot.

In addition to settlements, there is the possibility that lateral spreading of the soils could occur as a result of soil liquefaction. Lateral spreading involves lateral displacement of large, surficial blocks of non-liquefied soil, as well as the liquefied soil itself, as the underlying soil layer liquefies. Lateral spreading generally develops in areas where sloping ground is present or near a free face, such as the Whatcom Waterway. If liquefaction were to occur within the upper soils at the site, we anticipate that there would be a moderate potential for lateral spreading to occur, characterized by movement of the soils towards the Waterway. During a design earthquake, lateral spread displacements across the site are estimated to be several inches in some locations and over 1 foot near the Waterway. Most of the significant structures in the Downtown Waterfront Area and in the Log Pond Area will be at least 200 feet back from the Waterway such that this will help minimize the potential impacts. However, this condition may warrant site specific studies once individual projects have been identified.

Tsunami

The site is identified as impacted by potential tsunami from a large earthquake in the Pacific Ocean basin. The available modeling suggests that a magnitude 9.1 Cascadia Subduction Zone earthquake could generate a tsunami that could cause inundation up to about 0.5 meter (1.6 feet) across the site at present elevations. The EIS and planning completed to date suggests that the site will be filled on the order to 3 to 6 feet higher than present elevations which would likely mitigate the risk of inundation from a tsunami; however, this will need to be addressed for individual projects.

Mine Hazard Areas

Mine hazard areas are defined by the CAO as those areas underlain by or affected by historical mine workings. Specific hazard areas include:

- Areas depicted within the as *Coal Mine Hazard Areas* within the Geologic Hazard Areas Map Folio, Bellingham, Washington, 1991.

The northeastern limits of the project site, between Cornwall Avenue and the extension of Bay Street, and between Laurel and Chestnut Streets, is mapped within an area identified as “unknown hazard” by the Map Folio. The Sehome Mine was active in this portion of Bellingham between about 1853 and 1878. Reportedly, mining records and detailed maps were kept in San Francisco and subsequently destroyed in the earthquake and fire of 1906. The most likely location of the main opening to the mine was along the original bluff near the intersection of Railroad Avenue and Laurel Street or Railroad Avenue and Myrtle Street. The coal was extracted from a seam about 7 to 14 feet thick, with the general direction of the mining drift to the northeast.

GeoEngineers has previously reviewed available information and performed deep explorations in at various projects in the site vicinity to evaluate locations of the coal mine or mine associated voids. The depth to workings is likely on the order at least 200 and more likely 300 feet below the ground surface directly below the subject site. Therefore, no special mitigation measures are likely required. However, this consideration warrants site specific analysis once projects are identified.

FOUNDATION CONSIDERATIONS

Foundation support for structures will depend on land use, site specific subsurface conditions, building specific design elements, and other factors. Most of the site is underlain by fill and beach deposits with a high liquefaction potential. Therefore, special foundations and/or ground improvement will be required. Most of the existing and past buildings have been pile supported at the site. More recently, some ground improvement techniques have been used on commercial and public infrastructure in these types of waterfront environments. These two approaches are discussed in more detail below followed by individual discussion of the three sub-areas.

Pile Foundations

Pile foundations provide foundation support through the loose and liquefiable soils, and are considered adequate for mitigation of earthquake hazards associated with ground movement. This technique does not stabilize the ground around a pile supported building, so the connections to surrounding infrastructure can be compromised during a large earthquake.

Pile foundations at this site will extend through the potentially liquefiable soils, gaining capacity in skin friction in the clay and/or end-bearing on the underlying bedrock or possibly in the dense sand units encountered at various locations across the site. Any new pile foundation design must include the effects of downdrag loads. New piles will offer some lateral resistance for the buildings and to resist the effects of lateral spreading. Driven piles will likely be the preferred piles at the site because drilled piles generate cuttings, which could have potential soil contamination. Another advantage of driven piles is the ability to confirm adequate bearing considering the variable conditions across the site. Generally, timber, steel H-pile, or steel pipe piles have been used and will likely be preferred options depending on the type of building, building loads, and depth to the bearing strata. For the larger structures, it will be necessary to use sufficient piles to resist lateral loads or ground improvement may be required to densify the soil to provide better lateral resistance. We obtained pile foundation costs with several pile driving contractors in Puget Sound, including Whatcom Construction in Bellingham, McDowell Northwest, Inc. and Pile Contractors, Inc.

Ground Improvement

The purpose of ground improvement at waterfront sites can be four-fold: 1) to reduce settlement under static conditions resulting from the newly imposed structural loads; 2) to reduce settlement resulting from liquefaction of loose saturated foundation soils; 3) to densify loose/soft soils under structures or waterward of structures to provide suitable static and seismic stability and reduce deformation associated with lateral spreading; and 4) improve soils around pile foundations to provide additional lateral pile resistance.

We considered methods of ground improvement such as vibro-compaction, timber compaction piles, compaction grouting, soil mixing, deep dynamic compaction, wick drains, and blast densification for use at the site. Some of these techniques are likely not suitable considering the location, potential impacts of contamination, high groundwater and other factors. For this level of planning, we focused on a common waterfront ground improvement technique that mitigates liquefaction impacts and provides adequate foundation support for lightly loaded structures. A combination of these techniques has been used to allow creative and more cost-effective foundation alternatives; however, such evaluation was beyond the scope of this preliminary study.

Stone Columns

Vibro-replacement (more typically referred to as stone columns) is a common ground improvement technique that would meet all the mitigation goals/foundation support requirements for lightly loaded structures. This technique has been used previously on other projects in Whatcom County and is typically one of the more cost effective methods of ground improvement. Therefore, we have focused on this ground improvement technique . Additional analyses of stone columns will be required during design to determine the specific amount of ground improvement (replacement ratio and depth) that will be necessary to meet the project needs. This type of ground improvement method is typically satisfactory to mitigate liquefaction and support low-rise structures that can then be designed using conventional shallow foundation and slab-on-grade techniques. A slightly higher allowable bearing pressure can oftentimes be used to help off-set some of the ground improvement costs.

This technique is typically not adequate for support of heavy structures such as the tall structures envisioned in the Downtown Waterfront Area. However, it may be beneficial and/or necessary to use a ground improvement technique to stiffen the bearing soils for lateral resistance around the pile foundations and/or resist lateral spreading. That level of analysis was not appropriate for this preliminary study.

Construction of stone columns involves the partial replacement of loose/soft, unsuitable soils with a vertical column of compacted stone. Typically, a hollow tube or probe is vibrated, jetted or driven into the ground to the desired depth. As the tube or probe is withdrawn, crushed stone is fed to the bottom of the hole and compacted. The end result is a column of dense stone which penetrates through the loose/soft unsuitable soil and is capable of transferring loads into the underlying competent soils. Vibration resulting from the installation method also densifies surrounding granular soil deposits. The presence of the column creates a composite material of lower overall compressibility and higher shear strength than the native soil alone. Confinement of the stone is provided by the lateral stress within the densified loose/soft soils. As loads are applied, the stone column and soft soil move downward together, resulting in transfer of the majority of the load to the stone column.

Typically the stone columns will extend through the liquefiable fill and beach deposits. The depth of these units is discussed above and is variable across the site with the thicker deposits toward the Whatcom Waterway. For smaller, lightly loaded structures, it may not be necessary to install ground improvement the full depth of the liquefiable soils; an improved soil foundation area of 30 to 40 feet thick is likely sufficient for the warehouse structures envisioned at the site. Based on our experience, column spacing on the order of 7 to 10 feet is typical within a building footprint, providing about 25 percent to 15 percent (respective) replacement ratio for 42-inch diameter stone columns. A closer spacing is used under perimeter spread footings and columns to buildings. Typically, the area in which stone columns are installed extend at least 10 to 20 feet (one to two rows) laterally beyond the edges of buildings. We obtained stone column installation cost from Hayward Baker in Seattle, Washington.

Downtown Waterfront Area

The Downtown Waterfront Area is envisioned to be a mix of housing, office and institutional uses. Assumptions regarding structures that might be located in this area are presented previously. The institutional building could be a multi-story structure on the order of 100-feet and commercial mixed-use buildings could be multi-story on the order of 150 to 200 feet high. It is anticipated that these buildings will be supported on piles extending to bedrock.

Institutional Design

- University type project
  - We have assumed 24- to 30-inch diameter, ½-inch steel pipe piles driven into bedrock.
  - Structurally supported floor slab.
  - No ground improvement has been assumed, although some improvement could be required depending height and footprint of the building, potential for lateral spreading, and other factors.
  - Pile lengths for this structure could be quite variable. At the south end of this area, it appears that piles on the order of 50-feet long would be sufficient. However, extending toward the water, piles may need to be on the order of 80 feet or longer. For this planning level of effort, we have piles am average of 60 feet long. The estimated increased incremental cost for piles, pile caps, grade beams and structural slab for the described structure would be on the order of $27/sf to $32/sf of building footprint.

Commercial Mixed Use Design

- Multi-story mixed use
  - We have assumed 24- to 30-inch diameter, ½-inch steel pipe piles driven into bedrock.
  - Structurally supported floor slab.
  - No ground improvement has been assumed, although some improvement could be required depending on height and footprint of the building, and other factors.
  - The middle of this area (as shown in Figure 5) has shallower bedrock and we have assumed that piles on the order of 40-feet long could be used. The estimated increased incremental cost for piles, pile caps, grade beams and structural slab for the described structure would be on the order of $36/sf to $43/sf of building footprint.
  - The margins of this area (as shown in Figure 5) have deeper bedrock and we have assumed that piles on the order of 60-feet long could be used. The estimated increased incremental cost for piles, pile caps, grade beams and structural slab for the described structure would be on the order of $50/sf to $60/sf of building footprint.

Log Pond Area

The Log Pond Area is envisioned to be one to two story warehouse structures during this transitional period of discussion. Assumptions regarding structures that might be located in this area are presented previously. Bedrock was encountered at about 25 feet bgs at the south end of this sub-area, and a dense sand at about 30 feet bgs in the limited borings available.

- Pile Foundations. We have assumed timber piles to support the building including the floor slab.
  - No ground improvement has been assumed.
  - Assuming timber piles on the order of 30 feet long, the estimated increased incremental cost for piles, pile caps, grade beams and structural slab for the described structure would be on the order of $14/sf to $15/sf of building footprint for 250 psf floor loading.
  - With the same assumptions but a 500psf floor loading, the estimated increased incremental cost would be on the order of $20/sf to $22/sf of building footprint.
- Ground improvement. We have assumed stone columns could be used as described above. The estimated cost of the ground improvement would be on the order of $18/sf to $30/sf of building footprint, assuming a maximum depth of about 30 feet of ground improvement. The lower cost is appropriate for lightly loaded structures with a maximum design floor load of 250psf; the higher cost would be for a more heavily loaded two-story structure with a design floor load of 500psf.

Shipping Terminal Area

The Shipping Terminal Area already has some pile supported structures on it. The outer buildings are supported on driven timber piles, likely gaining their support by friction within the glaciomarine drift unit. The existing POB maintenance facility is supported on steel H-piles driven to bedrock, on the order of 30 to 75 feet bgs. The steel H-piles were used because almost all of the soil above the bedrock is liquefiable.

New development at the Shipping Terminal Area between the existing structures is envisioned to be one to two story warehouse structures during this transitional period of discussion. Assumptions regarding structures that might be located in this area are presented previously.

- Pile Foundations. We have assumed timber piles to support the building including the floor slab.
  - No ground improvement has been assumed.
  - Assuming timber piles on the order of 40 feet long for about the south undeveloped one-half of the Shipping Terminal Area, the estimated increased incremental cost for piles, pile caps, grade beams and structural slab for the described structure would be on the order of $15/sf of building footprint. With the same assumptions but a 500psf floor loading, the estimated increased incremental cost would be on the order of $22/sf of building footprint.
  - For the north undeveloped one-half of the Shipping Terminal Area, it appears that timber piles on the order of 60 feet long might be required, and the estimated increased incremental cost would be on the order of $22/sf of building footprint. With the same assumptions but a 500psf floor loading, the estimated increased incremental cost would be on the order of $34/sf of building footprint.
- Ground improvement. We have assumed stone columns could be used as described above. The estimated cost of the ground improvement would be on the order of $18/sf to $30/sf of

building footprint, assuming a maximum depth of about 40 feet of ground improvement. The lower cost is appropriate for lightly loaded structures with a maximum design floor load of 250psf; the higher cost would be for a more heavily loaded two-story structure with a design floor load of 500psf.

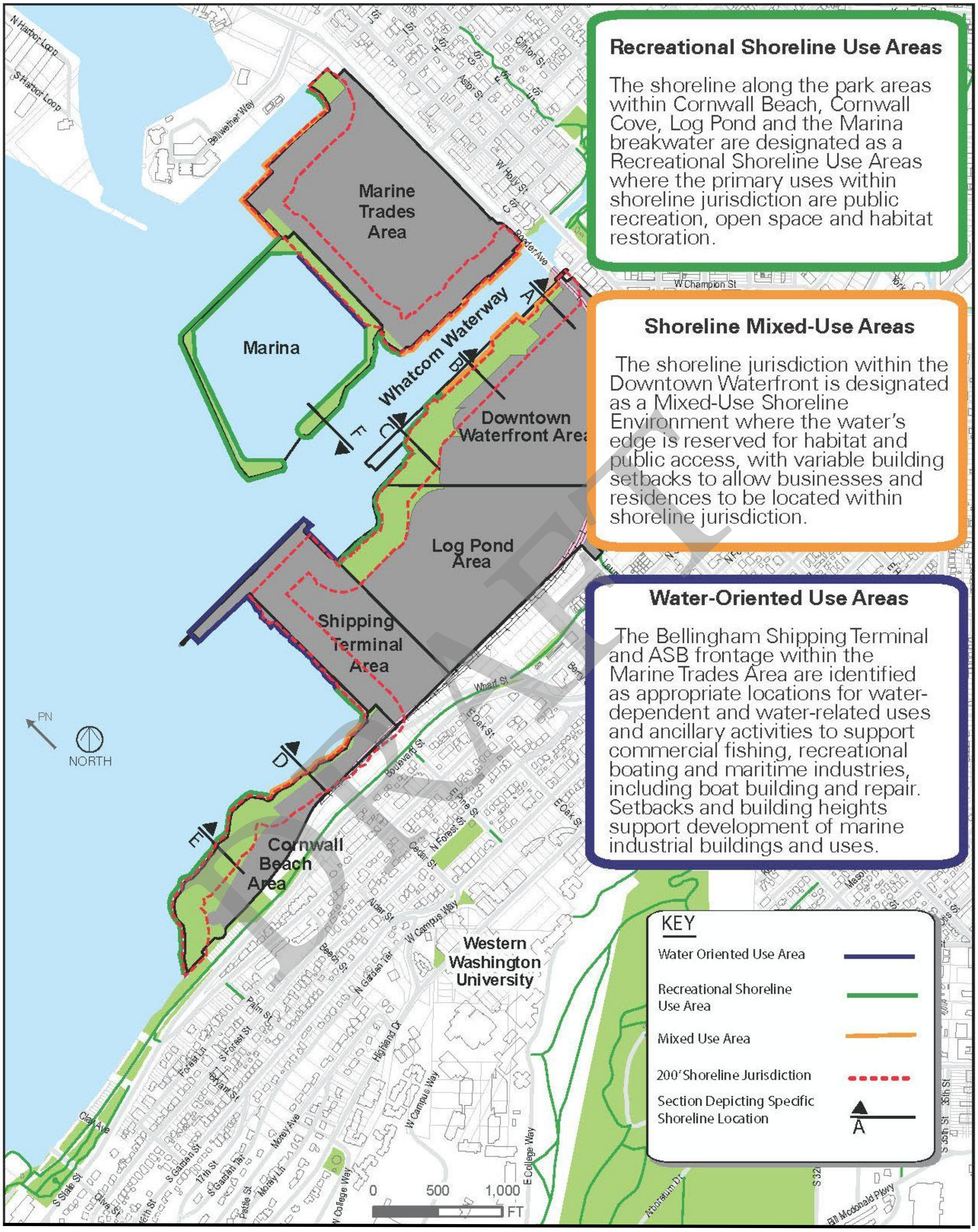
LIMITATIONS

We have prepared this report for the exclusive use of the Port of Bellingham and their authorized agents for the Waterfront District Sub-Area planning in Bellingham, Washington.

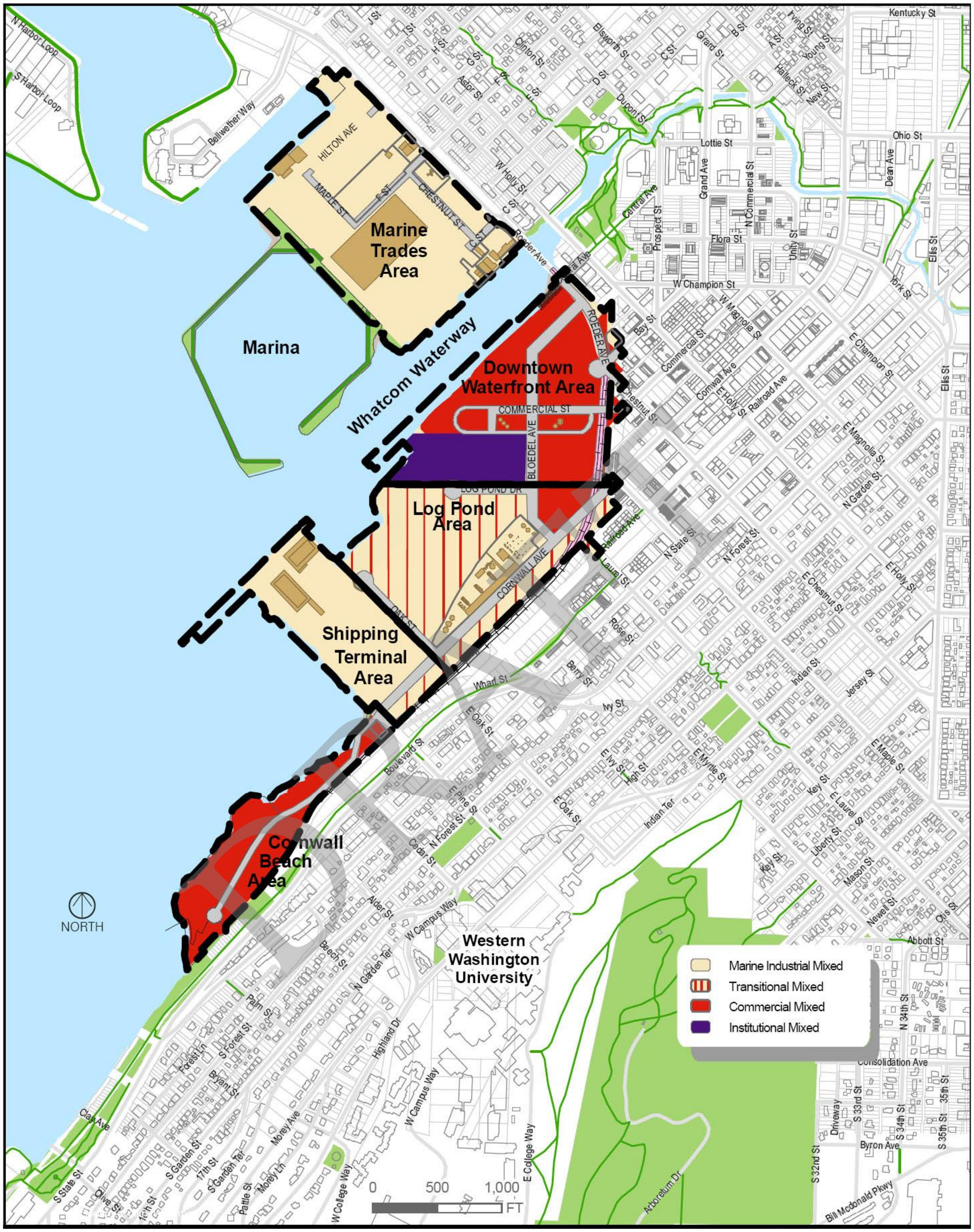
This report has been prepared based on review of the existing limited database of information, preliminary level analyses, and collaboration with a structural engineer and specialty contractors. In order to provide cost estimates, very broad assumptions were necessary regarding building type and size, subsurface conditions, ground response, and other considerations to assume a limited number of foundation and ground improvement strategies. Site specific investigations will be required to develop more accurate conclusions and recommendations for design, construction and cost estimating.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

Any electronic form, facsimile or hard copy of the original document (email, text, table, and/or figure), if provided, and any attachments are only a copy of the original document. The original document is stored by GeoEngineers, Inc. and will serve as the official document of record.



Sub-Area Plan	
The Waterfront District Bellingham, Washington	
GEOENGINEERS	Figure 1

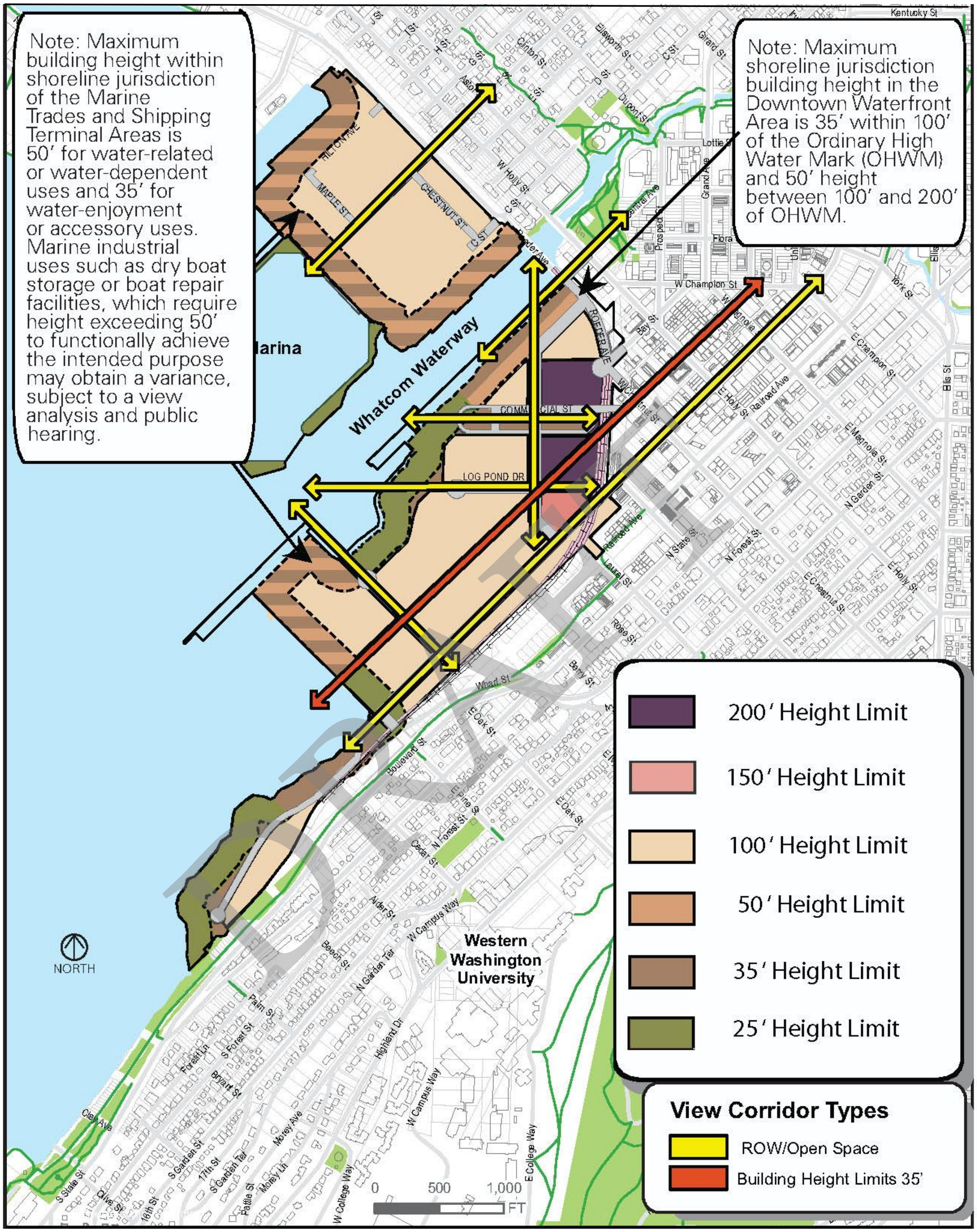


Land Use/Zoning

The Waterfront District
Bellingham, Washington

GeoEngineers

Figure 2



Height Limits and View Corridors

The Waterfront District
Bellingham, Washington

GeoEngineers

Figure 3

Notes

1. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. can not guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Legend

● BORING LOCATION

N

0 360 720
SCALE IN FEET

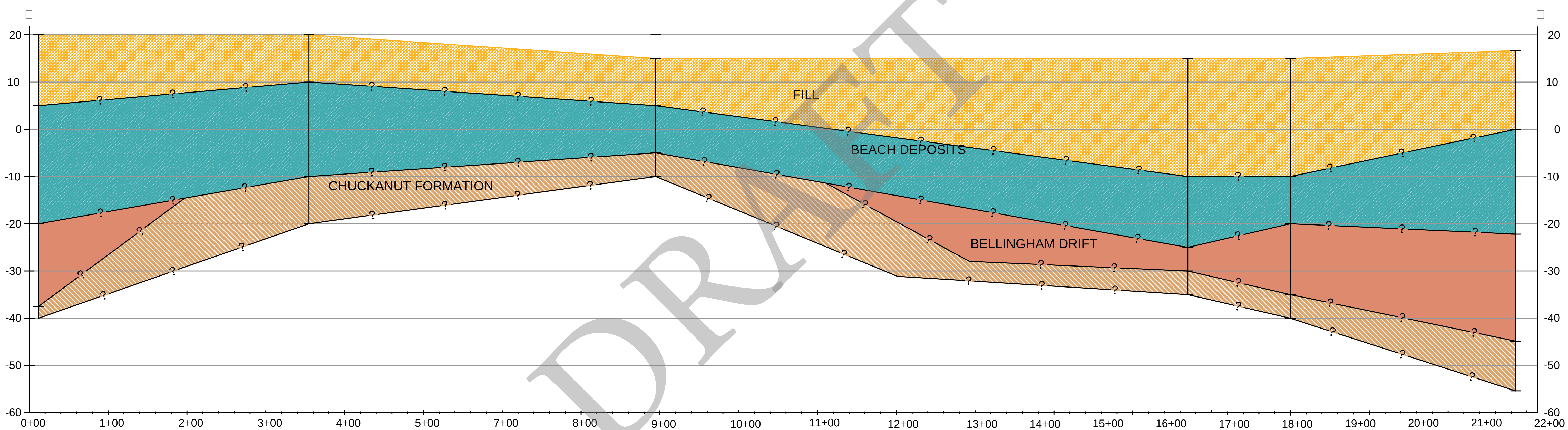
Waterfront District
Boring Locations

Bellingham, Washington

GEOENGINEERS

Figure 4

P:\0\0307056\02\CAD\0307560\02_AERIAL EXHIBIT IMS.DWG\TAB:11X17 EXST. SITE PLAN MODIFIED BY ANELSON ON JUN 15, 2011 - 18:04



HORIZONTAL SCALE: 1"= 120'
VERTICAL SCALE: 1"= 20'
VERTICAL EXAGGERATION: 6X

120 0 120
FEET

Legend

FILL; silty fine sand, fine sandy silt, or sandy clay with occasional wood debris, organic matter; loose/medium dense/soft to medium stiff

BEACH/INTERTIDAL DEPOSITS; gray silty fine sand to sand with silt with occasional shells and wood; very loose to medium dense clay with sand; soft to medium stiff

BELLINGHAM DRIFT; brown fine sandy clay with occasional fine gravel; medium stiff to stiff

CHUCKANUT FORMATION; gray to brown sedimentary bedrock consists of sandstone, coal and siltstone with depth; hard

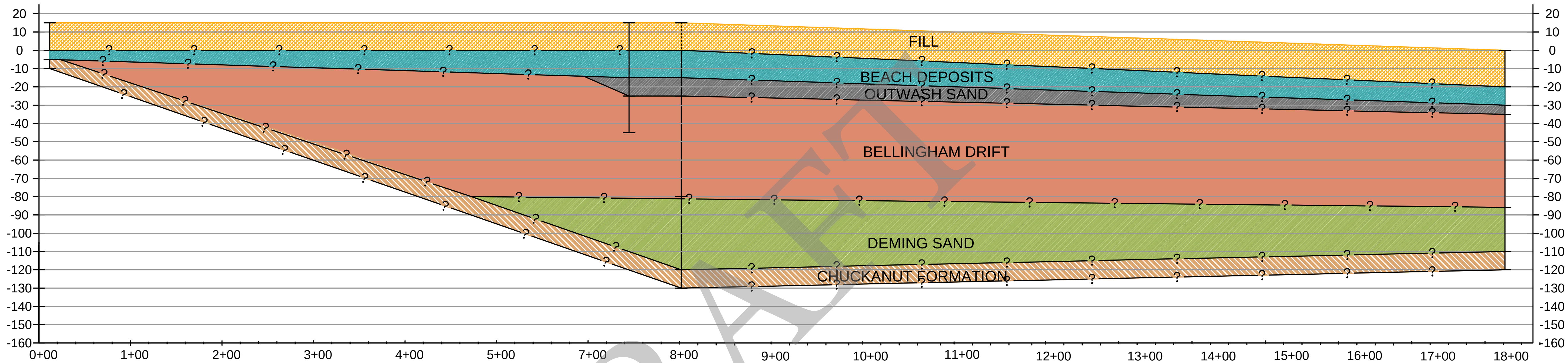
Boring Location

?—? Inferred Soil Contact

Notes:
1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. Refer to Figure 4 for location of cross section.
3. The locations of all features shown are approximate.
4. This figure is for informational purposes only. It is intended to assist in the identification of features discussed in a related document. Data was compiled from many different geotechnical firms and reports from 1960 to 2010. The data sources do not guarantee these data are accurate or complete. There may have been updates to the data since the publication of this figure. This figure is a copy of a master document. The master hard copy is stored by GeoEngineers, Inc. and will serve as the official document of record.

Cross-Section A-A'	
PORT OF BELLINGHAM BELLINGHAM, WASHINGTON	
GEOENGINEERS	Figure 5

P:\0\0307056\02\CAD\03075602 X-SECTIONS FINAL ACAD.DWG\TAB:AA MODIFIED BY ANELSON ON JUN 15, 2011 - 17:14



Legend

FILL; silty fine sand, fine sandy silt, or sandy clay with occasional wood debris, organic matter; loose/medium dense/soft to medium stiff

DENSE SAND: outwash and gravel; gray sand with gravel; dense

BEACH/INTERTIDAL DEPOSITS; gray silty fine sand to sand with silt with occasional shells and wood; very loose to medium dense; clay with sand; soft to medium stiff

BELLINGHAM DRIFT; brown fine sandy clay with occasional fine gravel; medium stiff to stiff

DEMING SAND: Sand dense / GLACIAL TILL (?); silty sand with gravel and cobbles; very dense

CHUCKANUT FORMATION; gray to brown sedimentary bedrock consists of sandstone, coal and siltstone with depth; hard

Boring Location

?——? Inferred Soil Contact

HORIZONTAL SCALE: 1"= 120'
VERTICAL SCALE: 1"= 60'
VERTICAL EXAGGERATION: 2X

120 0 120
FEET

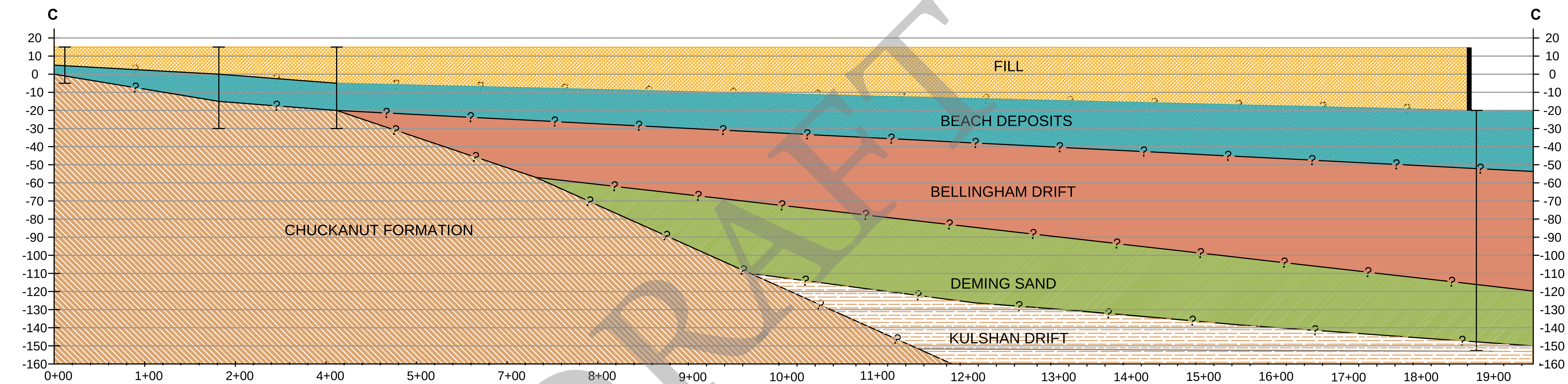
Notes:
1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. Refer to Figure 4 for location of cross section.
3. The locations of all features shown are approximate.
4. This figure is for informational purposes only. It is intended to assist in the identification of features discussed in a related document. Data was compiled from many different geotechnical firms and reports from 1960 to 2010. The data sources do not guarantee these data are accurate or complete. There may have been updates to the data since the publication of this figure. This figure is a copy of a master document. The master hard copy is stored by GeoEngineers, Inc. and will serve as the official document of record.

Cross-Section B-B'

Port of Bellingham
Bellingham, Washington

GEOENGINEERS

Figure 6



HORIZONTAL SCALE: 1"= 120'
VERTICAL SCALE: 1"= 60'
VERTICAL EXAGGERATION: 2X

120 0 120
FEET

Legend

FILL; silty fine sand, fine sandy silt, or sandy clay with occasional wood debris, organic matter; loose/medium dense/soft to medium stiff

BEACH/INTERTIDAL DEPOSITS; gray silty fine sand to sand with silt with occasional shells and wood; very loose to medium dense; clay with sand; soft to medium stiff

BELLINGHAM DRIFT; brown fine sandy clay with occasional fine gravel; medium stiff to stiff

DEMING SAND: sand dense / GLACIAL TILL (?); silty sand with gravel and cobbles; very dense

KULSHAN DRIFT: Silt and clay, medium stiff.

CHUCKANUT FORMATION; gray to brown sedimentary bedrock consists of sandstone, coal and siltstone with depth; hard

Boring Location

?—? Inferred Soil Contact

Notes:
1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. Refer to Figure 4 for location of cross section.
3. The locations of all features shown are approximate.
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Cross-Section C-C'

Port of Bellingham
Bellingham, Washington

GEOENGINEERS

Figure 7

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