Materials Testing & Consulting, Inc.
Geotechnical Engineering ● Materials Testing ● Special Inspection ● Environmental Consulting

March 23, 2015

City of Marysville, c/o:
Justin Clary, P.E.
Maul Foster Alongi
1329 North State Street, Suite 301
Bellingham, WA 98225

Subject: **Geotechnical Feasibility Investigation**
Proposed Site Redevelopment – Geddes Marina
1326 First Street, Marysville, Washington

MTC Project No.: 14B037-01

Dear Mr. Clary:

This letter transmits our Geotechnical Feasibility Report for the above-referenced project. Materials Testing & Consulting, Inc. (MTC) performed this geotechnical engineering study in accordance with our Proposal for Geotechnical Services, dated September 19, 2014, and the work order executed November 20th, 2014.

We would be pleased to continue our role as your geotechnical engineering consultants during the project planning and construction. We also have a keen interest in providing materials testing and special inspection during construction of this project. We will be pleased to meet with you at your convenience to discuss these services.

We appreciate the opportunity to provide geotechnical engineering services to you for this project. If you have any questions regarding this report, or if we can provide assistance with other aspects of the project, please contact me at (360) 755-1990.

Respectfully Submitted,
MATERIALS TESTING & CONSULTING, INC.

Leland Rupp

John R. Gillaspy, L.E.G.
Project Engineering Geologist

Leland B. Rupp, P.E.
Geotechnical Division Manager

Attachment: Geotechnical Feasibility Investigation Report – Geddes Marina Redevelopment

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GEOTECHNICAL
FEASIBILITY INVESTIGATION

PROPOSED REDEVELOPMENT – GEDDES MARINA

1326 FIRST STREET
MARYSVILLE, WASHINGTON

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March 23, 2015
MTC Project Number: **14B037-01**

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1.0 INTRODUCTION

1.1 GENERAL

This report presents the findings and recommendations of Materials Testing & Consulting, Inc.'s (MTC) geotechnical feasibility study conducted for the client's consideration in evaluating the suitability of proposed site redevelopment of the Geddes Marina property in Marysville, Washington. The subject project is located at the north end of Highway 529 on Ebey Slough in Marysville, Washington. The project vicinity and location, and an aerial photo of the project site, are provided in the Figures 1 and 2 of Appendices A and B respectively.

1.2 PROJECT DESCRIPTION

It is our understanding based on initial project scoping discussions that the client is considering redevelopment of the subject site, formerly a privately owned and operated marina. Presently the site is occupied with various small buildings and associated gravel or asphalt-surfaced access roads. Much of the site interior is occupied by the existing marina waterway. Existing structures are primarily concentrated among the north portion of the site near the fronting First Street, while a few isolated shops and storage outbuildings are spread along the perimeter and south end at the shoreline. MTC is informed that the site is under evaluation by the client for potential construction of one or multiple 3- to 5-story structures and associated general site improvements and marina repurposing. Underground parking is also considered due to space constraints. Primary building areas are proposed to be in the north part of the site near First Street. The scope of development is subject to change, and may be altered following the results of this study and the concurrent environmental investigation conducted by others.

MTC understands the project site has experienced prior uses, including a historic fueling station. In addition, the site is known to contain relatively thick historically deposited uncontrolled fills from unknown sources. Therefore, a goal of the geotechnical feasibility program has been to delineate the condition and extent of existing fills, addressing bearing suitability and potential difficulties for redevelopment.

This study constitutes a limited exploration program and cursory-level site analysis for general development consideration. Recommendations and interpretations herein are preliminary pending further exploration and analysis as needed for final design and construction, and will need to be re-evaluated once design details are available. MTC should be allowed to review proposed plans and provide recommendations for additional scope of study at that time.

1.3 PURPOSE AND SCOPE OF SERVICES

The purpose of our study at this time was to explore and characterize subsurface conditions at the site, summarize findings and interpretations, perform liquefaction analysis, and provide commentary on the geotechnical feasibility of site development for the client's use toward further site consideration and planning purposes. At the request of the client, the scope of study does not constitute a full-scale geotechnical engineering investigation. The findings presented are limited to the information available from the means and methods employed for site exploration and engineering analysis, the scope of which was determined in cooperation with the client representative prior to commencing the study.

MTC's scope of services was consistent with that presented in our Proposal for Geotechnical Engineering Services, dated September 19, 2014, and the subconsultant work order executed November 20th, 2014.

2.0 SITE EXPLORATION AND LABORATORY TESTING

2.1 SITE EXPLORATION ACTIVITIES

MTC's geotechnical site exploration activities were performed on December 29, 2014 and February 2 through 3, 2015. Field activities catalogued for this phase of study included Cone Penetrometer Testing (CPT) and Geoprobe (GP) direct-push continuous borings. Exploration locations were generally selected by Maul, Foster, Alongi (MFA) prior to commencing field work based on the environmental phase directives and potential building areas for the site. Test locations for CPT were nominally adjusted by MTC while on site during explorations as needed for access and coverage. Additional information on the site exploration program and field methods is provided with our exploration logs in Appendix C and D of this report. Test locations are shown approximately on the aerial photo site plan, Figure 2 of Appendix B.

Two (2) CPT tests were commenced on December 29, 2014 under subcontract to MTC and on-site direction by an MTC staff geologist. One CPT was performed in the holding yard of CJ Marine Supplies, Inc. located in the northeast corner of the site while a second CPT was advanced near the northwest corner of the marina in the existing road. CPTs were advanced to a termination depth of 100 feet below present grade (BPG). CPT results provided by the contractor are attached in Appendix C.

Ten (10) GP direct-push boreholes were advanced on February 2 and 3, 2015 under direction of MFA personnel. Each boring was advanced to termination at 15 feet BPG. MFA's team selected test locations, directed borehole advancement, and performed environmental sampling procedures, while an MTC staff geologist observed and logged continuous core samples as available. Soil cores were documented in accordance with the Unified Soil Classification System (USCS), and MTC staff made note of soil texture, color, consistency or density, and other geotechnical or geologically defining characteristics as possible. MTC was present to observe and log subsurface conditions encountered at test locations within the project area as they related to potential building development, labeled as GM-1 through GM-5 and GM-10 in accordance with MFA designations. Field data on other potentially relevant locations among the middle site perimeter, GM-6 and GM-9, was provided by MFA and the archaeological subconsultant and adapted for informational use herein. Complete GP borehole logs are attached as Appendix D.

2.2 LABORATORY TESTING

Due to the nature of the site conditions, potential for environmental contamination, and exploration activities at the time of this study, the scope of work did not include sampling of soils for geotechnical laboratory analysis. The classifications and resulting interpretations reported herein are based on MTC's field classifications and best available information gathered during the explorations.

3.0 EXISTING SITE CONDITIONS

3.1 SURFACE DESCRIPTION

The project site consists of a historically developed group of parcels located along the south side of First Street and north of Ebey Slough in Marysville, Washington. The north border of the site extends along First Street for approximately 390 feet, turning south for 690 feet along the roadway bordering the west edge of Ebey Waterfront Park. The southern edge of the project site is approximately 375 feet along Ebey Slough. The west boundary travels 560 feet along the railroad tracks extending from the shoreline to the intersection with First Street. The site is roughly rectangular in shape and flat, with typical surface level near sea-level elevation.

The site is bordered by other historically developed properties. To the west beyond that railroad tracks is a vacant industrial lot with dilapidated buildings. Directly north across First Street, occupied commercial and retail buildings of the Marysville Mall are present. The site shares its east boundary with Ebey Waterfront Park, a recently redeveloped public property consisting of open spaces, parking lots, and shoreline accesses. To the south, Ebey Slough separates the subject site from generally unimproved land and the convergence of Highway 529 and Interstate I-5.

The site has been historically utilized as a private marina, with improvements typically consisting of boathouses, docks and upland metal buildings of storage and warehouse-style construction. Present surface conditions at the site vary, with the entrance road along the eastern boundary consisting of pavement that becomes crushed gravel and aggregate approximately 300 feet south from First Street. Another gravel access road begins at the north end of the site and borders the west edge of the property adjacent to the railroad alignment. Small patches of grass and ground cover are present throughout the property, primarily found in areas along the edge of the marina and Ebey Slough. No large areas of vegetation or trees are present. In areas where buildings or boathouses remain or were apparently removed, evidence of local surface fills was observed. An aerial photo of recent conditions depicting remaining structures and general layout was provided by MFA and is attached as Figure 2, Appendix B.

3.2 AREA GEOLOGY

The *Geologic Map of the Marsyville Quadrangle, Snohomish County, Washington* published by the USGS (Miscellaneous Field Studies Map MF-1743, J.P. Minard, 1985), indicates that the site geology is comprised of alluvial and glacial outwash deposits. The southern majority of the site is mapped as *Holocene Younger Alluvial and Estuarine Deposits* (Qyal), with the exception of the north end near First Street which is mapped as *Quaternary Recessional Outwash* (Qvrm) - Marysville Sand Member. The Marysville Sand Member Recessional Outwash, dating from the Vashon Stade of the Fraser Glaciation, extends broadly within the valley to the north of the site and is generally described as stratified to massive outwash sand with minor amounts of gravel containing interbeds of silt and clay throughout. The Marysville Sand is up to approximately 100 feet thick, deepest in the middle of the valley, and is underlain by Vashon Till (Qvt) which is mapped on hills to the east and west of the regional valley. The younger alluvial deposits (Qyal) are mapped along present-day streams and waterways north of the site location, and primarily along Ebey Slough and other estuarine land to the south of the site. Deposits are described as stream-laid stratified sediment of varying sand, silt, and clay composition with common organic matter. The unit includes tidal flats, peat deposits, and historic shoreline fill zones, and may be up to 100 feet thick or more.

Shallow soils at the western and southern majority of the site are mapped by the NRCS Web Soil Survey as *Sumas Silt-Loam*, formed from alluvium on flood plains and generally consisting of silt loam to silty clay loam becoming coarse sand with depth. The *Sumas Silt-Loam* unit is poorly drained and classified as Hydrologic Group C, with the depth to seasonal high groundwater ranging from 1.0 to 3.0 feet BPG. The northeast portion of the site is mapped as *Field Silt-Loam*, generally silt loam becoming stratified loamy fine sand to sand with depth. *Field Silt-Loam* is typically moderately well drained and classified as Hydrologic Group B, with the depth to seasonal high groundwater ranging from 3.0 to 4.0 feet BPG.

Native soil conditions encountered in the field below approximately 10 to 15 feet to maximum depth explored consist generally of bedded sand to silty fine sand with locally interbedded silt horizons. By approximately 85 to 90 feet BPG, silt deposits were found to termination depth of 100 feet BPG. The silt appears correlative with alluvial conditions rather than glacial till based on recorded consistency. These conditions are typical of thick regional alluvium and glacial outwash deposits, and are thus generally consistent with local geology sources.

Near-surface conditions of the upper 10 to 15 feet as observed in detail via GP cores appear highly variable. Shallow layers likely represent imported or native grade fills anticipated to vary locally within the site. Deposits of silt and sand with abundant organic detritus were also encountered commonly. These horizons may represent near-shore organic-rich deposits, buried soil-slash cuttings, reworked native materials from on-site development or off-site sources, or a combination thereof.

3.3 SOIL CONDITIONS

A general characterization of on-site soil units interpreted from CPT explorations and observed within the upper 15 feet during GP borings is presented below. The exploration logs in Appendix C and D present details of soils encountered at each exploration location.

The on-site soils are generally characterized as follows:

- **Uncontrolled Fills and Deleterious Deposits (GP, SP, SM, ML, OL, Wood) – Surface to 12 feet BPG:**

  Uncontrolled fill or disturbed soils and underlying deleterious soils and organic remains were encountered at all GP borings extending from the surface to typically 10 to 12 feet BPG, and locally to the maximum GP depth of 15 feet BPG. Thickest locations were documented at GM-1 and GM-3, at the northeastern most portion of the site. Locations among the north-central area (GM-2, GM-4, and GM-10) reached apparent alluvial soils by approximately 10 to 13 feet BPG. Explorations along the southern fringe of the study area in the middle portion of the site (GM-5, GM-6, and GM-9) contacted apparent alluvial soils in the range of approximately 7 to 10 feet BPG. The high degree of variation in these deposits is reflected on the attached GP logs, which have not been simplified and thus record detailed field notes from continuous core observations for illustrative purposes.

  Fill soils generally consist of a thin layer of topsoil with vegetation to approximately 0.5 to 1.0 feet BPG at all GP borings observed except GM-3, GM-4, and GM-5. GM-3 and GM-5 had approximately 0.5 to 1.0 feet of asphalt overlying crushed gravel base course and a relic concrete slab with leveling crushed aggregate extending to depths between 1.25 and 2.25 feet BPG. GM-4 displayed a thick silt layer topping a layer of crushed gravel approximately 1.0 feet thick. All three locations encountered a thin layer of relic asphalt in the range of 3 to 4 feet BPG. Beneath, soils varied typically on the 6-inch to 1-foot scale, composed of apparent coarse-grained fills and native grade fills intermixed with common organics consisting of wood debris and occasional layers of abundant organic deposits. Organic content ranged from large woody branches to wood chips and sawdust, appearing to be of non-native origin in the upper deposits.

  With depth, soils became increasingly fine-grained and composed of organic-rich clays, silts and silty sands. Some layers may represent disturbed or redeposited native organic soils intermixed with wood detritus. With depth, the organic layers typically become more consistent with evidence of carbonized material and rhythmic lensing suggesting native deposits of estuarine origin.

- **Upper Alluvial Deposits (SP, SM, SM-ML) – 12 to 85 feet BPG:**

  Soils consisting of sand, silty sand, and silty sand to sandy silt of generally loose becoming medium dense or stiff consistency were encountered at both Cone Penetrometer Tests (CPT) exploration locations. The deposits were composed primarily of thickly bedded sands with interbedded silty sand horizons of 1 to 3 foot thickness. Less commonly, silt-rich interbeds occurred for 1 to 2 feet. Correlated SPT data indicates soils are loose to medium dense to approximately 15 to 17 feet BPG, becoming consistently medium dense below with lenses of dense sand. While no visual observations were made of the unit characteristics at depths greater than 15 feet BPG, interpretation of soil type and consistency is based upon the soil behavior type using data from the UBC-1873 during CPT probes to 100 feet BPG in the vicinity of GM-4 and GM-10. CPT logs indicate these deposits begin at approximately 11 to 12 feet BPG.

Apparent alluvial soils were encountered by end depth at a majority of GP explorations conducted among the north-central and central areas of the site. Observed in GP cores, deposits generally consisted of sands, silty sands, and silts interbedded on the 1 to 3 foot scale, consistent with CPT interpretations. Visually estimated consistencies ranged from loose to firm or medium dense. Some organic detritus was entrained within soils and locally occurring in concentrated thin lenses. At GM-6, larger cedar debris was encountered near end depth.

- **Lower Alluvial Deposits (SM, SP-SM, ML) – 85 to 100 feet BPG or greater:**
  CPT results indicate alluvial deposits differ past 85 to 90 feet BPG from those above. Soils become predominantly silty with thin silty sand interbeds. Soil strength ranges from medium stiff to stiff in the silts, and medium dense in the sands. CPT instrumentation measured elevated pore pressures during advancement in this depth range, suggesting some clay content and cohesive conditions are present locally. This unit extended to end depth of 100 feet BPG.

3.4 SURFACE WATER AND GROUNDWATER CONDITIONS

Aside from the existing marina footprint, no surface water features were observed within the site during the current site explorations conducted in the late winter season. Site topography is generally near sea level, and an approximately 1.5-acre open marina is located in the center of the subject parcel, with high waters estimated as 5.0 to 7.0 feet below existing grade of the adjacent site upland. In all observed GP borings, water was present during the time of drilling. Water conditions were higher in the borings furthest away from the marina, generally ranging from 0.8 feet in GM-5 at the east boundary to 4.95 feet in GM-1 at the north end of the property. Borings located around the perimeter of the marina (GM-6, GM-9 and GM-10) had recorded water levels at 7.95, 6.02 and 6.01 feet BPG, respectively. Based on regional conditions at the time of the December and February explorations, these conditions are interpreted to represent typical but not necessarily peak winter levels. The open water of the marina appears to have an effect on landward water levels at test locations in proximity, while locations further away at the edges of the site may be more affected by seasonal fluctuations and the presence or absence of restrictive horizons in the complex shallow stratigraphy.

Soil mottling was observed in some locations within a few feet of native grade. With generally high groundwater levels and heavy organics within the sampling matrices, soil mottling was observed below 2.0 feet BPG. Mottled soils and low-chroma colors are indicative of a high seasonal water table and/or soil wetting and drying cycles. MTC’s scope of investigation did not include observation and monitoring of seasonal variations or conclusive measurement of groundwater elevations at the time of exploration. Water levels noted above should be considered close approximations. Given the time of this investigation in the late winter, it is interpreted that measured groundwater levels represent a general, but not absolute wet-season condition. Actual groundwater conditions can vary locally as a consequence of complex shallow stratigraphy, especially in the winter months. It is important to note that past development of the property and adjacent sites, including stripping and drainage improvements in the vicinity, may have altered winter groundwater patterns or lowered seasonal levels since mottling was established.

Due to the close proximity of Ebey Slough and the Snohomish River system, water table levels may remain broadly consistent throughout the year. Groundwater may perch locally within the upper column, especially in areas near the north and east boundaries of the property where ground improvements, relic impermeable surfaces and fine-grained soils are present. Seepage from perched water horizons or confined coarse lenses at shallow levels should be anticipated in the wet season. Field observations suggest that the groundwater table will likely be encountered in excavations at the project site beginning around 6.0 feet BPG in areas adjacent to the marina. If earthwork occurs in the wet season, general wet conditions and free water should be anticipated to begin as shallow as 0.5 to 1.0 feet BPG.

4.0 GEOTECHNICAL ANALYSIS

This section addressed the results of site-specific geotechnical analysis and review of available relevant geologic data. In the current phase of study, this has included: 1) Determination of Seismic Design Parameters, and 2) Liquefaction analysis. The results described below form the basis for the geotechnical feasibility discussion and preliminary recommendations presented in Section 5.0.

4.1 SEISMIC DESIGN AND ACCELERATION PARAMETERS

According to the Washington State Department of Natural Resources *Site Class Map of Snohomish County, Washington* (Palmer et al., 2004), the site location is mapped as Seismic Site Class D to E, representing a relatively moderate to high potential for increased amplitude of ground shaking during a seismic event based on generalized soil conditions. Areas to the west and southwest along the shoreline of Port Gardner Bay are mapped as Site Class E and F. The Marysville valley north of the site is broadly mapped as Site Class D, which extends to the site location. For determination of site-specific seismic design parameters, the consistency of actual soil conditions encountered at the site was evaluated. Average soils in the upper 100 feet are considered. Assuming site design and preparation methods are employed to address the issues discussed herein, including mitigation of liquefaction susceptibility and surface fills, soils to 100 feet depth are on average medium dense to dense or stiff.

The *USGS Seismic Design Map Tool* was used to determine site coefficients and spectral response accelerations for the project site assuming design Site Class D, representing a generally dense or stiff soil profile.

Table 2. Seismic Design Parameters – Site Class D

Mapped Acceleration Parameters (MCE horizontal)	S_S	1.165 g
	S_1	0.450 g
Site Coefficient Values	F_a	1.034
	F_v	1.550
Calculated Peak SRA	S_{MS}	1.204 g
	S_{M1}	0.698 g
Design Peak SRA (*2/3 of peak*)	S_{DS}	0.803 g
	S_{D1}	0.465 g
Seismic Design Category – Short Period (0.2 Second) Acceleration		D
Seismic Design Category – 1-Second Period Acceleration		D

Chapter 20 of ASCE 7-10, Section 20.3.1, indicates that sites deemed susceptible to liquefaction shall be treated as Site Class F and a site response analysis shall be performed specific to the proposed improvements. Liquefaction potential is addressed in the section below. The above seismic parameters shall be considered for preliminary design purposes only, until the actual scope of proposed development is known and mitigating measures are taken into consideration. In some cases, liquefaction susceptibility can be reduced to an acceptable design range via installation of ground improvements such as stone columns, rammed aggregate piers, or surcharge loading. This type of improvement effectively densifies the subsurface and can contribute a positive net effect to site class designation. In the event that such improvements are determined successful in adequately mitigating liquefaction potential, and are incorporated into site construction, Seismic Site Class D appears appropriate. If another method of building design is selected to address liquefaction susceptibility, such as a pile or mat foundation, MTC recommends that we be contacted for further consultation and analysis as needed to assist in determining seismic site class and design parameters appropriate for the chosen improvements.

4.2 LIQUEFACTION SUSCEPTIBILITY AND SETTLEMENT POTENTIAL

According to the *Liquefaction Susceptibility Map of Snohomish County, Washington* (Palmer et al., 2004), the site is identified as having a moderate to high liquefaction susceptibility. This is generally consistent with the findings of MTC's investigation to date. The site is underlain by an alluvial deposition stratigraphy primarily composed of loose to medium dense sand and silty sand, with groundwater beginning at 5 to 7 feet depth. Liquefaction is a phenomenon associated with a subsurface profile of relatively loose, cohesionless soils saturated by groundwater. Under seismic shaking the pore pressure can exceed the soil's shear resistance and the soil 'liquefies', which may result in excessive settlements that are damaging to structures and disruptive to exterior improvements.

MTC performed analysis of site liquefaction potential and resulting ground subsidence from available site exploration data collected via the limited explorations conducted in the current project phase. CPT data are widely considered desirable for liquefaction analysis due to the continuous resolution of stratigraphy as well as the logging of various soil and driving parameters correlative with traditional borehole SPT data. For this feasibility-level study, CPT results were used independently without correlation to direct soil sampling and geotechnical blow-count data gathered from hollow-stem auger drilling and SPT sampling methods. It is possible that results will change or be refined based on additional data, or that analyses using solely CPT data present one viewpoint.

Analysis was completed using LiquefyPro, Version 5.8h, published by CivilTech Software©. LiquefyPro performs liquefaction analysis in accordance with the latest National Center for Earthquake Engineering Research (NCEER) Workshop recommended procedures and provides several options for the treatment of data inputs. CPT profiles were analyzed according to the Modified Robertson method which includes fines correction from CPT data. Settlement estimates were obtained after Ishihara and Yoshimine (1987). A 7.0 magnitude earthquake was applied for all analyses. Calculations were completed for maximum considered earthquake peak ground acceleration (0.485g) in accordance with ASCE 7-10 guidelines. To most accurately reflect liquefaction risk of existing conditions, no factor of safety was applied in the analyses. Final analysis should be conducted which takes into account proposed building design and any relevant ground improvements, as well as appropriate factor of safety. For complete graphical results, refer to Appendix E.

For initial calculations, both profiles were analyzed to assess site variability and substantiate data. Results were similar between CPT locations. The CPT-2 profile (Figure 7) yielded a marginally higher value; however data recovery failure at that location from approximately 63 to 66 feet BPG appears to affect the results by roughly the same degree. Therefore the results are considered broadly equivalent, and CPT-1 data was utilized for further calculations.

Soil column settlements of approximately 9.1 inches were tabulated using CPT-1 data, representing existing conditions with no ground improvements or surcharge loading from building pad preparations under maximum considered earthquake peak ground acceleration. This analysis is based on liquefaction-induced settlement in the upper 100 feet of the profile, to maximum depths explored. For purposes of assessing a conservative scenario of liquefaction potential, the relatively fine-grained silty members of the stratigraphy were not prohibited from liquefying. Table 3 summarizes the results of MTC’s liquefaction analysis represented graphically in Appendix E, Figures 6 to 9.

Table 3. Summary of Liquefaction-induced Settlement Estimates (in inches)

ANALYSIS SCENARIO	**Liquefaction-induced Settlement**
Peak Ground Acceleration	PGA-max = 0.485
CPT-1 – Existing Conditions	9.1
CPT-2 – Existing Conditions	9.8
CPT-1 – Surcharge (5-foot fill equivalent)	4.2
CPT-1 – Surcharge (10-foot fill equivalent)	2.5

Post-development liquefaction potential may be reduced as an effect of increased overburden pressures transferred from development features including blanket addition of fill soils as well as building loads if applied uniformly (such as with a structural mat foundation). In a preliminary attempt to demonstrate potential effects of surface development on liquefaction potential for discussion purposes, a hypothetical surcharge analysis was performed to simulate additional vertical loading at multiple degrees. LiquefyPro allows for assigning surface overburden fill thickness and density overtop of existing grade. For overburdens of 5 and 10 feet thickness at a bulk unit weight of 125 pounds per cubic foot, pressures at the existing surface equate to 625 and 1250 pounds per square foot respectively. Under these surface surcharges, potential settlements calculated via LiquefyPro are reduced to 4.2 and 2.5 inches.

5.0 FEASIBILITY DISCUSSION & RECOMMENDATIONS

MTC has prepared the following discussion and cursory-level recommendations for consideration by the client and their design team toward evaluation of site feasibility for the proposed development. The discussion items and recommendations presented are based on MTC's current understanding of general project scope and the client's interest at this time, but should not be considered an exhaustive address of potential considerations for site development. Additional work including further site exploration of building areas and geotechnical engineering analysis will be required to properly address specific geotechnical considerations as subsequent phases of project design are completed. We recommend that MTC be allowed to review and comment as project plans develop and, as necessary, provide additional explorations, consultation and engineering services as deemed appropriate for the evolving project.

5.1 FOUNDATION FEASIBILITY

Two requirements must be fulfilled in foundation design. First, loads must be less than the ultimate bearing capacity of foundation soils to maintain stability; and secondly, the differential settlement must not exceed an amount that will produce adverse behavior of the structure. The allowable settlement is usually exceeded before bearing capacity considerations become important; thus, the allowable bearing pressure is normally controlled by settlement considerations. Excess settlement due to adverse soil conditions may be a result of shallow or deep soils, static or dynamic factors, or a combination thereof.

5.1.1 Shallow Considerations

Explorations to date have shown on average the upper approximately 12 feet of the site subsurface to consist of various uncontrolled fills, organic soils, debris and deleterious matter unsuitable for direct building support. Uncontrolled fills and native grade fills of non-structural quality are generally not suitable for direct bearing of building foundations because of the inherent variability and high risk of differential settlement when placed under load. Organic soils and soils containing abundant plant and wood matter are highly susceptible to settlement via consolidation as well as decomposition over time, and are generally unsuitable beneath building areas. Thus, large-scale excavation of the building area may be required for either basement-level construction or removal of deleterious fills and backfill with imported structural fill, depending on other site improvement methods and foundation styles selected. As discussed in Section 5.2.1, excavations of this scale will most likely exceed depth to groundwater, even if work is completed in the summer months. Major dewatering efforts, lining of excavations, engineered shoring, and selective use of suitable backfill materials are anticipated to be involved.

MTC understands a subsurface parking garage or basement level is preliminarily considered. Undisturbed alluvial soils at depth, or a fill pad installed over suitably firm alluvial soils, may be eligible for support of relatively lighter loads from a shallow bearing perspective. The explorations conducted to date did not include direct testing of in-situ soil strength, but CPT provides correlative SPT data suggesting upper native soils are in the lower range of stiff or medium dense. If a contribution of support from these soils is proposed, building locations should be further explored with equipment capable of directly sampling and confirming soil consistency at proposed depth.

Consideration of shallow bearing contribution, however, is further complicated by deep settlement concerns. MTC's liquefaction analysis has estimated the project site may be subject to potential seismic-induced settlements on the order of 9 inches based on existing vacant conditions, with equal or lesser settlement potential remaining after development surcharges are applied depending on building and site design. The site's liquefaction susceptibility is an effect of the combined occurrence of shallow groundwater and thick alluvial soils composed primarily of sand to silty sand of medium dense consistency. Because of liquefaction potential, bearing suitability and settlement from shallow soils is considered a secondary risk and design factor compared to liquefaction-induced settlement. In other words, a successful mitigation of liquefaction may also generally address shallow foundation suitability.

5.1.2 Ground Improvements

MTC recommends the client consider the option of proprietary ground improvement systems, possibly coupled with surcharge loading and raising of site grades, as a means of mitigating liquefaction potential at the site building area. Typically ground improvements, such as described below, have the additional benefit of increasing the allowable bearing capacity of shallow soils which can facilitate shallow foundation construction in existing marginal situations.

Ground improvement systems utilize variations of proprietary methods in order to improve bearing conditions and densify subsurface conditions both laterally and vertically. Rammed aggregate piers (RAPs) and stone columns are popular ground improvement methods for this region involving placement or injection of dense aggregate into the subsurface. RAPs are typically placed using a hollow mandrel, which transports the aggregate from a hopper to the desired depth. The mandrel is retracted about every foot to allow for aggregate placement, then vibrated to compact the material. This process proceeds upward until reaching the surface, resulting in a bulging rock pier resembling a pile. Stone column installation is broadly similar to RAPs. The aggregate is placed using a mandrel and hopper, except densification is performed using air pressure. Air-pressure method is considered effective for both vertical compaction as well as lateral injection of rock fill into soft or loose soil horizons at depth. Comparatively, stone columns tend to provide more of a physical mixing and densification with the surrounding native materials whereas RAPs densify to some extent while also functioning more similarly to traditional piles. Maximum depth range for both of these systems is typically around 50 feet, although entry into a prepared excavation zone may permit increasing effective installation depth.

If ground improvements are considered an option for site development, MTC recommends contacting ground improvement design-build specialty contractors to preliminarily discuss liquefaction mitigation strategies. The contractor should reassess liquefaction settlement in light of the benefits of their proprietary system and the results presented herein. MTC may be contacted for additional information or clarifications to be provided at consent of the client.

5.1.3 Pile Foundation Option

A pile foundation may be a feasible option in addressing the risk of liquefaction settlement for building construction. Engineering analysis toward assessing the feasibility and design of pile foundation elements is beyond the scope of this phase of study. For the purpose of general discussion, MTC anticipates piles may need to extend to 60 to70 feet BPG or greater to be founded in suitably dense bearing sands while surpassing the extent of most identified liquefiable soils to eliminate the need for other deep ground improvements. Commonly, pile systems are considered the more expensive option for liquefaction mitigation compared to stone columns or RAPs. However in this case, piles may be a more economically viable alternative when accounting for associated site improvements. For instance, if the project elects to construct at or near existing grade due to the difficulties and costs involved with mass excavation and related dewatering, a pile system may be designed to provide foundation support extending through existing uncontrolled and deleterious fill soils. Comparatively, ground improvements may still require removal and replacement of deleterious soils, or may call for an increased array density or alternative methods such as sheathing to account for the unsuitable conditions. If piles are of interest, MTC may be contacted to provide additional engineering analysis and geotechnical consultation towards assessing the relative feasibility of deep foundations. The below discussion is provided solely for the client’s general consideration of pile options, and does not constitute an endorsement or recommendation of a specific type of pile or dismissal of options not presented.

Driven piles are typically useful in soil profiles composed primarily of cohesive materials, or in cases where an obvious ‘floor’ exists for end bearing below loose or soft conditions. Driven piles gain a secondary skin friction capacity from cohesive soils as water pore pressure equilibrates after the pile is driven (known as pile setup). In the case of the granular soil profile at the site, driven piles may be effective at achieving design capacity by target depth. However, driven piles, if designed for the loads needed near the surface, may encounter early refusal or difficult driving conditions in intermediate zones of higher density sands, such as exist at this site. Also, setup due to skin friction in the site’s primarily granular soils is not expected to be a significant factor. The design would, therefore, rely almost totally on end-bearing with a smaller contribution of granular friction for the length of the pile.

Cast-in-place (CIP) piles, installed by auguring methods to reach desired depth, can achieve pre-determined bearing stratum. Grout is applied centrally through the augur as it is retracted. Installation and cost of CIP piles may be significantly affected by the saturated sandy profile. Theoretically, if grouting is performed during augur retraction, it may be sufficient to avoid caving, however as a precautionary measure casing may be required in order to counteract flowing sand situations.

Costs per pile type can vary based on required depths and the materials and equipment necessary to successfully install them. Since larger sizes of driven piles often require hammer and crane assemblies, these are anticipated to be of relatively higher cost compared to ground improvement methods for some design scenarios. For concerns of potential early refusal on intermediate dense sands, a large enough pile and hammer system can be utilized to overcome resistance and achieve desired bearing depth. Although, this may ultimately result in piles that have a greater capacity than is required at the surface and an increase in cost. CIP piles may pose the need for casing or other specialized installation, which can also drive cost higher than typical. Schedule can also be affected most detrimentally by CIP installation, as it is generally not a fast method. Cost and time efficiency of driven piles or ground improvement methods may be more attractive comparatively.

5.2 SITE PREPARATIONS DISCUSSION

Site preparation design and construction methods are anticipated to be heavily affected by the existing fill soils present from the surface at all locations explored, the character of which was highly variable between test locations. The below discussion addresses general concerns anticipated to be relevant to project planning at this time.

5.2.1 Excavations and Dewatering

Excavations can generally be performed with conventional earthmoving equipment such as bulldozers, scrapers, and excavators. Larger excavators may be needed to efficiently mitigate existing debris fill if present to a greater size and quantity than encountered to date.

As discussed above for foundation feasibility, existing soils containing highly organic and deleterious materials are considered generally unsuitable to remain below building footprints unless a pile foundation is utilized to support at-grade construction. Underground parking or basement level construction will necessitate mass excavation of the building footprint. Use of a ground improvement system for liquefaction mitigation may also require excavation removal of unsuitable materials and backfilling with structural fill to proposed foundation grade, unless specialized methods are employed to allow for installation and structural support through organic and deleterious conditions.

Excavations of this scale will most likely exceed depth to groundwater, even if work is completed in the summer months. Major dewatering efforts, lining of excavations, engineered shoring, and selective use of suitable backfill materials are anticipated to be involved. Dewatering efforts may also be necessary for utility trenches, depending on total excavation depth, season of construction, and weather conditions during earthwork. MTC recommends major earthwork activities take place during the dry season if possible to minimize the potential for encountering perched groundwater at shallow depth, and to reduce the burden of dewatering due to temporary storm conditions. Some amount of water seepage from shallow sources or perched lenses may be unavoidable year-round. We recommend that if large-scale excavations are proposed, a groundwater level monitoring program should be conducted prior to commencing construction to fully assess seasonal fluctuations.

5.2.2 Exterior Surface Improvements

MTC anticipates structural exterior improvements may include access road and parking pavements, walls or barriers, large sign foundations and other free-standing non-occupied structures. We recommend bearing and foundation suitability be evaluated on a case-by-case basis for the type and location of structure proposed. Additional site investigation of proposed improvement locations should be conducted to verify conditions and design or modify foundations as needed.

In areas where uncontrolled fills are thick but consistently firm and free of deleterious materials, the soils may be suitable to remain beneath light exterior loads such as walkway pavements and exterior slab-on-grade constructions but not beneath building foundations. Design of structural base materials and thicknesses must be modified accordingly to account for the marginal subgrade suitability of remaining fills. In addition, confirmation of subgrade suitability prior to and during construction will be integral. Some amount of overexcavation and backfill with structural fill materials should be anticipated, either accounted for in design or performed as needed during construction.

Foundations for large signs, awnings, and free-standing exterior covers or shelters will likely call for installing column supports which extend through the fill section to bear on underlying suitably firm alluvial strata encountered on average by 10 to 12 feet BPG, but as deep as 14 to 15 feet BPG.

Road and parking pavement sections should be specifically designed for the site conditions along the proposed alignment. A more economical section will be achievable if a greater-than-normal regularity of repairs is acceptable resulting from settlement in underlying uncontrolled fills and unsuitable soils. For a more durable pavement installation, we anticipate excavation and replacement of existing fill soils with structural fill to a thickness of multiple feet or greater as well as use of a geotextile ground stabilization product, the actual extent of which depends on local subgrade conditions.

5.2.3 Site Materials

All material placed below structures or pavement areas should be considered structural fill. In addition to standard practices and project material specifications, structural fill shall be free of deleterious material, have a maximum particle size of 6 inches, and be compactable to the required compaction level. Excavated uncontrolled fills, debris, organic soils, and soils with deleterious materials are not suitable for re-use as structural fill. Organic-rich soils stockpiled during site preparations may be suitable or amended for reuse as top-dressing landscape fill to promote vegetative growth.

Historic coarse-grained fills resembling pit run may not be present to an amount suitable for planned re-use. However, if such soils are carefully excavated and stockpiled while limiting cross-contamination and degredation, these materials may be eligible for re-use as structural fill provided the materials are confirmed prior to placement, properly moisture-conditioned and reinstalled in accordance with industry standards and project requirements for placement and compaction of structural materials. During warm, dry weather, it will likely be necessary to add water to these soils after residing in stockpiles. The condition and suitability of stockpiled on-site materials should be verified prior to reuse as structural fill.

5.2.4 Wet Weather Construction Considerations

Uncontrolled fills near the site surface are generally moisture sensitive and will become soft and difficult to traverse with construction equipment when wet. Since on-site soils will be difficult to work with during periods of wet weather, and frozen soil is not suitable for use as structural fill, near-surface earthwork activities should generally take place in late spring, summer or early fall. In addition, late summer may be the most preferable time for construction involving building excavations and deep utility trenching, corresponding to the period of generally lowest perched water and groundwater occurrences. Although as discussed above, major dewatering efforts and specialized shoring methods are anticipated to be required for deep excavations throughout the year.

5.3 ON-SITE STORMWATER DISPOSAL FEASIBILITY

Shallow soil conditions consisting of highly variable uncontrolled fills, organic soils, and deleterious materials generally do not appear suitable for design of on-site infiltration of stormwater. Infiltration and transmission capacity of fill soils is anticipated to be on average low or very low to nil, with lateral transmission within confined coarse-grained horizons being the primary means of infiltration. As an additional constraint to stormwater infiltration design, evidence of perched water was encountered locally near the surface during our field explorations. The groundwater table was reliably encountered by approximately 5 to 7 feet BPG, apparently related to water elevation in the marina. It is possible that some practical infiltration value can be gained by employing shallow bioretention facilities (such as rain gardens) equipped with underdrains and overflow safeguards, or a similar style of feature intended primarily for on-site treatment and flow control but with the added benefit of an open base to allow for local infiltration to occur as possible.

If exterior site development strategies and/or site environmental remediation activities call for removal of existing fills and replacement with imported material, this may allow for incorporation of engineered soils and stormwater control measures not feasible for current site conditions. In this case, potential systems are expected to be restrained primarily by spatial allowance and shallow groundwater depth. Options may include permeable pavement, shallow bio-infiltration swales, subsurface storage and disposal beds, or a combination thereof.

6.0 LIMITATIONS

Recommendations contained in this report are based on our understanding of the proposed development and construction activities, our field observations and exploration and our laboratory test results. It is possible that soil and groundwater conditions could vary and differ between or beyond the points explored. If soil or groundwater conditions are encountered during construction that vary or differ from those described herein, we should be notified immediately in order that a review may be made and supplemental recommendations provided. If the scope of the proposed construction, including the proposed loads or structural locations, changes from that described in this report, our recommendations should also be reviewed.

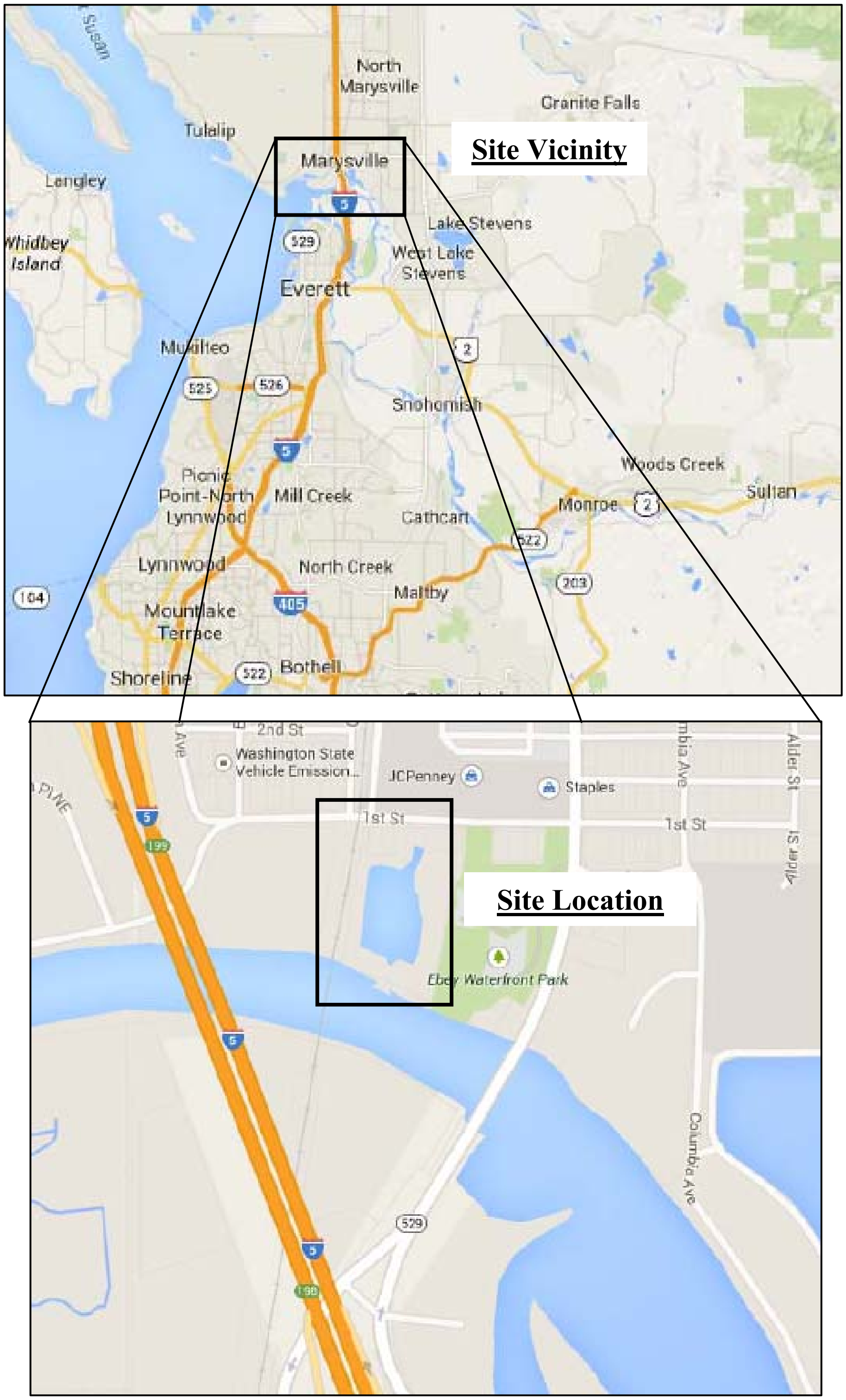
We have prepared this report in substantial accordance with the generally accepted geotechnical engineering practice as it exists in the site area at the time of our study. No warranty, express or implied, is made. The recommendations provided in this report are based on the assumption that an adequate program of tests and observations will be conducted by MTC during the construction phase in order to evaluate compliance with our recommendations. Other standards or documents referenced in any given standard cited in this report, or otherwise relied upon by the author of this report, are only mentioned in the given standard; they are not incorporated into it or “included by referenced”, as that latter term is used relative to contracts or other matters of law.

This report may be used only by City of Marysville and their design consultants and only for the purposes stated within a reasonable time from its issuance, but in no event later than 18 months from the date of the report. Note that if another firm assumes Geotechnical Engineer of Record responsibilities they need to review this report and either concur with the findings, conclusions, and recommendations or provide alternate findings, conclusions and recommendation under the guidance of a professional engineer registered in the State of Washington. The recommendations of this report are based on the assumption that the Geotechnical Engineer of Record has reviewed and agrees with the findings, conclusion and recommendations of this report.

Land or facility use, on- and off-site conditions, regulations, or other factors may change over time, and additional work may be required with the passage of time. Based on the intended use of the report, MTC may recommend that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by City of Marysville or anyone else will release MTC from any liability resulting from the use of this report by any unauthorized party and City of Marysville agrees to defend, indemnify, and hold harmless MTC from any claim or liability associated with such unauthorized use or non-compliance. We recommend that MTC be given the opportunity to review the final project plans and specifications to evaluate if our recommendations have been properly interpreted. We assume no responsibility for misinterpretation of our recommendations.

The scope of work for this subsurface exploration and geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this site.

Appendix A. SITE LOCATION & VICINITY

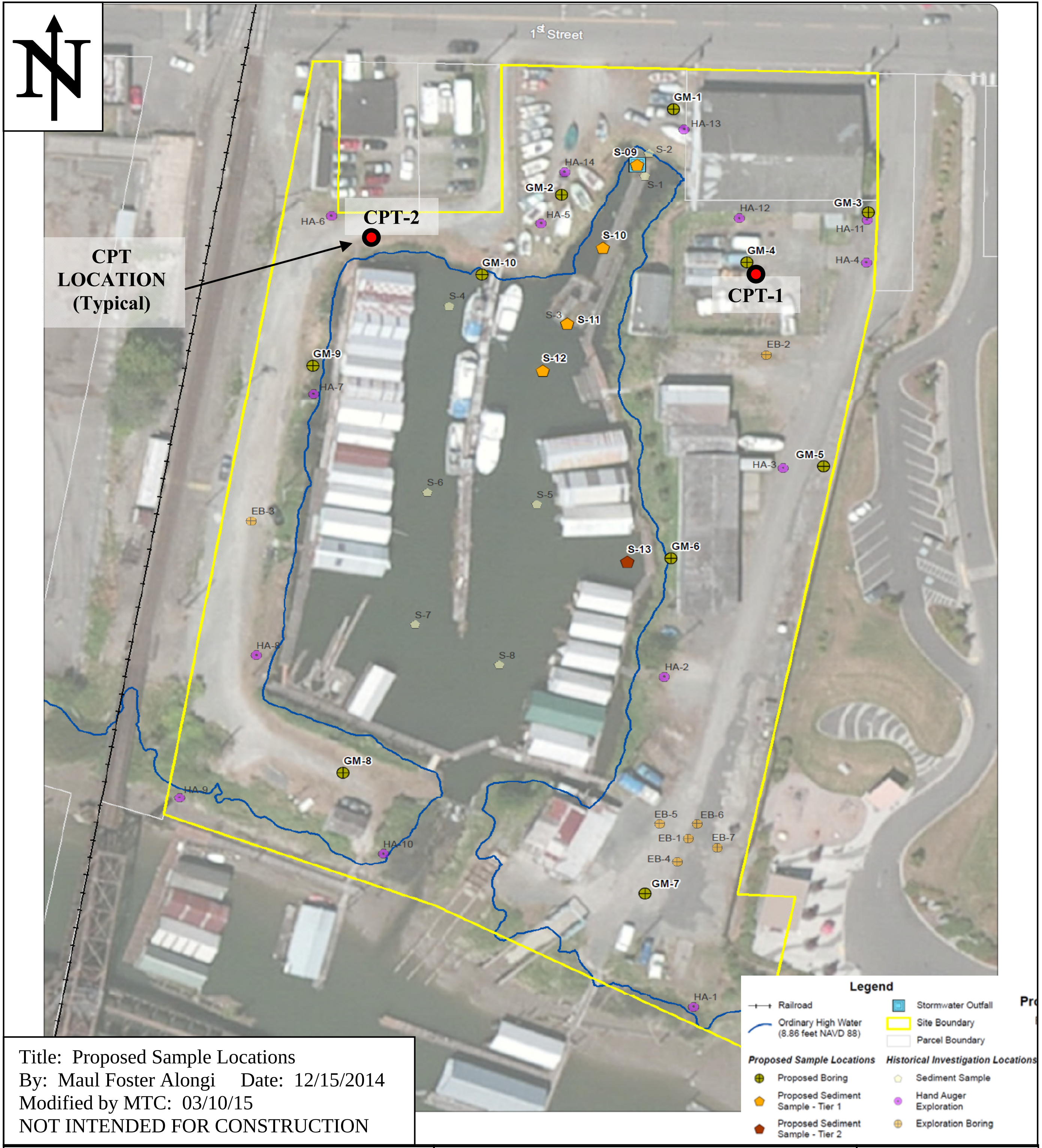


Materials Testing & Consulting, Inc.
777 Chrysler Drive
Burlington, WA 98233

Location & Vicinity Map
Geddes Marina Redevelopment
1326 First Street
Marysville, WA

FIGURE
1

Appendix B. SITE MAP OF TEST LOCATIONS



Title: Proposed Sample Locations
By: Maul Foster Alongi Date: 12/15/2014
Modified by MTC: 03/10/15
NOT INTENDED FOR CONSTRUCTION

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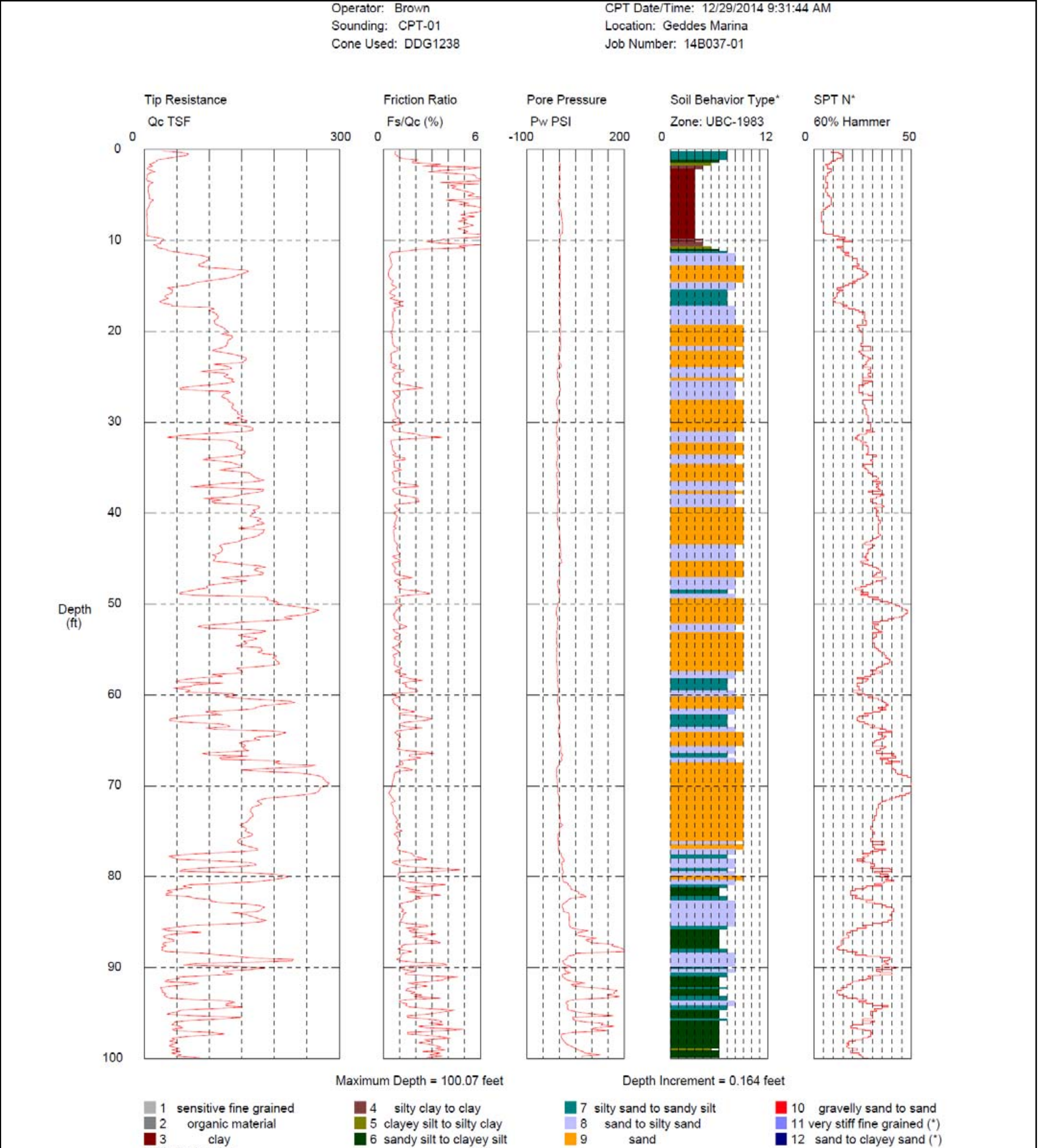
Proposed Development Map
Geddes Marina Redevelopment
1326 First Street
Marysville, WA

FIGURE
2

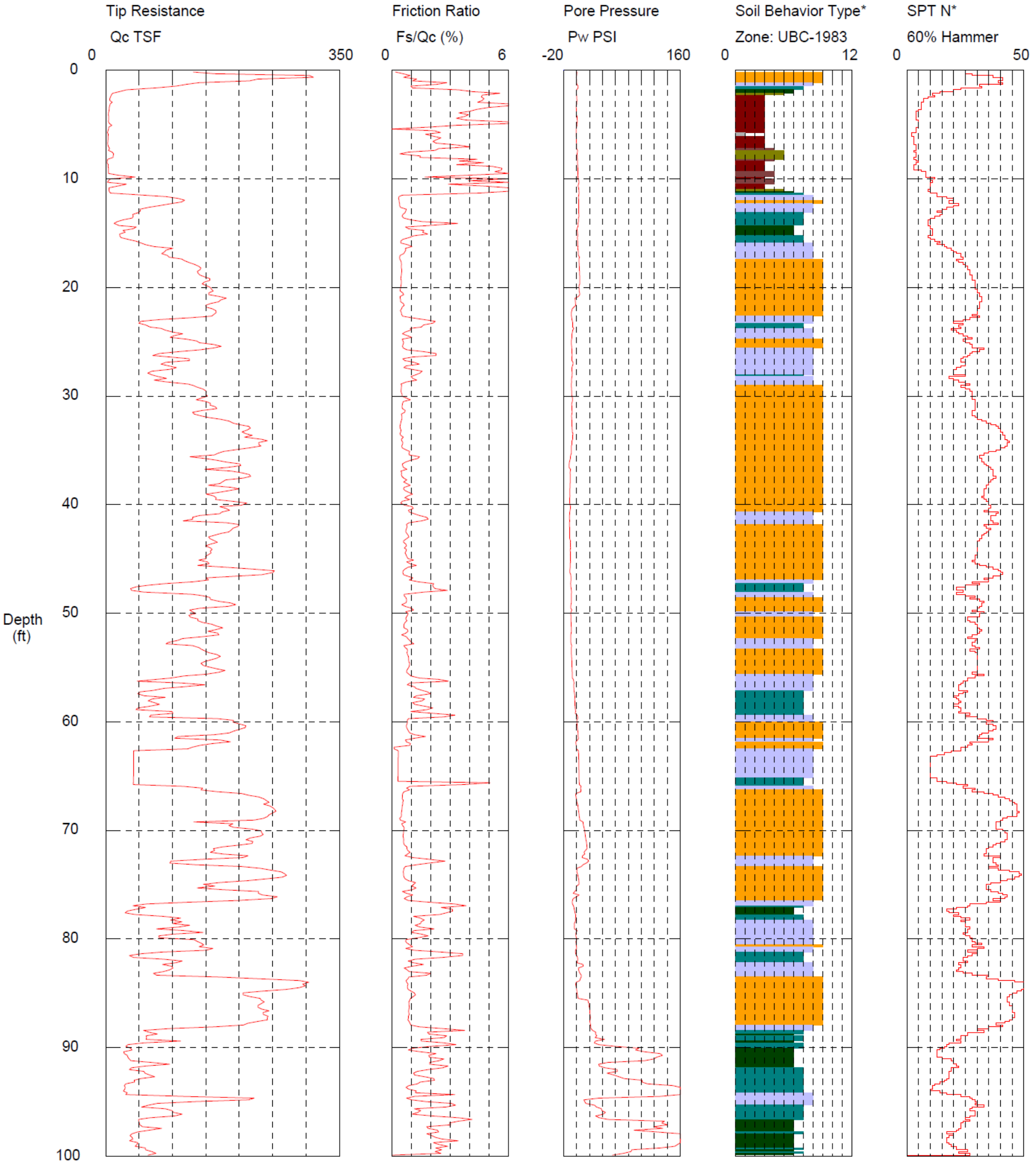
Appendix C. CPT RESULTS



Materials Testing & Consulting, Inc. 777 Chrysler Drive Burlington, WA 98233	**CPT-1 Log** Geddes Marina Redevelopment 1326 First Street Marysville, WA	**FIGURE** **3**

Operator: Brown
Sounding: CPT-02
Cone Used: DDG1238

CPT Date/Time: 12/29/2014 10:51:11 AM
Location: Geddes Marina
Job Number: 14B037-01



Maximum Depth = 99.90 feet Depth Increment = 0.164 feet

- ⬜ 1 sensitive fine grained
- ⬛ 2 organic material
- 🟥 3 clay
- 🟫 4 silty clay to clay
- 🟫 5 clayey silt to silty clay
- 🟩 6 sandy silt to clayey silt
- 🟦 7 silty sand to sandy silt
- 🟪 8 sand to silty sand
- 🟧 9 sand
- 🟥 10 gravelly sand to sand
- 🟪 11 very stiff fine grained (*)
- 🟦 12 sand to clayey sand (*)

InSitu Engineering Data from 62.6 to 65.8 feet is not representative due to inoperative depth counter.

Materials Testing & Consulting, Inc. 777 Chrysler Drive Burlington, WA 98233	**CPT-2 Log** Geddes Marina Redevelopment 1326 First Street Marysville, WA	**FIGURE** **4**

Appendix D. GEOPROBE EXPLORATION LOGS

Exploration logs are shown in Appendix C, representing observations by MTC personnel collected during Geoprobe exploration activities in support of environmental sampling conducted by the client. Select representative explorations were monitored by our field geologist who examined and classified the materials encountered in accordance with the Unified Soil Classification System (USCS), and recorded pertinent information including soil depths and stratigraphy, observed soil engineering characteristics, inclusion of refuse and deleterious materials as encountered, and groundwater occurrence.

Due to the nature of the site conditions, potential for environmental contamination, and exploration activities at the time of this study, the scope of work did not include sampling of soils for geotechnical laboratory analysis. The classifications and resulting interpretations reported herein are based on MTC's field classifications and best available information gathered during the explorations.

The stratification lines shown on the individual logs represent the approximate boundaries between soil types; actual transitions may be either more gradual or more severe. The conditions depicted are for the date and location indicated only, and it should not necessarily be expected that they are representative of conditions at other locations and times.

Unified Soil Classification System Chart

Major Divisions			Graph	USCS	Typical Description
Coarse Grained Soils More Than 50% Retained On No. 200 Sieve	**Gravel** More Than 50% of Coarse Fraction Retained On No. 4 Sieve	Clean Gravels	▨	GW	Well-graded Gravels, Gravel-Sand Mixtures
			▨	GP	Poorly-Graded Gravels, Gravel-Sand Mixtures
		Gravels With Fines	▨	GM	Silty Gravels, Gravel-Sand-Silt Mixtures
			▨	GC	Clayey Gravels, Gravel-Sand-Clay Mixtures
	Sand More Than 50% of Coarse Fraction Passing No. 4 Sieve	Clean Sands	▨	SW	Well-graded Sands, Gravelly Sands
			▨	SP	Poorly-Graded Sands, Gravelly Sands
		Sands With Fines	▨	SM	Silty Sands, Sand-Silt Mixtures
			▨	SC	Clayey Sands, Clay Mixtures
Fine Grained Soils More Than 50% Passing The No. 200 Sieve	**Silts & Clays**	Liquid Limit Less Than 50	▥	ML	Inorganic Silts, rock Flour, Clayey Silts With Low Plasticity
			▨	CL	Inorganic Clays of Low To Medium Plasticity
			▥	OL	Organic Silts and Organic Silty Clays of Low Plasticity
	Silts & Clays	Liquid Limit Greater Than 50	▥	MH	Inorganic Silts of Moderate Plasticity
			▨	CH	Inorganic Clays of High Plasticity
			▨	OH	Organic Clays And Silts of Medium to High Plasticity
Highly Organic Soils			▤	PT	Peat, Humus, Soils with Predominantly Organic Content

Sampler Symbol Description

◩ Standard Penetration Test (SPT)
▥ Shelby Tube
☒ Grab or Bulk
▤ California (3.0" O.D.)
■ Modified California (2.5" O.D.)

Stratigraphic Contact

─ Distinct Stratigraphic Contact Between Soil Strata
╲ Gradual Change Between Soil Strata
- - - Approximate location of stratagraphic change

▼ Groundwater observed at time of exploration
▽ Measured groundwater level in exploration, well, or piezometer
◆ Perched water observed at time of exploration

Modifiers

Description	%
Trace	>5
Some	5-12
With	>12

Soil Consistency

Granular Soils		**Fine-grained Soils**	
Density	***SPT Blowcount***	***Consistency***	***SPT Blowcount***
Very Loose	0-4	Very Soft	0-2
Loose	4-10	Soft	2-4
Medium Dense	10-30	Firm	4-8
Dense	30-50	Stiff	8-15
Very Dense	> 50	Very Stiff	15-30
		Hard	> 30

Grain Size

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		> 12"	> 12"	Larger than a basketball
Cobbles		3 - 12"	3 - 12"	Fist to basketball
Gravel	Coarse	3/4 - 3"	3/4 - 3"	Thumb to fist
	Fine	#4 - 3/4"	0.19 - 0.75"	Pea to thumb
Sand	Coarse	#10 - #4	0.079 - 0.19"	Rock salt to pea
	Medium	#40 - #10	0.017 - 0.079"	Sugar to rock salt
	Fine	#200 - #40	0.0029 - 0.017"	Flour to Sugar
Fines		Passing #200	< 0.0029"	Flour and smaller

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Exploration Log Key
Geddes Marina Redevelopment
1326 First Street
Marysville, WA

FIGURE
5

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Log of Boring GM-1

Former Geddes Marina Property
Marysville, WA

MTC Project No. 14B037-01

Date Started : 02/03/2015
Date Completed : 02/03/2015
Sampling Method : Geoprobe, Continuous
Location : GM-1
Logged By : MH

Depth in Feet	USCS	GRAPHIC	DESCRIPTION	Samples	Water Level
0	SM		SILTY SAND with organics, loose, moist, grass and roots. BROWN **TOPSOIL/SUBSOIL**		
	SP		SILTY SAND with angular gravel and carbonized wood throughout, dense moist. LIGHT BROWN becoming DARK BROWN		
	GP		GRAVEL hard, moist. DARK GRAY		
	SM		SILTY SAND with angular gravell, wood debris present in lower 0.2 feet.dense, moist. LIGHT BROWN becoming DARK BROWN		
	GP		GRAVEL with sand, crushed gravel, dense, moist. GRAY		▼
5	ML-SM		SILT with some fine-grained SAND, common organics and wood debris throughout, firm, moist. DARK BROWN.		
	ML-OL		SILT with wood chips/sawdust and some to trace gravel, firm, very moist to wet. DARK BROWN		
	SM		SILTY SAND with angular gravel, carbonized wood debris in lower 0.1', firmly packed, moist. DARK BROWN		
	ML		SILT and large wood debris, firm, moist. Large wood debris is fibrous and carbonized. BLUE-GRAY		
10	ML-OL		SILT with wood debris, firm, moist to wet. Predominantly silt in upper 0.2 feet, grading to sawdust and wood chips with depth. GREY-BROWN		
	SS		SILTY SAND with wood debris, firm, very wet. BROWN		
	ML-OL		SILT with wood debris, firm, very wet. GRAY		
	OL-ML		SAWDUST with SILT, some sand, trace gravel, firm, very wet. BROWN		
	SP		SAND with some SILT, well packed, very wet. BROWN		
	OL-ML		SILT with heavy wood debris, Wood debris includes sawdust and some carbonized wood in lower 0.2 feet, well packed, very wet. BROWN		
			Raw wood,very moist to wet. Stick lodged in core upon recovery.		
15			TD 15.0' Boring terminated at contracted depth. No refusal or hard conditions encountered. Groundwater at 4.95' BPG.		

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Log of Boring GM-2

Former Geddes Marina Property
Marysville, WA

MTC Project No. 14B037-01

Date Started : 02/03/2015
Date Completed : 02/03/2015
Sampling Method : Geoprobe, Continuous
Location : GM-2
Logged By : MH

Depth in Feet	USCS	DESCRIPTION	Samples	Water Level
0	SM	SILTY SAND with organics consisting of grass and roots, loose, moist. MEDIUM to DARK BROWN **TOPSOIL/SUBSOIL**		
	GP	GRAVEL with sand, crushed angular clasts, dense, dry. GRAY		
	ML	SANDY SILT, orange mottling and common organics throughout, medium dense, moist. LIGHT BROWN		▼
	SP	SAND with some silt, orange mottling and common orgaincs throughout, loose, moist, fine grained. BROWN-GRAY		
	OL			
	ML-SM	WOOD DEBRIS with silt, black carbonized deposits at upper contact. loose, moist. BROWN-GRAY		
		SILT with some fine-grained sand,firm, moist. BLUE-GRAY		
	OL-ML	HEAVY ORGANICS in silt matrix, firm, very moist. DARK BROWN to BLACK		
	SP	SAND,medium grained, loose, very moist. BROWN		
5	ML-OL	SILT with rhythmic heavy wood lenses, very loose, very wet. BROWN		
	GP	GRAVEL, angular, medium dense, wet. GRAY		
	ML-OL	SILT with wood debris, very loose, moist to wet. BLUE-GRAY		
	GP	GRAVEL with sand, angular gravel clasts, very loose, wet. BLUE-BROWN-GRAY		
	SP	SAND with silt, predominantly medium sand, loose, very wet. BROWN		
	ML-OL	SILT with heavy wood debris, rhythmic carbonized wood lenses, firm, very moist. GRAY becoming BROWN-GRAY		
	ML-OL	SILT with wood debris in matrix, firm, very wet. BROWN		
	OL-ML	WOOD DEBRIS with SILT, heavy wood in matrix to 9 feet, carbonized wood in matrix to 9.3 feet, fresh wood to 9.5, firm, very moist. BROWN		
	SP	SAND, some to trace gravel, Medium to fine grained sand, well packed, heavy organics in matrix, firm, very moist. DARK GRAY		
10	SP	SAND, medium sand grading to coarse sand at lower contact, loose, very moist. BROWN-GRAY		
	GP	GRAVEL, angular clasts, loose, very moist. BROWN-GRAY		
	SP	SAND with trace gravel and wood debris, predominantly medium to coarse sand with gravel in matrix, wood lenses present in 3-4 inch intervals, firm, very moist.. GRAY		
	SM	SILTY SAND, well packed silty sand with rhythmic lenses of medium sand, firm, very moist. LIGHT BROWN		
15	ML-SM	SILT with some sand, fine grained sand, orange mottling throughtout, firm, moist. LIGHT BROWN		

TD 15.0' Boring terminated at contracted depth.
No refusal or hard conditions encountered.
Groundwater at 1.47' BPG.

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Log of Boring GM-3

Former Geddes Marina Property
Marysville, WA

MTC Project No. 14B037-01

Date Started : 02/02/2015
Date Completed : 02/02/2015
Sampling Method : Geoprobe, Continuous
Location : GM-3
Logged By : MH

Depth in Feet	USCS	GRAPHIC	DESCRIPTION	Samples	Water Level
0	FB		**ASPHALT**		
	GP		GRAVEL with sand and some silt, medium dense, moist. GRAY		▼
			CONCRETE SLAB = 4" THICK		
			WOOD DEBRIS and decomposing wood litter, loose, moist. BROWN		
	FB		**ASPHALT, loose**		
	GP		GRAVEL with silt, loose, dry. GRAY		
	SP		SAND with gravel and silt, crushed asphalt in matrix, firm, dry. DARK BROWN		
	ML		SILT,firm, moist. BROWN		
	OL-ML		WOOD CHIPS in silt matrix, clean wood chips at upper contact, wood content lessens with depth, firm, moist. RED-BROWN		
	ML		SILT with some sand, white granular deposits in matrix of unknown origin, loose, moist. . DARK BROWN		
5	OL		HEAVY ORGANICS, decomposing sod and wood chips. loose, moist.		
	GP		GRAVEL with sand and silt, loose, very moist to wet. GRAY		
	GP		GRAVEL with silt and sand, heavy wood debris and organics in matrix, loose, very moist. . GRAY-BROWN		
	ML		SILT, wood fibers in matrix and rhythmic wood lenses, firm, moist. BROWN		
			HEAVY ORGANICS, wood chips and sawdust increasing with depth, loose, moist. BROWN Carbonized Wood for 4"		
10	GP		GRAVEL with sand and silt, firm, very moist. GRAY-BROWN		
			HEAVY ORGANICS, wood debris and sawdust, loose, moist. RED-BROWN		
	SP		SAND, medium to fine grained sand, clean, firm, very moist. GRAY		
			HEAVY ORGANICS with silt, wood chunks up to 2", loose, very moist. ORANGE-BROWN		
	SP-SM		SAND with some silt and trace gravel and sawdust, predominantly fine grained sand grading to medium sand at lower contact. Sawdust at 14.2 ft to lower contact, firm, very moist. . GRAY		
15			T.D. 15.0' Boring terminated at contracted depth. No refusal or hard conditions encountered. Groundwater at 1.94' BPG.		

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Geotechnical & Environmental Engineering

Log of Boring GM-4

Former Geddes Marina Property
Marysville, WA

MTC Project No. 14B037-01

Date Started : 02/03/2015
Date Completed : 02/03/2015
Sampling Method : Geoprobe, Continuous
Location : GM-4
Logged By : MH

Depth in Feet	USCS	DESCRIPTION	Samples	Water Level
0	ML	SILT with some gravel, firm, moist. BROWN		
	GP	GRAVEL with silt, angular gravel clasts, medium dense, moist. GRAY		▼
	FB	ASPHALT, crushed aggregate in black oil matrix, loose. BLACK		
	SM	SILTY SAND with gravel, loose, dry to slightly moist. BROWN		
	SM	SILTY SAND with gravel, firm, Dry to 4 feet, grading from moist to wet at lower contact. moist. BROWN-GRAY		
	SP	SAND with gravel, Predominantly medium to coarse sand, firm, very moist. GRAY		
5	ML-OL	HEAVY ORGANICS, Decaying wood chips and sawdust, loose, very moist. BROWN		
		SILT with some sand and organics, Wood chips and decaying organic material in matrix at lower contact, firm, very moist. BROWN		
	SP	SAND with some gravel, Predominantly coarse grained sand, loose, very moist. . GRAY		
	OL-ML	HEAVY ORGANICS with silt, wood chips and sawdust with silt in matrix, loose, very moist. BROWN		
	ML	SILT with sand, Fine grained sand and increasing organics with depth, loose, very moist. . GRAY-BROWN		
	OL-ML	HEAVY ORGANICS with silt, Wood chips and sawdust with silt in matrix, loose to firm, very wet. BROWN		
	OL-ML	HEAVY ORGANICS with silt, Roots, wood chips and sawdust increasing with depth, Rhythmic lenses of wood chips and sawdust, organics disappear at 9.6 feet, loose, moist. BROWN		
10	GP	GRAVEL with refuse, Some wood chips and sawdust present, loose, very wet. DARK BROWN		
	SP	GRAVELLY SAND with some silt, wood chips within matrix, medium dense, very moist to wet. BROWN		
	OL	HEAVY ORGANICS with silt, sawdust with silt in matrix, tightly packed.medium dense, very wet. DARK BROWN		
	ML	SILT with sand, very loose, very wet. BROWN		
	SP	SAND with sawdust, some silt and trace gravel, predominantly medium grained sand grading to coarse sand at 13.1 feet, sawdust and wood chips present throughout, medium dense, very moist. GRAY		
15				

TD 15.0' Boring terminated at contracted depth.
No refusal or hard conditions encountered.
Groundwater at 2.45' BPG.

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Materials Testing & Consulting, Inc.
Burlington, WA
Geotechnical & Environmental Engineering

Log of Boring GM-5

Former Geddes Marina Property
Marysville, WA

MTC Project No. 14B037-01

Date Started	: 02/02/2015
Date Completed	: 02/02/2015
Sampling Method	: Geoprobe, Continuous
Location	: GM-5
Logged By	: MH

Depth in Feet	USCS	GRAPHIC	DESCRIPTION	Samples	Water Level
0	FB	■	ASPHALT. Crushed aggregate in black oil matrix. BLACK.		
	GP	░	GRAVEL with sand and silt, crushed surfacing base course, firm, moist. GRAY.		▼
		░	CONCRETE SLAB		
	GP	▒	GRAVEL with sand and silt, hard, dry. BROWN		
	SP	░	GRAVELLY SAND with some silt, dense, moist. GRAY		
	SM	▒	SILTY SAND with trace gravel, firm, dry. BROWN		
	FB	■	ASPHALT. BLACK		
	SM	▒	SANDY SILT with trace gravel, firm, moist to very wet at depth. BROWN.		
	GP	▒	GRAVEL with sand and silt, crushed aggregate, loose to medium dense, very wet. BROWN		
5			**NO RECOVERY**		
10	SP-SM	░▒	SAND with gravel and some silt, fine grained becoming medium grained with depth, firm, moist. BROWN		
	ML-OL	░	SILT with organics, some sand, wood fibers and fine grained sand in matrix, firm, moist. BROWN		
	ML	░	SILT with organics, rhythmic wood lenses firm, moist. BROWN GRAY becoming GRAY		
15					

TD 15.0' Boring terminated at contracted depth.
No refusal or hard conditions encountered.
Groundwater at 0.8' BPG.

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Materials Testing & Consulting, Inc.
Burlington, WA
Geotechnical & Environmental Engineering

Log of Boring GM-6

Former Geddes Marina Property
Marysville, WA

MTC Project No. 14B037-01

Date Started	: 02/02/2015
Date Completed	: 02/02/2015
Sampling Method	: Geoprobe, Continuous
Location	: GM-6
Logged By	: Adapted from log notes: Camille - Caldera Archaeology

Depth in Feet	USCS	GRAPHIC	DESCRIPTION	Samples	Water Level
0	ML		GRAVELLY SILT, loamy texture. GRAY-BROWN		
	ML-OL		SILT LOAM with heavy organics, wet. Decaying wood chips in matrix. RED-BROWN		
	ML-OL		ORGANIC SILT with heavy organics. BLACK-GRAY		
	ML-CL		CLAY LOAM. GRAY		
5	ML		SILT with gravel. Angular clasts and wood chips in matrix. Uncontrolled fill. GRAY		
	ML		CLAY LOAM with large natural wood. Interpreted as estuary deposits. GRAY		▼
	ML-OL		SILT with clay and organics. GREY-BROWN		
10	SW		SILT LOAM with heavy organics and some gravel, wet. Lenses of wood chips. GRAY BROWN		
	ML		SILT LOAM with clay and organics. Decaying wood chips and natural wood, strong wood odor. GRAY BROWN. Large cedar log at 14.5 feet.		
15			TD 15.0' Boring terminated at contracted depth. No refusal or hard conditions encountered. Groundwater at 7.95' BPG.		

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Materials Testing & Consulting, Inc.
Burlington, WA
Geotechnical & Environmental Engineering

Log of Boring GM-9

Former Geddes Marina Property
Marysville, WA

MTC Project No. 14B037-01

Date Started : 02/02/2015
Date Completed : 02/02/2015
Sampling Method : Geoprobe, Continuous
Location : GM-9
Logged By : Adapted from log notes: Mike Murray - Maul Foster Alongi

Depth in Feet	USCS	GRAPHIC	DESCRIPTION	Samples	Water Level
0			**NO RECOVERY**		
			NO RECOVERY		
	ML-GM	▨	TOPSOIL: SAND with silt and organic debris. BROWN		
	ML	▨	SILT with fine sand and organics, damp. Organic debris partly decaying. BROWN		
	ML	▨	SANDY SILT with organic debris, moist. GRAY		
5			**NO RECOVERY**		▼
	ML	▨	SANDY SILT with organic debris, moist. GRAY		
10			**NO RECOVERY**		
	SM	▨	SILTY SAND with organic debris, loose, moist. GRAY		
	SM	▨	SILTY SAND, loose, wet. GRAY-BROWN		
		▨	HEAVY ORGANICS. Unweathered wood chips.		
	ML	▨	SILT with sand, firm, damp. GRAY		
15					

TD 15.0' Boring terminated at contracted depth.
No refusal or hard conditions encountered.
Groundwater at 6.02' BPG.

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Materials Testing & Consulting, Inc.
Burlington, WA

Geotechnical & Environmental Engineering

Log of Boring GM-10

Former Geddes Marina Property
Marysville, WA

MTC Project No. 14B037-01

Date Started : 02/03/2015
Date Completed : 02/03/2015
Sampling Method : Geoprobe, Continuous
Location : GM-10
Logged By : MH

Depth in Feet	USCS	GRAPHIC	DESCRIPTION	Samples	Water Level
0	SM		SILTY SAND with gravel and common organics, loose, moist. DARK BROWN **TOPSOIL**		
	GP		GRAVEL with sand and some silt, angular crushed clasts, medium dense, dry. GRAY		
	ML-OL		SILT with some fine grained sand, trace organics, organics predominantly carbonized wood, firm, moist. BROWN		
	ML		SILT with some sand, well packed fine to medium sand grains, firm, moist. BLUE-GRAY		
	ML-OL		SILT with heavy organics, wood debris in silt matrix, some debris carbonized, dense, moist. BLUE-GRAY		
	GP		GRAVEL with sand, some silt and wood debris, sand fraction predominantly medium grained, wood debris partially carbonized, well packed, very wet. DARK BROWN		
5	SM		SILTY SAND with gravel, loose, very wet. Trace organics. DARK BROWN.		▼
			HEAVY ORGANICS, well packed wood chips and sawdust in a silt matrix, firm, very wet. BROWN		
	ML		SILT with fine-grained sand, some organics, wood debris and sawdust, firm to loose, very moist. BROWN		
10			HEAVY ORGANICS with silt, loosely packed wood debris and sawdust, loose, very moist. RED-BROWN		
			HEAVY ORGANICS, very firmly packed wood debris and sawdustfirm, very moist. RED-BROWN		
	SP		SAND, firmly packed, medium grained, orange mottling from 13.2 feet to 13.9 feet, decreases with depth, rhythmically deposited lenses of well sorted medium grained sand, generally fining upwards, firm, very wet. GRAY		
15			TD 15.0' Boring terminated at contracted depth. No refusal or hard conditions encountered. Groundwater at 6.01' BPG.		

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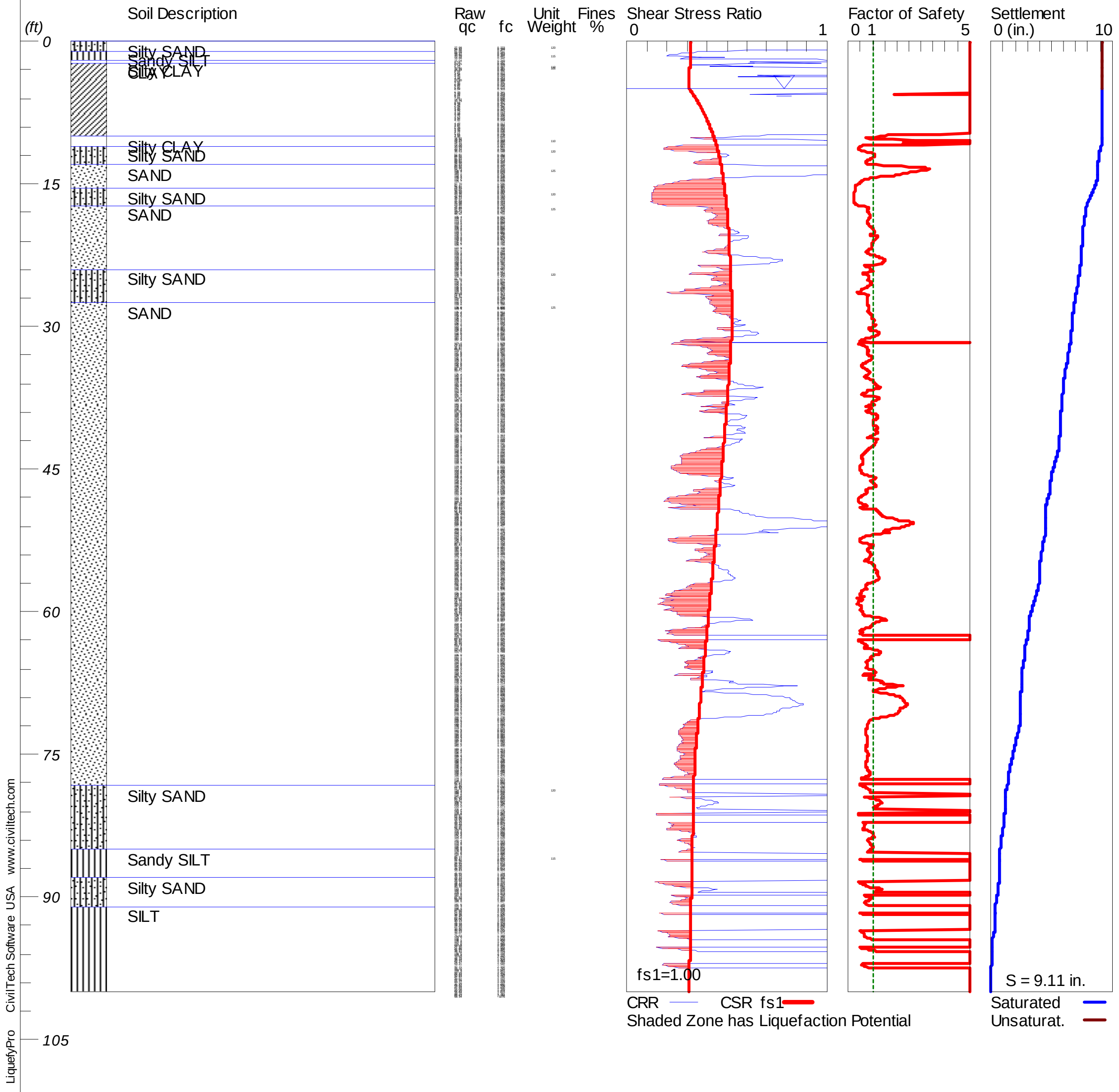
Appendix E. LIQUEFACTION ANALYSIS

LIQUEFACTION ANALYSIS

Geddes Marina

Hole No.=CPT-1 Water Depth=5 ft

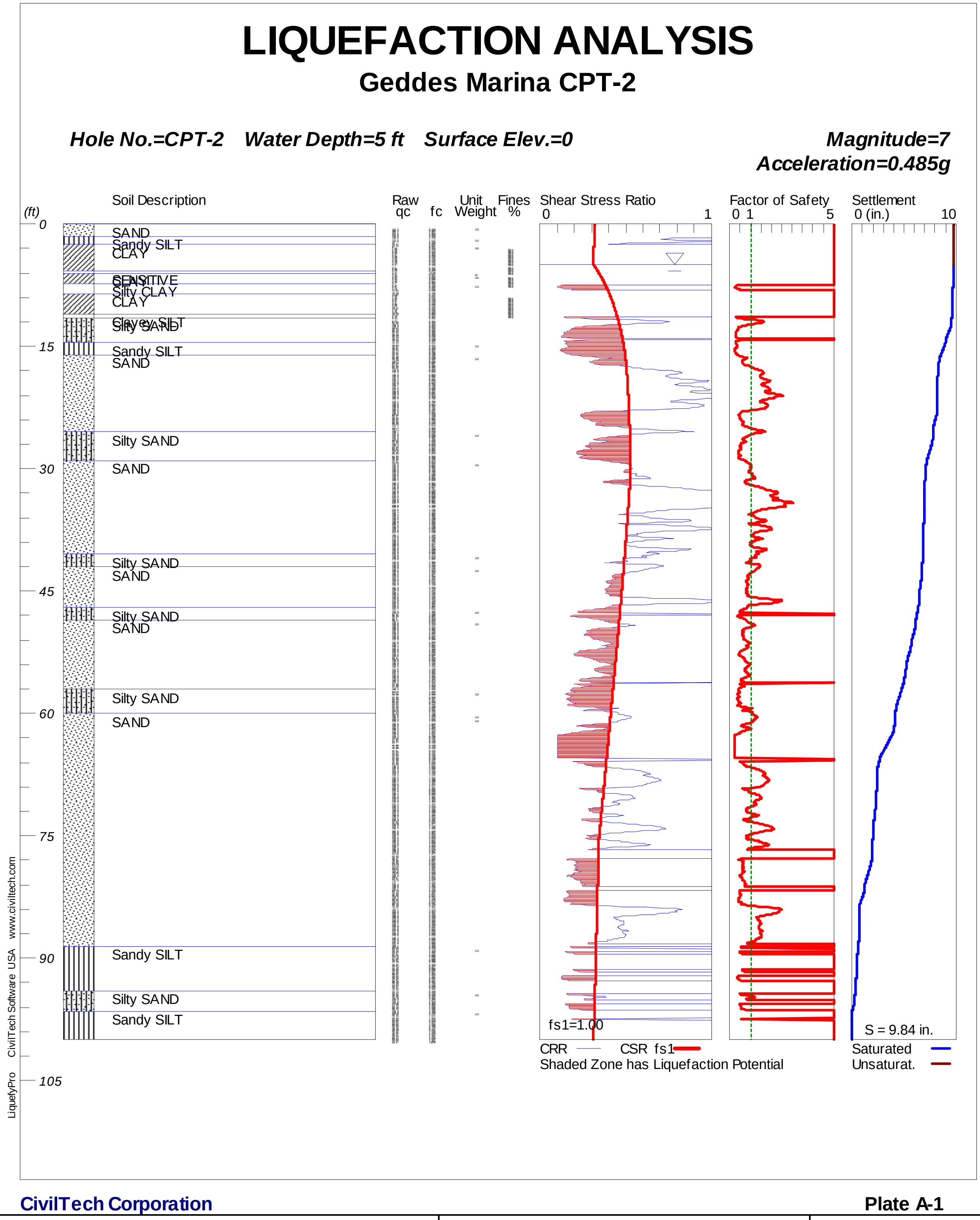
Magnitude=7
Acceleration=.485g



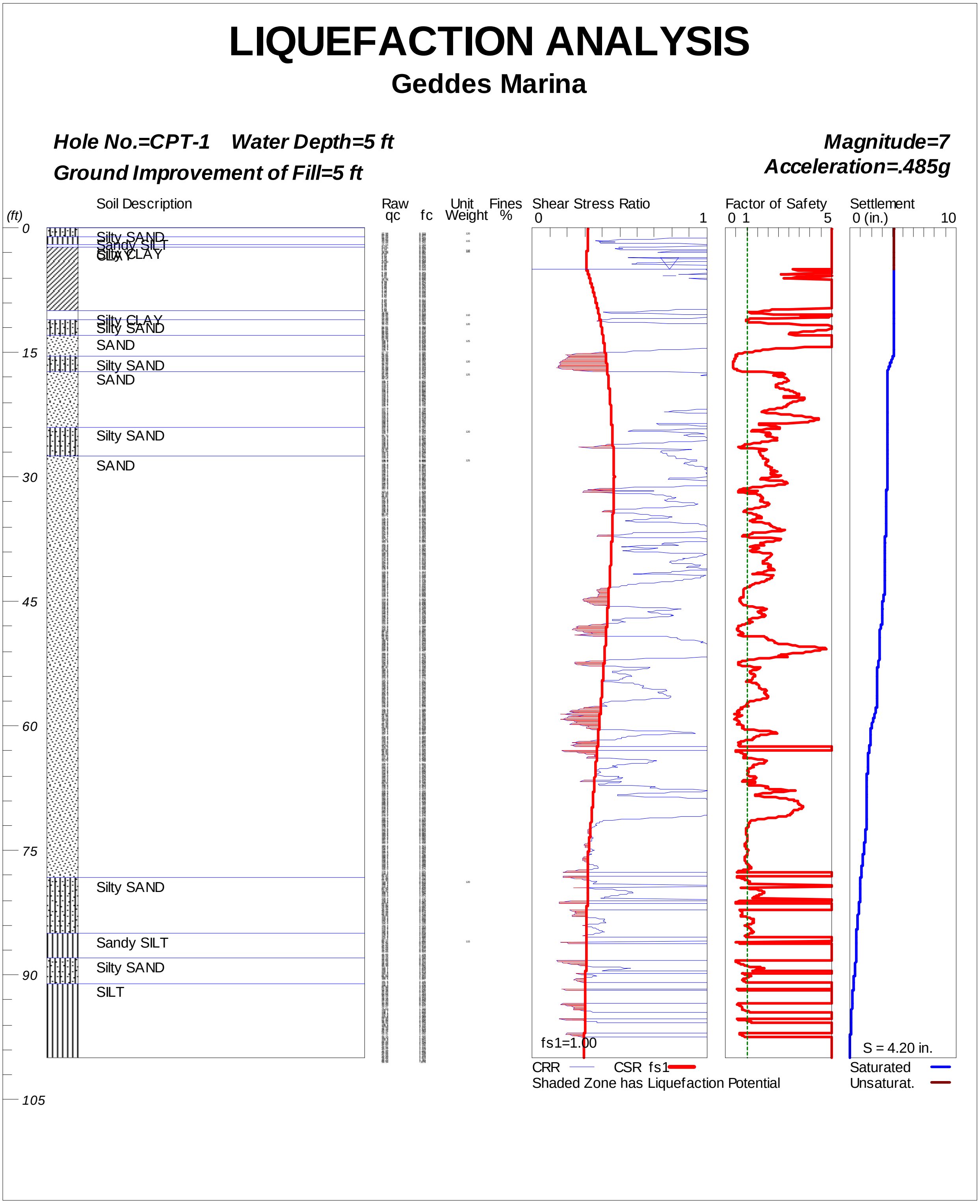
Materials Testing & Consulting, Inc.
777 Chrysler Drive
Burlington, WA 98233

LiquefyPro – CPT-1
Geddes Marina Redevelopment
1326 First Street
Marysville, WA

FIGURE
6



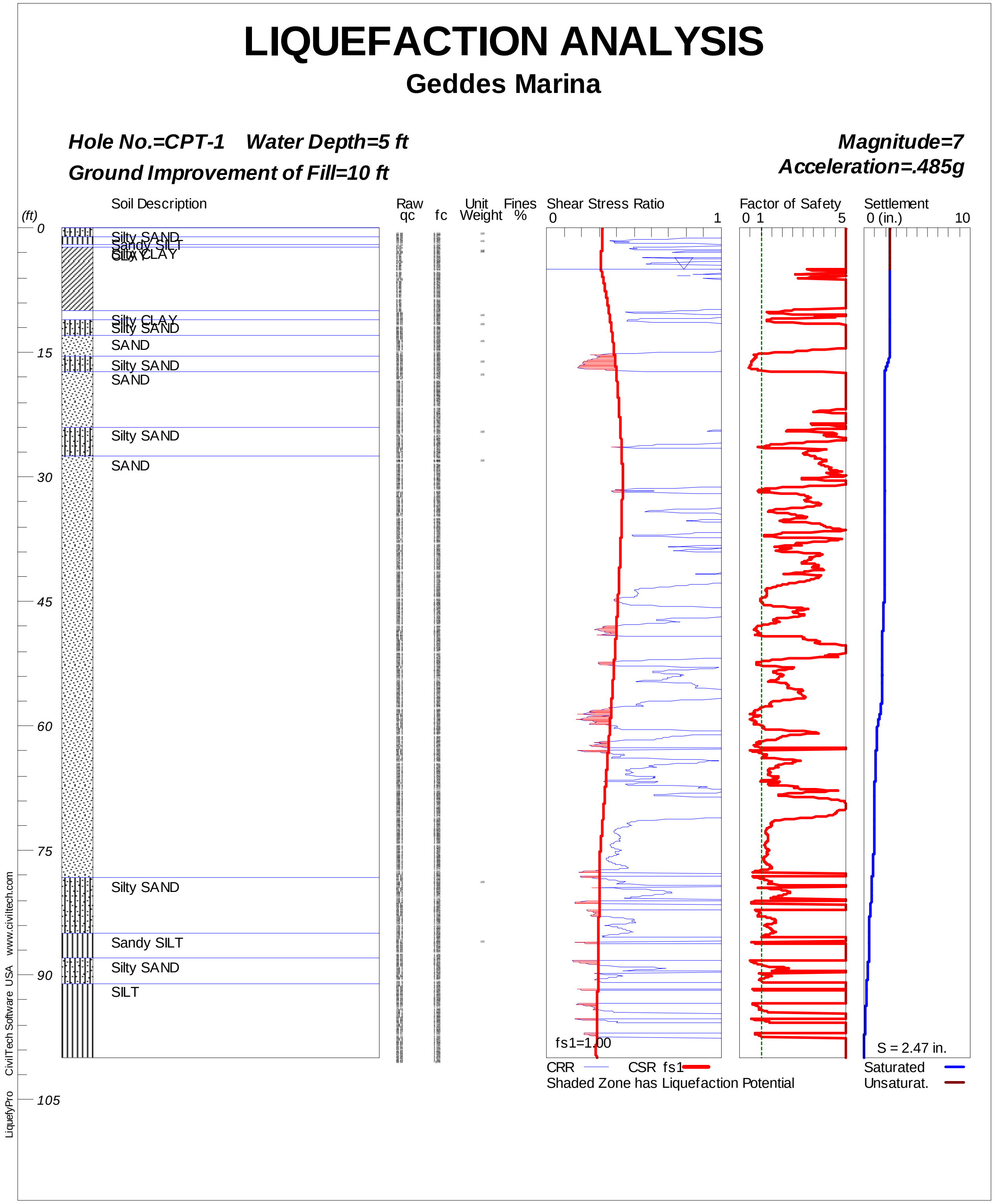
Materials Testing & Consulting, Inc. 777 Chrysler Drive Burlington, WA 98233	**LiquefyPro – CPT-2** Geddes Marina Redevelopment 1326 First Street Marysville, WA	**FIGURE** **7**



CivilTech Corporation

Plate A-1

Materials Testing & Consulting, Inc. 777 Chrysler Drive Burlington, WA 98233	**LiquefyPro – CPT-1, Fill = 5 ft** Geddes Marina Redevelopment 1326 First Street Marysville, WA	**FIGURE** **8**



LIQUEFACTION ANALYSIS

Geddes Marina

Hole No.=CPT-1 Water Depth=5 ft
Ground Improvement of Fill=10 ft

Magnitude=7
Acceleration=.485g

CivilTech Corporation

Plate A-1

Materials Testing & Consulting, Inc. 777 Chrysler Drive Burlington, WA 98233	**LiquefyPro – CPT-1, Fill = 10 ft** Geddes Marina Redevelopment 1326 First Street Marysville, WA	**FIGURE** **9**