GEOTECHNICAL REPORT
TRAILHEAD APARTMENTS
1550 NEWPORT WAY NORTHWEST
ISSAQUAH, WASHINGTON

PROJECT NO. 24-484
June 2025

Credit: Google Earth

Prepared for:

Trailhead Apartments LLLP

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June 24, 2025
File No. 24-484

Trailhead Apartments LLLP
600 Andover Park West
Tukwila, WA 98188
Attn: Nathan Kraus (King County Housing Authority)

Subject: GEOTECHNICAL ENGINEERING REPORT
Trailhead Apartments
1550 Newport Way Northwest, Issaquah, Washington

Dear Nathan:

As requested, PanGEO, Inc. completed a geotechnical report for the proposed Trailhead Apartments in Issaquah, Washington. In summary, the site is underlain by a shallow groundwater table and thick layer of compressible soil that is susceptible to liquefaction. It is our opinion that the site may be developed generally as planned, provided the effects of compressible soils and the risk of liquefaction are properly considered into the design of the building foundation. As currently planned, we understand that the proposed building will be supported on a mat foundation supported on aggregate piers. It is our opinion that this design approach is appropriate.

Because of shallow groundwater the loose/wet soil conditions, the contractor should be prepared to control groundwater and improve existing subgrade to provide a firm working surface.

We appreciate the opportunity to work with you on this project. Please call if there are any questions regarding this report.

Sincerely,

Siew L. Tan, P.E.
Principal Geotechnical Engineer
(STan@pangeoinc.com)

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1.0 INTRODUCTION

This report presents the results of geotechnical studies to support the design and construction of the proposed development at the subject site. Our study was performed in accordance with our mutually agreed scope of work as outlined in our agreements dated November 11 and 19, 2024, and authorized December 11, 2024. Our current scope of work includes reviewing and collecting readily available and published geologic map, subsurface data and geotechnical reports in the vicinity of the site, conducting a site reconnaissance, advancing five test borings, installing five groundwater monitoring wells, performing laboratory testing, performing engineering analyses, and developing the conclusions and recommendations presented in this geotechnical report.

2.0 SITE AND PROJECT DESCRIPTION

The subject property is located at 1550 Newport Way Northwest, in Issaquah, Washington, as shown on the attached Figure 1 – Vicinity Map. The project site (King County Parcel #2924069002) is a rectangular-shaped parcel with an area of about 174,189 square-foot (4 acres). The subject site is bound to the north by Northwest Maple Street, to the south by Newport Way Northwest, to the east by a business park, and to the west by the Sound Transit Issaquah Transit Center.

The site is currently occupied by a Centurylink operations center in the approximately south half of the site, which includes a one-story approximately 33,680 square-foot building of concrete construction with a slab-on-grade floor that we understand was constructed in 1981. Asphalt paved parking and storage areas surround the existing building and a stormwater detention pond is located in the northwestern portion of the site. The remainder of the site consists of asphalt paved parking areas and driveways, along with landscaping planters. Based on our field observations and the review of the site topographic survey prepared by David Evans and Associates dated 09/26/2024, the site grades gently slope down from the south (about elevation 77 feet) to the north at about 70 feet (NAVD88).

We understand that the current development is limited to the north half of the site where it is currently vacant. The south half of the site where the existing building is located will be developed in the future and not part of our current studies.

The proposed site layout is shown in the attached Figure 2 – Site and Exploration Plan. We understand that the proposed development will consist of a new mixed-use workforce housing project. The proposed building will be an at-grade building with five stories of wood-frame construction over a two to three floors of concrete podium. A typical north-south building cross section is shown in Plate 1, below, for reference. The proposed finished floor will be near the existing grade.

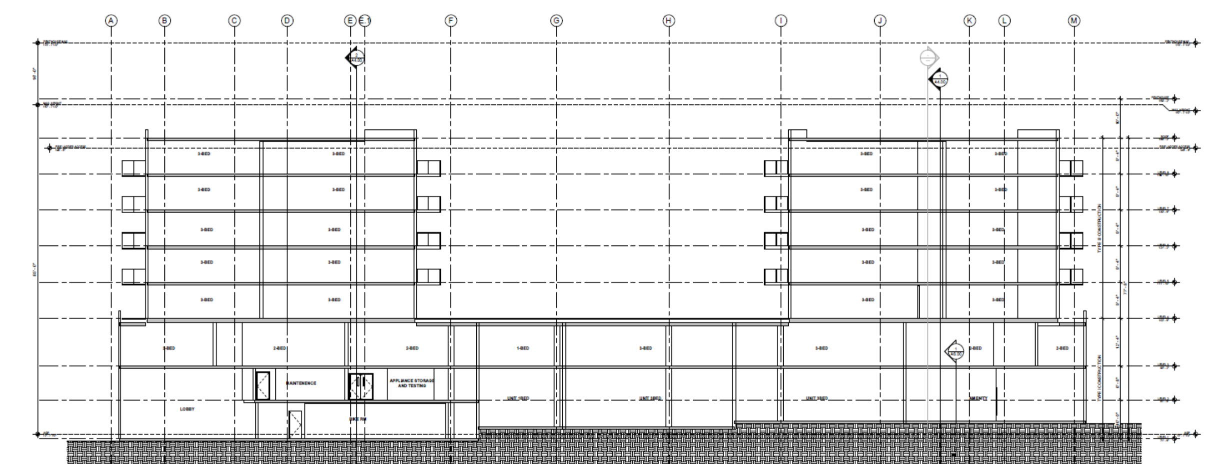

Plate 1. Typical north-south building section (source: Weber Thompson)

The conclusions and recommendations in this report are based on our understanding of the proposed development, which is in turn based on the project information provided. If the above project description is incorrect, or the project information changes, we should be consulted to review the recommendations contained in this study and make modifications, if needed. In any case PanGEO should be retained to provide a review of the final design to confirm that our geotechnical recommendations have been correctly interpreted and adequately implemented in the construction documents.

3.0 SUBSURFACE EXPLORATIONS

3.1 TEST BORINGS (CURRENT STUDY)

Five test borings (PG-1 through PG-5) were drilled at the project site on January 15, 2025. The approximate boring locations are indicated on the attached Figure 2 – *Site and Exploration Plan*. The principal objective of these test borings was to evaluate the groundwater levels at the site.

The borings were drilled to about 16½ feet below the existing grades using an Acker Recon track drill rig owned and operated by Geologic Drill Partner Inc. The drill rig was equipped with 8-inch outside diameter hollow stem augers, and soil samples were obtained from the borings at 2½- and 5-foot depth intervals in general accordance with Standard Penetration Test (SPT) sampling methods (ASTM test method D-1586) in which the samples are obtained using a 2-inch outside diameter split-spoon sampler. The sampler was driven into the soil a distance of 18 inches using a 140-pound automatic hammer falling a distance of 30 inches. The number of blows required for each 6-inch increment of sampler penetration was recorded. The number of blows required to achieve the last 12 inches of sample penetration is defined as the SPT N-value. The N-value provides an empirical measure of the relative density of cohesionless soil, or the relative consistency of fine-grained soils.

Following the completion of drilling, a nominal 2-inch diameter PVC standpipe with a 10-foot screen interval was installed for each of the five borings from a depth of 5 feet to 15 feet measured from the existing ground surface as part of monitoring well construction.

PanGEO personnel were present throughout the field exploration program to observe the drilling, assist in sampling, and to document the soil samples obtained from the borings. The completed boreholes were backfilled with bentonite chips and cold-patched with asphalt at the ground surface. The soil samples retrieved from the borings were described using the system outlined on Figure A-1 of Appendix A, and the summary boring logs are included as Figures A-2 through A-6.

3.2 PREVIOUS CPTS AND TESTING BORINGS

As part of our study, we collected and reviewed readily available subsurface data and summary logs of previous subsurface investigations in vicinity of the project site. The approximate locations of these previous explorations are also shown in Figure 2. Specifically, the following previous subsurface data was reviewed:

- PanGEO, Inc. (2018) previously completed four cone penetration tests (CPT-1 through CPT-4) at the project site. The tests were performed by In-Situ Engineering of Snohomish, Washington on January 2, 2018. The CPT consisted of pushing an approximately one-inch diameter piezometer-equipped cone into a soil deposit from a truck mounted reaction frame and measuring the resistance and pore water pressure on the top and side of the cone. The CPTs were advanced approximately 61 to 72 feet below grade before encountering practical refusal in a dense gravelly sand deposit. Higher tip resistance measurements indicate the soil deposit has a higher strength or density than lower tip resistance measurements. The resistances to continuous penetration encountered by the cone tip and adjacent friction sleeve also exhibit high sensitivity to changes in soil type, which may be correlated to differing soil types and strength parameters. The principal advantages of using a CPT are minimum site disturbance and continuous profiling of the underlying soil.

- Zipper Zeman Associates, Inc. (ZZA, June 2004; and May 2005) previously completed six test borings (B-4, B-6, B-8; B-13, B-14, and B-15) for the Issaquah Transit Center located immediately west next to the current project site. The borings were advanced between 21½ and 74 feet below grade.

- Rittenhouse-Zeman & Associates, Inc. (RZA, 1980) previously completed two test borings (B-1 and B-2) for the construction of a one-story, concrete, tilt-up structure on the project site (i.e., the current CenturyLink operations center). The borings were advanced with a truck-mounted, hollow-stem power auger. Standard penetration test (SPT) samplers were used to collect the samples up to 52½ and 14 feet deep below the grade for B-1 and B-2, respectively.

The summary logs of previous the above-mentioned explorations are included in **Appendix B** of this report.

3.3 Laboratory Testing

For the current study, selected soil samples were tested in general accordance with test methods of the American Society for Testing and Materials (ASTM). The tests include samples for moisture content, P-200 passing and grain size determination.

Moisture Content Testing – Moisture content tests were performed in general accordance with ASTM D2216.

P-200 Passing Testing – The determination of the amount of material by mass finer than 75-μm (No. 200) sieve in soils was performed in general accordance with ASTM D1140.

Grain Size Analyses – Grain size distribution analyses were performed on representative samples obtained from the test borings. The grain size distribution tests were performed in general accordance with the procedure outlined in ASTM D6913.

The test results are included on the appropriate summary boring logs in Appendix C.

3.4 Geophysical Survey

Global Geophysics of Redmond, Washington, under a subcontract to PanGEO, conducted one local and regional Microtremor Array Measurements (MAM, S-1 and S-2), one Multichannel Analysis of Surface Waves (MASW), and one Horizontal over Vertical Spectral Ratio (HVSR) to determine a shear wave velocity profile that extended to various depths. The approximate Geophysical survey locations are indicated on the attached Figure 2 – *Site and Exploration Plan*. The results are included in Appendix D of this report. The principal objective of this geophysical survey is to provide a basis to support the site-specific ground response analysis.

4.0 SUBSURFACE CONDITIONS

4.1 Site Geology

Based on review of the *Geologic Map of the Issaquah 7.5-minute quadrangle, Washington, U.S. Geological Survey* (Booth et al., 2012), the project site is underlain by Holocene Alluvium (map unit Qal). Alluvium typically consists of interbedded, loose sand and silty sand, and soft to medium stiff silt and clay with occasional thin peat seams and some organics. Locally, alluvium includes sediments of similar texture and age found in low-lying areas adjacent to Lake Sammamish, particularly where the beach and shallow lacustrine deposits presents. Alluvium can also consist of cobble gravel and pebbly sand. The results of the subsurface exploration program confirmed the mapped geology and encountered alluvial soils under the surficial fill.

4.2 Soil Conditions

Based on the results from our test borings and our review of the previous subsurface explorations advanced at the project site, the site is underlain by a sequence of recent fill over the mapped alluvium. The following is a summary of the subsurface conditions encountered in the explorations.

Unit 1: Pavement/Fill – Below an approximately 4-inch-thick layer of asphalt, the test borings encountered about 3 to 7 feet of fill, except for the test boring PG-2 at the middle of the site that encountered about 12 feet of fill. The fill generally consisted of loose to medium dense silty sand with variable amount of gravel and scattered organics. Boring PG-2 encountered a very loose pea gravel layer from 7 to 12 feet below the ground surface and drilling on a thin obstruction layer was observed at the bottom of the pea gravel layer. Fill was encountered in recent CPT locations (PanGEO, 2018), as well as the locations of the previous explorations on the site.

Unit 2: Alluvium/Lacustrine Deposits – Below the fill, a very loose/soft to medium dense/stiff silty sand, sandy silt, clay-silt, and silty clay layer was encounter in all the current test borings, and extended to the depth between 35 and 48 feet below grade at the CPT locations (PanGEO, 2018). Due to the generally fine-grained nature of this soil unit, we interpret it as a lacustrine (i.e., lake) or an alluvium deposit from a low-energy stream. This soil unit is generally consistent with the mapped geology of the area compiled by Booth et al., (2012). The previous ZZA test borings ZZA B-13, B-14, and B-15 and RZA test boring B-2 were terminated in this soil unit.

Unit 3: Old Alluvium – Underlying the Alluvium/Lacustrine soil unit, medium dense to dense relative clean sand to silty sand with a varying gravel content was encountered to the maximum explored depth at all the CPT locations, as well as ZZA test borings B-4, B-6, and B-8 and RZA test boring B-1. This soil unit contained occasionally stiff to very stiff silt lenses.

Based on previous CPT explorations onsite (PanGEO 2018), soil encountered from around 4 to 16½ feet was mapped as clay and silty clay with some silty sand, which was observed in each of the four CPT tests. However, all our recent five test borings samples retrieved from this depth range indicated mainly silty sand and sandy silt with occasional silt and clay interbeds. Therefore, the interpretation of the soil properties at the project site at greater depths (e.g., deeper than 16½ feet) from the CPT tests will be adjusted based on the difference between the CPT and SPT interpretations at shallower depths. For example, soils at greater depth that are mapped as clay on CPT summary log may be interpreted as sandy silt, if the same soil mapping was observed in CPT as clay but interpreted as sandy silt in our test borings as shallower depth.

In general, our subsurface descriptions are based on the conditions encountered at the time of exploration. Soil conditions between our exploration locations may vary from those encountered. The nature and extent of variations between our exploratory locations may not become evident until construction. If variations do appear, PanGEO should be requested to reevaluate the recommendations in this report and to modify or verify them in writing prior to proceeding with earthwork and construction.

4.3 Groundwater Conditions

Groundwater was encountered in each of the test borings (PanGEO, 2025) that we recently advanced at the site. As discussed previously, groundwater monitoring wells were installed in each of the five borings and initial readings were taken on January 16, 2025. Three of the monitoring wells, PG-1 to PG-3, are located generally around the perimeter of the proposed Trailhead Apartments, and the other two monitoring wells, PG-4 and PG-5, are located south of the exiting warehouse building. As indicated in Table 1, at the time of the water levels were measured on January 16, 2025, the groundwater levels were the highest along the south side of the site (i.e., along Newport Way NW) and gradually dropping to the north (i.e., along NW Maple Street), with an approximately 5 feet of change in groundwater level elevation.

Data loggers were installed on January 16, 2025 in all five piezometers to monitor the fluctuation of groundwater. The data loggers were retrieved on February 28, 2025 and the results were summarized and plotted as shown in Figures 3A and 3E, along with the rainfall precipitations during that period. The data indicates that the groundwater levels at the site are highly sensitive to the amount of rainfall.

Table 1. Highest Measured Groundwater Levels between 1/16/2025 and 2/28/2025

Boring No.	Approx. Ground Surface El. (NAVD88) (ft)	Approx. Highest Measured Groundwater Elevation (ft) during Monitoring Period
PG-1	72.3	69.4
PG-2	75.1	70.6
PG-3	72.7	68.4
PG-4	77.1	74.1
PG-5	76.1	74.3

For design purposes, based on the results for PG-1 to PG-3, it is our opinion that it is reasonable to assume a groundwater level of Elevation 70 feet at the sound end of the proposed Trailhead building and Elevation 72 feet at the north end of the building. PG-4 and PG-5 are located further away from the proposed building.

It should be noted that groundwater elevations may vary depending on the season, local subsurface conditions, and other factors. Groundwater levels are normally highest during the winter and early spring (typically October through May).

5.0 SEISMIC CONSIDERATIONS

5.1 SEISMIC CLASS AND RESPONSE SPECTRA

We anticipate that the seismic design of the building will be accomplished based on the 2021 edition of International Building Code (IBC) and ASCE 7-16, which specifies a design earthquake having a 2% probability of occurrence in 50 years (return interval of 2,475 years). The IBC seismic design parameters are in part based on the site soil conditions and site classifications defined in Chapter 20 of ASCE 7-16. According to Chapter 20 of ASCE 7-16, the site soil should be classified as Site Class F because of its liquefaction potential (see discussions in Section 5.2 of this report). Section 20.3.1 of ASCE 7-16 indicates that for Site Class F a site-specific ground response analysis in accordance with Section 21.1 shall be performed unless the exception to Section 20.3.1 is applicable.

Section 20.3.1 of ASCE 7-16 states that "*For structures having fundamental periods of vibration equal to or less than 0.5s, site response analysis is not required to determine spectral accelerations* *for liquefiable soils. Rather, a site class is permitted to be determined in accordance with Section 20.3 and the corresponding values of F_a and F_v determined from Tables 11.4-1 and 11.4-2*." In other words, for structures with a period of vibration equal to or less than 0.5 second and situated on liquefiable soils, the ASCE-7 exception allows the values of F_a and F_v for liquefiable soils be taken equal to the values of site class determined without regard to soil liquefaction.

Based on input from the structural engineer, we understand that the vibration building for the proposed building is 0.6 seconds, thus the aforementioned exception does not apply. The site-specific ground response analysis was performed by Atlas Geotechnical, Inc. The summary report of the analysis is included in Appendix E of this report.

5.2 SOIL LIQUEFACTION EVALUATION

Liquefaction occurs when saturated predominately sand and silt soils are subjected to cyclic loading. This causes the porewater pressure to increase in the soil, thereby reducing the inter-granular stresses. As the inter-granular stresses are reduced, the shearing resistance of the soil decreases. If pore pressures develop to the point where the effective stresses acting between the grains become zero, the soil particles will be in suspension and behave like a viscous fluid. The liquefaction potential of saturated sands is evaluated mainly on soil gradation, relative density, and the depth of deposit. Typically, loose, saturated sand and silt that have a low enough permeability to prevent drainage during cyclic loading have the greatest potential for liquefaction, while more dense soil deposits with higher silt or clay contents have a lesser potential. The effect of liquefaction can range from reduced shear strength to viscous fluid behavior which may cause the temporary loss/reduction of foundation capacity and settlement.

5.2.1 Liquefaction Analysis Procedure

We performed liquefaction analyses to evaluate the liquefaction potential of the site soils based on the results of our recent test borings and our previous CPT explorations (PanGEO, 2018) at the project site. The analyses were conducted using the computer liquefaction assessment software programs: 1) *LiqSVs 2.0.2.1* based on both the test borings and CPT explorations, and 2) *Cliq* based on the CPT data only. The two liquefaction assessment programs adopted the same assessment method proposed by Boulanger & Idriss (2014), the same vertical settlement evaluation approach utilizing depth correction per Cetin et al. (2009), and input ground parameters. The vertical settlement is estimated by using Cetin et al. (2019), which is a probabilistically based model for the assessment of cyclically induced straining of saturated cohesionless soils. This approach includes a depth correction factor that assumes contribution of layers to surface settlement diminishes as the depth of layer increases, and the settlement of an individual layer that is up to about 18 meters (about 60 feet) deep below the ground surface will manifest at the ground surface. The input ground motion parameters in our analyses include a Magnitude (M_w) of 7.5 earthquake and a site modified Peak Ground Acceleration of 0.74*g*, which was provided by Atlas Geotechnical, Inc (2025) from the site-specific response analysis (see Appendix E).

5.2.2 Liquefaction Analysis Results

Ground settlement should be expected to occur in the event of soil liquefaction. The results of the liquefaction analysis and induced vertical settlements of the two liquefaction assessments are presented as follows:

CPT based *Cliq* program: Partial liquefaction was observed at the uppermost 40 to 50 feet of the alluvium. The calculated liquefaction-induced settlement of free-field settlements as the ground surface ranged from around 2 to 4 inches among the four CPTs completed at the site. The estimated free-field differential settlement is about 1½ to 2 inches across the site, indicating relatively uniform settlement pattern. The results of the liquefaction analysis (i.e., factors of safety against liquefaction versus depths) and the calculated settlement with depth are included in Appendix F.

SPT and CPT based *LiqSVs* program: Extensive liquefaction was observed at the uppermost 40 to 50 feet of the alluvium. The calculated liquefaction-induced settlement of free-field settlements as the ground surface ranged from around 6½ to 8½ inches among the four CPTs completed at the site. The estimated free-field differential settlement is about 1½ to 2 inches across the site, indicating relatively uniform settlement pattern. The results of the liquefaction analysis (i.e., factors of safety against liquefaction versus depths) and the calculated settlement with depth are included in Appendix G.

Based on our experience of the local geology and the review of subsurface explorations nearby, in our opinion the SPT and CPT combined approach (i.e., *LiqSVs* program), which provides the mutual-verified soil profile along the depth, is more reliable to present the actual soil conditions underground, and is proposed as a more conservative and a more appropriate approach for the liquefaction evaluation and the corresponding vertical settlement calculations at the project site.

6.0 GEOTECHNICAL RECOMMENDATIONS

6.1 Foundation Support Options

The alluvial soils beneath the site are subject to compression and settlement upon an increase in overburden stress under static conditions. In addition, as discussed above, the soils below the site are prone to liquefaction during the IBC-level earthquake. The estimated total liquefaction-induced settlement is on the order of 6½ to 8½ inches, and the differential settlement is about 2 inches across the site. Due to the shallow groundwater table and the presence of very loose to loose granular alluvium material at shallow depths, the occurrence of soil liquefaction could also result in partial loss of bearing capacities and significant settlement.

Options to mitigate the effects of soil liquefaction typically consist of a deep foundation system such as piles extending through the liquefiable soils, or implementation of ground improvements to mitigate the risk of soil liquefaction to allow the use of shallow foundations. A deep foundation system typically provides the best performance but is also the costliest. For this project, after discussions with the project team and contractor, we understand that the use of aggregate piers and a mat foundation will be used to support the proposed building. It is our opinion that this is an appropriate option.

6.2 Aggregate Piers

We anticipate that aggerate piers could be a feasible and cost-effective ground improvement approach for this site. Aggregate piers consist of compacting columns of well-graded crushed rock to increase the bearing capacity of poor soils, to mitigate liquefaction potential within the improved zone and reduce settlement. Because the aggregate piers increase the stiffness of the subsurface soils and provide additional drainage pathways for excess pore pressure during a seismic event; thus, the potential for earthquake induced liquefaction in the improved soils is reduced.

The actual depth of improvements and their diameter and spacing should be determined by the aggregate pier contractor/designer, based on the settlement criteria provided by the structural engineer.

Because specialty contractors install aggregate piers using a proprietary system, the contractor determines the lengths/depths and spacing of piers, the allowable soil bearing pressure of the improved soil, improved soil characteristics and anticipated settlements. Discussions with the ground improvement designer/contractor should be made so that the ground improvement design provides the desired level of performance. The aggregate pier contractor will base their design on the settlement criteria provided by the project owner and the project structural engineer.

After the aggregate piers are installed, a mat foundation or structural slab with thickened edges is constructed directly on the improved soils.

6.3 Mat and Structural Slab Foundation

A mat foundation or a structural slab should be designed so that it is sufficiently stiff to spread the concentrated column loads. The foundation should be constructed over improved subgrade soils as discussed in Section 6.3.1, below. A mat/slab foundation will reduce the effects of potential differential settlement of adjacent columns and will perform better than isolated column footings during an earthquake, especially in the event that the underlying soils liquefy.

6.3.1 Foundation Subgrade Preparation

The mat or structural slab foundation should be supported on at least 2 feet of structural fill. Depending on the finished floor elevation, over-excavation below the foundation level may be needed in order to place the 2 feet of structural fill. The structural fill to be placed below the building foundation should consist of 2 inch minus crushed rock, or approved equivalent.

The soils exposed at the bottom of the foundation excavation should be compacted to a firm and unyielding condition prior to placement of geogrid and structural fill. Any soft/loose and pumping subgrade soil detected during compaction should be removed and replaced with structural fill.

6.3.2 Lateral Resistance

Lateral loads from un-balanced soil loads, wind or seismic loading may be resisted by a combination of passive earth pressures acting against the embedded portions of the foundations and walls, and by friction acting on the base of the foundations.

- For foundations bearing on compacted structural fill, a frictional coefficient of 0.45 may be used to evaluate sliding resistance. However, if waterproofing measures are installed directly below the concrete slab, the frictional resistance will need to be reduced or not relied upon for base friction.

- Passive soil resistance may be calculated using an equivalent fluid unit weight of 350 pcf, assuming properly re-compacted native sandy soil or compacted structural fill will be placed against the footings, the footings are located above groundwater table (groundwater at 4 feet below existing grade), and level ground surface. In addition, unless it is covered by pavements or slabs, the passive resistance in the upper 12 inches of soil should be neglected. Below the water table, the passive resistance should be reduced to 175 pcf.

The above values include a factor of safety of approximately 1.5.

6.3.3 Estimated Settlement

The ground improvements should be designed to meet the settlement criteria determined by the project team.

6.3.4 Buoyancy

Portions of the buildings such as elevator pits may be positioned below the groundwater table. Building elements extending below the groundwater table should be designed to resist the hydrostatic uplift pressure and the bending stress from the uplift pressure. The weight of the structure and friction along the sides of the structure will resist uplift forces. In needed, based slab of the below-grade structures may be extended outside its wall to increase its uplift resistance.

For design purposes, based on the results for PG-1 to PG-3, it is our opinion that it is reasonable to assume a groundwater level of Elevation 70 feet at the sound end of the proposed Trailhead building and Elevation 72 feet at the north end of the building.

6.4 RETAINING WALLS

We understand the proposed building will be built at grade level; however, we suppose there are structures that would be built below grade level, e.g., elevator pits or dentation vaults, etc. We expect retaining walls should be designed to assist the underground structures construction by resisting the lateral earth pressures exerted by the soils behind the walls. Proper drainage provisions should be provided behind the walls to intercept and remove groundwater that may be present behind the walls unless the walls are designed for potential hydrostatic conditions as discussed in Section 7.3, above.

Our geotechnical recommendations for the design and construction of the new retaining walls are presented below.

6.4.1 Wall Foundation

For foundation walls supported on aggregate piers, the recommended parameters outlined in Section 6.3.2 of this report remain applicable for retaining wall design and construction.

For short site retaining walls, if needed, wall footings should be supported on at least two feet of granular structural fill such as crushed rock. An allowable bearing pressure of 2,000 psf may be used to size site retaining wall footings.

6.4.2 Lateral Earth Pressures

For walls that will be braced, such as basement walls, we recommend a lateral earth pressure of 50 pcf for design. Cantilevered site retaining walls with level backslopes should be designed for a static active earth pressure based upon an equivalent fluid weight of 35 pcf.

The design values assume drainage provisions will be provided behind the walls. In addition, we also recommend a seismic surcharge of 9H psf be included for design (where H is the height of the below grade portion of the wall). The recommended lateral pressures assume that the backfill behind the wall consists of a free draining and properly compacted fill with adequate drainage provisions to prevent the development of hydrostatic pressure. PanGEO is available to provide additional recommendations if needed.

Buried Structures (Elevator Pits, Detention Vaults, etc.) - There is potential for groundwater to accumulate next to buried structures such as elevator pits and detention vaults. If it is not feasible to incorporate footing drains for elevator pits, detention vaults, etc., we recommend that an equivalent fluid weight of 90 pcf be applied for wall design. The recommended 90 pcf includes both the soil pressure and the effects of hydrostatic pressure. Buoyancy force should also be considered in the design of these structures.

6.4.3 Surcharge

Retaining walls should be designed to accommodate surcharges from nearby structures and traffic. Where traffics will be located within a horizontal distance equal or less than the wall weight, a uniform lateral pressure of 80 psf should be applied to the walls. If the traffic will be limited to light weight passenger vehicles, a lower uniform lateral pressure of 25 psf is considered adequate. Other surcharge loads located within a horizontal distance equal or less than the wall weight, the lateral pressure on the wall can be calculated as 35% of the vertical surcharge loads.

In addition, if the proposed basement wall footing will extend below the 1H:1V (horizontal:vertical) downward projection line from a new or existing footing or loaded slab, the proposed basement walls should be designed to support the surcharge pressures from the adjacent footings and slabs.

6.4.5 Wall Drainage

Provisions for permanent control of subsurface water should be incorporated into the design and construction of walls. Prefabricated composite drainage mats, such as Mirafi 6000 or equivalent, may be installed behind the walls and the collected water should be directed through weep pipes at the base of the walls, spaced about 8 feet on center, to a 4-inch diameter perforated collector pipe located along the interior of the perimeter footings and discharged to an appropriate outlet. The 4-inch diameter perforated drainpipe should be embedded in 12 to 18 inches of clean crushed rock or pea gravel wrapped with a layer of filter fabric. The composite drainage material should be installed per the manufacturer's recommendations.

Alternatively, a minimum 18-inch-wide zone of open-graded free-draining granular soils (i.e., pea gravel or washed rock) is recommended to be placed adjacent to the wall for the full height of the wall, in lieu of the composite drainage material mentioned above.

PanGEO will provide additional project-specific recommendations when design details become available.

6.4.6 Wall Backfill

Given the relatively high fines content of the alluvium soils anticipated in the site excavation, we do not recommend using the on-site soils for wall backfill. Where wall backfill will be needed, the backfill should consist of free draining granular soils such as City of Seattle (COS) Mineral Aggregate Type 17, Gravel Borrow (Section 9.03.14 (1) of the 2025 WSDOT Standard Specifications) or an approved equivalent. In areas where the space is limited between the wall the face of excavation, pea gravel may be used as backfill without compaction.

In structural areas, wall backfill should be moisture conditioned to near its optimum moisture content, placed in loose, horizontal lifts less than 8 inches in thickness, and systematically compacted to a dense and relatively unyielding condition. If density tests will be performed, the test results should demonstrate at least 95 percent of the maximum dry density, as determined using test method ASTM D 1557. In landscaping areas and within 5 feet of the wall, the backfill should be compacted to at least 90 percent of its laboratory determined maximum dry density.

6.4.7 Damp Proofing

The exterior of all foundation walls should be protected with a damp proofing compound. Recommendations for damp proofing are beyond our area of expertise. A building envelope specialist or product vendors may be consulted for specific recommendations regarding this matter.

6.5 FLOOR SLABS

As discussed above, due to the soil liquefaction potential, and the needs to mitigate the risk of excessive differential settlements between columns, we recommended the proposed building be supported on a concrete mat foundation on improved ground. As such, the top of the mat foundation or the structural slab foundation can serve as the finished floor.

In spaces where moisture may be sensitive, the slabs should be constructed on a minimum 4-inch-thick capillary break. Capillary break is optional in areas that are not moisture sensitive, such as parking stalls and drive aisles. Where needed, the capillary break should consist of open-graded, free-draining, crushed rock compacted to a firm and unyielding condition. The capillary break material should have no more than 10 percent passing the No. 4 sieve and less than 5 percent by weight of the material passing the U.S. Standard No. 100 sieve.

Due to the proximity of groundwater to the finished floor, it may be prudent to consult a building envelope consultant regarding the waterproofing of the building.

6.6 PAVEMENT

New asphalt pavement will be constructed as part of the proposed building. Because the site soils are prone to settlement, we recommend that the fill soil in the pavement areas be placed as early in the project as possible and allowed to settle prior to final grading and pavement construction.

Assuming the pavement will generally be used by light passenger vehicles, with only occasional heavy truck, bus, or garbage truck use, as a minimum, we recommend that the new pavement section consists of 3-inch hot mix asphalt (HMA), overlying a 6-inch-thick layer of crushed surfacing base course (CSBC), overlying a minimum of 12 inches of properly compacted granular structural fill. Both the structural fill and crushed rock base should be compacted to a minimum of 95% of the material's maximum dry density (Modified Proctor ASTM D-1557).

It should be noted that actual pavement performance will depend on a number of factors, including the actual traffic loading conditions. The recommended pavement section will need to be revised if the traffic level will be more or less than our assumed value.

6.7 Underground Utilities

6.7.1 Pipe Support and Bedding

We anticipate the exposure of variable, but generally adequate fill subsoil conditions at pipe invert elevations less than about 3 to 4 feet below the existing ground surface. Below about 4 feet, very loose to loose silty sand and sandy silt with silt and clay interbeds, were encountered. In our opinion, the fill material, consisting of medium dense silty sand with gravel, should provide suitable support for the proposed pipelines; however, for utilities deeper than about 4 feet, if very loose to loose/soft silty sand, sandy silt, silt, clay, or organic-rich soil is exposed along the bottom of any trench, we recommend about 6 to 12 inches of the soft soils be removed and replaced with additional bedding material.

In general, pipe bedding materials should be placed in loose lifts not exceeding 6 inches in thickness, and compacted to a minimum relative compaction of 95 percent maximum dry density, per ASTM D1557. Bedding materials and thickness provided should be suitable for the utility system and materials installed, and in accordance with any applicable manufacturer's recommendations. Pipe bedding materials should be placed on relatively undisturbed native soils, or compacted structural fill soils. If the native subgrade soils are disturbed, the disturbed material should be removed and replaced with compacted structural fill or bedding material.

6.7.2 Trench Backfill

Beneath structural or paved areas, we recommend that trench backfill will be selected granular material, meeting the requirements for structural fill. During placement of the initial lifts, the trench backfill should not be bulldozed into the trench or dropped directly on the pipe. Furthermore, heavy vibratory equipment should not be permitted to operate directly over the pipe until a minimum of 3 feet of backfill has been placed.

In order to minimize subsequent settlement of the trench backfill, it is recommended that the trench backfill be placed in 8- to 12-inch, loose lifts and compacted using mechanical equipment to about 90 percent maximum dry density, as determined by Standard Proctor, per ASTM D698. In structural or paved areas, the upper 2 feet of the backfill should be compacted to at least 95 percent maximum dry density, per ASTM D1557.

It is anticipated that selected excavation spoils may be used as trench backfill if they are placed at or near the optimum moisture content and proper compaction control is utilized. In our opinion, the top approximately 3 to 4 feet of soil at the site (sand and silty sand with gravel) may be potentially re-used as trench backfill. However, some of the soils may be too wet to achieve the recommended compaction requirements. If the material is not compacted as recommended, the potential for backfill settlement will be increased. Below a depth of about 3 to 4 feet, the sandy silt with silt and clay interbeds will not be suitable for re-use as trench backfill.

Underground utilities should be designed to accommodate differential and total settlements on the order of several inches over the design life of the project.

6.8 PERMANENT CUT AND FILL SLOPES

We recommend that the permanent cut and fill slopes be constructed no steeper than 2H:1V (horizontal:vertical). For fill slopes constructed at the angles recommended above, and the comprised of soils placed and compacted as recommended in this report, we anticipate that adequate factors of safety against global failure will be maintained.

Measures should be taken to prevent surficial instability and/or prevent erosion on slopes. For a permanent fill slope, this can be accomplished by conscientious compaction of the fills all the way out to the slope face, by maintaining adequate drainage, and planting the slope face as soon as possible after construction. To achieve the specified relative compaction at the slope face, it may be necessary to overbuild the slopes several feet, and then trim back to design finish grade. In our experience, compaction of slope faces by "track-walking" is generally not as effective.

6.9 Infiltration Considerations

Based on our understanding of the site soil conditions, it is our opinion that the site soils are conducive to infiltration of stormwater. However, the shallow groundwater may limit the use of infiltration facilities. Specifically, the City of Issaquah Stormwater Design Manual specifies a minimum vertical separation from the bottom of the infiltration facilities of 3 feet from the seasonal high groundwater table (October to April). Due to the shallow groundwater table at the site, the site may not meet this requirement.

In summary, the feasibility to infiltrate collected stormwater on site requires additional studies and will need to overcome several limiting factors.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 Site Conditions and Preparation

Site preparation for the proposed project includes stripping and clearing of topsoil and sod, surface vegetation, root balls, existing foundations and pavements, and any other deleterious materials within the proposed development areas, and excavating to the design subgrade. All stripped materials should be properly disposed off-site or be "wasted" on site in non-structural landscaping areas. Soil disturbed during stripping and clearing activities should be compacted to a firm and unyielding condition.

Based on the results of our test borings, we anticipate that the existing ground is underlain by about 4 to 7 feet of fill. As such, there is the potential that unknown structures or debris, such as bricks, concrete or wood fragments, and boulders may be present within the fill.

The contractor should be aware that the groundwater is quite shallow at the site and wet/soft soil conditions should be anticipated. Contractor may need to bring in one to two feet of quarry spalls to provide a stable working surface, especially to support the installation of aggregate piers.

7.2 TEMPORARY EXCAVATIONS

Temporary excavations greater than 4 feet deep should be properly sloped or shored, however, vertical excavations of 4 feet deep or less will likely not remain stable, and will slough or collapse, due to the very loose nature of the sandy soils anticipated at the site. Along property lines, excavations less than 4 feet deep may also need to be shored unless space is available for an unsupported excavation, a temporary construction easement or a street use permit is approved by the City to allow excavation to encroach beyond the property lines.

All temporary excavations should be performed in accordance with Part N of WAC (Washington Administrative Code) 296-155. The contractor is responsible for maintaining safe excavation slopes and/or shoring. For planning purposes, the temporary excavation less than 4 feet in the upper fill of loose to medium dense silty sand maybe sloped as steep as 1.5H:1V (Horizontal:Vertical). Cuts deeper than 4 feet, if needed, will likely need to be cut back at a shallower angle of 2H:1V, to maintain stability.

To stabilize the toe of excavation slopes below the groundwater table level, such as elevator pit excavations, the soils at the toe of the slope need to be replaced with angular rock such as 2- to 4-inch quarry spalls. A sheet of geotextile separator should be placed below the quarry spalls to prevent the native fine sand and silt from migrating into the spalls.

All cuts must be re-evaluated in the field during construction based on actual observed soil conditions and the presence of groundwater seepage. If groundwater seepage is encountered the temporary slope will likely need to be cut to shallower angles to maintain stability. During wet weather, runoff water should be prevented from entering excavations. The cut slopes may need to be flattened to reduce potential erosion or should be covered with plastic sheets. We also recommend that heavy construction equipment, building materials and excavated soil should not be allowed within a distance equal to 1/3 the slope height from the top of any excavation.

7.3 DEWATERING

As discussed previously, our explorations encountered shallow groundwater that fluctuates seasonally and highly depends of the amount of precipitations. The level of dewatering efforts will depend on the groundwater conditions at the time of construction and the depth of the excavation. Provided that the excavation is no than about 2 feet into groundwater, we anticipate that a passive dewatering system should be adequate. However, for deeper excavations such as for the elevator pits or underground vaults, an active dewatering system such as well and/or well points may be needed.

In addition, excavations for underground utilities may extend into groundwater, depending on the depth of excavation. Where excavations extend below groundwater, “running” sand conditions should be anticipated. As a result, the needs for dewatering should be taken into considerations for planning purposes.

Where groundwater is encountered, the exposed subgrade will likely be wet and possible unstable. To provide a firm working subgrade, the groundwater should be lowered to at least 2 feet below the bottom of the excavation. Over-excavation of 1 to 2 feet and replaced with crushed rock underlain by a geotextile may also be needed in order to provide a firm working subgrade.

Excessive lowering of groundwater in the area could lead to settlement. As such, where needed, the dewatering efforts should be minimized. PanGEO can provide additional assessment when the excavation depths for the project are known.

7.4 Material Reuse

In the context of this report, structural fill is defined as compacted fill placed under foundation elements, concrete stairs and landings, slabs, or other load-bearing areas. Based on our SPTs, CPTs explorations and review of the previous nearby testing borings, the top 3 to 7 feet of on-site soils consist of silty to relatively clean sand with gravel (fill) that may be suitable for use as structural fill at the site. Below the 3- to 7-foot-thick layer of granular fill, the site soils consist of soft, wet, silty sand and sandy silt with interbeds of silt and clay, which will likely not be suitable as structural fill.

The re-use of on-site materials as structural fill may be possible only if the material are properly handled and can be compacted to the required density. The re-use of the on-site soils during wet times of the year will be more difficult or impossible. In addition, if the site soils become wet, they will likely not be suitable to support the movement of the conventional construction equipment without improvement such as the installation of quarry spalls over geotextile fabric.

In areas where existing foundations, slab-on-grade floors, and pavements are removed, it may be possible to crush the existing material for use as structural fill. Materials reclaimed by crushing and used as fill should have a maximum size of four inches and should be mixed with soil to provide a well-graded material.

Suitable material for use as structural fill as described in Section 7.5 below.

The on-site soil can be used as a general fill in the non-structural and landscaping areas. If the use of the on-site soil is planned, the excavated soil should be stockpiled and protected with plastic sheeting to prevent softening from rainfall in the wet season.

7.5 Structural Fill Placement and Compaction

Unless otherwise noted, structural fill below footing should consist of imported, well-graded, granular material, such as Gravel Borrow (Section 9.03.14 (1) of the 2025 WSDOT Standard Specifications), Seattle Mineral Aggregate Type 17 (*Seattle Standards and Specifications*, 2024, Section 9-03.14), Crushed Surfacing Base Course (CSBC), or other approved equivalent. Fill for use during wet weather should consist of well graded granular material having a maximum size of three inches and no more than 5 percent fines passing the US No. 200 sieve based on the minus 3/4-inch fraction.

Following the removal of deleterious and unsuitable materials, the exposed subgrade within the development area, such as building foundation, slab, and pavement areas, should be proof-rolled with fully loaded dump truck or a smooth roller compactor. The proof-rolling operation should be observed by a representative of PanGEO. The subgrade soil in the improvement areas, if recompacted and still yielding, should also be over-excavated and replaced with compacted structural fill of CDF/lean-mix concrete.

In the area where structural fill is to be used, the structural fill should be moisture conditioned to near its optimum moisture content, placed in loose, horizontal lifts less than 12 inches in thickness, and systematically compacted to a dense and relatively unyielding condition. If density tests will be performed, the test results should indicate at least 95 percent of the maximum dry density, as determined using test method ASTM D 1557. In non-structural areas, the recommended compaction level may be reduced to 90 percent. Heavy compaction equipment should not be operated directly over utilities until a minimum of 2 feet of backfill has been placed.

The procedure to achieve proper density of the compacted fill depends on the size and type of the compaction equipment, the number of passes, thickness of the lifts being compacted, and certain soil properties. If the excavation to be backfilled is constricted and limits the use of heavy equipment, smaller equipment can be used, but the lift thickness will need to be reduced to achieve the required relative compaction.

Generally, inadequately compacted soils are a result of poor construction technique or improper moisture content. Soils with high fines contents are particularly susceptible to becoming too wet and coarse-grained materials easily become too dry, for proper compaction. Silty or clayed soils with a moisture content too high for adequate compaction should be dried as necessary, or moisture conditioned by mixing with drier materials, or other methods.

In no case should the stripped organic rich soils be used as structural fill or mixed with material to be used as structural fill. The stripped material may be “wasted” on site in nonstructural landscaping, or they should be exported.

PanGEO can provide additional recommendations regarding structural fill and compaction during construction.

7.6 WET WEATHER EARTHWORK

In our opinion, the proposed site construction may be accomplished during wet weather (such as in winter) without adversely affecting the site stability. However, earthwork construction performed during the drier summer months likely will be more economical. General recommendations related to earthwork performed in wet weather or in wet conditions are presented below. The following procedures are best management practices recommended for use in wet weather construction:

- Earthwork should be performed in small areas to minimize subgrade exposure to wet weather. Excavation or the removal of unsuitable soil should be followed promptly by the placement and compaction of clean structural fill. The size and type of construction equipment used may have to be limited to prevent soil disturbance.
- The ground surface within the construction area should be graded to promote run-off of surface water and to prevent the ponding of water.
- Bales of straw and/or geotextile silt fences should be strategically located around the site to control erosion and the movement of soil.

- During wet weather, the allowable fines content of the structural fill should be reduced to no more than 5 percent by weight based on the portion passing the 0.075-mm sieve. The fines should be non-plastic.
- Excavation slopes and soils stockpiled on site should be covered with plastic sheeting.

7.7 Surface Drainage and Erosion Considerations

Adequate drainage provisions are imperative to improve the performance of the proposed developments. We recommend both short-term and long-term drainage measures be incorporated into the project design and construction. Surface runoff can be controlled during construction by careful grading practices. Typically, this includes the construction of shallow, upgradient perimeter ditches or low earthen berms in conjunction with silt fences to collect runoff and prevent water from entering excavations or to prevent runoff from the construction area from leaving the immediate work site.

Special care should be taken to avoid surface water on open cut excavations, and exposed slopes should be protected with plastic sheeting. Temporary erosion control may require the use of hay bales on the downhill side of the project to prevent water from leaving the site and potential storm water detention to trap sand and silt before the water is discharged to a suitable outlet. All collected water should be directed under control to a positive and permanent discharge system.

Permanent control of surface water and roof runoff should be incorporated in the final grading design. In addition to these sources, irrigation and rainwater infiltrating into landscape and planter areas adjacent to paved areas or building walls should also be controlled. Water should not be allowed to pond immediately adjacent to buildings or paved areas. All collected runoff should be directed into conduits that carry the water away from the proposed developments and existing structures and into the storm drain systems or other appropriate outlets. Adequate surface gradients should be incorporated into the grading design such that surface runoff is directed away from structures. Collected water from surface runoff should not drain into retaining wall drain systems. Potential problems associated with erosion may also be reduced by establishing vegetation within disturbed areas immediately following grading operations.

8.0 LIMITATIONS

We have prepared this report for use by Trailhead Apartments LLLP and the project team. Recommendations contained in this report are based on a site reconnaissance, a site-specific subsurface exploration program, review of pertinent subsurface information, and our understanding of the project. The study was performed using a mutually agreed-upon scope of work.

Variations in soil conditions may exist between the explorations and the actual conditions underlying the site. The nature and extent of soil variations may not be evident until construction occurs. If any soil conditions are encountered at the site that are different from those described in this report, we should be notified immediately to review the applicability of our recommendations. Additionally, we should also be notified to review the applicability of our recommendations if there are any changes in the project scope.

The scope of our work does not include services related to construction safety precautions. Our recommendations are not intended to direct the contractors' methods, techniques, sequences or procedures, except as specifically described in our report for consideration in design. Additionally, the scope of our work specifically excludes the assessment of environmental characteristics, particularly those involving hazardous substances. We are not mold consultants nor are our recommendations to be interpreted as being preventative of mold development. A mold specialist should be consulted for all mold-related issues.

This report may be used only by the client and for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both off and on-site), or other factors including advances in our understanding of applied science, may change over time and could materially affect our findings. Therefore, this report should not be relied upon after 24 months from its issuance. PanGEO should be notified if the project is delayed by more than 24 months from the date of this report so that we may review the applicability of our conclusions considering the time lapse.

It is the client's responsibility to see that all parties to this project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk. Any party other than the client who wishes to use this report shall notify PanGEO of such intended use and for permission to copy this report. Based on the intended use of the report, PanGEO may require that additional work be performed and that an updated report be reissued. Noncompliance with any of these requirements will release PanGEO from any liability resulting from the use of this report.

Within the limitation of scope, schedule and budget, PanGEO engages in the practice of geotechnical engineering and endeavors to perform its services in accordance with generally accepted professional principles and practices at the time the Report or its contents were prepared. No warranty, express or implied, is made.

We appreciate the opportunity to be of service to you on this project. Please feel free to contact our office with any questions you have regarding our study, this report, or any geotechnical engineering related project issues.

Sincerely,

PanGEO, Inc.

June24, 2025

Hao Wang, Ph.D.
Staff Geotechnical Engineer

Siew L. Tan, P.E.
Principal Geotechnical Engineer

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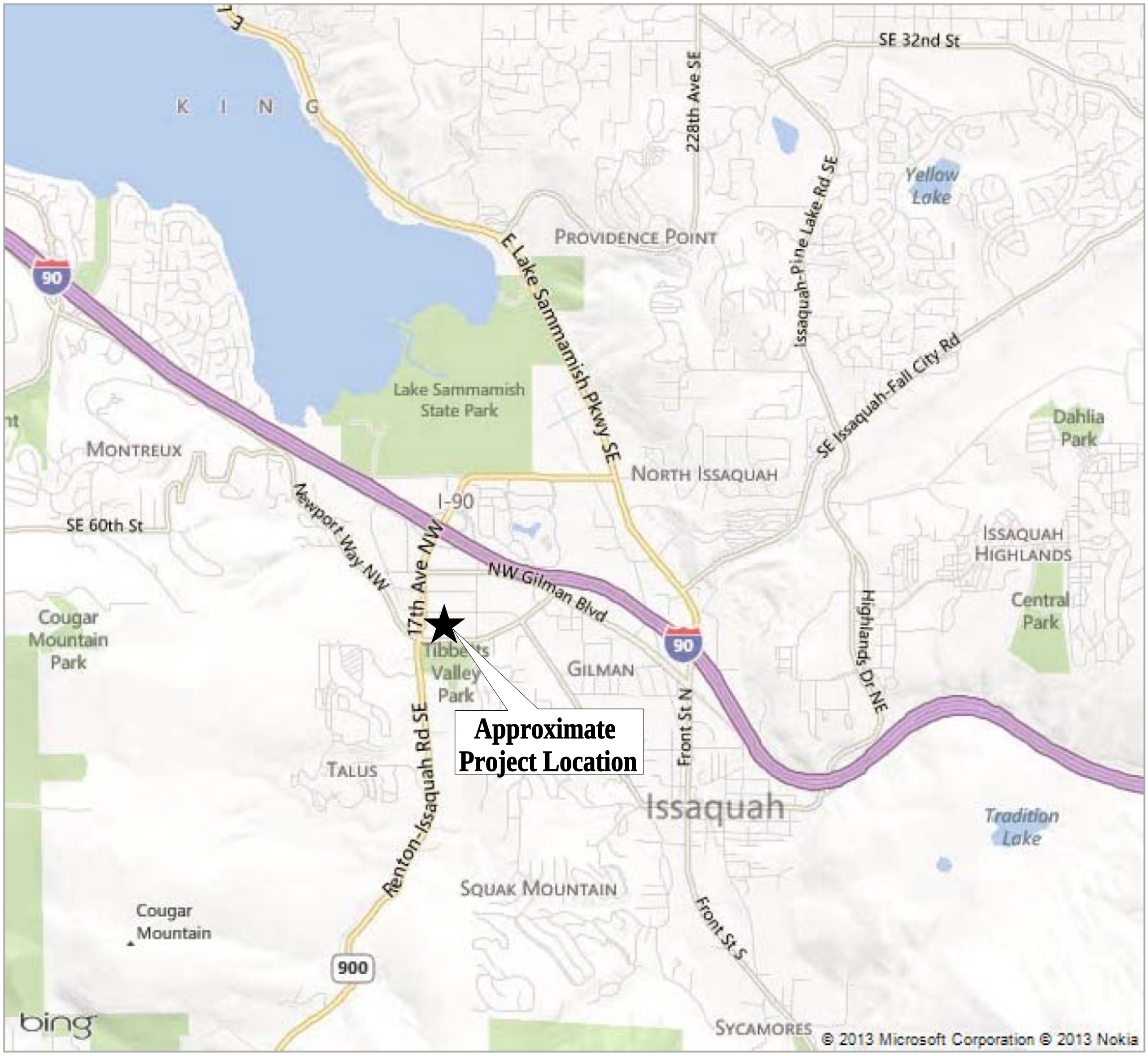
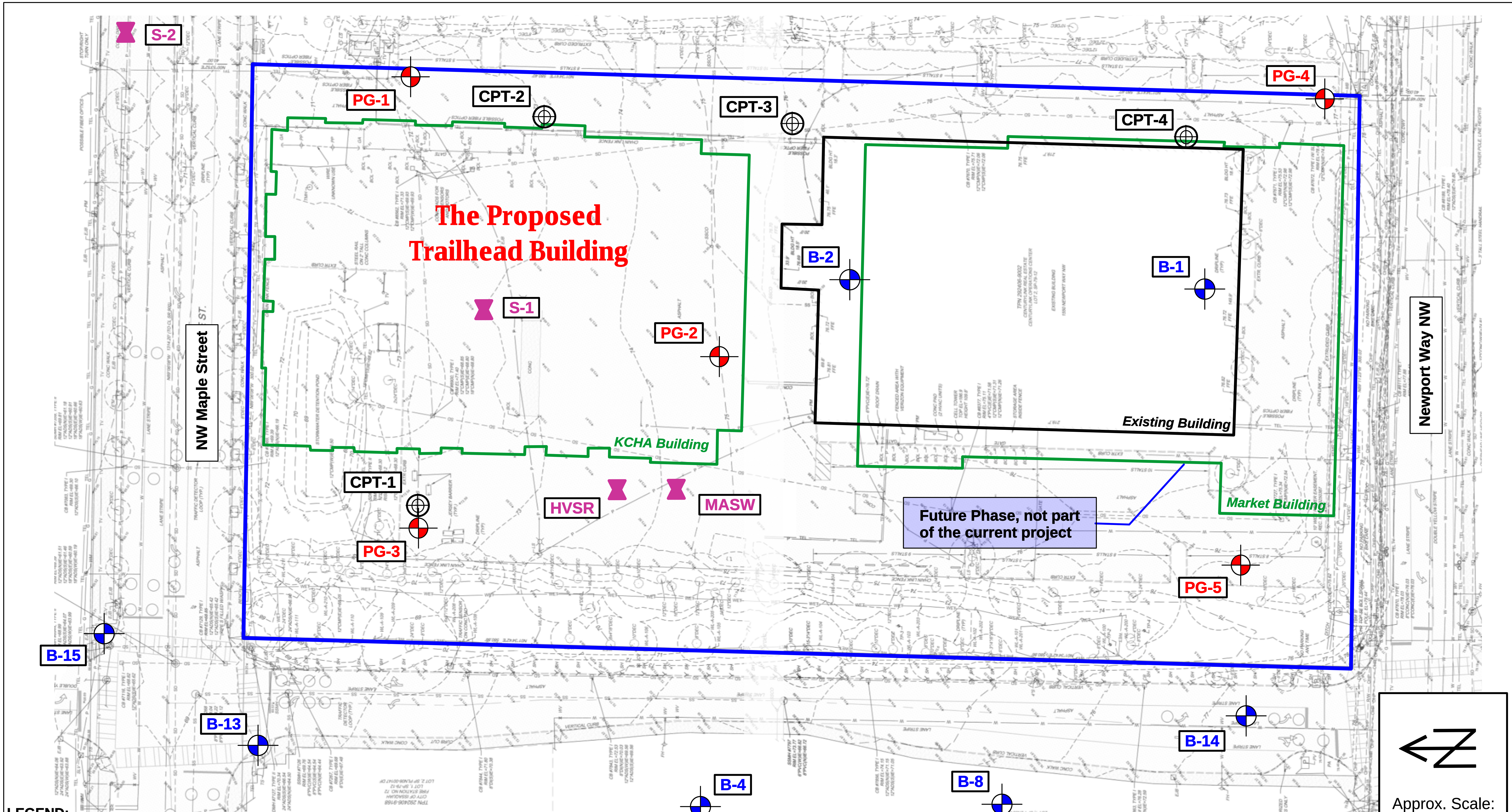


N

Scale
1 inch = 4,000 feet
(Approximate)

17-296 Fig 1 Vicinity Map.grf 1/16/25 (11:41:35)

PanGEO INCORPORATED	**Trailhead Apartments** **1505 Newport Way NW** **Issaquah, Washington**	**VICINITY MAP**	
		Project No. **24-484**	Figure No. **1**



LEGEND:

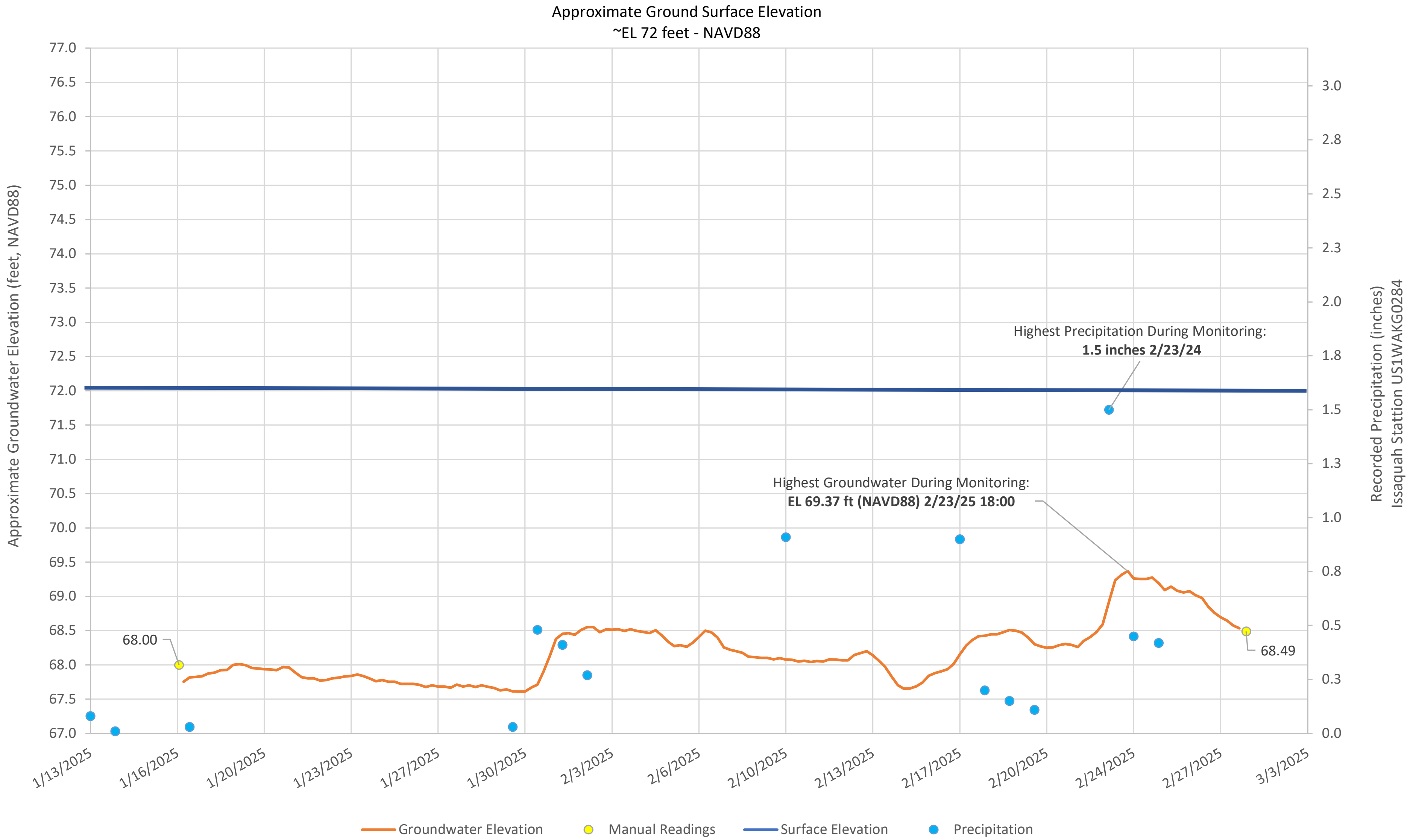
- Approx. Test Boring Location *This Study, 2025*
- Approximate Geophysical Survey *Global Geophysics, 2025*
- Site Boundary
- Previous Test Boring Location *RZA, Inc., 1980 | ZZA, Inc., 2004*
- Previous CPT Location *PanGEO Inc., 2018*
- Proposed Buildings

PanGEO INCORPORATED

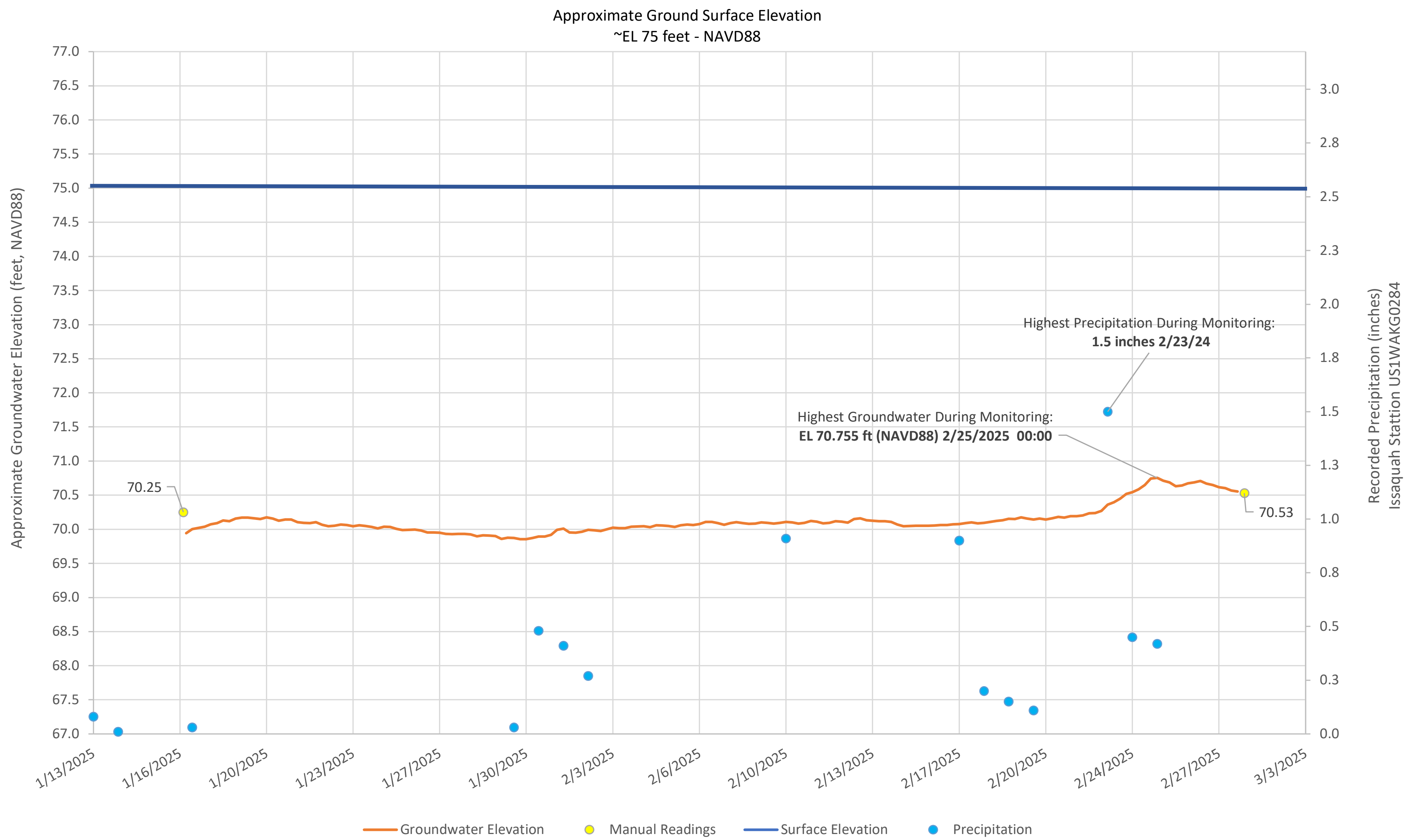
Trailhead Apartments
1550 Newport Way NW
Issaquah, Washington

SITE AND EXPLORATION PLAN

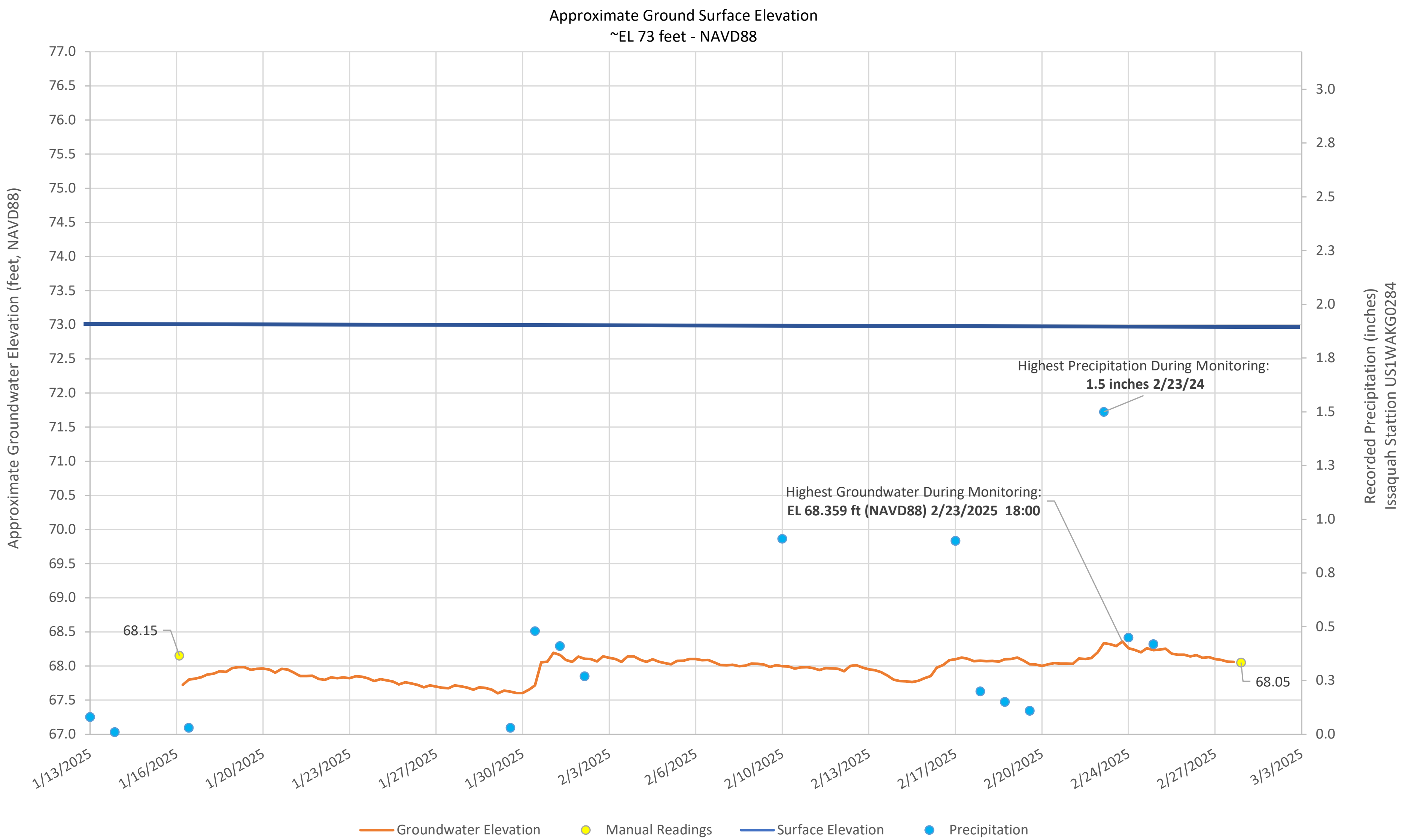
Project No. **24-484** Figure No. **2**



PanGEO INCORPORATED	Trailhead Apartments 1550 Newport Way NW Issaquah, Washington	**GROUNDWATER ELEVATION DATA MONITORING WELL PG-1**	
		Project No. **24-484**	Figure No. **3A**



PanGEO INCORPORATED	Trailhead Apartments 1550 Newport Way NW Issaquah, Washington	GROUNDWATER ELEVATION DATA MONITORING WELL PG-2
		Project No. 24-484 / Figure No. 3B

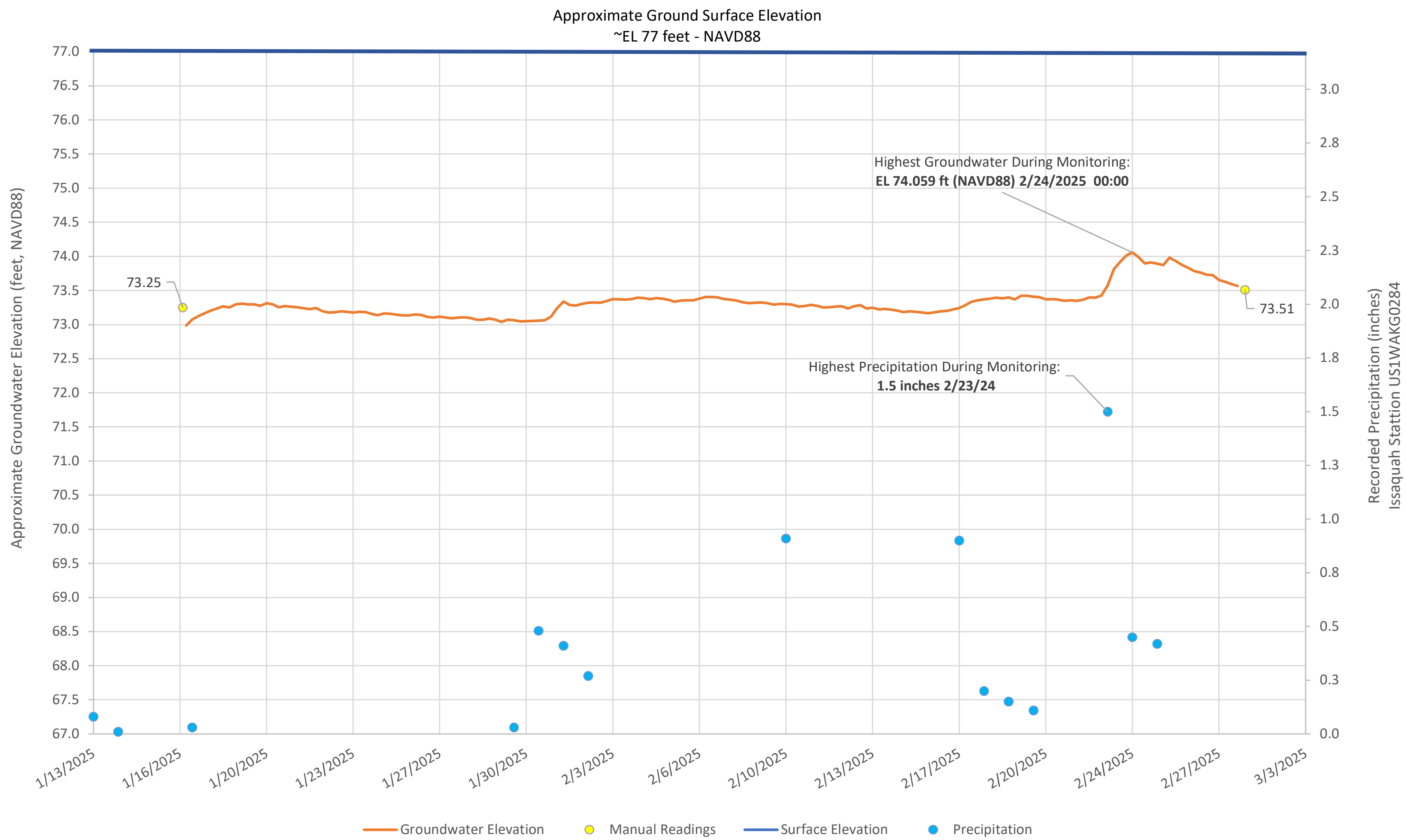


PanGEO INCORPORATED

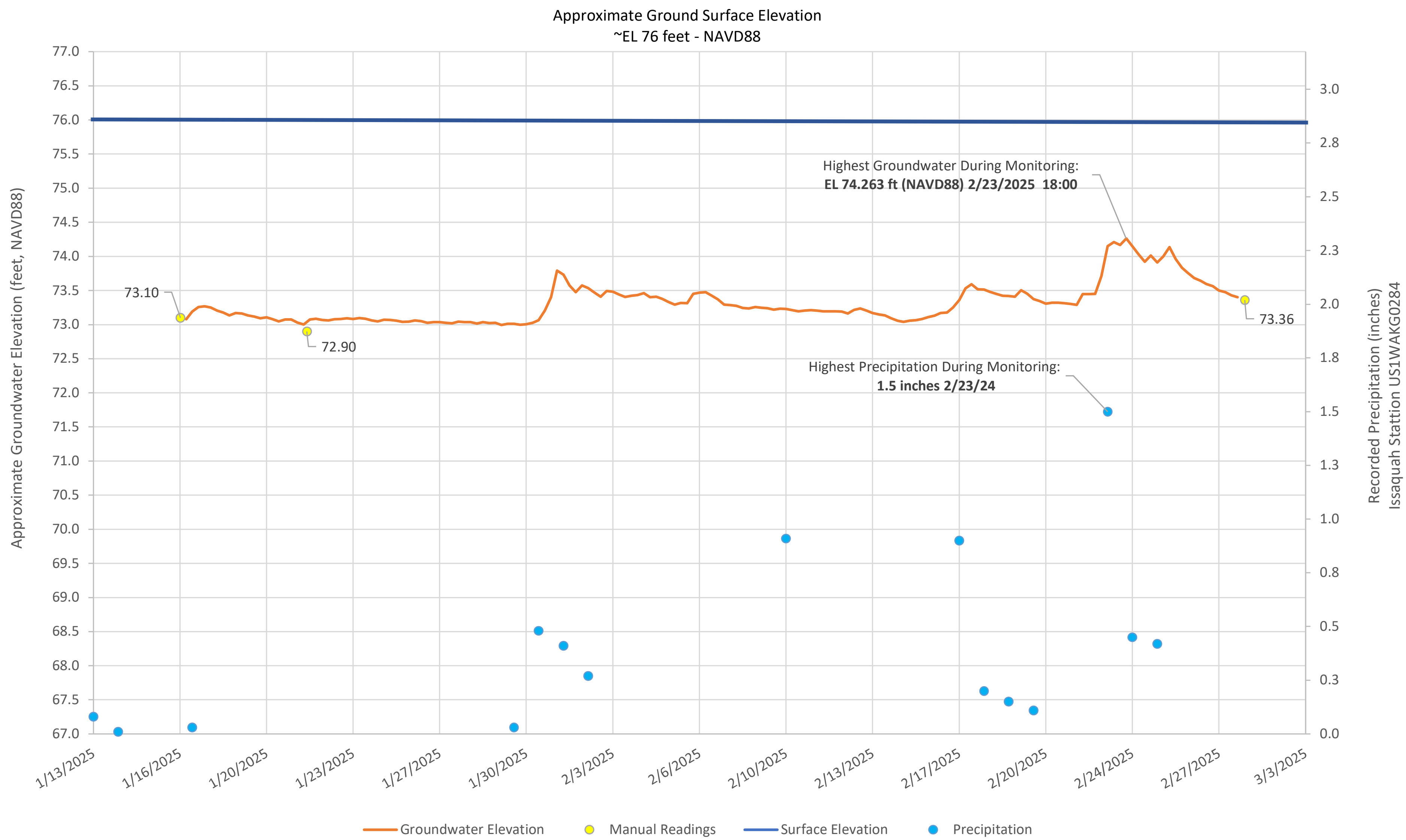
Trailhead Apartments
1550 Newport Way NW
Issaquah, Washington

GROUNDWATER ELEVATION DATA
MONITORING WELL PG-3

Project No.	Figure No.
24-484	3C



PanGEO INCORPORATED	**Trailhead Apartments** **1550 Newport Way NW** **Issaquah, Washington**	**GROUNDWATER ELEVATION DATA** **MONITORING WELL PG-4**	
		Project No. **24-484**	Figure No. **3D**



PanGEO INCORPORATED	Trailhead Apartments 1550 Newport Way NW Issaquah, Washington	**GROUNDWATER ELEVATION DATA MONITORING WELL PG-5** Project No. **24-484** Figure No. **3E**

APPENDIX A

SUMMARY TESTING BORING LOGS
(CURRENT STUDY)

RELATIVE DENSITY / CONSISTENCY

SAND / GRAVEL			SILT / CLAY		
Density	SPT N-values	Approx. Relative Density (%)	Consistency	SPT N-values	Approx. Undrained Shear Strength (psf)
Very Loose	<4	<15	Very Soft	<2	<250
Loose	4 to 10	15 - 35	Soft	2 to 4	250 - 500
Med. Dense	10 to 30	35 - 65	Med. Stiff	4 to 8	500 - 1000
Dense	30 to 50	65 - 85	Stiff	8 to 15	1000 - 2000
Very Dense	>50	85 - 100	Very Stiff	15 to 30	2000 - 4000
			Hard	>30	>4000

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		GROUP DESCRIPTIONS	
Gravel 50% or more of the coarse fraction retained on the #4 sieve. Use dual symbols (eg. GP-GM) for 5% to 12% fines.	GRAVEL (<5% fines)	GW	Well-graded GRAVEL
		GP	Poorly-graded GRAVEL
	GRAVEL (>12% fines)	GM	Silty GRAVEL
		GC	Clayey GRAVEL
Sand 50% or more of the coarse fraction passing the #4 sieve. Use dual symbols (eg. SP-SM) for 5% to 12% fines.	SAND (<5% fines)	SW	Well-graded SAND
		SP	Poorly-graded SAND
	SAND (>12% fines)	SM	Silty SAND
		SC	Clayey SAND
Silt and Clay 50%or more passing #200 sieve	Liquid Limit < 50	ML	SILT
		CL	Lean CLAY
		OL	Organic SILT or CLAY
	Liquid Limit > 50	MH	Elastic SILT
		CH	Fat CLAY
		OH	Organic SILT or CLAY
Highly Organic Soils		PT	PEAT

Notes:
1. Soil exploration logs contain material descriptions based on visual observation and field tests using a system modified from the Uniform Soil Classification System (USCS). Where necessary laboratory tests have been conducted (as noted in the "Other Tests" column), unit descriptions may include a classification. Please refer to the discussions in the report text for a more complete description of the subsurface conditions.

2. The graphic symbols given above are not inclusive of all symbols that may appear on the borehole logs. Other symbols may be used where field observations indicated mixed soil constituents or dual constituent materials.

DESCRIPTIONS OF SOIL STRUCTURES

- **Layered:** Units of material distinguished by color and/or composition from material units above and below
- **Laminated:** Layers of soil typically 0.05 to 1mm thick, max. 1 cm
- **Lens:** Layer of soil that pinches out laterally
- **Interlayered:** Alternating layers of differing soil material
- **Pocket:** Erratic, discontinuous deposit of limited extent
- **Homogeneous:** Soil with uniform color and composition throughout
- **Fissured:** Breaks along defined planes
- **Slickensided:** Fracture planes that are polished or glossy
- **Blocky:** Angular soil lumps that resist breakdown
- **Disrupted:** Soil that is broken and mixed
- **Scattered:** Less than one per foot
- **Numerous:** More than one per foot
- **BCN:** Angle between bedding plane and a plane normal to core axis

COMPONENT DEFINITIONS

COMPONENT	SIZE / SIEVE RANGE	COMPONENT	SIZE / SIEVE RANGE
Boulder:	> 12 inches	Sand	
Cobbles:	3 to 12 inches	Coarse Sand:	#4 to #10 sieve (4.5 to 2.0 mm)
Gravel		Medium Sand:	#10 to #40 sieve (2.0 to 0.42 mm)
Coarse Gravel:	3 to 3/4 inches	Fine Sand:	#40 to #200 sieve (0.42 to 0.074 mm)
Fine Gravel:	3/4 inches to #4 sieve	Silt	0.074 to 0.002 mm
		Clay	<0.002 mm

TEST SYMBOLS

for In Situ and Laboratory Tests listed in "Other Tests" column.

ATT	Atterberg Limit Test
Comp	Compaction Tests
Con	Consolidation
DD	Dry Density
DS	Direct Shear
%F	Fines Content
GS	Grain Size
Perm	Permeability
PP	Pocket Penetrometer
R	R-value
SG	Specific Gravity
TV	Torvane
TXC	Triaxial Compression
UCC	Unconfined Compression

SYMBOLS

Sample/In Situ test types and intervals

- 2-inch OD Split Spoon, SPT (140-lb. hammer, 30" drop)
- 3.25-inch OD Spilt Spoon (300-lb hammer, 30" drop)
- Non-standard penetration test (see boring log for details)
- Thin wall (Shelby) tube
- Grab
- Rock core
- Vane Shear

MONITORING WELL

- Groundwater Level at time of drilling (ATD)
- Static Groundwater Level
- Cement / Concrete Seal
- Bentonite grout / seal
- Silica sand backfill
- Slotted tip
- Slough
- Bottom of Boring

MOISTURE CONTENT

Dry	Dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water

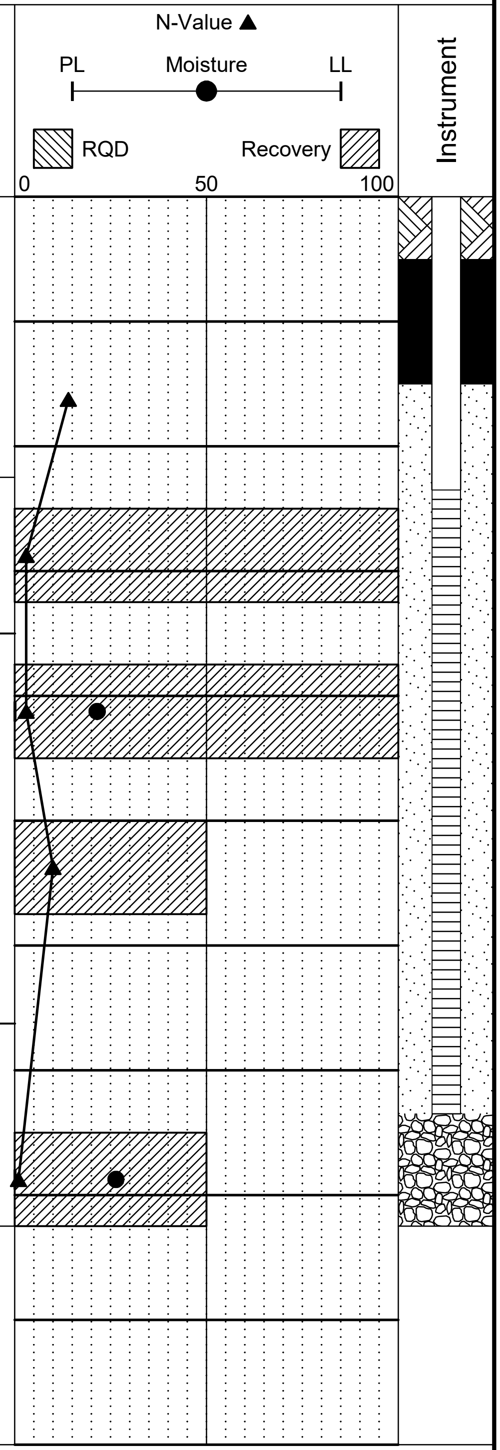
PanGEO INCORPORATED
Phone: 206.262.0370

Terms and Symbols for Boring and Test Pit Logs

Figure A-1

Project:	Trailhead Apartments	Surface Elevation:	~72 ft
Job Number:	24-484	Top of Casing Elev.:	NOT SURVEYED
Location:	1550 Newport Way NW, Issaquah, WA	Drilling Method:	HSA
Coordinates:	Northing: 200360, Easting: 1337505	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0						**ASPHALT AND FILL** Approximately 4 inches of asphalt above: Medium dense, gray, silty SAND with GRAVEL and COBBLE; subround to subangular; 4- to 6-inch cobbles; moist to wet.
2	S-1		4 8 6			*--groundwater measured at 4.3 feet below grade on 01/16/2025.* *--groundwater observed at approximately 5 feet below grade at time of drilling.*
4	S-2		2 1 2			**ALLUVIUM - Qal** Very loose, gray, silty SAND, trace organics (wood debris and burnt wood fragments), occasional silt to clay interbeds; massive texture; moist to wet (saturated).
6 / 8	S-3		3 2 1	MC GS		Very loose to loose, gray to gray-brown, silty SAND with GRAVEL, trace organics (wood debris and burnt wood fragments); round to subround gravels, iron-oxide staining; very moist to wet. *SAMPLE S3: GRAVEL 28.0%, SAND 51.9%, FINES 20.1%* *MOISTURE CONTENT = 21.5%.*
10 / 12	S-4		3 5 5			
14 / 16	S-5		1 0 1	MC GS		Very loose, gray to gray-brown, silty SAND; some gravel, trace organics (wood debris and burnt wood fragments); wet (saturated). *SAMPLE S5: GRAVEL 14.0%, SAND 38.3%, FINES 47.7%* *MOISTURE CONTENT = 26.4%.*
18 / 20						Boring terminated at approximately 16.5 feet below grade. Groundwater was observed beginning at about 5 feet below grade at time of drilling. 2-inch Monitoring Well Installled with Department of Ecology Well Tag # BQN-133. Groundwater was measured at approximately 4.3 feet below grade on 01/16/2025.



Completion Depth:	16.5ft
Date Borehole Started:	1/15/25
Date Borehole Completed:	1/15/25
Logged By:	S. Scott
Drilling Company:	Geologic Drill Partners

Remarks: Boring drilled using an Acker Recon tracked drill rig. Standard penetration test (SPT) sampler driven with a 140 lb. automatic trip hammer with 90% Efficiency Rating. Surface elevation is approximate and based the provided Topographic Survey. This information is provided for relative information only and is not a substitution for field survey.
HORIZONTAL DATUM - WA State Plane - North | VERTICAL DATUM - NAVD88

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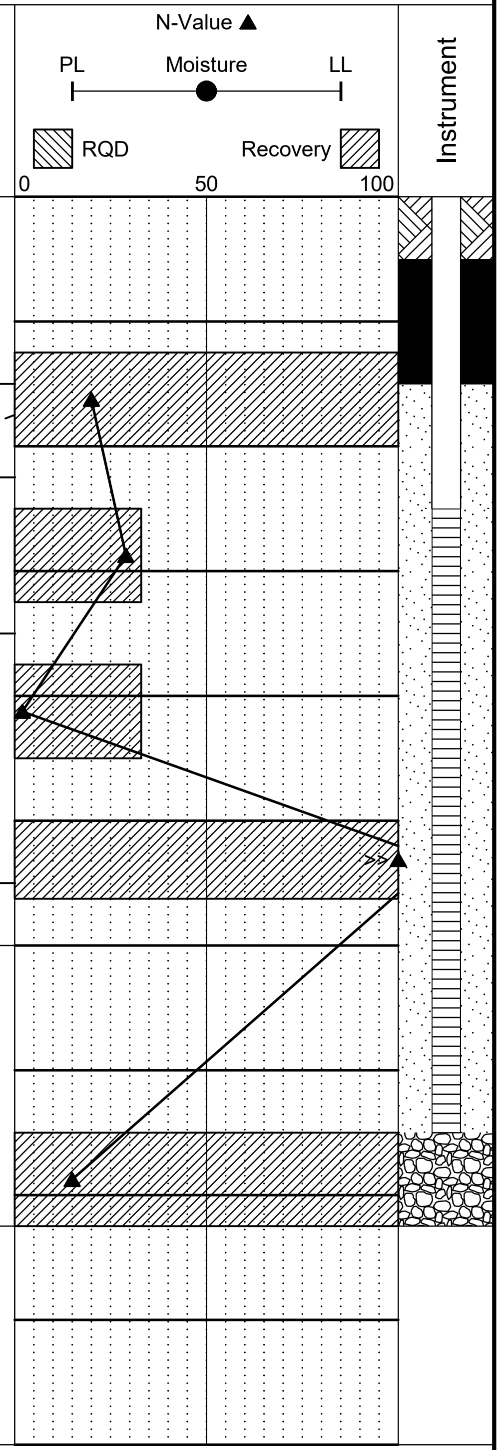
LOG OF TEST BORING PG-1

Figure A-2

The stratification lines represent approximate boundaries. The transition may be gradual.

Project:	Trailhead Apartments	Surface Elevation:	~75 ft
Job Number:	24-484	Top of Casing Elev.:	NOT SURVEYED
Location:	1550 Newport Way NW, Issaquah, WA	Drilling Method:	HSA
Coordinates:	Northing: 200201, Easting: 1337355	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0						**ASPHALT AND FILL** Approximately 3 inches of asphalt above: Medium dense, light gray, sandy GRAVEL; angular, CSBC-like; moist.
2						
3	S-1		8, 10, 10			Medium dense, brown, silty SAND with GRAVEL; round to subround, pit-run like; moist.
4						Medium dense, black, CREOSOTE; moist.
4.75						Medium dense, gray-brown, silty SAND with GRAVEL; angular to subround, pit-run and CSBC-like; moist. *--groundwater measured at 4.75 feet below grade on 01/16/2025.* *--groundwater observed at approximately 6.5 feet below grade at time of drilling.*
6	S-2		14, 18, 11			
7.25						Very loose, gray, GRAVEL; trace silt and sand, round, pea-gravel-like; wet (saturated).
8	S-3		2, 1, 1			
10	S-4		14, 18, 50/3			
11						*-- large flat obstruction from about 11 to 12 feet (concrete slab?); inflated blow counts.*
12						**ALLUVIUM - Qal** Medium dense, gray, silty SAND with GRAVEL, round to subround, trace iron-oxide staining; slightly reworked texture; wet (saturated).
14						
16	S-5		6, 8, 7			
16.5						Boring terminated at approximately 16.5 feet below grade. Groundwater was observed beginning at about 6.5 feet below grade at time of drilling. 2-inch Monitoring Well Installled with Department of Ecology Well Tag # BQN-137. Groundwater was measured at approximately 4.75 feet below grade on 01/16/2025.
18						
20						



Completion Depth:	16.5ft
Date Borehole Started:	1/15/25
Date Borehole Completed:	1/15/25
Logged By:	S. Scott
Drilling Company:	Geologic Drill Partners

Remarks: Boring drilled using an Acker Recon tracked drill rig. Standard penetration test (SPT) sampler driven with a 140 lb. automatic trip hammer with 90% Efficiency Rating. Surface elevation is approximate and based the provided Topographic Survey. This information is provided for relative information only and is not a substitution for field survey.
HORIZONTAL DATUM - WA State Plane - North | VERTICAL DATUM - NAVD88

PanGEO INCORPORATED

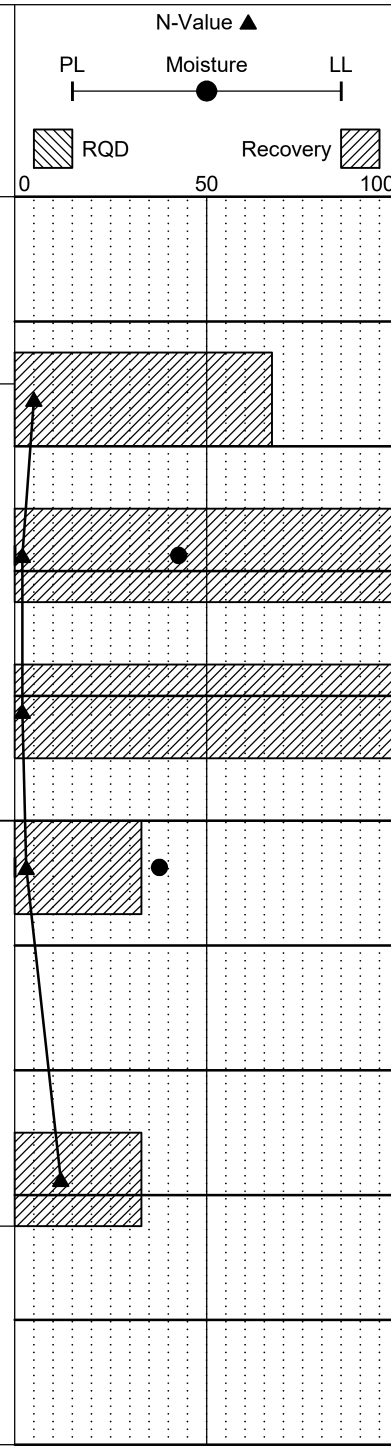
LOG OF TEST BORING PG-2

Figure A-3

The stratification lines represent approximate boundaries. The transition may be gradual.

Project:	Trailhead Apartments	Surface Elevation:	~73 ft
Job Number:	24-484	Top of Casing Elev.:	NOT SURVEYED
Location:	1550 Newport Way NW, Issaquah, WA	Drilling Method:	HSA
Coordinates:	Northing: 200363, Easting: 1337261	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0						**ASPHALT AND FILL** Approximately 4 inches of asphalt above: Medium dense to loose, gray, silty SAND with GRAVEL and COBBLE; subround; 4- to 6-inch cobbles; moist.
3	S-1		7, 3, 2			**ALLUVIUM - Qal** Loose to very loose, dark brown to gray, interbedded organic SILT and sandy SILT; rootlets and wood debris, trace gravel; non-plastic; moist to wet (saturated). *--groundwater measured at 4.45 feet below grade on 01/16/2025.*
6	S-2		1, 1, 1	MC GS		*SAMPLE S2: GRAVEL 1.0%, SAND 39.9%, FINES 59.1% MOISTURE CONTENT = 42.7%.* *--groundwater observed at approximately 6.5 feet below grade at time of drilling.*
8	S-3		0, 1, 1			
10	S-4		1, 1, 2	MC GS		Very loose, dark brown to gray, interbedded organic SILT and sandy SILT; rootlets and wood debris; trace iron-oxide staining, non-plastic, moist to wet (saturated). *SAMPLE S4: GRAVEL 0.0%, SAND 33.7%, FINES 66.3% MOISTURE CONTENT = 37.7%.*
15	S-5		2, 5, 7			*--becomes medium dense; increase in gravel content.*
16.5						Boring terminated at approximately 16.5 feet below grade. Groundwater was observed beginning at about 6.5 feet below grade at time of drilling. 2-inch Monitoring Well Installled with Department of Ecology Well Tag # BQN-136. Groundwater was measured at approximately 4.45 feet below grade on 01/16/2025.



Instrument

Completion Depth:	16.5ft
Date Borehole Started:	1/15/25
Date Borehole Completed:	1/15/25
Logged By:	S. Scott
Drilling Company:	Geologic Drill Partners

Remarks: Boring drilled using an Acker Recon tracked drill rig. Standard penetration test (SPT) sampler driven with a 140 lb. automatic trip hammer with 90% Efficiency Rating. Surface elevation is approximate and based the provided Topographic Survey. This information is provided for relative information only and is not a substitution for field survey.
HORIZONTAL DATUM - WA State Plane - North | VERTICAL DATUM - NAVD88

PanGEO INCORPORATED

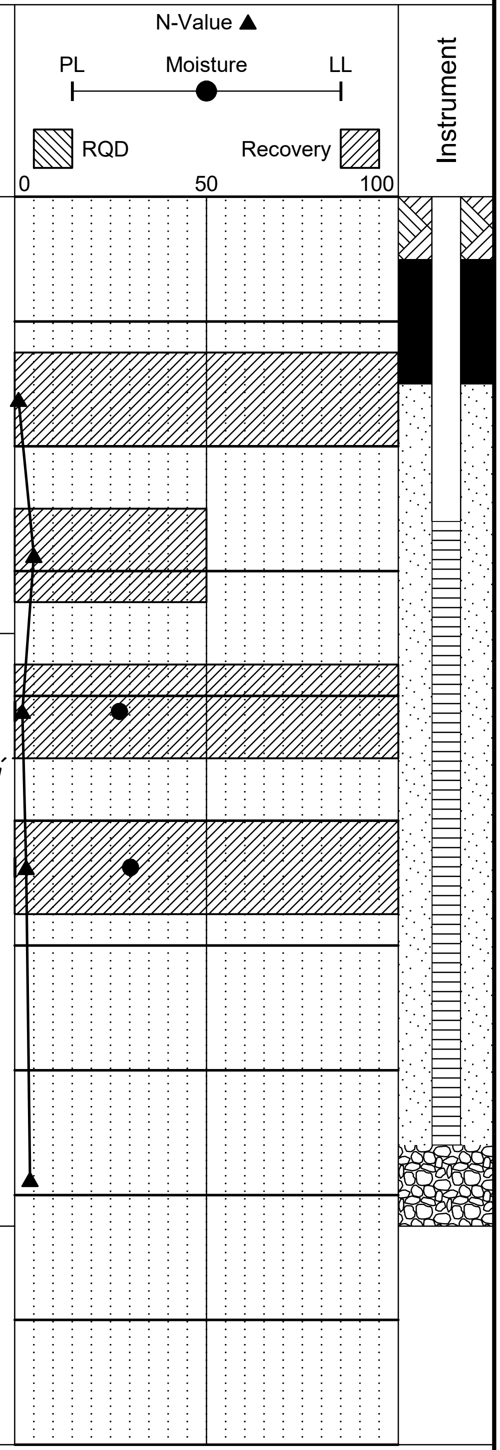
LOG OF TEST BORING PG-3

Figure A-4

The stratification lines represent approximate boundaries. The transition may be gradual.

Project:	Trailhead Apartments	Surface Elevation:	~77 ft
Job Number:	24-484	Top of Casing Elev.:	NOT SURVEYED
Location:	1550 Newport Way NW, Issaquah, WA	Drilling Method:	HSA
Coordinates:	Northing: 199927, Easting: 1337246	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	MATERIAL DESCRIPTION
0					**ASPHALT AND FILL** Approximately 4 inches of asphalt above: Very loose, gray-brown to dark brown, silty SAND with GRAVEL; interbedded organics, subround to subangular; moist to wet (saturated).
3	S-1	SPT	1 0 1		*--groundwater measured at 3.75 feet below grade on 01/16/2025.*
5.5	S-2	SPT	4 3 2		*--groundwater observed at approximately 5 feet below grade at time of drilling.*
7					**ALLUVIUM - Qal** Very loose, gray, silty SAND, trace organics (wood debris and burnt wood fragments); massive texture; iron-oxide staining; moist to wet (saturated).
8.5	S-3	SPT	0 1 1	MC GS	*SAMPLE S3: GRAVEL 9.0%, SAND 54.5%, FINES 36.5%* *MOISTURE CONTENT = 27.3%.*
9					Very loose, gray, sandy SILT, trace organics (wood debris and burnt wood fragments); laminated texture, trace iron-oxide staining; non-plastic, wet (saturated).
11	S-4	SPT	1 2 1	MC GS	*SAMPLE S4: GRAVEL 0.0%, SAND 41.3%, FINES 58.7%* *MOISTURE CONTENT = 30.2%.*
16	S-5	SPT	0 1 3		
16.5					Boring terminated at approximately 16.5 feet below grade. Groundwater was observed beginning at about 5 feet below grade at time of drilling. 2-inch Monitoring Well Installled with Department of Ecology Well Tag # BQN-134. Groundwater was measured at approximately 3.75 feet below grade on 01/16/2025.



Completion Depth:	16.5ft
Date Borehole Started:	1/15/25
Date Borehole Completed:	1/15/25
Logged By:	S. Scott
Drilling Company:	Geologic Drill Partners

Remarks: Boring drilled using an Acker Recon tracked drill rig. Standard penetration test (SPT) sampler driven with a 140 lb. automatic trip hammer with 90% Efficiency Rating. Surface elevation is approximate and based the provided Topographic Survey. This information is provided for relative information only and is not a substitution for field survey.
HORIZONTAL DATUM - WA State Plane - North | VERTICAL DATUM - NAVD88

PanGEO INCORPORATED

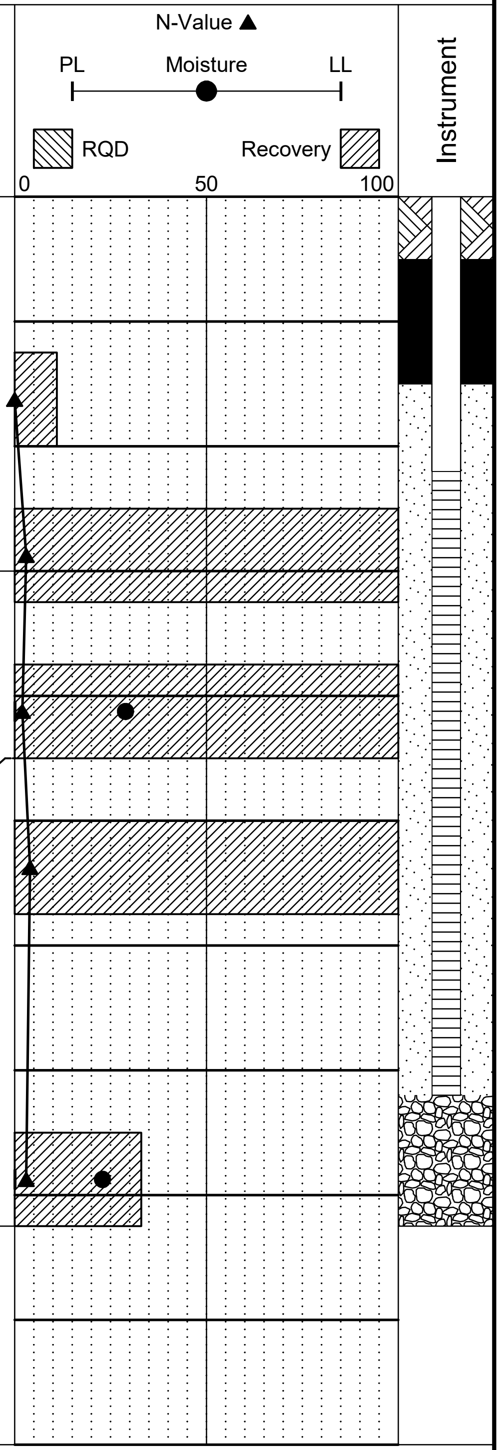
LOG OF TEST BORING PG-4

Figure A-5

The stratification lines represent approximate boundaries. The transition may be gradual.

Project:	Trailhead Apartments	Surface Elevation:	~76 ft
Job Number:	24-484	Top of Casing Elev.:	NOT SURVEYED
Location:	1550 Newport Way NW, Issaquah, WA	Drilling Method:	HSA
Coordinates:	Northing: 199879, Easting: 1337490	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	MATERIAL DESCRIPTION
0					**ASPHALT AND FILL** Approximately 4 inches of asphalt above: Very loose, brown, silty SAND to sandy SILT; trace gravel, heavy iron-oxide staining, reworked texture; moist. *-- driller able to push sampler with weight of hammer; likely very loose to soft soils not encountered in sampler.*
3	S-1	X	0, 0, 0		*--groundwater measured at 3 feet below grade on 01/16/2025.*
5.5	S-2	X	0, 1, 2		*--groundwater observed at approximately 6 feet below grade at time of drilling.*
6					**ALLUVIUM - Qal** Very loose, gray, sandy SILT, trace organics (wood debris and burnt wood fragments), trace gravel; trace iron-oxide staining, massive to laminated texture; non-plastic, moist to wet (saturated).
8	S-3	X	2, 1, 1	MC GS	*SAMPLE S3: GRAVEL 3.0%, SAND 45.4%, FINES 51.6%* *MOISTURE CONTENT = 28.9%.*
9					Loose to very loose, gray-brown to brown, silty SAND; trace to some gravel, occasional interbedded silt-clay lens, trace burnt wood fragments, massive texture; non-plastic, wet (saturated).
10.5	S-4	X	0, 2, 2		
15.5	S-5	X	1, 1, 2	MC GS	*SAMPLE S5: GRAVEL 9.0%, SAND 63.6%, FINES 27.4%* *MOISTURE CONTENT = 22.9%.*
16.5					Boring terminated at approximately 16.5 feet below grade. Groundwater was observed beginning at about 6 feet below grade at time of drilling. 2-inch Monitoring Well Installled with Department of Ecology Well Tag # BQN-135. Groundwater was measured at approximately 3 feet below grade on 01/16/2025.



Completion Depth:	16.5ft
Date Borehole Started:	1/15/25
Date Borehole Completed:	1/15/25
Logged By:	S. Scott
Drilling Company:	Geologic Drill Partners

Remarks: Boring drilled using an Acker Recon tracked drill rig. Standard penetration test (SPT) sampler driven with a 140 lb. automatic trip hammer with 90% Efficiency Rating. Surface elevation is approximate and based the provided Topographic Survey. This information is provided for relative information only and is not a substitution for field survey.

HORIZONTAL DATUM - WA State Plane - North | VERTICAL DATUM - NAVD88

PanGEO INCORPORATED

LOG OF TEST BORING PG-5

Figure A-6

The stratification lines represent approximate boundaries. The transition may be gradual.

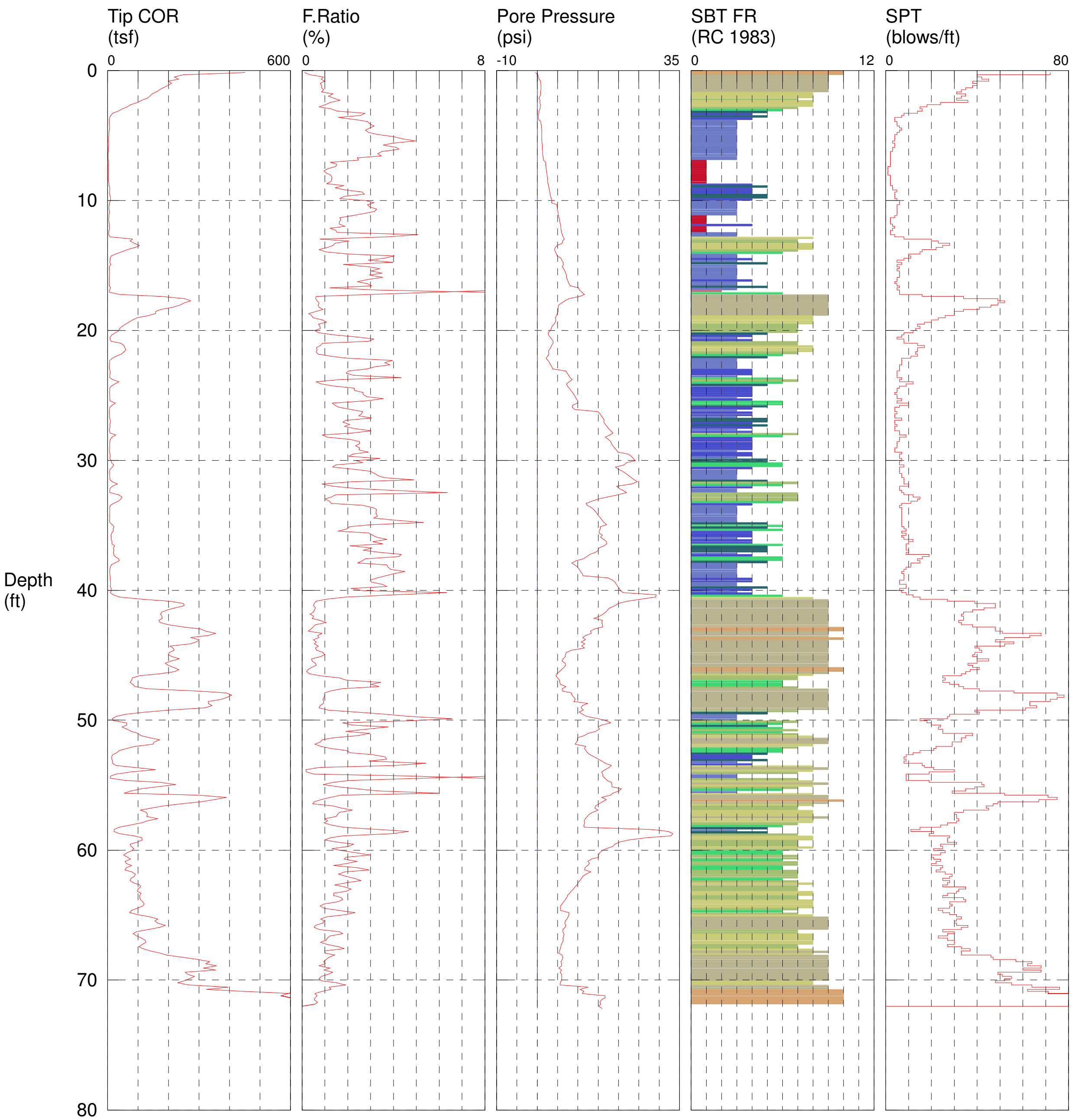
APPENDIX B

PREVIOUS CPT AND BORING LOGS

CPT-01

CPT CONTRACTOR: InSitu Engineering
CUSTOMER: PanGEO
LOCATION: Issaquah
JOB NUMBER: 17-296

OPERATOR: Romanelli
CONE ID: DDG1424
TEST DATE: 1/2/2018 10:42:29 AM
PREDRILL: N/A
BACKFILL: 20% Bentonite Grout
SURFACE PATCH: Granular Bentonite Chip



COMMENT:

1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

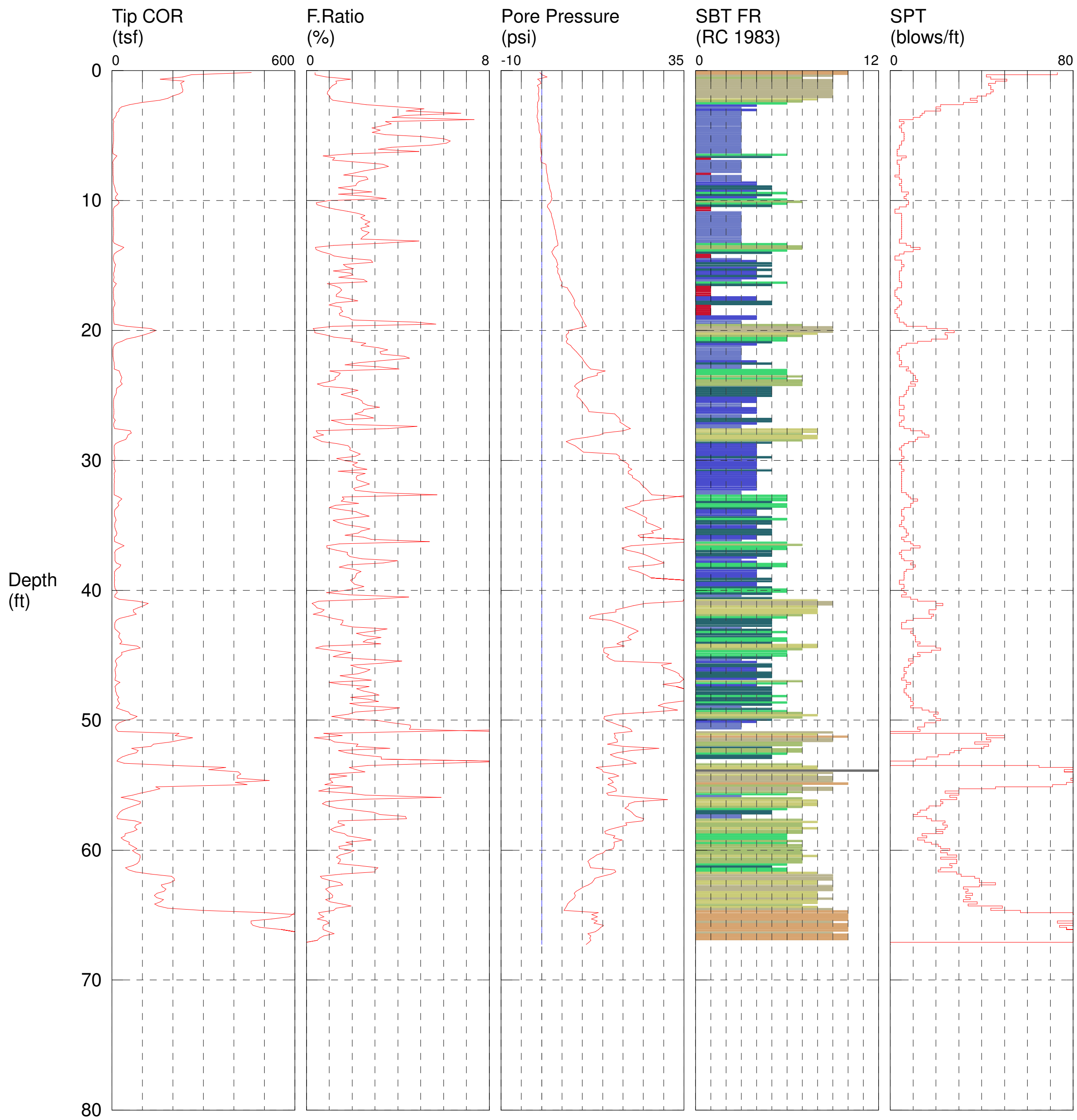
*SBT/SPT CORRELATION: UBC-1983

Figure A-1

CPT-02

CPT CONTRACTOR: InSitu Engineering
CUSTOMER: PanGEO
LOCATION: Issaquah
JOB NUMBER: 17-296

OPERATOR: Romanelli
CONE ID: DDG1424
TEST DATE: 1/2/2018 9:45:30 AM
PREDRILL: N/A
BACKFILL: 20% Bentonite Grout
SURFACE PATCH: Granular Bentonite Chip



COMMENT:

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 (*)

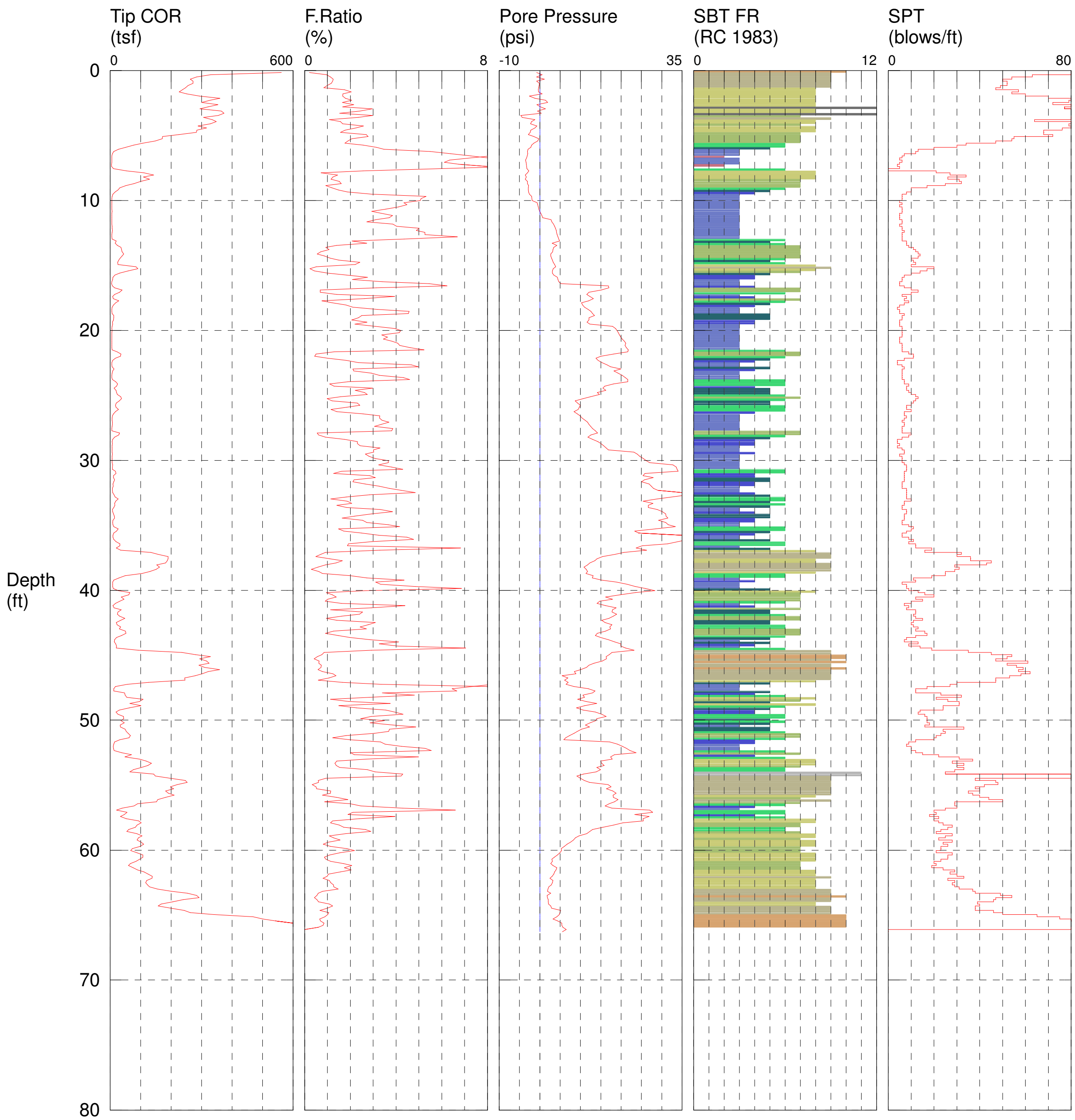
*SBT/SPT CORRELATION: UBC-1983

Figure A-2

CPT-03

CPT CONTRACTOR: InSitu Engineering
CUSTOMER: PanGEO
LOCATION: Issaquah
JOB NUMBER: 17-296

OPERATOR: Romanelli
CONE ID: DDG1424
TEST DATE: 1/2/2018 8:34:46 AM
PREDRILL: N/A
BACKFILL: 20% Bentonite Grout
SURFACE PATCH: Granular Bentonite Chip



COMMENT:

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 (*)

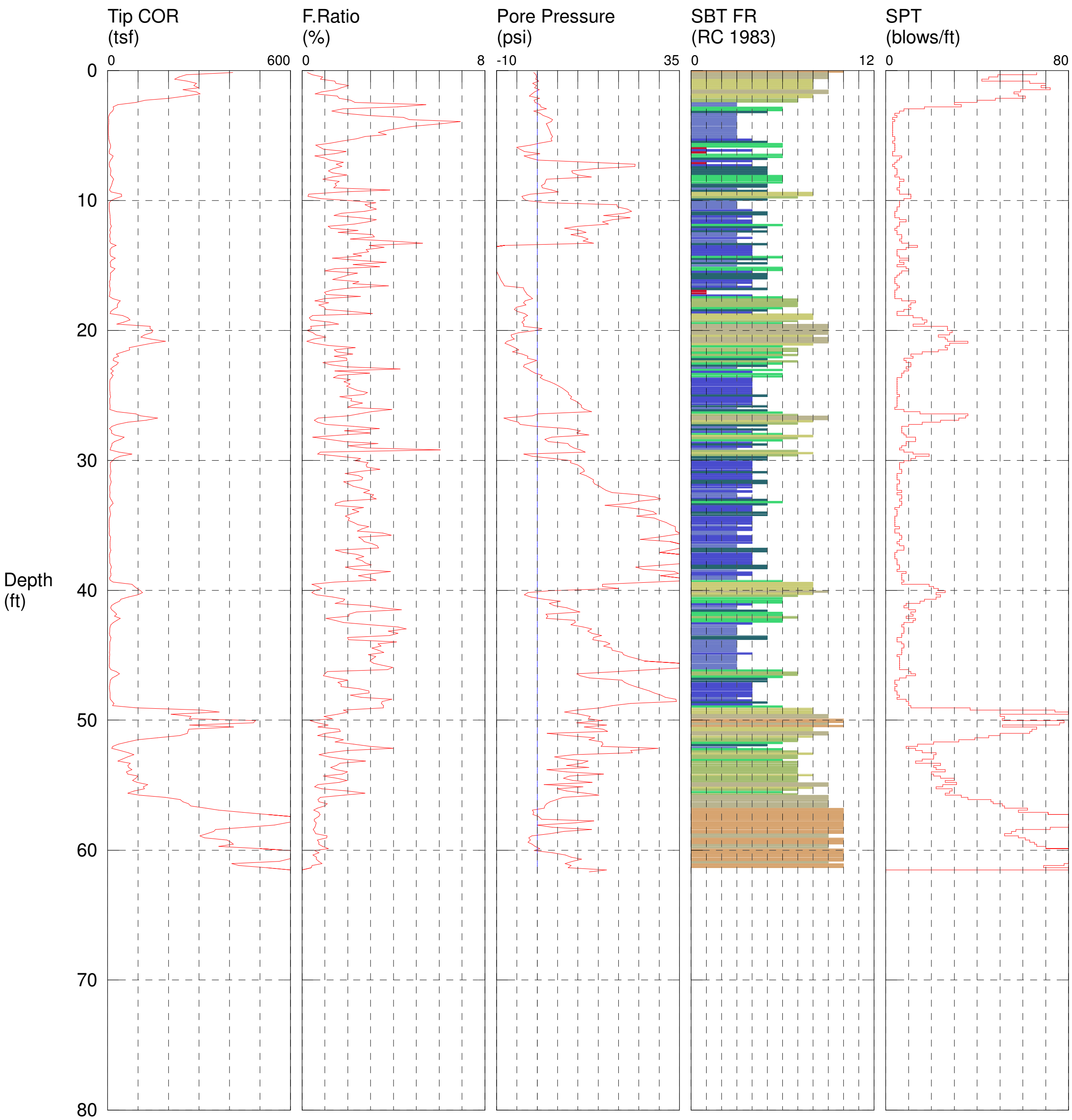
*SBT/SPT CORRELATION: UBC-1983

Figure A-3

CPT-04

CPT CONTRACTOR: InSitu Engineering
CUSTOMER: PanGEO
LOCATION: Issaquah
JOB NUMBER: 17-296

OPERATOR: Romanelli
CONE ID: DDG1424
TEST DATE: 1/2/2018 7:14:16 AM
PREDRILL: N/A
BACKFILL: 20% Bentonite Grout
SURFACE PATCH: Granular Bentonite Chip



COMMENT:

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 (*)

*SBT/SPT CORRELATION: UBC-1983

Figure A-4

PROJECT: Issaquah Transit Center | **JOB NO.:** J-1875 | **BORING:** B-4 | **PAGE 1 OF 3**

Location: Issaquah, Washington | **Approximate Elevation:** 74 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (Blows per foot)	N-values	Testing
0	4 inches asphalt						
	Medium dense, moist, brown-gray, silty, gravelly, SAND (fill)	I	S-1			25	
5	Very soft, saturated, black, sandy, organic SILT			7/7/04			
	Very soft, staurated, gray with orange, sandy, clayey SILT with trace fine organic material	I	S-2			1	
10							
	Grades to gray, sandy SILT with some clay and trace fine organic material	I	S-3			1	
15	Medium dense, saturated, gray, silty, sandy GRAVEL with trace organics			ATD			
		I	S-4			16	
20							
	Very soft, saturated, gray, clayey SILT with trace fine organic material	I	S-5			2	
25							

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

Explanation

- I 2-inch O.D. split spoon sample, 140 lb hammer with 30-inch free fall
- 3-inch O.D. Dames & Moore sample, 300 lb hammer with 30-inch free fall
- 3-inch I.D Shelby tube sample
- ⊗ No Recovery
- ▼ ATD Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

- GSA = Grain Size Analysis
- 200W = 200 Wash Analysis
- Att. = Atterberg Limits
- Con. = Consolidation Test

Zipper Zeman Associates, Inc.
Geotechnical and Environmental Consulting

BORING LOG	Figure A-4
Date Drilled: 6/23/2004	Logged By: KTH

PROJECT: Issaquah Transit Center | **JOB NO.:** J-1875 | **BORING:** B-4 |

Location: Issaquah, Washington | **Approximate Elevation:** 74 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (Blows per foot; ▲ Standard, △ Other; 0–50)	N-values	Testing
25							
	Grades to soft, sandy SILT with some gravel and trace clay and fine organic material	2-inch O.D. split spoon	S-6	Clean Sand / Screened Casing	▲	5	
30				Bentonite			
	Grades to very soft, gray-black, sandy SILT with some clay and trace fine organic material	2-inch O.D. split spoon	S-7	Bentonite	▲	2	
35				Bentonite			
	Grades to medium stiff, sandy SILT with 1/8-inch thick organic layers	2-inch O.D. split spoon	S-8	Bentonite	▲	6	
40	Loose, saturated, gray, gravelly SAND with trace silt			Bentonite			
	Very soft, saturated, dark gray, sandy SILT with organics and trace gravel	2-inch O.D. split spoon	S-9	Bentonite	▲	2	
45	Medium dense, saturated, gray- black, organic, silty, fine SAND with trace gravel			Bentonite			
		2-inch O.D. split spoon	S-10	Bentonite	▲	12	
50							

Explanation

- 2-inch O.D. split spoon sample; 140 lb hammer with 30-inch free fall
- 3-inch O.D. Dames & Moore sample; 300 lb hammer with 30-inch free fall
- 3-inch I.D Shelby tube sample
- No Recovery
- ATD Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

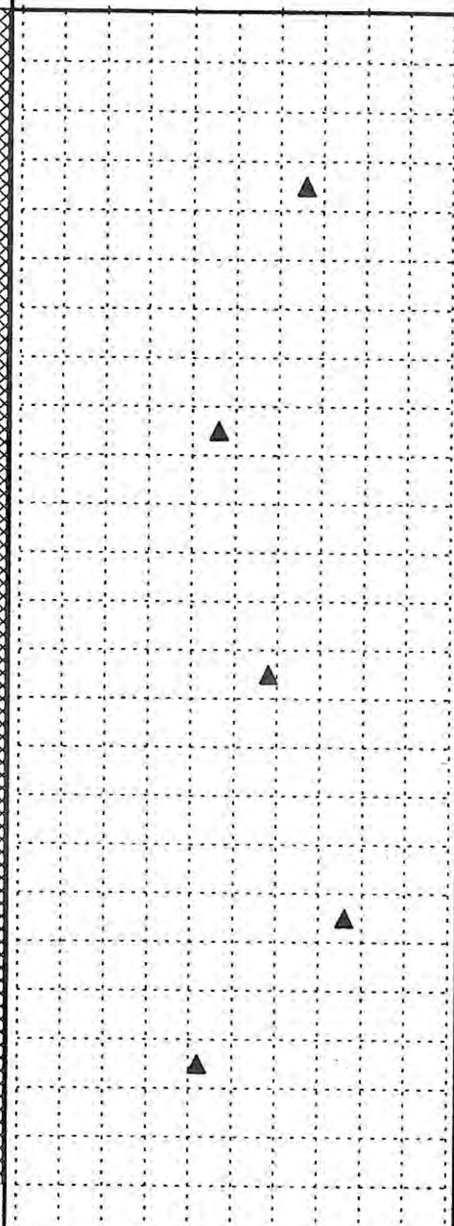
Testing Key

- GSA = Grain Size Analysis
- 200W = 200 Wash Analysis
- Att. = Atterberg Limits
- Con. = Consolidation Test

PROJECT: Issaquah Transit Center | **JOB NO.:** J-1875 | **BORING:** B-4 | **PAGE 3 OF 3**

Location: Issaquah, Washington | **Approximate Elevation:** 74 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	N-values	Testing
50				Bentonite		
52.5	Medium dense, saturated, dark gray, sandy, silty GRAVEL with trace organics	2-inch O.D. split spoon	S-11	Bentonite	33	
55				Bentonite		
57.5		3-inch O.D. Dames & Moore	S-12	Bentonite	23	
60				Bentonite		
62.5		2-inch O.D. split spoon	S-13	Bentonite	29	
65				Bentonite		
67.5	Grades to silty, sandy GRAVEL	2-inch O.D. split spoon	S-14	Bentonite	38	
70	(2' heave)	2-inch O.D. split spoon	S-15	Bentonite	21	
75						



Boring completed at 74 feet on 6/23/04
Groundwater observed at 15 feet at time of drilling
Groundwater measured at 5.4 feet on 7/7/04

Explanation

- 2-inch O.D. split spoon sample, 140 lb hammer with 30-inch free fall
- 3-inch O.D. Dames & Moore sample, 300 lb hammer with 30-inch free fall
- 3-inch I.D Shelby tube sample
- No Recovery
- ATD: Groundwater level at time of drilling or date of measurement

Monitoring Well Key
- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content
Plastic Limit — Natural — Liquid Limit

Testing Key
- GSA = Grain Size Analysis
- 200W = 200 Wash Analysis
- Att. = Atterberg Limits
- Con. = Consolidation Test

Zipper Zeman Associates, Inc.
Geotechnical and Environmental Consulting

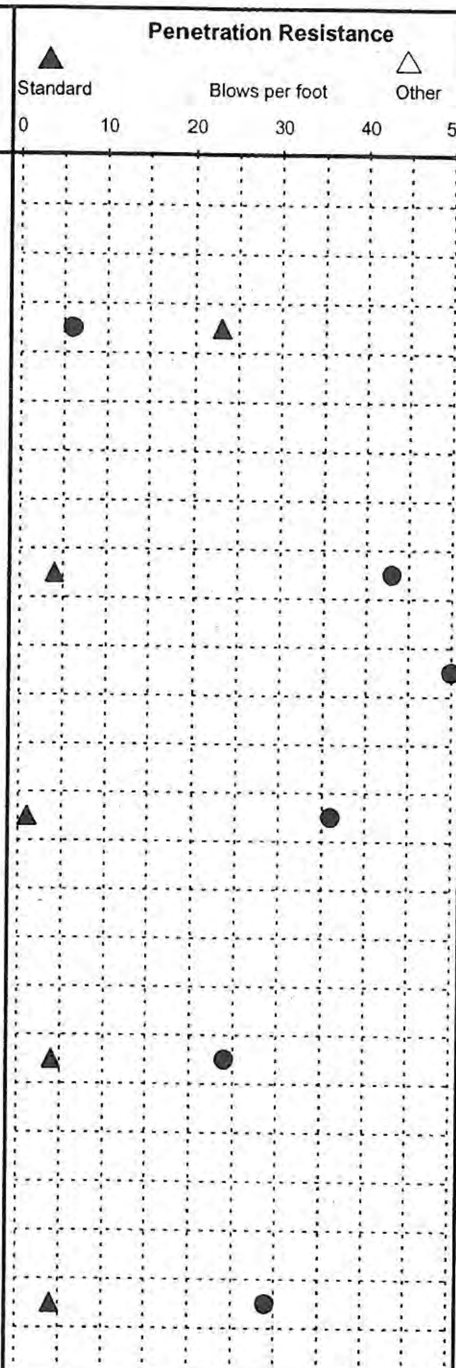
BORING LOG	Figure A-4
Date Drilled: 6/23/2004	Logged By: KTH

PROJECT: Issaquah Transit Center | **JOB NO.:** J-1875 | **BORING:** B-6 | **PAGE 1 OF 3**

Location: Issaquah, Washington | **Approximate Elevation:** 79 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	N-values	Testing
0	4 inches asphalt					
	Medium dense, moist, brown, silty, gravelly SAND (Fill)					
		2-inch O.D. split spoon	S-1		23	
5	Soft, wet, black, sandy, organic SILT					
	Soft, saturated, brown-gray, clayey SILT with some sand and trace organics and gravel			ATD		
		2-inch O.D. split spoon	S-2		4	
10		3-inch I.D Shelby tube	S-3			Con. Att.
	Grades to very soft, wet to saturated, orange-gray, sandy SILT with some clay and with trace fine organic material					
	Very soft, saturated, gray, sandy SILT to silty SAND with trace organics	2-inch O.D. split spoon	S-4		1	200W
15						
		2-inch O.D. split spoon	S-5		4	
20						
	Grades to soft, clayey SILT with some sand and fine organic material	2-inch O.D. split spoon	S-6		4	
25						

Penetration Resistance



Explanation

- 2-inch O.D. split spoon sample, 140 lb hammer with 30-inch free fall
- 3-inch O.D. Dames & Moore sample, 300 lb hammer with 30-inch free fall
- 3-inch I.D Shelby tube sample
- No Recovery
- ATD: Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

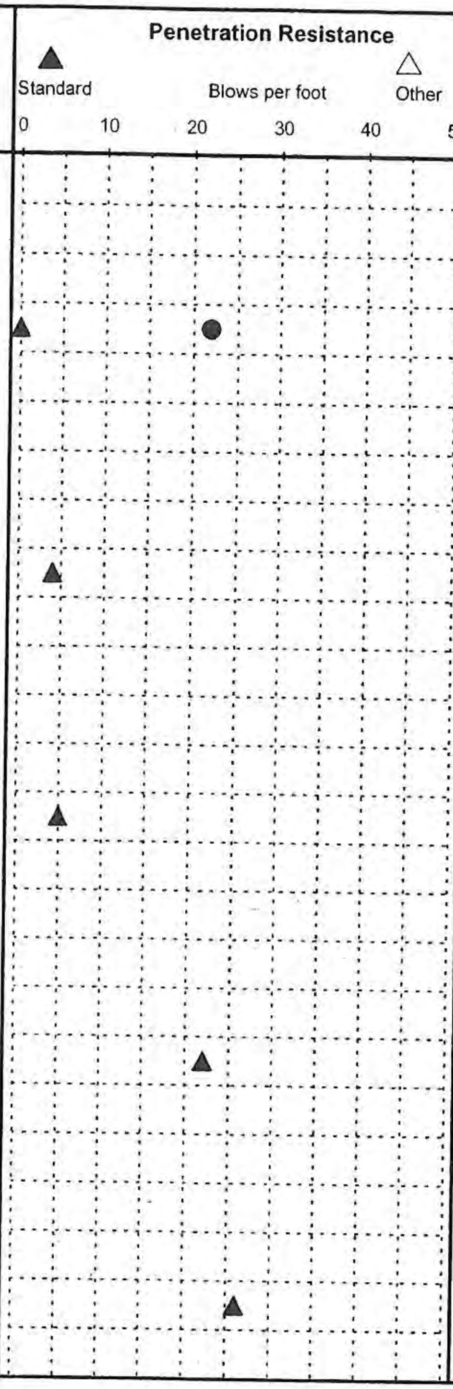
GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Con. = Consolidation Test

Zipper Zeman Associates, Inc.
Geotechnical and Environmental Consulting

BORING LOG	Figure A-6
Date Drilled: 6/24/2004	Logged By: KTH

PROJECT: Issaquah Transit Center	JOB NO.: J-1875	BORING: B-6	PAGE 2 OF 3
Location: Issaquah, Washington	Approximate Elevation: 79 Feet		

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (▲ Standard, △ Other; Blows per foot 0–50)	N-values	Testing
25							
	Very loose, saturated, dark gray, clayey, fine SAND with gravel lenses	I	S-7			0	200W
30							
	Grades to fine to medium SAND with trace organics	I	S-8			4	
35							
	Medium stiff, wet to saturated, dark gray, sandy SILT with 1/8-inch thick organic lenses and medium sand lens with trace gravel	I	S-9			5	
40							
	Medium dense, saturated, gray, sandy GRAVEL	I	S-10			22	
45							
	Grades to silty, gravelly, SAND with 2-inch thick silt lenses	I	S-11			26	
50							



Explanation

- I 2-inch O.D. split spoon sample, 140 lb hammer with 30-inch free fall
- I 3-inch O.D. Dames & Moore sample, 300 lb hammer with 30-inch free fall
- II 3-inch I.D Shelby tube sample
- ⊗ No Recovery
- ▼ ATD Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

- GSA = Grain Size Analysis
- 200W = 200 Wash Analysis
- Att. = Atterberg Limits
- Con. = Consolidation Test

Zipper Zeman Associates, Inc. Geotechnical and Environmental Consulting	BORING LOG	Figure A-6
	Date Drilled: 6/24/2004	Logged By: KTH

PROJECT: Issaquah Transit Center	JOB NO.: J-1875	BORING: B-6	PAGE 3 OF 3
Location: Issaquah, Washington	Approximate Elevation: 79 Feet		

Depth (ft)	Soil Description	Sample Number	N-values	Testing
50				
~52.5	Grades to dense silty, sandy GRAVEL (7 ' of heave flushed from augers)	S-12	40	
55				
~57.5	Grades to medium dense, silty, gravelly SAND	S-13	26	
60				
~62.5	Grades to sandy GRAVEL with some silt	S-14	27	
65				
~67.5	Grades to very dense, gravelly SAND with trace silt	S-15	74	
69	Boring completed at 69 feet on 6/24/04 Groundwater observed at 7 feet at time of drilling			
70				
75				

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

Explanation

- 2-inch O.D. split spoon sample, 140 lb hammer with 30-inch free fall
- 3-inch O.D. Dames & Moore sample, 300 lb hammer with 30-inch free fall
- 3-inch I.D Shelby tube sample
- ⊗ No Recovery
- ▼ ATD Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

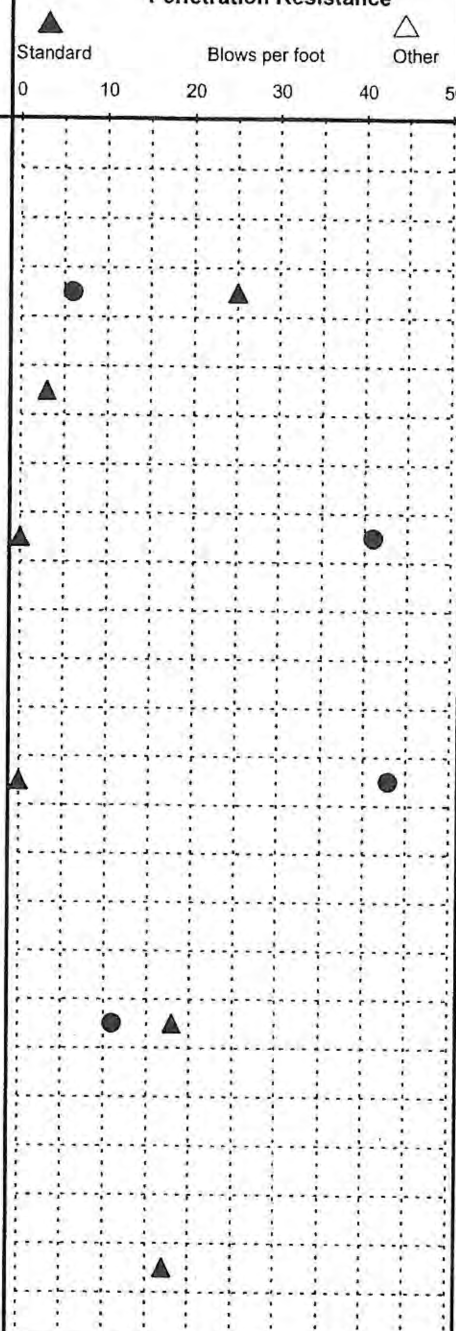
- GSA = Grain Size Analysis
- 200W = 200 Wash Analysis
- Att. = Atterberg Limits
- Con. = Consolidation Test

PROJECT: Issaquah Transit Center | **JOB NO.:** J-1875 | **BORING:** B-8 |

Location: Issaquah, Washington | **Approximate Elevation:** 77 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	N-values	Testing
0	3 inches asphalt					
	Medium dense, moist, brown, sandy GRAVEL with some silt (Fill)	2-inch O.D. split spoon	S-1		25	GSA
5	Soft, wet, black, sandy, organic SILT	3-inch O.D. Dames & Moore	S-2		3	
	Very soft, wet, brown-gray, clayey SILT with trace to some sand and organics	2-inch O.D. split spoon	S-3		0	
10						
	Grades to saturated, gray, sandy, clayey SILT with trace organics	2-inch O.D. split spoon	S-4	▼ ATD	0	
15						
	Medium dense, saturated, gray, silty, sandy GRAVEL	2-inch O.D. split spoon	S-5		18	
20						
	Grades to sandy GRAVEL with trace silt	2-inch O.D. split spoon	S-6		17	
25						

Penetration Resistance



Explanation

- 2-inch O.D. split spoon sample, 140 lb hammer with 30-inch free fall
- 3-inch O.D. Dames & Moore sample, 300 lb hammer with 30-inch free fall
- 3-inch I.D Shelby tube sample
- ⊗ No Recovery
- ▼ ATD Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

- GSA = Grain Size Analysis
- 200W = 200 Wash Analysis
- Att. = Atterberg Limits
- Con. = Consolidation Test

Location: Issaquah, Washington **Approximate Elevation:** 77 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (Standard ▲ / Other △, Blows per foot 0–50)	N-values	Testing
25							
	Soft, saturated, gray, sandy SILT with some clay and organics	I	S-7		▲	4	
30							
	Loose, saturated, dark gray, clayey, fine SAND with gravel and organics	I	S-8		▲	7	
35							
	Soft, saturated, gray, sandy SILT with organics	I	S-9		▲	5	
40							
	Very loose, saturated, gray, silty, gravelly, fine to medium SAND with organics	I	S-10		▲	2	
45							
	Dense, saturated, gray, silty, sandy GRAVEL to gravelly SAND	I	S-11		▲	31	
50							

Explanation

- 2-inch O.D. split spoon sample, 140 lb hammer with 30-inch free fall
- 3-inch O.D. Dames & Moore sample, 300 lb hammer with 30-inch free fall
- 3-inch I.D Shelby tube sample
- ⊗ No Recovery
- ▼ ATD Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Con. = Consolidation Test

Location: Issaquah, Washington

Approximate Elevation: 77 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (Blows per foot)	N-values	Testing
50							
~53	Grades to sandy GRAVEL with trace silt	2-inch O.D. split spoon	S-12		▲ 40	40	
55							
~58	Grades to medium dense, gravelly SAND with trace silt and organics	3-inch O.D. Dames & Moore	S-13		▲ 26	26	
59	Boring completed at 59 feet on 06/25/04 Groundwater observed at 14 feet at time of drilling						
60							
65							
70							
75							

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

Explanation

- 2-inch O.D. split spoon sample, 140 lb hammer with 30-inch free fall
- 3-inch O.D. Dames & Moore sample, 300 lb hammer with 30-inch free fall
- 3-inch I.D Shelby tube sample
- ⊗ No Recovery
- ▼ ATD: Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

- GSA = Grain Size Analysis
- 200W = 200 Wash Analysis
- Att. = Atterberg Limits
- Con. = Consolidation Test

PROJECT: Issaquah Transit Center | **JOB NO.:** J-1875 | **BORING:** B-13 | **PAGE 1 OF 1**

Location: Issaquah, Washington | **Approximate Elevation:** 69.5 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (Standard, Blows per foot, 0–50)	N-values	Testing
0	Loose, moist, brown, silty SAND. (Fill)						
	Grades to medium dense, damp, brown, gravelly SAND.	I	S-1		▲	16	
5	Very loose, wet, mottled gray, silty SAND with some clay.	I	S-2		▲	3	
10	Very soft, moist, gray, silty CLAY. (PP = 0.0-0.25 tsf)	I	S-3		▲	2	
15	Grades to silty CLAY to sandy, silty CLAY (PP = 0.0 tsf)	I	S-4		▲	2	
	Very loose, moist to wet, gray, silty SAND.						
20	Medium dense, saturated, gray, sandy GRAVEL.	I	S-5	▼ ATD	▲	12	
	Boring completed at 21.5 feet on 5/15/05. Groundwater observed at 20.0 feet at time of drilling						
25							

Explanation

- I — 2-inch O.D. split spoon sample, 140 lb hammer with 30-inch free fall
- I — 3-inch O.D. Dames & Moore sample, 300 lb hammer with 30-inch free fall
- II — 3-inch I.D Shelby tube sample
- ⊗ — No Recovery
- ▼ ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

- GSA = Grain Size Analysis
- 200W = 200 Wash Analysis
- Att. = Atterberg Limits
- Con. = Consolidation Test
- PP = Pocket Penetrometer

ZZA Zipper Zeman Associates, Inc.
Geotechnical and Environmental Consulting

BORING LOG	Figure A-13
Date Drilled: 5/15/2005	Logged By: BAG

PROJECT: Issaquah Transit Center **JOB NO.:** J-1875 **BORING:** B-14 **PAGE 1 OF 1**

Location: Issaquah, Washington **Approximate Elevation:** 79 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	N-values	Testing
0	4 inches asphalt over loose to medium dense, damp, brown, sandy GRAVEL with trace silt. (Fill)					
	Grades to very dense.	2-inch O.D. split spoon	S-1		59	
5	(gravelly drilling action)	2-inch O.D. split spoon, No Recovery	S-2		6	
	Very loose, moist to wet, mottled gray-brown, fine to medium silty SAND with 2 inch thick lense of sandy, clayey SILT.					
10		2-inch O.D. split spoon	S-3		3	
15	Grades to loose, saturated, gray, clayey SAND with 2 inch thick sandy, clayey silt lense. (PP = 0.0 tsf)	2-inch O.D. split spoon	S-4	ATD	5	
20	Grades to very loose, saturated, gray, silty SAND.	2-inch O.D. split spoon	S-5		3	
	Very soft, saturated, gray, sandy, clayey SILT.					
	Very loose, saturated, gray, silty SAND.					
	Boring completed at 21.5 feet on 5/15/05. Groundwater observed at 15.0 feet at time of driling.					
25						

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

Explanation

- 2-inch O.D. split spoon sample, 140 lb hammer with 30-inch free fall
- 3-inch O.D. Dames & Moore sample, 300 lb hammer with 30-inch free fall
- 3-inch I.D Shelby tube sample
- ⊗ No Recovery
- ▼ ATD Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

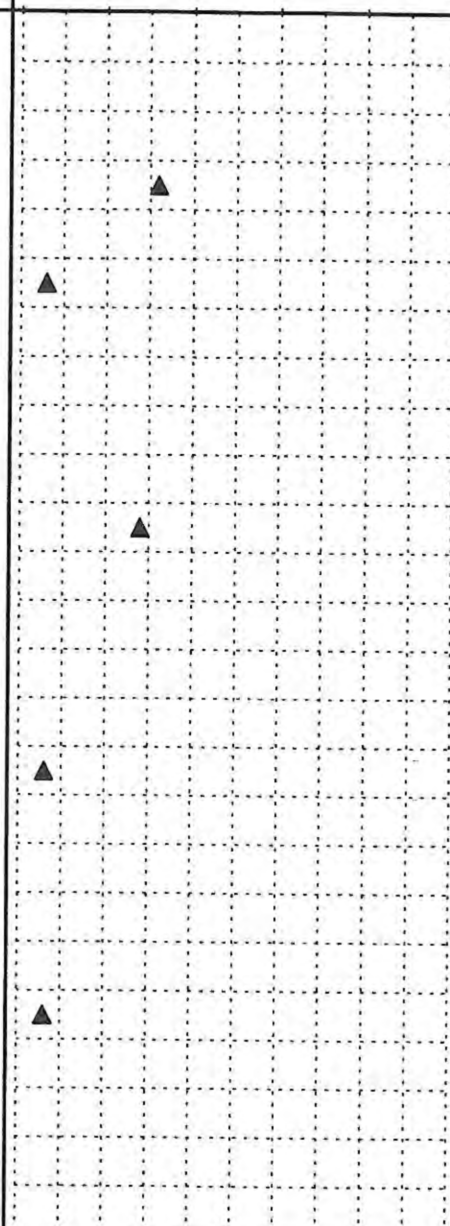
GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Con. = Consolidation Test
PP = Pocket Penetrometer

PROJECT: Issaquah Transit Center **JOB NO.:** J-1875 **BORING:** B-15 **PAGE 1 OF 1**

Location: Issaquah, Washington **Approximate Elevation:** 70 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	N-values	Testing
0	4-3/4 inches concrete over loose to medium dense, damp, brown, sandy GRAVEL with trace silt. (Fill)					
	Grades to medium dense, moist, brown, gravelly SAND.	I	S-1		16	
5	Very loose, wet, mottled gray, silty SAND with sandy, clayey SILT interbeds. (PP - 0.0 tsf)	I	S-2		3	
10	Grades to medium dense, saturated, gray, gravelly SAND with some silt and a 6" thick clayey SILT interbed. (PP = 0.0 tsf)	I	S-3	▼ ATD	14	
15	Grades to very loose silty SAND.	I	S-4		3	
20	Grades to silty SAND interbedded with very soft, saturated, gray, sandy SILT. (PP = 0.0 tsf)	I	S-5		3	
	Boring completed at 21.5 feet on 5/15/05. Groundwater observed at 10.0 feet at time of drilling.					
25						

Penetration Resistance



Explanation

- I 2-inch O.D. split spoon sample, 140 lb hammer with 30-inch free fall
- I 3-inch O.D. Dames & Moore sample, 300 lb hammer with 30-inch free fall
- II 3-inch I.D Shelby tube sample
- ⊗ No Recovery
- ▼ ATD Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Con. = Consolidation Test
PP = Pocket Penetrometer

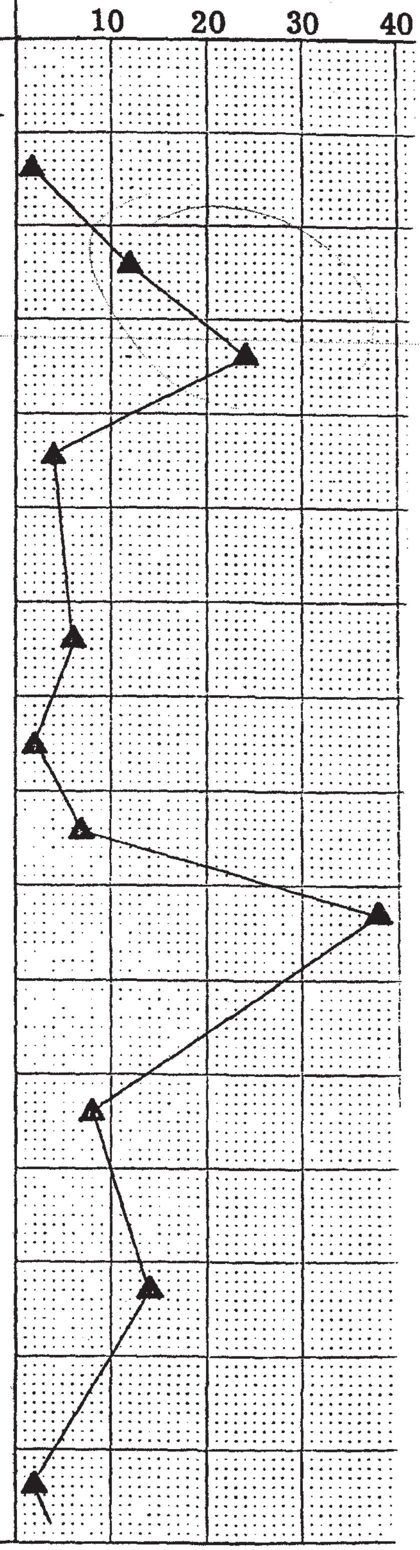
Zipper Zeman Associates, Inc.
Geotechnical and Environmental Consulting

BORING LOG | **Figure A-15**

Date Drilled: 5/15/2005 | **Logged By: BAG**

DEPTH, FEET	SOIL DESCRIPTION Surface Elevation:	SAMPLES	GROUNDWATER
	Sod over topsoil		∇
5	Very loose to loose, saturated, brown, clayey, silty, fine to coarse sand with a trace of gravel	I I	
	Medium dense, saturated, gray, sandy gravel	I	
10	Soft to medium stiff, wet, gray, clayay silt with interbedded sand below 15 feet	I II I I	
15			
20	Loose to medium dense, saturated, gray, silty, fine sand	I	
	Dense, saturated, gray, sandy gravel	I	
25			
30	Medium stiff to very stiff, saturated, gray, clayey, sandy silt with occasional gravel	I I I	
35			
40			

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



● % Water Content

LEGEND

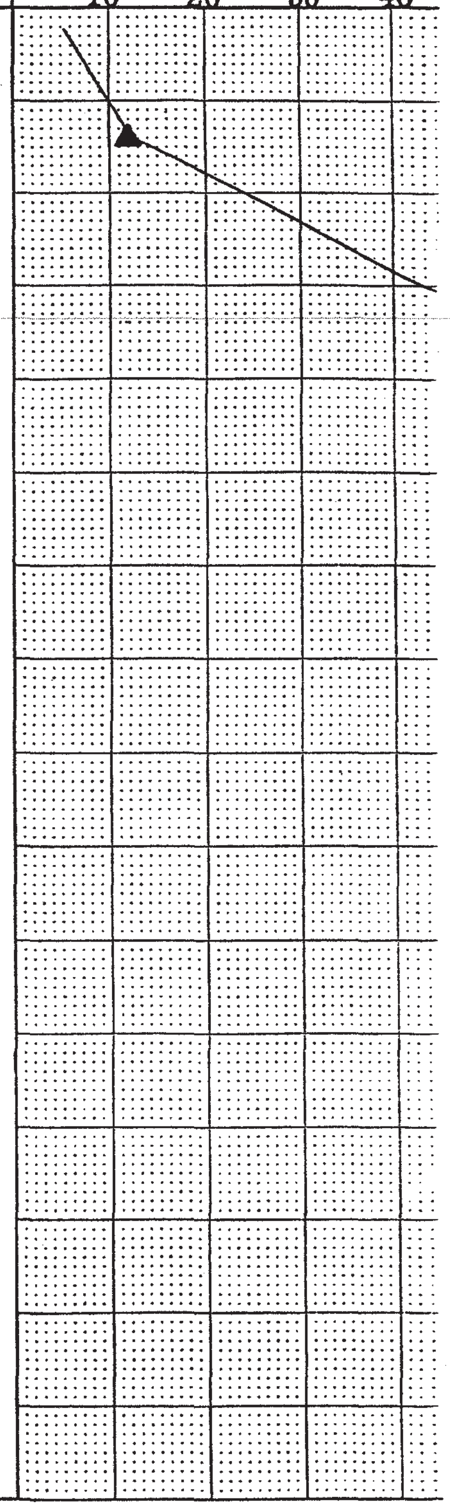
I 2.0" O.D. split spoon sampler
II 3.0" O.D. undisturbed sampler
P Sampler pushed
Atterberg limits: ├─●─┤ ← Liquid limit; Natural water content; Plastic Limit
• Sample not recovered
▮ Piezometer tip
∇ Water level

W-3240

LOG OF BORING NO. 1

RITTENHOUSE-ZEMAN & A
SOILS ENGINEERING AND GEOLOGY

DEPTH, FEET	SOIL DESCRIPTION Surface Elevation:	SAMPLES	GROUNDWATER	STANDARD PENETRATION RESISTANCE ▲ Blows per foot (140 lb. hammer, 30" drop) 10 20 30 40
	As above			
	Medium dense, moist, gray, silty, fine sand bedded with silt and gravelly sand	I		
45				
	Dense, saturated, gray, gravelly, fine to coarse sand	I		
50				
	4 feet of heave at 52.5 feet			
	Boring terminated at 52.5 feet			
55				
60				
65				
70				
75				
80				



● % Water Content

LEGEND

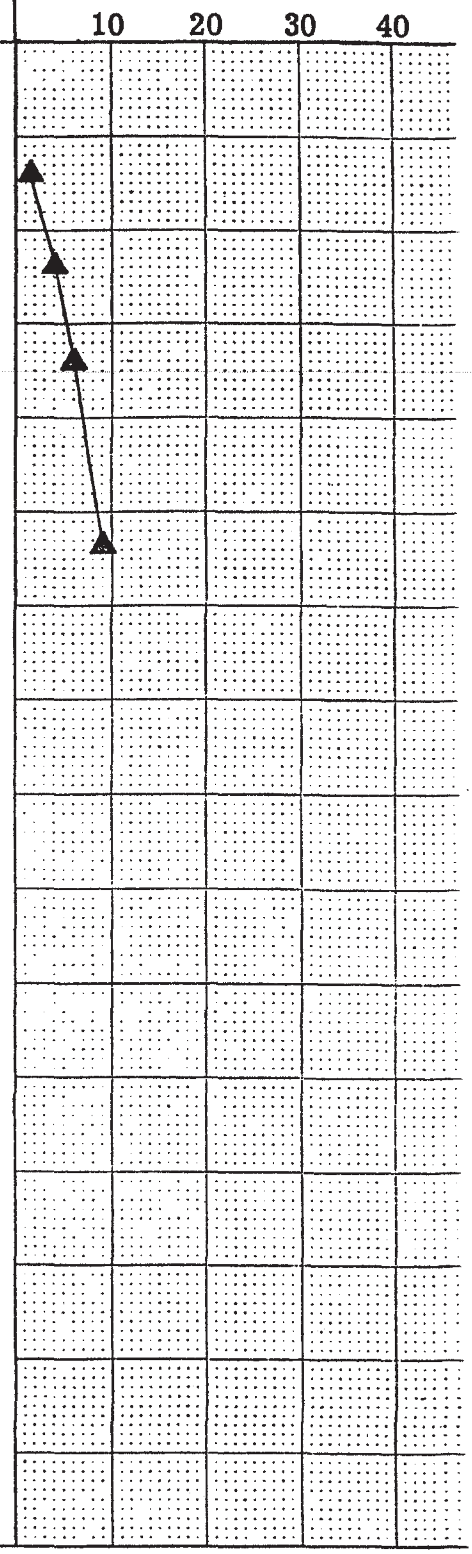
I 2.0" O.D. split spoon sampler
II 3.0" O.D. undisturbed sampler
P Sampler pushed
* Sample not recovered
▮ Piezometer tip
∇ Water level
Atterberg limits: ├─●─┤ ← Liquid limit; Natural water content; Plastic Limit

W-3240
LOG OF BORING NO. 1 (con't.

RITTENHOUSE-ZEMAN & ASS
SOILS ENGINEERING AND GEOLOGY

DEPTH, FEET	SOIL DESCRIPTION Surface Elevation:	SAMPLES	GROUNDWATER
	Sod over topsoil		
	Very soft to medium stiff, wet, mottled brown to gray, sandy, clayey silt	I	▽
5		I	
		I	
10			
	Loose, saturated, sandy gravel (driller's interpretation)	I *	
15	Boring terminated at 14.0 feet		
20			
25			
30			
35			
40			

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



● % Water Content

LEGEND

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

W-3240

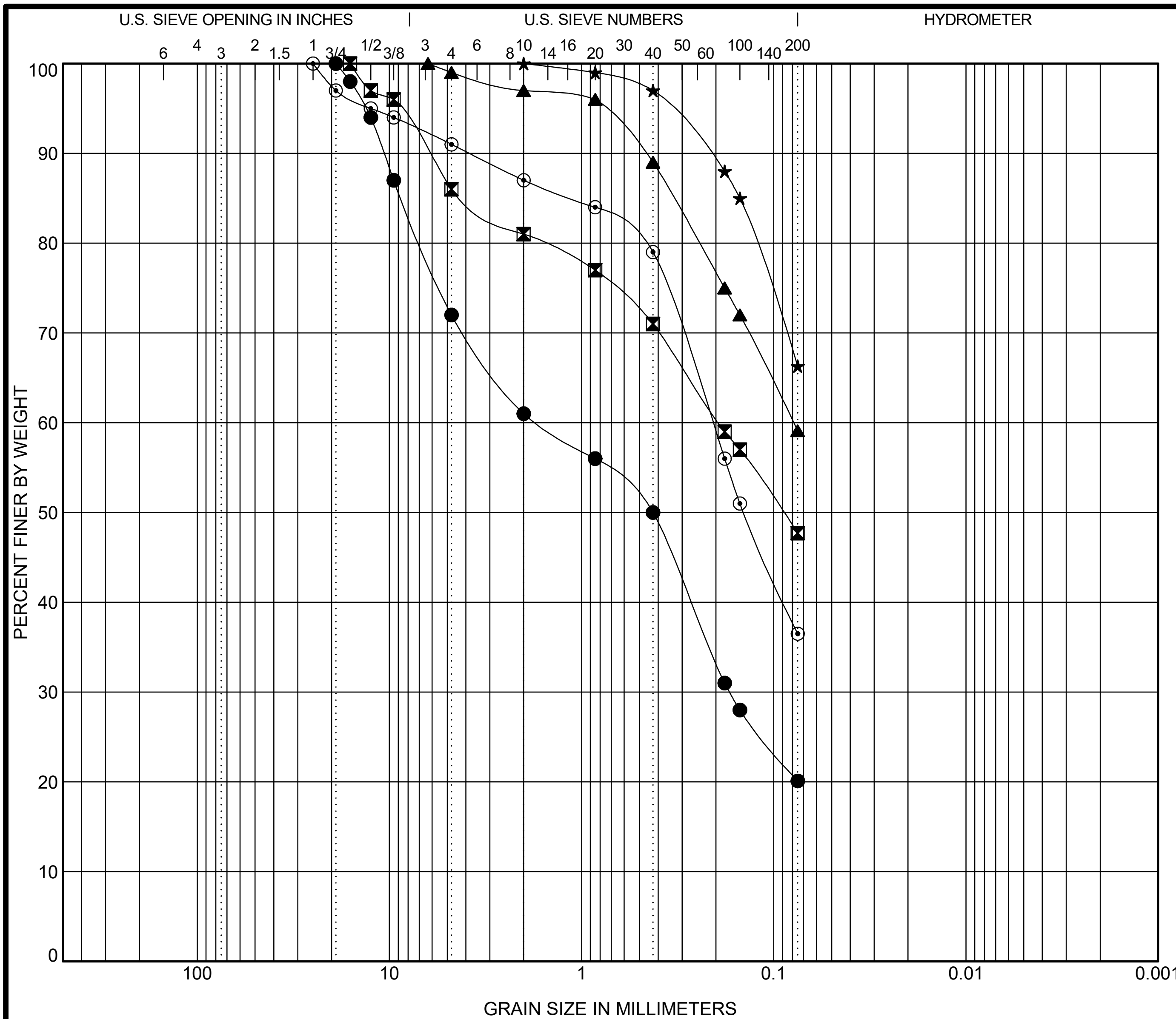
LOG OF BORING NO. 2

RITTENHOUSE-ZEMAN & ASSOC

SOILS ENGINEERING AND GEOLOGY

APPENDIX C

LABORATORY TEST RESULTS



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification		Classification	LL	PL	PI	Cc	Cu
● PG-1	@ 7.5 ft.	**SILTY SAND with GRAVEL(SM)**	NP	NP	NP		
⧗ PG-1	@ 15.0 ft.	**SILTY SAND(SM)**	NP	NP	NP		
▲ PG-3	@ 5.0 ft.	**SANDY SILT(ML)**	NP	NP	NP		
★ PG-3	@ 10.0 ft.	**SANDY SILT(ML)**	NP	NP	NP		
⊙ PG-4	@ 7.5 ft.	**SILTY SAND(SM)**	NP	NP	NP		

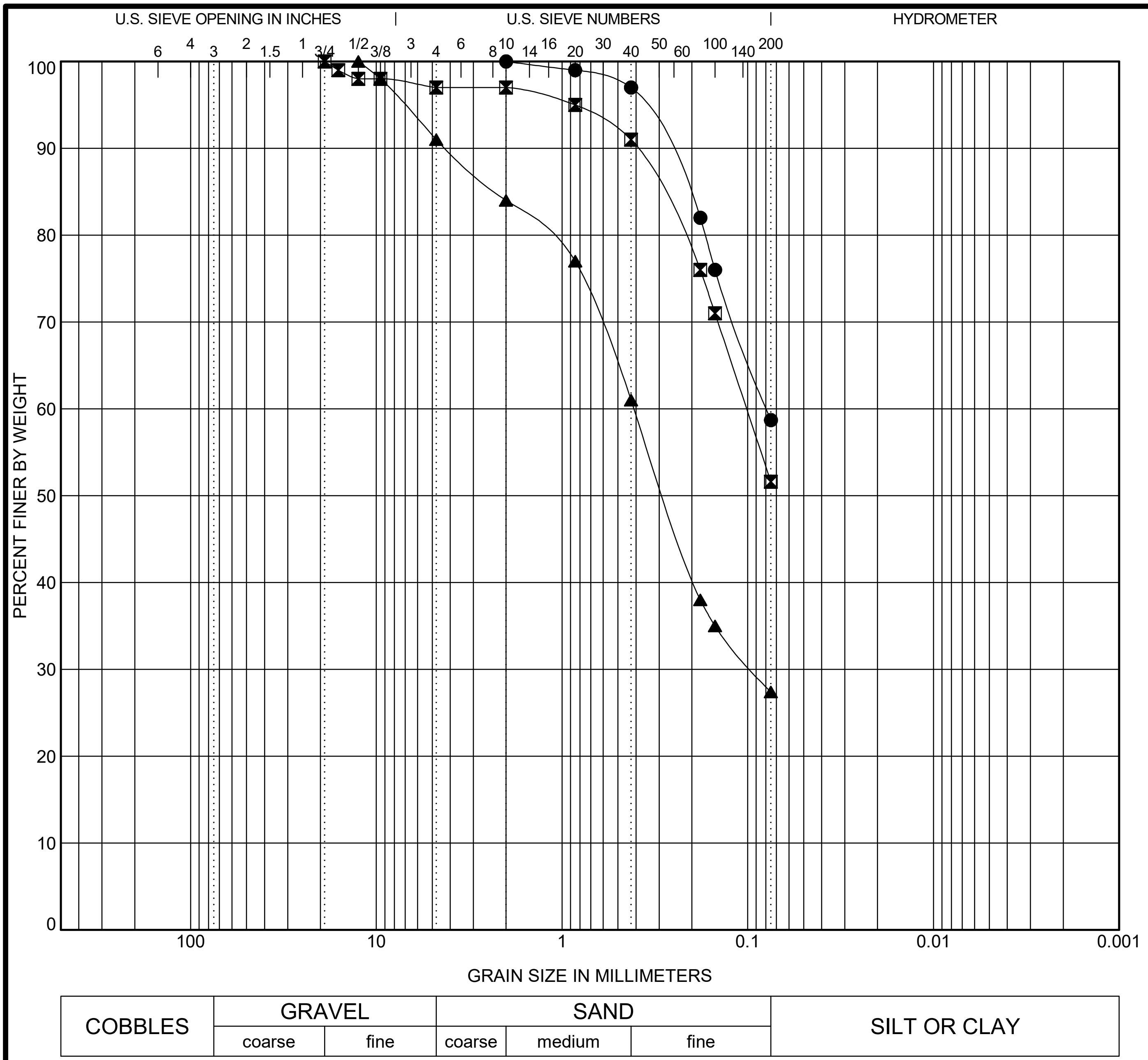
Specimen Identification		D100	D90	D60	D10	%Gravel	%Sand	%Silt	%Clay
● PG-1	7.5	19	10.686	1.685		28.0	51.9	20.1	
⧗ PG-1	15.0	16	6.268	0.193		14.0	38.3	47.7	
▲ PG-3	5.0	6.3	0.469	0.079		1.0	39.9	59.1	
★ PG-3	10.0	2	0.218			0.0	33.7	66.3	
⊙ PG-4	7.5	25	3.826	0.209		9.0	54.5	36.5	

GRAIN SIZE DISTRIBUTION

PanGEO INCORPORATED
Phone: 206.262.0370

Project: Trailhead Apartments
Job Number: 24-484
Location: 1550 Newport Way NW, Issaquah, WA

Figure B-1



	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	**PG-4**	**@ 10.0 ft.**	**SANDY SILT(ML)**	**NP**	**NP**	**NP**		
⊠	**PG-5**	**@ 7.5 ft.**	**SANDY SILT(ML)**	**NP**	**NP**	**NP**		
▲	**PG-5**	**@ 15.0 ft.**	**SILTY SAND(SM)**	**NP**	**NP**	**NP**		

	Specimen Identification		D100	D90	D60	D10	%Gravel	%Sand	%Silt	%Clay
●	**PG-4**	**10.0**	**2**	**0.285**	**0.079**		**0.0**	**41.3**	**58.7**	
⊠	**PG-5**	**7.5**	**19**	**0.401**	**0.101**		**3.0**	**45.4**	**51.6**	
▲	**PG-5**	**15.0**	**12.5**	**4.198**	**0.409**		**9.0**	**63.6**	**27.4**	

PanGEO INCORPORATED
Phone: 206.262.0370

GRAIN SIZE DISTRIBUTION

Project: Trailhead Apartments
Job Number: 24-484
Location: 1550 Newport Way NW, Issaquah, WA

Figure B-2

APPENDIX D

GEOPHYSICAL SURVEY REPORT

Report on 1D MAM, MASW, and HVSR Survey

Near 1505 Newport Way NW, Issaquah, WA 98027

Global Geophysics Project No. 114-1230.000

Prepared for
Siew L. Tan, P.E.
PanGEO, Inc.
3213 Eastlake Avenue East, Suite B
Seattle, WA 98102

Prepared by
Global Geophysics LLC
P.O. Box 2229
Redmond, WA 98073-2229

January 2, 2025

CONTENTS

1. OBJECTIVE

Global Geophysics, LLC conducted one passive local MAM survey, one passive regional MAM survey, one MASW survey, and one HVSR survey near 1505 Newport Way NW, Issaquah, WA 98027 in the evening of December 30th 2024. The goal of this investigation was to determine the average shear wave velocity profile from near surface to the base layer of 2500 ft/s. This report provides the methods, instrumentation, data collection and processing procedures, results, and analysis of this investigation.

2. INTRODUCTION

Surface waves are a special type of seismic wave whose propagation is confined to the near surface medium. The depth of subsurface penetration of a surface wave is directly proportional to its wavelength. In a non-homogeneous medium, surface waves are dispersive, i.e. each wavelength has a characteristic velocity stemming from subsurface variations in the soils and rocks. The velocity that the surface waves' wavelengths propagate through the subsurface is related to the shear wave (S-Wave) velocity of the subsurface. If the S-Wave velocity varies with depth, so will the surface wave's wavelength velocity. Analysis of how the wavelength varies, or dispersion, allows us to estimate the S-Wave velocity as it passes through the subsurface. The S-Wave velocity of the subsurface can then be used to infer useful characteristics such as the rock/soil type, stratigraphy, and soil conditions.

Average S-Wave velocities to a depth of 100ft (30m) are known as V_{S100} (V_{S30}) and are sorted into classes by the International Building Code (IBC) to provide valuable earthquake engineering design information. These classes are shown here:

Class Name	**Ground Description**	**V_{S100}**	**V_{S30}**
A	Hard Rock	>5000ft/s	>1500m/s
B	Rock	5000ft/s to 2500ft/s	1500m/s to 760m/s
C	Dense Soil or Soft Rock	2500ft/s to 1200ft/s	760m/s to 360m/s
D	Stiff Soil	1200ft/s to 600ft/s	360m/s to 180m/s
E	Soft Soil	<600ft/s	<360m/s
F	Needs site specific evaluation	NA	NA

Surface waves can be utilized in both active and passive deployments. Multichannel Analysis of Surface Waves (MASW) comprises most active deployments while Microtremor Array Measurements (MAM) are the primary method to collect passive data. MASW arrays are typically linear while MAM arrays can be linear (often known as refraction microtremor, or ReMi, when linear) but generally perform better when deployed in 2D orientations (triangular, circular, T-shaped, or L-shaped arrays). Another passive method employed is the Horizontal over Vertical Spectral Ratio (HVSR) which utilizes a single geophone sensitive to motion in three directions (vertical, east-west, and north-south).

For this project, seismic surveys were deployed at each of the locations shown in Figure 1.

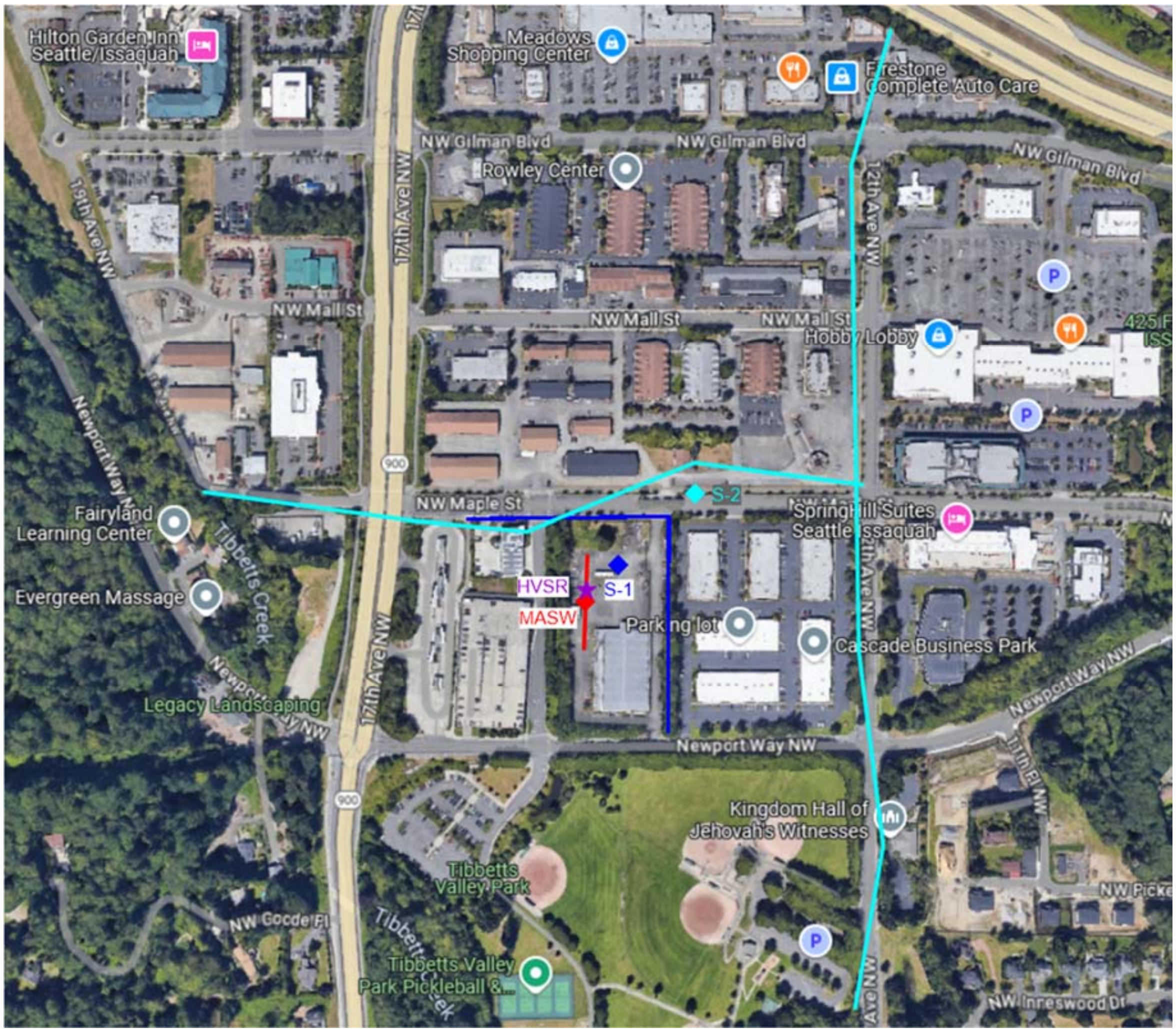


Figure 1. Site Plan

3. METHODOLOGY AND INSTRUMENTATION

3.1 Microtremor Array Measurements (MAM) Method

A detailed description of the MAM method can be found in Okada, 2003. MAM arrays generally have a greater degree of flexibility with their design and in addition to linear arrays, can be deployed in 2D arrays such as the circular, triangular, T and L arrays. Since this is a passive survey, the ambient vibrations of the surroundings are utilized rather than deliberately generated. These passive sources can come from all directions and include traffic, ocean waves, cultural noise, and construction. MAM arrays should utilize an array size equal to or greater than the depth of investigation (Geometrics, 2009) and record the ambient vibrations for a minimum of 30 seconds and collect a minimum of 10 minutes of data.

Wireless sensors, such as the SmartSolo IGU-16 or the Geometrics Atom enable passive arrays to be deployed to sizes much larger than wired arrays. Wireless MAM arrays can span several hundred, or even several thousand feet; enabling much deeper investigations into the subsurface.

3.2 Multichannel Analysis of Surface Waves (MASW) Method

A detailed description of the MASW method is given by Park, 1999b. Typically, an MASW deployment contains a linear array of at least 24 geophones spaced 5 to 10ft apart and connected to a seismograph. The MASW method is an active survey, meaning that seismic waves are intentionally generated to be recorded by the array. Common sources of seismic waves for shallow investigations are various sized hammers and accelerated weight drops from vehicle-mounted devices. MASW arrays should be twice the length of the depth of interest due to surface waves sampling to a depth of half of their wavelength (Geometrics, 2009). However, when combined with passive arrays (which have a deeper sampling depth), the array may be designed to be shorter.

Data is collected by generating a seismic wave (shot) at a known location along the array and recording the response of each geophone as the seismic waves arrive with the seismograph. A dispersion curve is generated from the data and then inverted to create a 1D profile of the subsurface located at the center of the array (Park, Miller, Xia, & Ivanov, 2007). A single 1D profile survey is sometimes referred to as an active source ReMi. When multiple shots are made at set intervals along the seismic array, the subsequent 1D profiles can be interpolated to create a 2D profile along the length of the array. Off-end shots at either end of the array at around 10 – 20% and 40% of the array length are also collected when possible, although space constraints can limit the collection of off-end shots.

3.3 HVSR

The H/V spectral ratio (i.e. the ratio between the Fourier amplitude spectra of the horizontal and the vertical component of ambient noise vibrations recorded at one single station) was first introduced by Nogoshi & Igarashi (1971), and widespread by Nakamura (1989, 1996, 2000). Its inversion process includes the models from surface wave methods to model the shear wave velocities and depths to largest impedance contrast (i.e. soil/rock).

3.4 Surface Wave Dispersion Curve Modeling

Dispersion curves are useful for determining S-Wave velocities of the subsurface and are generated with the help of specialized software. Data files are added to the software and their traces displayed by location versus time, showing the seismic waves that arrive at each geophone over the course of the record.

For MAM surveys, the data are transformed with a fast Fourier transform to the frequency domain. Then the coherence (or similarity between traces or waveforms) is calculated. If the coherences are averaged over a long period of time or over many data blocks, the data are considered to be Spatially Auto-Correlated (SPAC) ((Aki, 1957). From here, the phase velocity can be calculated from each frequency and fundamental and higher modes can be picked. From the fundamental mode, the dispersion curve can be created and edited (Roesset, 1991). The dispersion curve is used to create an inversion model that displays the S-Wave velocities at the desired range of depths (Xia, Miller, & Park, 1999). Theoretical dispersion curves are generated via a matrix method (Saito & Kabasawa, 1993) and compared against the observed dispersion curve. The model is updated until the observed and theoretical dispersion curves converge. The resulting model is the delivered S-Wave velocity model for the array.

4. INSTRUMENTS AND EQUIPMENT

4.1 MAM

For this investigation, Global Geophysics used 24 wireless Geometrics Atom seismographs and 2 Hz Sunfull geophones at varied spacings. MAM data were collected using ambient seismic waves recorded from the surroundings. Data processing was done with Geometrics SeisImager software package.

Figure 2. Seismograph and Geophone

4.2 MASW

For this investigation, Global Geophysics used a 24-channel array of 4.5 Hz geophones at a spacing of 10 ft connected to a Geometrics Geode seismograph. Seismic waves were generated with a 20lb. sledgehammer. Data recording was triggered by an accelerometer attached to the hammer and data were recorded through Geometrics Seismodule Controller. Data processing was done with Geometrics SeisImager software package.

Figure 3. Geometrics Geode Seismograph

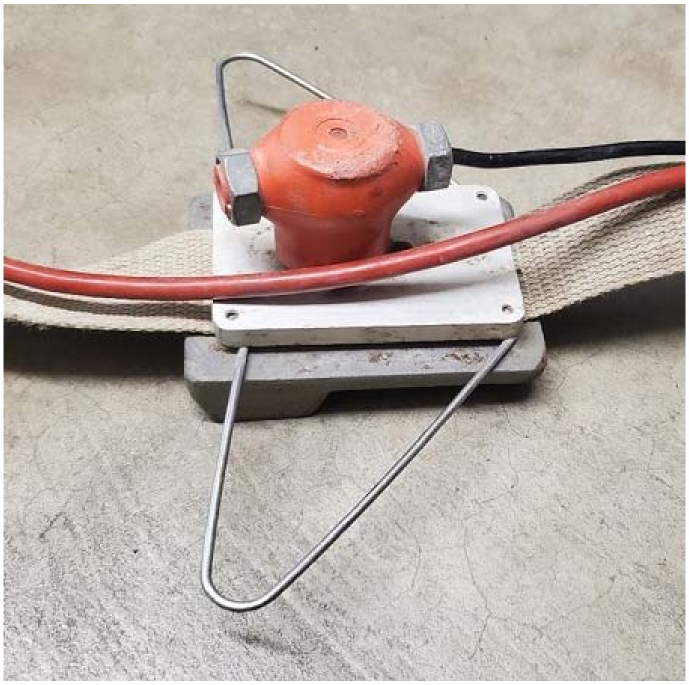


Figure 4. Land streamer geophone

4.3 HVSR

A Trillium tri-axial sensors was also used to obtain H/V ratios (Figure 5).

Figure 5. Nanometrics Trillium and Centaur

5. PROCEDURES

5.1 Field Deployment

MAM Sounding

The Local MAM Sounding, S-1, was centered near 1505 Newport Way NW, Issaquah, WA 98027 in the Parking lot near the Radio Tower. The MAM sounding was a modified L-array that was 525 ft by 570 ft. Sensor deployment can be found in Figure 7. Data were recorded for over 120 min.

The Regional MAM Sounding, S-2, was a modified T-Array that was roughly 2,700 ft by 1800 ft. Sensor Deployment can be found in Figure 8. Data were recorded for over 90 min.

1D MASW Sounding

The 1D MASW data were collected using passive and active source. The linear array was 230 feet in length with a sensor spacing of 10 feet. Sensor deployment for the sounding can be seen in Figure 6.

HVSR

The sensor was leveled and oriented to the north at 47.542263, -122.060655. After the unit was turned on, it collected data continuously for over 1 hour.

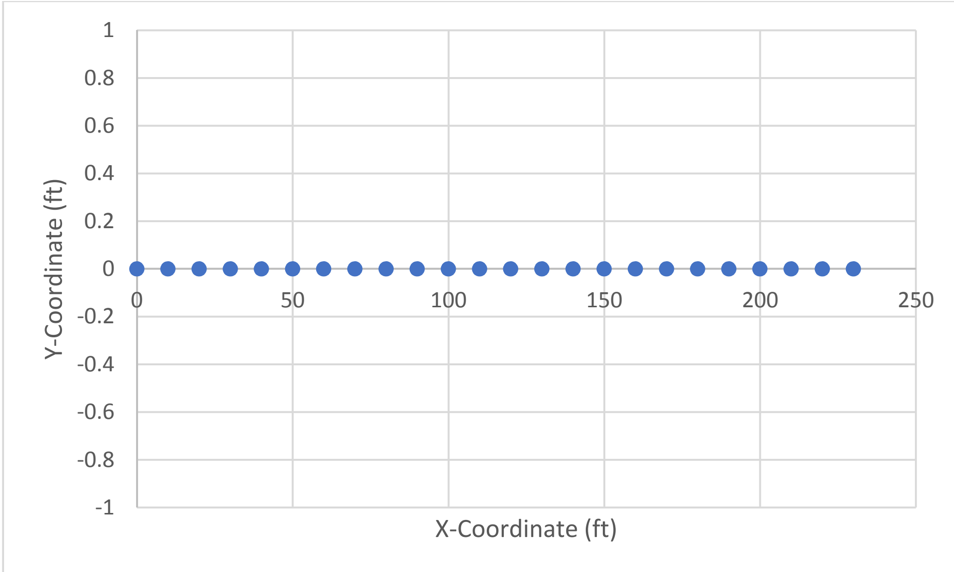


Figure 6. MASW Sounding Sensor Deployment

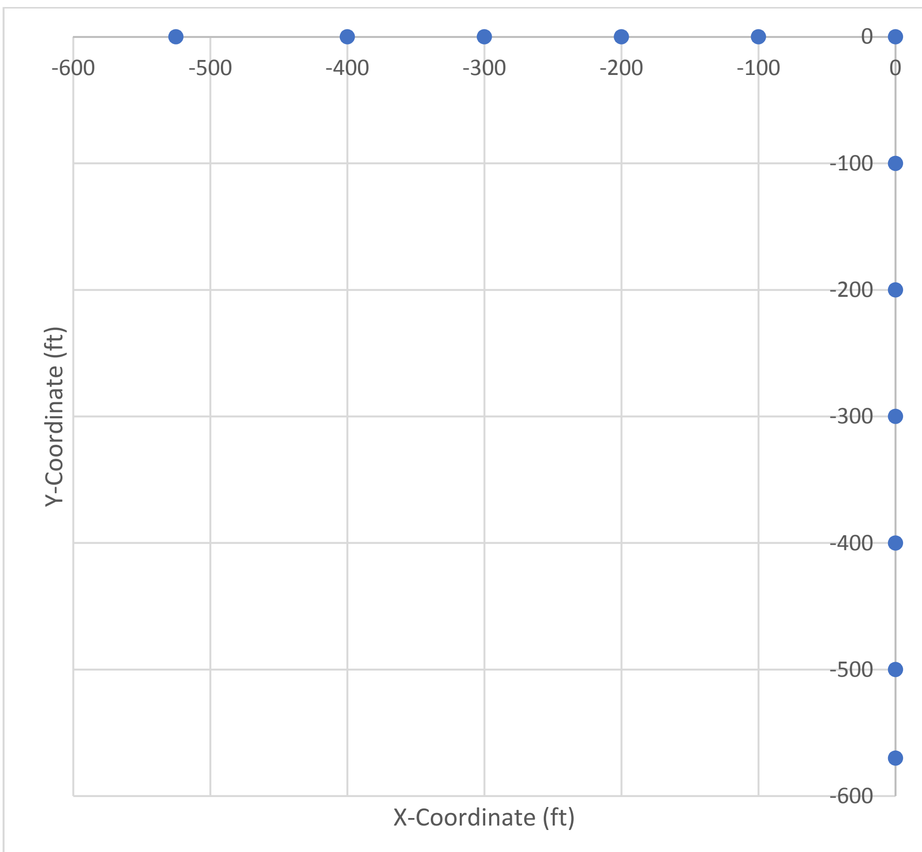


Figure 7. S-1 Sounding Wireless Sensor Deployment

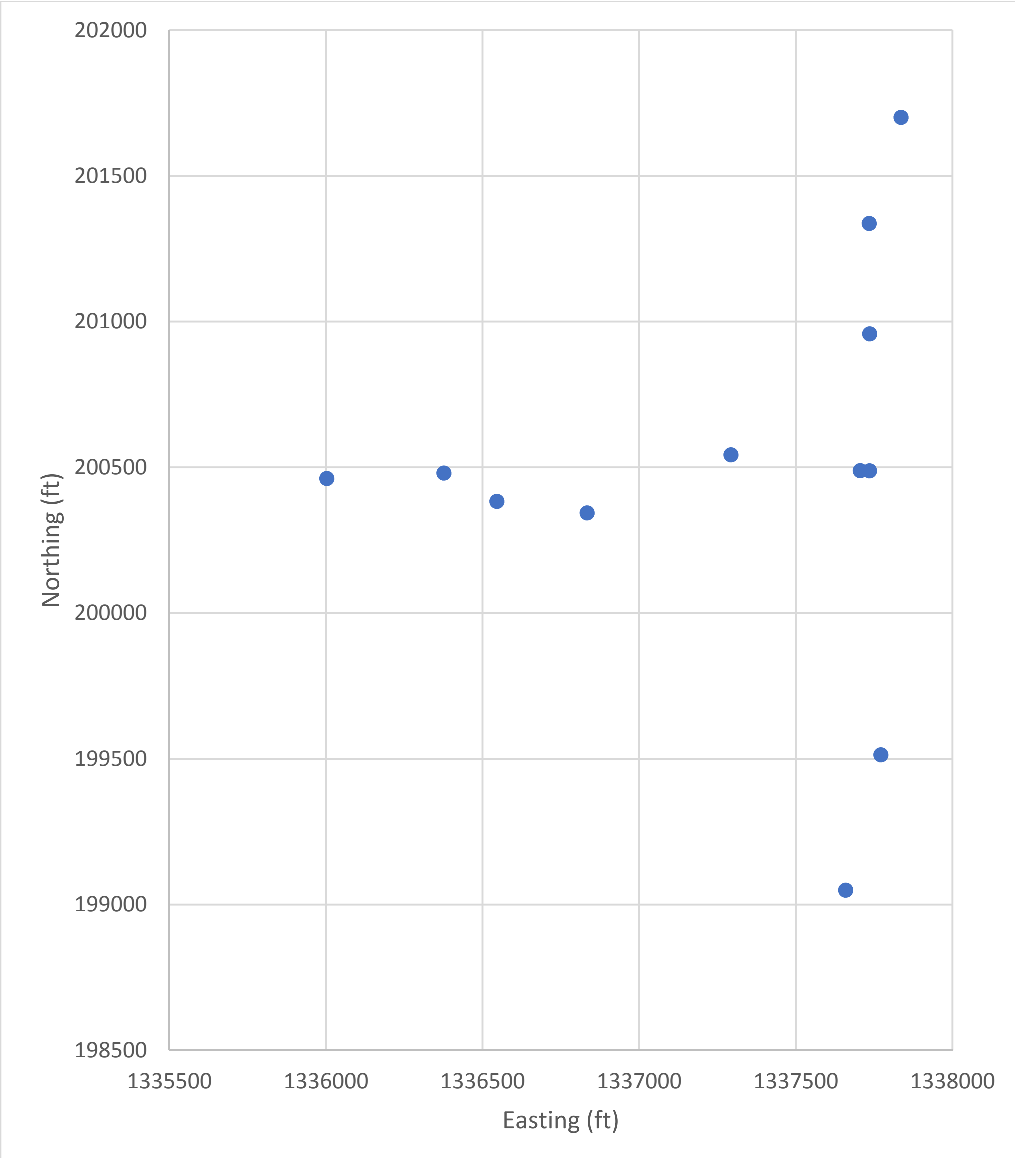


Figure 8. S-2 Sounding Wireless Sensor Deployment

5.2 Quality Control and Data Processing

5.2.1 Quality Control
Wireless MAM Arrays

Data were recorded for over 60 minutes to record enough vibrations at various frequencies.

2D MASW Array

Data were recorded for 10 minutes to record enough vibrations at various frequencies. A 20 lb sledge hammer was used to generate an active source signal.

5.2.2 Data Processing
Wireless MAM Arrays

Data were opened in the Surface Wave Analysis Wizard within SeisImager using the Passive Source (Microtremor) tool. A dispersion curve is generated from the data and the fundamental mode is picked with the assistance of the software. Uncertain data at high and low frequencies are clipped. The dispersion curve is inverted with the Wave EQ program within SeisImager and an initial model is generated. The model is improved by using a Least Square Method inversion with at least 5 iterations.

2D MASW Array

Data were opened in the Surface Wave Analysis Wizard within SeisImager using the 2D MASW tool. A dispersion curve is generated from the data and the fundamental mode is picked with the assistance of the software. Uncertain data at high and low frequencies are clipped. The dispersion curve is inverted with the Wave EQ program within SeisImager and an initial model is generated. The model is improved by using a Least Square Method inversion with at least 5 iterations.

HVSR

The data were opened in the SPAC+, filtered and processed to generate H/V ratio vs frequency. Models from MAM sounding were incorporated into the H/V data for inversion.

6. RESULTS

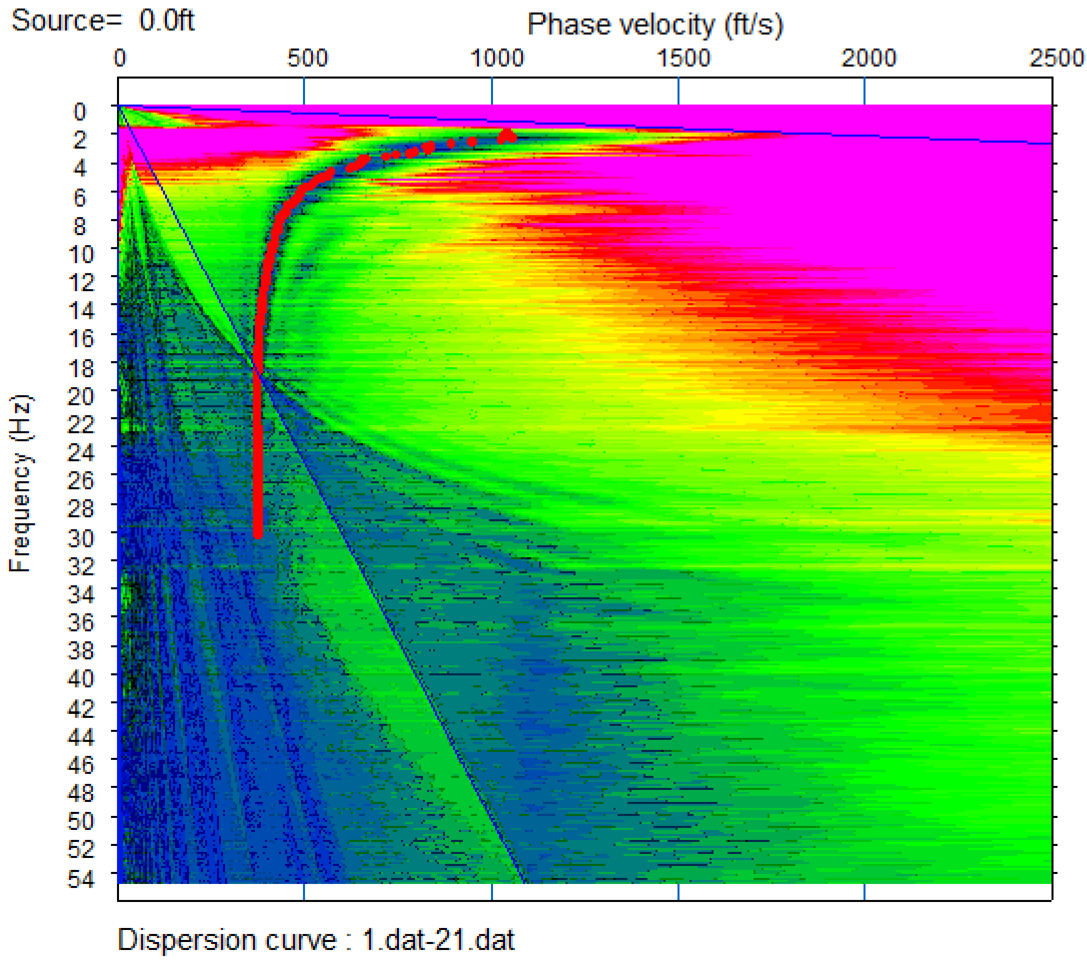


Figure 9. MASW Sounding Dispersion Curve

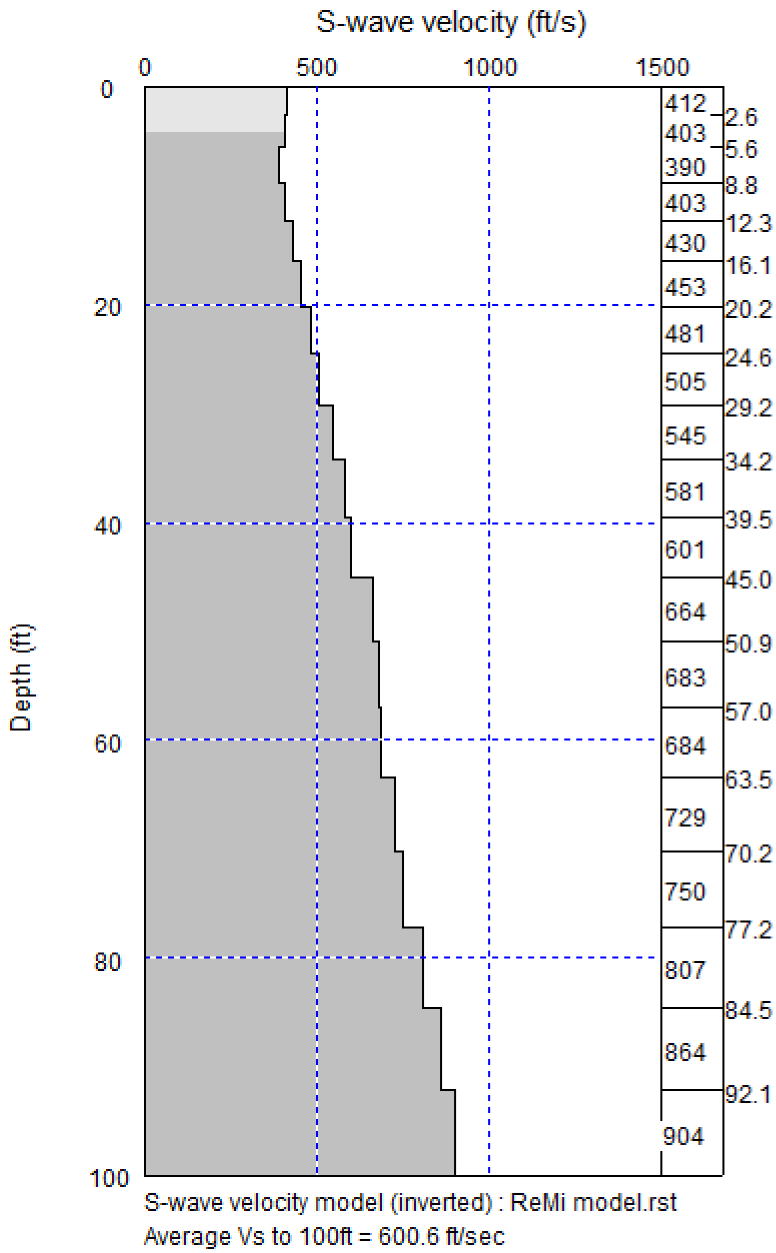


Figure 10. MASW S-Wave Model

Depth(ft)	S-wave velocity(ft/s)
0	412
3	404
6	391
9	404
12	430
16	454
20	481
25	505
29	545
34	582
39	601
45	665
51	683
57	685
63	730
70	751
77	808
85	864
92	905
100	904

Table 1. MASW S-Wave Velocities at Specific Depths

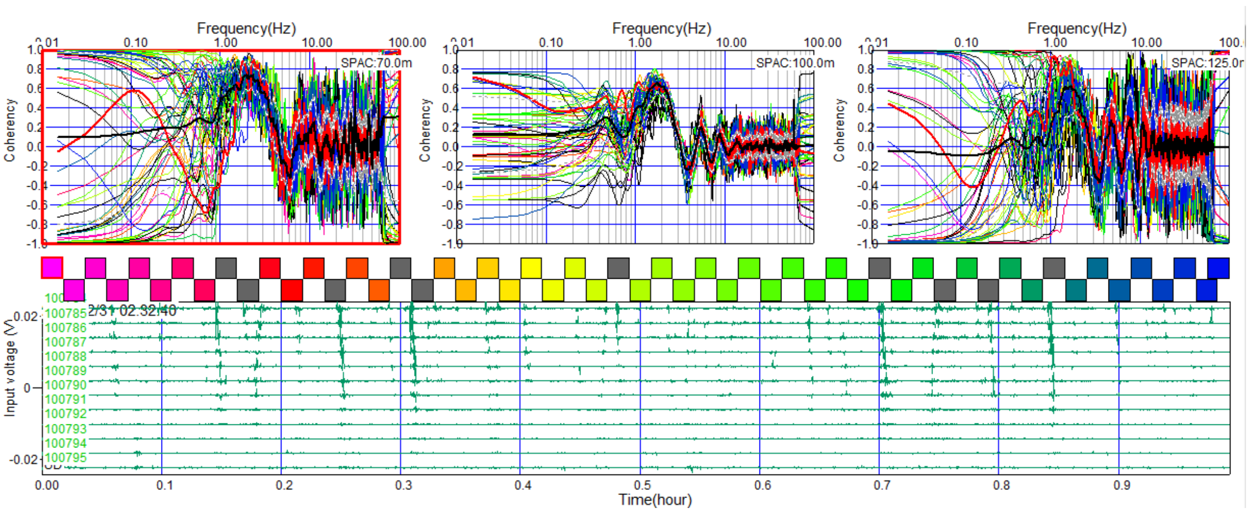


Figure 11. S-1 Sounding Coherency

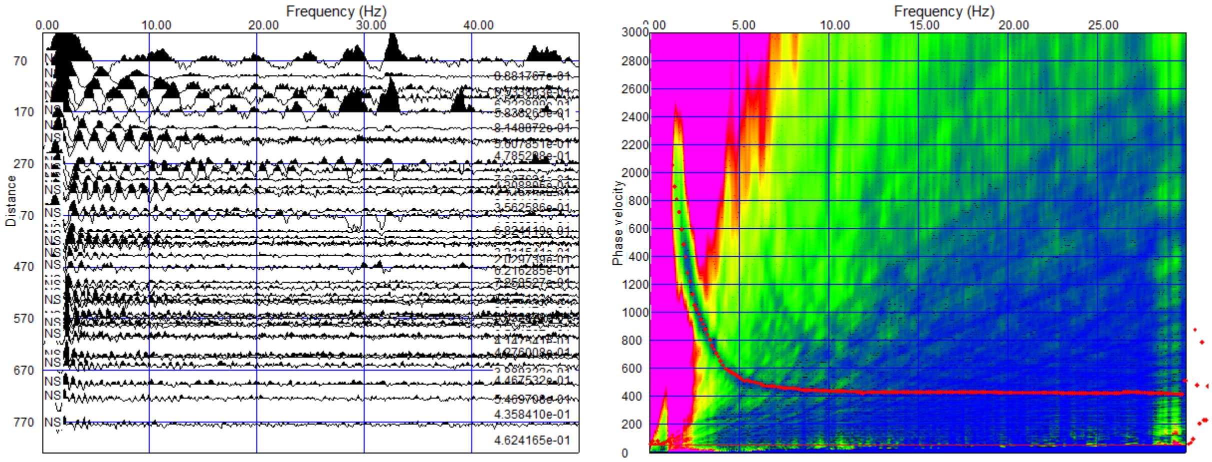


Figure 12. S-1 Sounding Dispersion Curve

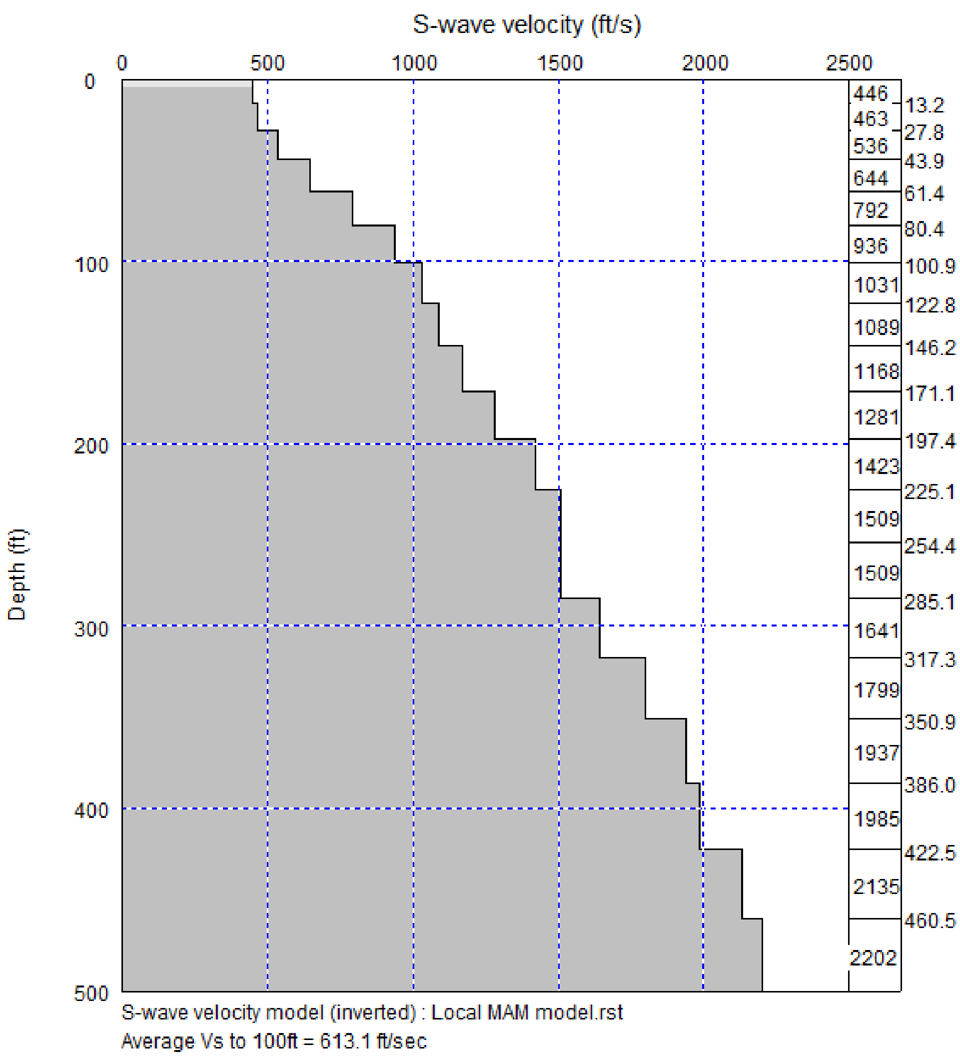


Figure 13. S-1 S-Wave Model

Depth(ft)	S-wave velocity(ft/s)
0	447
13	464
28	537
44	644
61	792
80	937
101	1031
123	1089
146	1169
171	1282
197	1423
225	1509
254	1509
285	1642
317	1799
351	1938
386	1985
423	2135
461	2202
500	2202

Table 2. S-1 S-Wave Velocities at Specific Depths

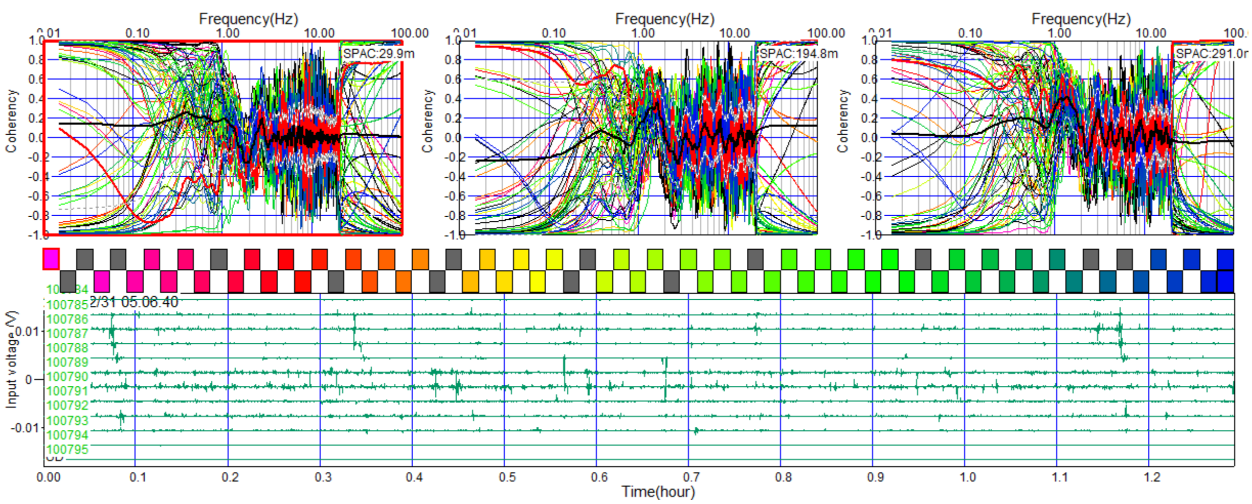


Figure 14. S-2 Sounding Coherency

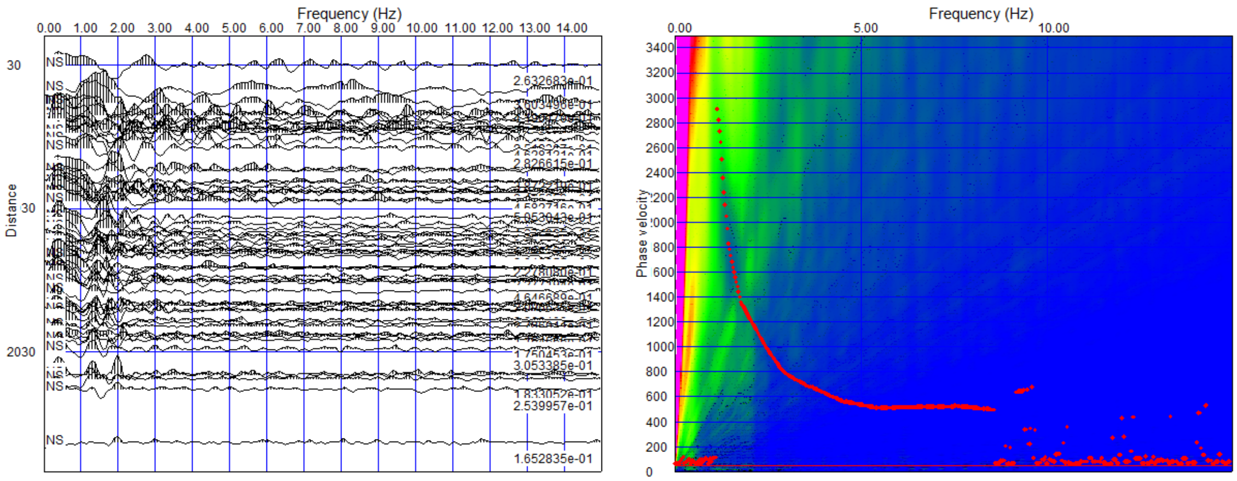


Figure 15. S-2 Sounding Dispersion Curve

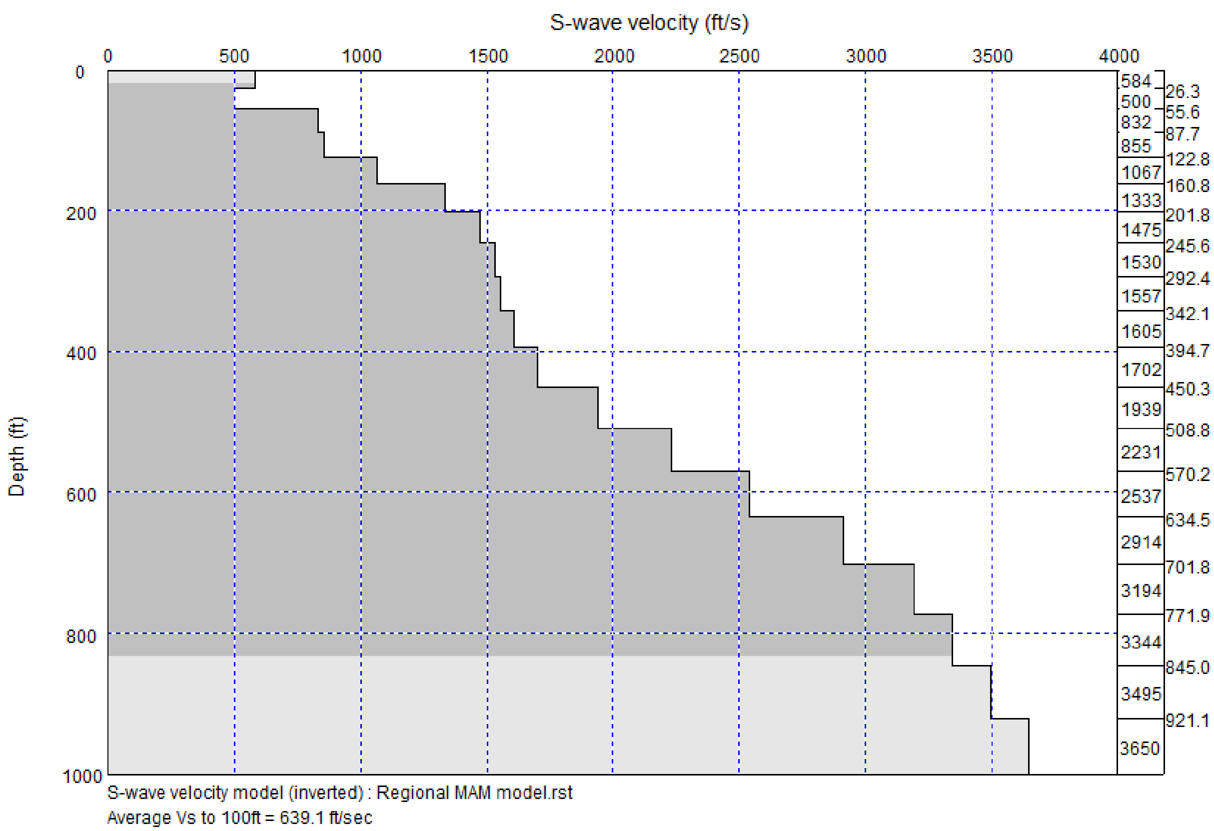
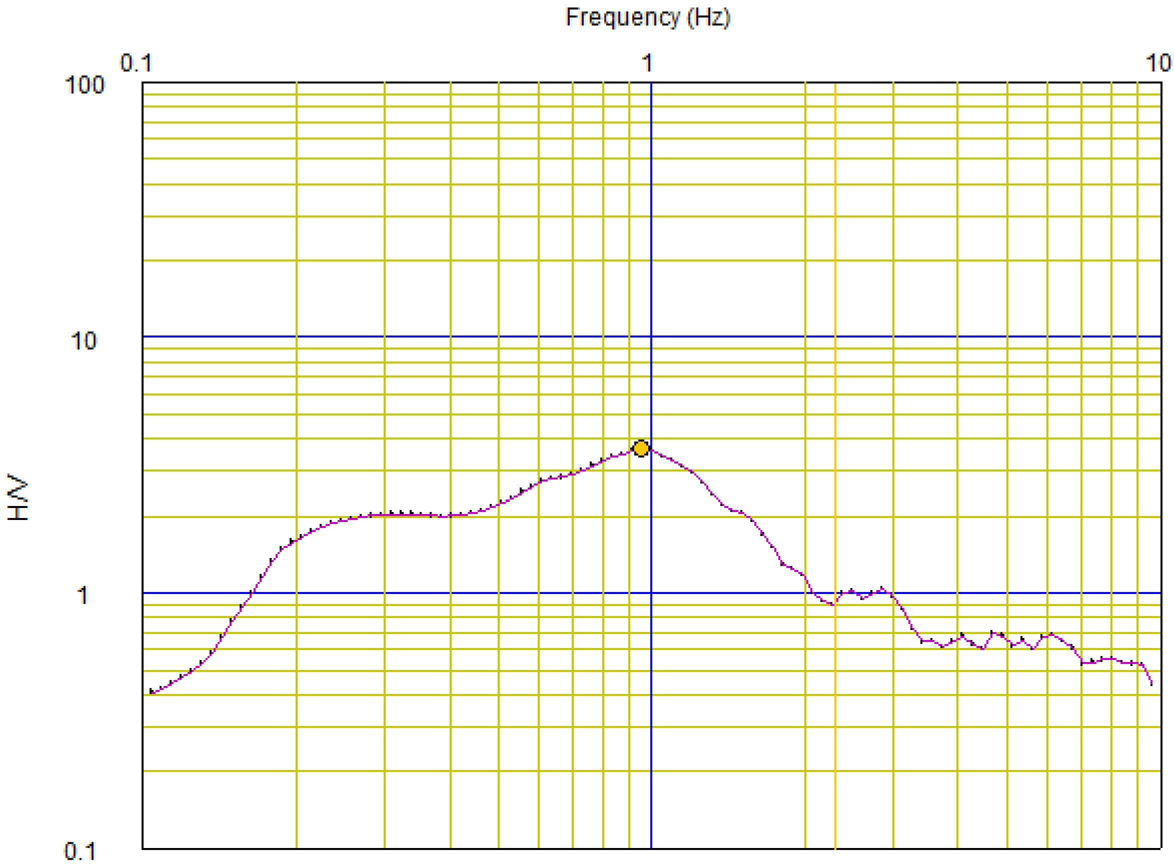


Figure 16. S-2 S-Wave Model

Depth(ft)	S-wave velocity(ft/s)
0	585
26	500
56	832
88	856
123	1067
161	1333
202	1475
246	1531
292	1558
342	1605
395	1703
450	1939
509	2231
570	2537
635	2914
702	3194
772	3344
845	3495
921	3651
1000	3650

Table 3. S-2 S-Wave Velocities at Specific Depths



H/V spectrum : HVSR+Regional MAM.pvs

H/V peak frequency = 0.959 Hz (obs.) 0.800 Hz (cal.)
H/V peak amplitude = 3.645
H/V peak clarity = 5.986

Figure 17. HVSR Curve

The H/V peak frequency is 0.959 Hz.

7. ANALYSIS

Figure 9 shows the MASW dispersion curve with the fundamental mode picked in red. Figure 10 shows the MASW shear wave velocity profile. Table 1 shows the MASW modeled shear wave velocities of the subsurface at specific depths. Figure 11 shows the Local MAM S-1 sounding coherency. Figure 12 shows the Local MAM S-1 dispersion curve with the fundamental mode picked in red. Figure 13 shows the Local MAM S-1 shear wave velocity profile. Table 2 shows the Local MAM S-1 modeled shear wave velocities of the subsurface at specific depths. Figure 14 shows the Regional MAM S-2 sounding coherency. Figure 15 shows the Regional MAM S- dispersion curve with the fundamental mode picked in red. Figure 16 shows the Regional MAM S-2 shear wave velocity profile. Table 3 shows the Regional MAM S-2 modeled shear wave velocities of the subsurface at specific depths. Figure 17 shows the HVSR curve.

The V_{S100} value for the MASW Sounding is 600.6 ft/s.

The V_{S100} value for the S-1 Sounding is 613.1 ft/s.

The V_{S100} value for the S-2 Sounding is 639.1 ft/s.

8. LIMITATIONS OF THE GEOPHYSICAL METHOD

Global Geophysics' services are conducted in a manner consistent with the level of care and skill ordinarily exercised by other members of the geophysical community currently practicing under similar conditions and are subject to the time limits, financial and physical constraints applicable to the services. MAM and 1D MASW are remote sensing geophysical methods that may not detect all subsurface conditions due to the limitations of the method, soil conditions, size of features, and their depths.

Sincerely,

Global Geophysics, LLC.

Evangeline Johnston
Field Operation Manager

John Liu, Ph.D., R.G.
Principal Geophysicist

9. REFERENCES

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APPENDIX E

SITE SPECIFIC GROUND RESPONSE ANALYSIS

Site Response Analysis

Issaquah Transit-Oriented Development Project

Issaquah, Washington

Prepared for:

PanGEO Incorporated

3213 Eastlake Avenue East, Suite B
Seattle, WA 98102

Prepared by:

Atlas Geotechnical

PO Box 2338
Santa Cruz, CA 95063

This work was prepared by me
or under my supervision.

Rev. No.	Date	Description
1	01/28/2025	Revised to discuss total stress analysis condition.
0	01/17/2025	Submittal

Contents

Appendices

Introduction

Atlas Geotechnical performed a Site Response Analysis in general accordance with ASCE 7 [2016] for the purpose of providing:

1. A seismic site class,
2. A design-level, site-specific acceleration response spectrum (ARS),
3. Seismic design parameters, S_{DS} and S_{D1}, and
4. A Site-Specific maximum considered earthquake geometric mean (MCE_G) peak ground acceleration (PGA).

Site-Specific Earthquake Design Parameters

Table 1 summarizes the results of this site-specific earthquake ground motion analysis and compares them to values determined for a non-liquefiable Site Class D site, which the site would be classified as if not for the liquefiable soils. In summary:

- The site-specific design spectral acceleration at the building period, S_{D1}, is reduced by about 40%, decreasing seismic design forces.
- The MCE_G PGA increases by about 9%, which could increase the liquefaction potential.

The remainder of this report describes the planned construction, the site conditions, and how the analyses were performed.

Table 1 – Summary of Results

Parameter	Value	
	ASCE 7-16 Site-Specific	ASCE 7-22 Non-Liquefiable Site
Seismic Site Class	F	D
Short Period Seismic Design Parameter, S_{DS} (g)	0.637	1.07
Long Period Seismic Design Parameter, S_{D1} (g)	1.05	0.71
MCE_G Peak Ground Acceleration (g)	0.74	0.68

Planned Construction

The planned Transit-Oriented Development Project (TOD) will consist of a new workforce housing project featuring two at-grade eight-story buildings. Each building will have five levels of wood frame construction over three levels of concrete construction, with foundation excavations less than four feet deep.

Borings at this and adjacent sites indicate up to 46 feet of soft liquefiable lacustrine/alluvial deposits and a shallow groundwater table, forcing a Seismic Site Class F classification according to ASCE 7-16 Table 20.3-1. ASCE 7 does provide an exception to the site response analysis requirement for structures with a fundamental period equal to or less than 0.5 s, however, the structural engineer reports fundamental building periods of 0.6 s, which is the motivation for the analysis summarized in this report.

Scope of Analysis

Our scope of work followed the procedure outlined in Chapters 20 and 21 of ASCE 7-16, and included these principal tasks:

1. Review the mapped geology of the area, the six geotechnical boring logs, the four cone penetration tests (CPTs), and the three measured shear wave velocity profiles provided to us by PanGEO.
2. Assign appropriate site classes in accordance with Chapter 20 of ASCE 7-16 based on the data included in the geotechnical borings and the shear wave velocity (V_s) measurements.
3. Estimate the sedimentary basin depth for use in ground motion model (GMM)-based site response analysis and developing target site response spectra in the absence of soil liquefaction.
4. Evaluate the seismic hazard with both probabilistic (PSHA) and deterministic (DSHA) seismic hazard analyses at the site coordinates: 47.542125°, -122.060294°.
    - 4.1. The PSHA is based on the United States Geological Survey (USGS) [2024] source code, which is incorporated in ASCE 7-16. The PSHA includes GMMs that account for basin effects local to the project site [Peterson et al. 2024].
    - 4.2. The DSHA uses the GMMs that are consistent with the 2023 USGS National Seismic Hazard Model [Peterson et al. 2024]. We de-aggregated the probabilistic seismic hazard at 0.5 s, the available option closest to the expected building periods.
5. Develop a target acceleration response spectrum as the lesser of:
    - 5.1. The probabilistic risk-targeted maximum considered earthquake (MCE_R) acceleration response spectrum (ARS) in accordance with Sections 21.2, 21.2.1, and 21.2.1.1 [ASCE 2016].
    - 5.2. The deterministic MCE_R ARS in accordance with Sections 21.2 and 21.2.2. [ASCE 2016] which shall not be less than the deterministic lower limit presented in Section 21.2.2 of the ASCE 7-16 Supplement 1.
6. Review the USGS seismic hazard disaggregation data and select 11 appropriate ground motion recordings[1] that reflect the prevailing sources of earthquake hazard at the site.
    - 6.1. Scale each selected record such that the response spectra of the recorded motion matches the target spectrum over the period range of interest.

[1] ASCE 7-16 requires using at least 5 ground motions that represent the seismic hazard at the site. We elect to use 11 at this site (and at most sites in Puget Sound) because there are three important seismic sources, and more records allows better representation.

6.2. Review the scaled records to confirm reasonable secondary ground motion characteristics like significant duration and energy content.

7. Perform total stress, nonlinear 1-D dynamic site response analyses for the idealized soil column subjected to each of the 11 scaled time histories. Evaluate sensitivity to the interpreted profile by analyzing soil profiles 15% stiffer and 15% softer than the best estimate profile.
8. Compute the MCE_R ground-surface and design level acceleration response spectra.
9. Compute the site-specific MCE_G peak ground acceleration (PGA) in accordance with Section 21.5 [ASCE 2016] as the lesser of the probabilistic geometric mean PGA and the deterministic geometric mean PGA, but not less than 80% of the PGA determined using Equation 11.8-1. The deterministic and probabilistic PGA values were determined using the same methods used to perform the PSHA and DSHA analyses in Step 4 and with the site's weighted average shear wave velocity to a depth of 100 feet.
10. Summarize the analyses and results in this report.

Site Characterization

This section summarizes the site characterization efforts that preceded our site-specific seismic hazard analysis (SSHA). The focus of the site characterization was on establishing an idealized set of representative shear wave velocity profiles at the site, computing the associated average small-strain shear wave velocity in the upper 100 feet, $\bar{v}_s$, and estimating basin depth for use in GMM-based site response analysis.

Site characterization for the SSHA focused on three types of data:

1. Shear wave velocity, v_s, data measured by three geophysical soundings at the site [Global Geophysics 2025],
2. Four CPT logs provided by PanGEO, and
3. Geotechnical boring logs performed for the Issaquah Transit Center located immediately west of the site. Specifically, we reviewed the logs of test borings B-4, B-6, B-8, B-13, B-14, and B-15, which Zipper Zeman Associates, Inc. completed between June 2004 and May 2005.

Site Stratigraphy

In general, the subsurface consists of the following soil profile:

1. 3 to 6 feet of granular fill material. The fill generally consists of dense relatively clean to silty sand with gravel.
2. 35 to 46 ft of very soft to medium stiff silty clay to clayey silt.
3. Medium dense to dense clean sand to silty sand with varying gravel content to the maximum exploration depth of 74 feet.

Global Geophysics used the microtremor array measurement method (MAM) and the multichannel analysis of surface waves (MASW) method to collect surface wave data and define a shear wave velocity profile from near surface to a depth where the materials were stiff enough to have a velocity of at least 2,500 feet per second.

- Two MAM surveys using passive sources were conducted.
    - The local MAM sounding (S-1) utilized a modified L-array measuring 525 by 570 feet.
    - The regional MAM sounding (S-2) employed a modified T-array approximately 2,700 by 1,800 feet.
- A linear MASW survey was performed using passive and active sources. The linear array was 230 feet in length with a 10-foot sensor spacing.
- Figure 1 shows the measured shear wave velocity profiles at each sounding.

Seismic Site Classification

Figure 1 shows the measured shear wave velocity profiles along with profiles of the corrected SPT blow counts ($N_{1,60}$) and correlated undrained shear strength. The MASW survey was used to define the site's shear wave velocity to a depth of 100 feet. S-1 data was used to define the velocity profile for the next 400 feet. S-2, representing regional average shear wave velocities, was used to establish input parameters for deep basin effects. Best-estimate and upper and lower-bound profiles at +/- 15% of the best-estimate velocity profile are shown in Figure 1. The data indicates the expected gradual stiffness increase with depth.

A 100-ft deep shear wave velocity profile ($\bar{v}_s$) calculated from the ground surface is necessary to designate a site class in accordance with Chapter 20 of ASCE 7-16. Considering the building's lack of basements, this parameter was calculated using the average shear wave velocity profile from the ground surface. The calculated $\bar{v}_s$ is 601 feet per second which would indicate Site Class D if the liquefiable soils were not present. However, due to the presence of liquefiable soils and a shallow groundwater table, the site is classified as Class F.

The site is located in the Seattle Basin where bedrock is deeper than 10,000 feet. Soil profiles for this site response analysis were terminated at a depth where the shear wave velocity is 1,850 feet per second, representing the midpoint for Site Class C classification. This results in analyzing soil profiles with depths between 285 and 465 feet.

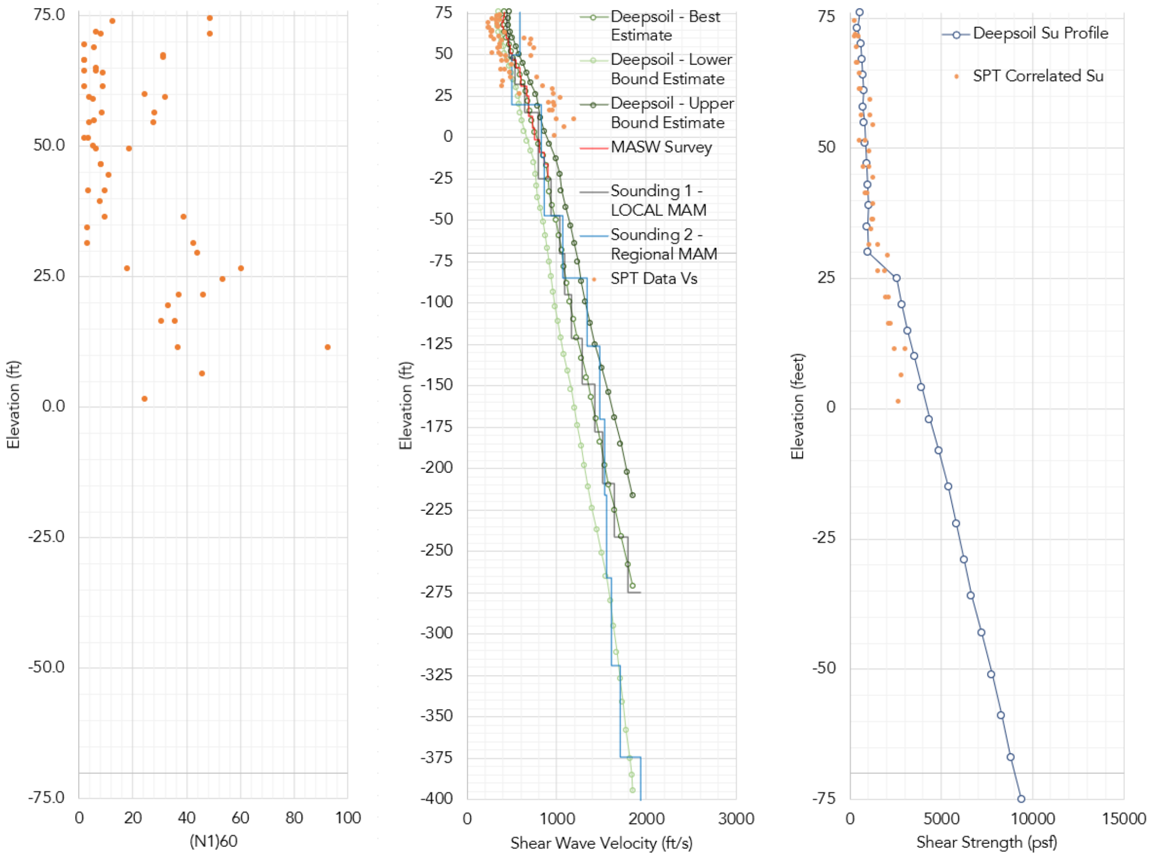

Figure 1 – Site characterization with SPT $N_{1,60}$ blow counts, shear wave velocity, and shear strength.

Basin Depth

The extent of the Seattle basin and the approximate location of the Issaquah TOD site are shown in Figure 2 [adapted from Moschetti et al. 2024].

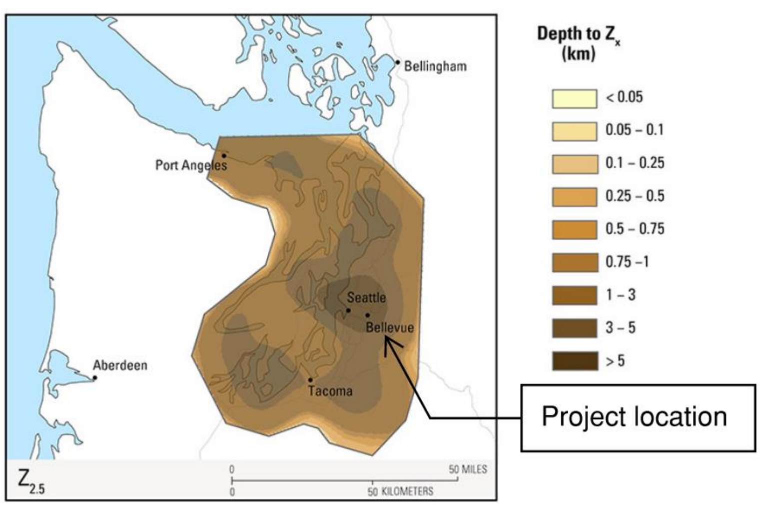

Figure 2 – Seattle Basin Extents

The basin depth beneath the project site was characterized according to the commonly used parameters of $Z_{1.0}$ and $Z_{2.5}$, which represent the depths at which the shear wave velocity reaches 1.0 and 2.5 km/s, respectively.

1. $Z_{1.0}$ was estimated directly from S2 MAM survey data as 0.23 kilometers.
2. Z2.5 was estimated by interpolating the gridded data in Figure 2 at the project location. The contour map indicates that Z2.5 for this site is approximately 4.0 kilometers.

Site-Specific Ground Motion Hazard Analysis

The following sections describe how we arrived at the target acceleration response spectrum (ARS) that define site-specific ground shaking in the absence of liquefaction.

Probabilistic MCE_R

The site-specific uniform hazard (ARS) at the model base (1,850 ft/sec) was developed for the MCE_R level seismic hazard (2% in 50 years probability of exceedance) using the published Site Class C spectrum in the USGS Earthquake Hazard Toolbox.[2] We used the NSHM Conterminous U.S. 2023 hazard curves which incorporate GMMs that account for basin effects local to the project site [Peterson et al. 2024 and Moschetti et al. 2024]. The hazard curves were scaled to maximum direction and uniform risk using the appropriate factors. The resulting probabilistic MCE_R ARS is shown in Figure 3.

Maximum Direction Factors

Maximum direction adjustment factors were applied to convert the geomean ordinates provided by the GMMs used in the PSHA to maximum direction motions as required by ASCE 7-16 Section 21.2. We used the mean value of the maximum direction adjustment factors proposed by Shahi and Baker [2014], which differ from those recommended in ASCE 7-16 but are consistent with the recommendations in PEER TBI [2017], ASCE 7-22, and FEMA [2020], all of which were more recently revised and/or published.

Uniform Risk Factors

ASCE 7-16 Section 21.2.1.1 requires that the spectral ordinates obtained from the PSHA be multiplied by corresponding risk coefficients to adjust the response spectrum from uniform hazard to uniform risk. We used the C_{Rs} and C_{R1} risk coefficients that were computed for the site using the ASCE 7 Hazard Tool.[3] The resulting risk coefficients are shown in Figure 5.

[2] https://earthquake.usgs.gov/nshmp/
[3] https://asce7hazardtool.online/

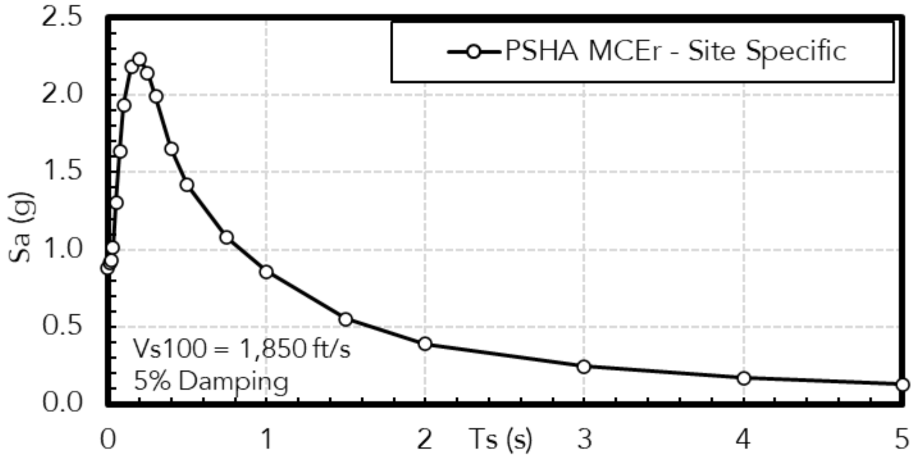

Figure 3 – Site-specific PSHA MCE_R target spectrum.

Near Fault Effects

Per section 11.4.1 of ASCE 7-16, an adjustment to the response spectrum for near-fault effects was not required at this site because the only mapped active faults within 9.5 km of the site, the Seattle Fault Zone and the Whidbey Island fault, both have slip rates less than 0.04 inches per year [Johnson et al. 2016].

Deterministic MCE_R

Based on the USGS seismic hazard disaggregation, three principal seismic sources were represented as magnitude-distance pairs, termed "scenario earthquakes." These were:

1. M=7.1 on the Seattle Fault at 3.2 km from the site,
2. M=7.1 on the deep subducting slab portion of the (CSZ) at 67.9 km, and
3. M=9.2 on the interface portion of the Cascadia Subduction Zone (CSZ) at 107.1 km.

Figure 4 shows the results of the disaggregation at the site for a period of 0.5 seconds. Using the information from the seismic hazard disaggregation, we proceeded to compute the deterministic MCE_R spectrum.

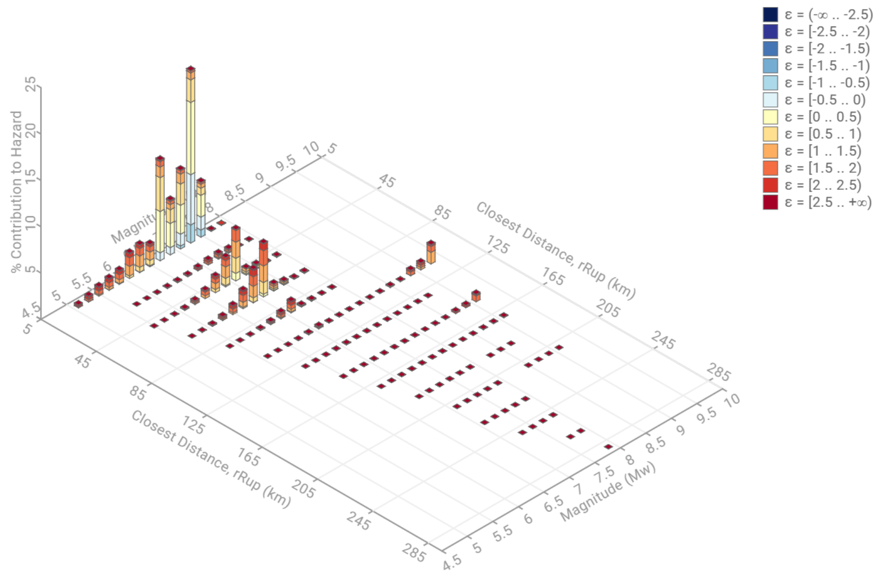

Figure 4 – Disaggregation for T=0.5 s and Site Class C.

We computed the median plus one sigma (i.e., 84th percentile) ARS for each scenario earthquake using the same GMMs and weighting factors used by the USGS for the 2023 update of the NSHM [Moschetti et al. 2024]. The base model shear wave velocity of 1,850 ft/s was used directly in the GMMs to account for site effects. Computed values of $Z_{1.0}$ and $Z_{2.5}$ were used in the GMMs to account for basin amplification.

At this site, the two CSZ deterministic spectra are inconsequential to the development of the site-specific target spectrum because ground shaking from the closer crustal source exceeds the other two ground motions. Section 21.2 of ASCE 7-16 defines the DSHA target spectrum as the envelope of all deterministic scenarios affecting the site, and at this site the crustal source is higher at all periods.

Maximum direction adjustment factors were applied to convert the geomean motions provided by the GMMs to maximum direction motions as required by Section 21.2 of ASCE 7-16. The maximum direction factors are the same ones used in the PSHA.

The deterministic ARS for the scenario earthquake was then computed as the product of the average spectral ordinates obtained from the USGS GMMs multiplied by the corresponding maximum direction factors. Figure 5 shows the results of the deterministic hazard analysis.

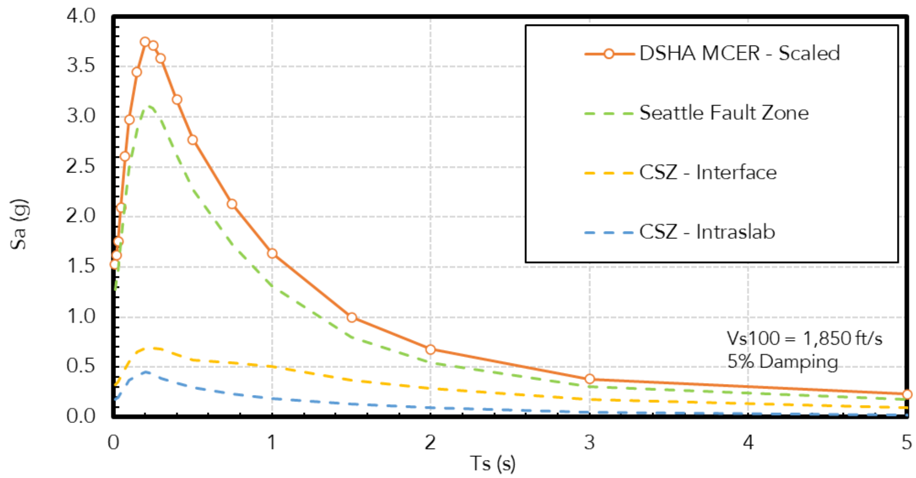

Figure 5 – Deterministic spectrum scaled for maximum direction.

Design Target Spectrum

Figure 6 shows a comparison of the probabilistic MCE_R spectrum and the deterministic MCE_R spectrum. The target spectrum (red line) is defined by the minimum of the probabilistic and deterministic spectra (Section 21.2.3 of ASCE 7-16). The site's proximity to the Seattle Fault Zone leads to deterministic MCE_R spectral accelerations greater than the PSHA MCE_R at all periods. Therefore, the PSHA was utilized to compute the MCER.

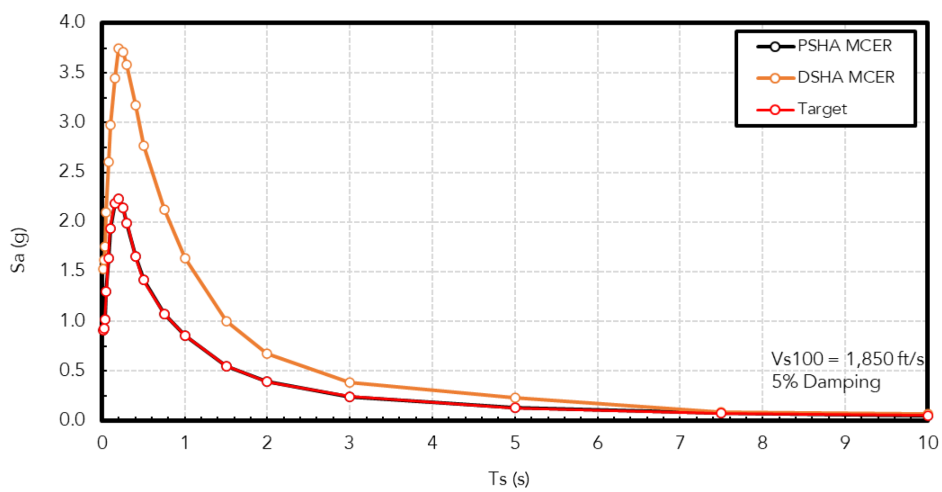

Figure 6 – Probabilistic and deterministic MCE_R spectra with the design target spectrum.

Site Response Analysis

This section describes the ground motion selection and modification process that was used to develop a suite of eleven ground motion records appropriate for use in a 1-D nonlinear, total stress site specific response analysis. The ground motion records were selected and amplitude scaled in accordance with Chapter 16 of ASCE 7-16 as follows:

1. Use the USGS Unified Hazard Tool to deaggregate the seismic hazard, based on spectral acceleration at the building period.
2. Bin the seismic hazard contributions according to their seismic source type: shallow crustal, interface (subduction), and intraslab (deep subduction), and compute the total percentage contribution per source type.
3. Select 11 ground motions in accordance with Section 21.1.1 of ASCE 7-16. The selected ground motions should:
    a. Have spectral shapes similar to that of the target spectrum across the period range of interest; and
    b. be from earthquakes that are generally similar to the earthquakes expected to cause the spectral acceleration at the conditioning period of interest.
4. Determine amplitude-scale factors for each record such that the geometric mean of the eleven maximum direction response spectra is no less than 90% of the target spectrum in the period range of interest while being mindful of the energy content of the scaled records. Apply the amplitude scale factors to the entire acceleration time history for the selected horizontal component of the eleven selected ground motions.

Input Ground Motions

We selected a suite of eleven recorded acceleration time histories and applied uniform scaling to each so that the geometric mean spectrum of all eleven ground motions closely matched the target spectrum across the period range of interest. The period range of interest was defined as 20% to 200% of the building period, or 0.12 to 1.2 seconds.

The proportion of records selected for each earthquake type was determined based on the USGS [2024] hazard disaggregation and the 2024 NHSM [Peterson et al. 2024] at a spectral period of 0.5 seconds, summarized in Table 3.

Crustal motions were downloaded from the PEER NGA-West2 database [Ancheta et al. 2014] while subduction (intraslab and interface) motions were downloaded from the NGA-Subduction database [Mazzoni 2024]. In addition to the fault rupture mechanism, the representative seed motions were selected based on factors such as spectral shape, peak ground acceleration (PGA), earthquake magnitude, rupture distance, V_s at the recording station, and whether they were pulse-like motions.

Since the Seattle Fault Scenario earthquake is greater than magnitude 6 at a distance less than 10 km, we included pulse-like motion records for 3 of the 11 selected time-histories. Table 4 summarizes the selected ground motion records. Figure 7 shows the geometric mean of the suite of ground motions scaled to the target spectrum for the base of our 1-D site response models. The geometric mean of the suite of scaled motions was at least 90% of the target spectrum in the period range of interest.

Table 3 – USGS Hazard tool disaggregation for this site at PGA.

Source	Contribution	Type	M	R (km)	# Motions
Seattle Fault Zone	45.93%	Crustal	7.1	3.2	7
CSZ – Intraslab	30.29%	Intraslab	7.1	67.9	3
CSZ – Interface	4.48%	Interface	9.2	107.1	1
Gridded	18.80%	Gridded	6.3	9.3	0[4]

[4] The gridded seismicity source was combined with the crustal source for ground motion selection.

Table 4 – Summary of Selected Time Histories

Earthquake Name	Tectonic Setting	Year	Station Name	Mw	R_{rup} (km)	Comp.
San Fernando California	Crustal	1971	Lake Hughes #12	6.61	19.30	021
Nahanni Canada	Crustal	1985	Site 2	6.76	4.93	330
Loma Prieta California	Crustal	1989	Gilroy Array #6	6.93	18.33	000
Northridge California	Crustal	1994	LA Dam	6.69	5.92	064
Cape Mendocino California	Crustal	1992	Bunker Hill FAA	7.01	12.24	360
Niigata Japan	Crustal	2004	NIGH12	6.63	10.72	NS
Chuetsu-oki Japan	Crustal	2007	Kashiwazaki Nishiyamacho Ikeura	6.80	12.63	EW
Tohoku Japan	Interface	2011	Naganuma	9.12	98.10	EW
Nisqually Washington	Intraslab	2001	BHD	6.80	67.57	000
Geiyo Japan	Intraslab	2001	Mikawa	6.83	55.85	EW
Pingtung Doublet Taiwan	Intraslab	2006	KAU049	6.94	65.85	NS

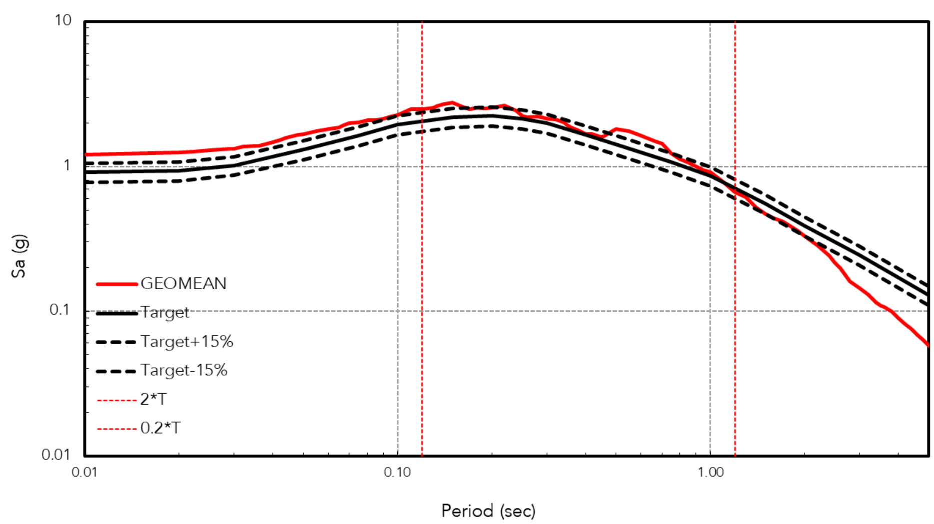

Figure 7 – Geometric mean response spectrum of eleven selected ground motions and the target spectrum +/- 15%.

Dynamic Site Response Modeling

The computer program DEEPSOIL [Hashash et al. 2018] was used to compute the response of the three soil profiles to the selected input motions. A total stress condition was chosen because excess pore pressure generation has minimal effect on surface motions for peak pore pressure ratios under 0.6, whereas for ratios above 0.6, excess pore pressure generation typically leads to a significant reduction in surface motion amplitudes [Markham et al. 2014 and Matasovic et al. 2024]. Additionally, effective stress analyses, which capture pore pressure generation and dissipation, involve greater uncertainty in input parameters. The total stress approach provides a slightly conservative estimate of surface motion amplitudes especially at the period of interest (0.6 seconds). The stress-strain behavior of the soil was modeled using the nonlinear, General Quadratic/Hyperbolic (GQ/H) Model with non-Masing hysteretic behavior. We used the Darendeli [2001] model, as implemented in DEEPSOIL, to represent the strain-dependent modulus reduction and damping curves for sandy and clayey soils.

Results

Figure 8 shows the geometric mean profiles of:

- maximum strain,
- shear stress ratio (shear stress normalized by effective overburden stress), and
- the peak ground acceleration,

computed for the best-estimate shear wave velocity profile and the upper and lower bound profiles when subjected to the eleven scaled input motions. Maximum soil strains across all soil profiles above a depth of 50 feet are about 4%. As expected, the nonlinear soil behavior associated with these soil strains damps short-period near surface accelerations, including PGAs.

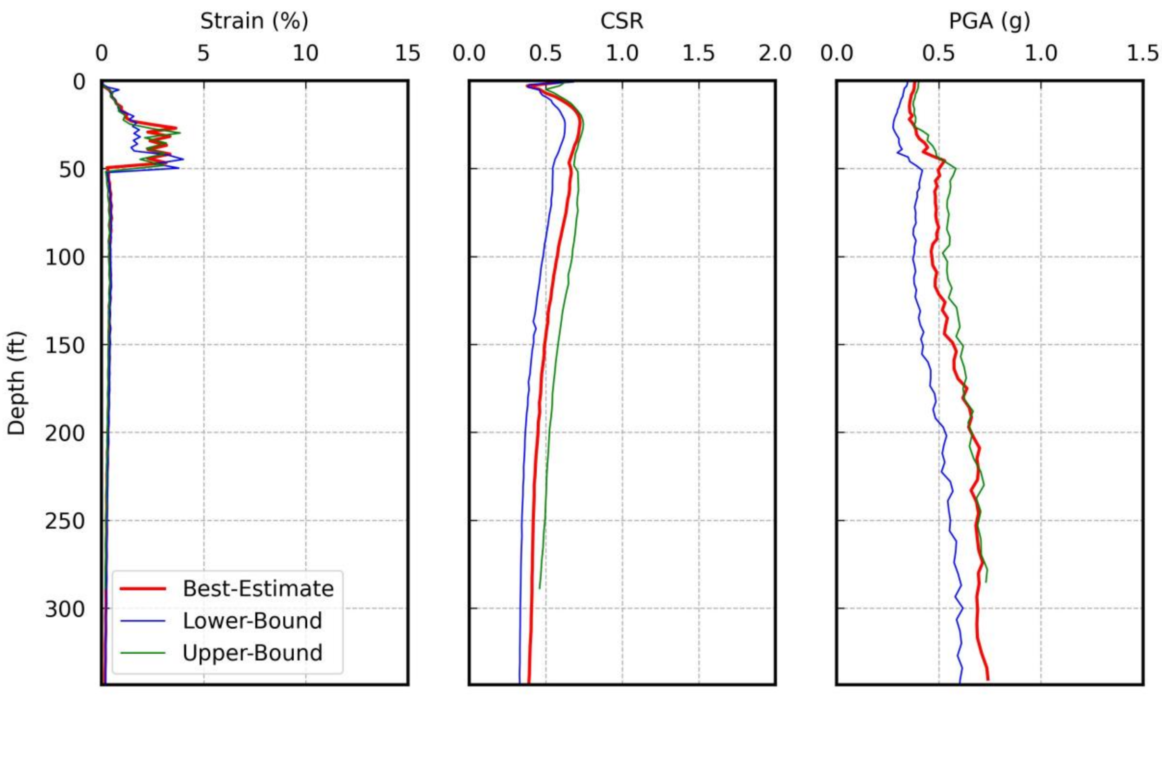

Figure 8 – Geometric mean shear strain, stress ratio, and maximum acceleration for best estimate v_s

Figure 9 shows the spectral modification factors (SMF's) of the weighted average site response analysis results for each considered V_s profile. The design SMF is the smoothed envelope of maximum average SMF ordinates across the three profiles and at each period. Ground motion amplification is expected where the spectral modification factor is greater than 1 (e.g., at periods greater than about 1 second). Short-period deamplification at this site is associated with nonlinear behavior of the very soft, liquefiable, silty clay to clayey silt. The high spectral amplification at long periods is due to both the deep soil column at this site (including the basin effects) and the nonlinearity of the soft liquefiable layers.

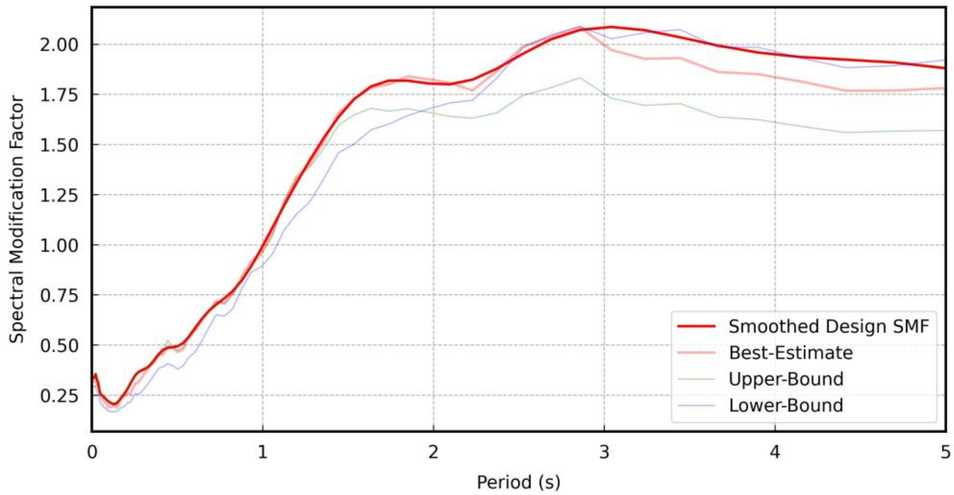

Figure 9 – Spectral amplification ratios for each soil profile and a smoothed envelope of maximum ordinates

We computed the site-specific MCE_R ground-surface spectrum by multiplying the target spectrum ordinates by the corresponding idealized SMF's in Figure 9. For Site Class F sites, the lower limit is 80% of the Site Class E response in accordance with section 11.4.5 of ASCE 7-16. Accordingly, the design response spectrum was computed as 2/3 of the MCE_R surface spectrum, but not less than 80% of the Site Class E design spectrum. Figure 10 shows the recommended design spectrum. We included the ordinates of the design spectrum in Appendix B.

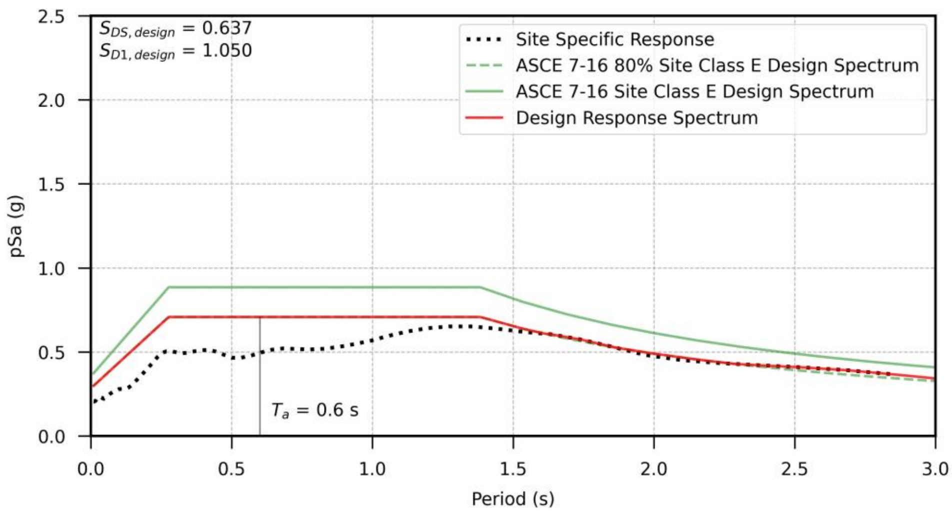

Figure 10 – Recommended acceleration response spectrum from site response analysis

The 80% Site Class E Design Spectrum will control the earthquake loads in structural design because it is greater than the result from the site response analysis at periods below about 1.5 seconds.

- The de-amplification of short-period motions, as reflected by the site-specific values being lower than the lower-bound code limit, is the expected result wherever deep, soft sediments are exposed to relatively strong earthquake shaking.
- This nonlinear soil behavior causes a long-period shift in earthquake energy so that the site specific response converges with the 80% Site Class E Design Spectrum at longer periods. This shift and amplification at long periods are a direct result of amplification from the soft liquefiable layers.

Maximum Considered Earthquake Peak Ground Acceleration

The MCE_G peak ground acceleration (PGA) was determined according to Section 21.5 of ASCE 7-16 as the lesser of the probabilistic geometric mean PGA and the deterministic geometric mean PGA. The site-specific MCE_G PGA is limited to at least 80% of the peak ground acceleration determined from Equation 11.8-1 of ASCE 7-16. We determined the probabilistic and deterministic geometric mean peak ground accelerations using the measured and calculated site $\bar{v}_s$ value of 601 feet/second and the same procedures used to compute the probabilistic and deterministic MCE_R values without adjustments for targeted risk and maximum direction. In summary:

- Probabilistic MCE_G PGA=0.74 g
- Deterministic MCE_G PGA =0.78 g
- ASCE 7-16 site-specific lower limit=0.50 g

Conclusions and Recommendations

The design spectral acceleration values S_{DS} and S_{D1} are calculated in accordance with Section 21.4 of ASCE 7-16 and using Figure 10. The MCE_g PGA is calculated in accordance with section 21.5 of ASCE 7-16.

- S_{DS} = 90% of the maximum spectral acceleration for periods from 0.2 to 5 seconds.
- S_{D1} = the maximum value of the product of $T \times S_a$ for periods from 1 to 5 seconds.

Accordingly, the design values are S_{DS} = 0.637 g and S_{D1} = 1.05 g, and the MCE_g PGA = 0.74 g. We recommend the MCE_G be used to further evaluate the liquefaction potential of the soft clayey silt to silty clay material and to estimate seismic settlements in accordance with ASCE 7-16.

It is a pleasure working with PanGEO, Inc. on these interesting site-specific analyses. Please call us at 603-704-0871 if you have any questions or if we can be of further assistance.

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Appendix A - Report Limitations and Guidelines for Use

This appendix explains how to understand and manage the risks inherent in using this report.

This Report is for a Specific Purpose & Project

This report should not be applied for any purpose or project except the one originally contemplated. Atlas Geotechnical prepared this report specifically to address the needs of Olsson and their client for the proposed landfill expansion. Because each geotechnical or geologic effort is unique, each geotechnical engineering or geologic report is unique, prepared solely for the specific client, project site, and proposed construction. No one except the parties specifically named should rely on this report without first conferring with Atlas Geotechnical.

Atlas Geotechnical considered a number of unique, project-specific, and client-specific factors when establishing our scope of services. Unless this report specifically indicates otherwise, do not rely on this report if it was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

If important changes are made after the date of this report, Atlas Geotechnical should be retained to review our interpretations and recommendations and provide written modifications or confirmation, as appropriate.

Subsurface Conditions and Regulations Can Change

This geotechnical or geologic report is based on conditions that existed at the time the analyses were performed. The findings and conclusions of this report may be affected by the passage of time, by man-made events such as construction on or adjacent to the site, or by natural events such as floods, earthquakes, slope instability, or ground water fluctuations. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur because of governmental action and the broadening of knowledge. The findings of this report may be invalidated wholly or in part by such changes, over which Atlas Geotechnical has no control. Always contact Atlas Geotechnical before applying this report to determine if it remains applicable.

Most Geotechnical and Geologic Findings are Professional Opinions

Our interpretations of subsurface conditions are based on geologic maps and the boring logs from a nearby site. Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Atlas Geotechnical reviewed the collected data and then applied our professional judgment to render an opinion about subsurface conditions as they affect the planned construction. Actual subsurface conditions may differ, sometimes significantly, from those indicated in this report. Our report, conclusions, and interpretations should not be construed as a warranty of the subsurface conditions.

Geotechnical Engineering Reports can be Misinterpreted

Misinterpretation of this report by other design team members can result in costly problems. You could lower that risk by having Atlas Geotechnical confer with appropriate members of the design team after submitting the report. Contractors can also misinterpret a geotechnical engineering or geologic report. Reduce that risk by having Atlas Geotechnical participate in pre-bid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Exploration Logs

Geotechnical engineers and geologists prepare final exploration logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable but recognize that separating logs from the report can increase risk.

Give Contractors a Complete Report and Guidance

Limiting information available for bidding in an attempt to transfer responsibility for unanticipated subsurface conditions onto the Contractor is both ineffective and disingenuous. To help prevent costly problems, give contractors the complete geotechnical engineering or geologic report, but preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with Atlas Geotechnical and/or to conduct additional study to obtain the specific types of information they need or prefer. A pre-bid conference can also be valuable. Be sure contractors have sufficient time to perform additional study. Only then might an owner be in a position to give contractors the best information available, while requiring them to at least share the financial responsibilities stemming from unanticipated conditions. Further, a contingency for unanticipated conditions should be included in your project budget and schedule.

Contractors are Responsible for Safety on their Construction Sites

Our geotechnical recommendations are not intended to direct the contractor's procedures, methods, schedule, or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to personnel and to adjacent properties.

Read These Provisions Closely

Some clients, design professionals, and contractors may not recognize that the practice of geotechnical engineering and geology are far less exact than other engineering and natural science disciplines. This lack of understanding can create unrealistic expectations that could lead to disappointments, claims, and disputes. Atlas Geotechnical includes these explanatory "limitations" provisions in our reports to help reduce such risks. Please confer with Atlas Geotechnical if you are unclear how these "Report Limitations and Guidelines for Use" apply to your project or site.

Geotechnical, Geologic and Environmental Reports Should not be Interchanged

The equipment, techniques, and personnel used to perform an environmental study differ significantly from those used to perform a geotechnical or geologic evaluation, and vice versa. For that reason, a geotechnical engineering or geologic report does not usually relate any environmental findings, conclusions, or recommendations, e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. Similarly, environmental reports are not used to address geotechnical or geologic concerns regarding a specific project.

Appendix B – Design Spectral Ordinates

T (s)	pSa (g)	T (s)	pSa (g)	T (s)	pSa (g)	T (s)	pSa (g)	T (s)	pSa (g)	T (s)	pSa (g)
0.010	0.298	1.010	0.708	2.010	0.487	3.010	0.326	4.010	0.244	5.010	0.195
0.020	0.314	1.020	0.708	2.020	0.485	3.020	0.324	4.020	0.244	5.020	0.195
0.030	0.329	1.030	0.708	2.030	0.483	3.030	0.323	4.030	0.243	5.030	0.195
0.040	0.344	1.040	0.708	2.040	0.481	3.040	0.322	4.040	0.242	5.040	0.194
0.050	0.360	1.050	0.708	2.050	0.478	3.050	0.321	4.050	0.242	5.050	0.194
0.060	0.375	1.060	0.708	2.060	0.476	3.060	0.320	4.060	0.241	5.060	0.194
0.070	0.391	1.070	0.708	2.070	0.474	3.070	0.319	4.070	0.241	5.070	0.193
0.080	0.406	1.080	0.708	2.080	0.471	3.080	0.318	4.080	0.240	5.080	0.193
0.090	0.421	1.090	0.708	2.090	0.469	3.090	0.317	4.090	0.239	5.090	0.192
0.100	0.437	1.100	0.708	2.100	0.467	3.100	0.316	4.100	0.239	5.100	0.192
0.110	0.452	1.110	0.708	2.110	0.465	3.110	0.315	4.110	0.238	5.110	0.192
0.120	0.467	1.120	0.708	2.120	0.462	3.120	0.314	4.120	0.238	5.120	0.191
0.130	0.483	1.130	0.708	2.130	0.460	3.130	0.313	4.130	0.237	5.130	0.191
0.140	0.498	1.140	0.708	2.140	0.458	3.140	0.312	4.140	0.237	5.140	0.191
0.150	0.513	1.150	0.708	2.150	0.455	3.150	0.311	4.150	0.236	5.150	0.190
0.160	0.529	1.160	0.708	2.160	0.453	3.160	0.310	4.160	0.235	5.160	0.190
0.170	0.544	1.170	0.708	2.170	0.451	3.170	0.309	4.170	0.235	5.170	0.189
0.180	0.559	1.180	0.708	2.180	0.449	3.180	0.308	4.180	0.234	5.180	0.189
0.190	0.575	1.190	0.708	2.190	0.448	3.190	0.307	4.190	0.234	5.190	0.189
0.200	0.590	1.200	0.708	2.200	0.446	3.200	0.306	4.200	0.233	5.200	0.188
0.210	0.605	1.210	0.708	2.210	0.444	3.210	0.305	4.210	0.233	5.210	0.188
0.220	0.621	1.220	0.708	2.220	0.442	3.220	0.304	4.220	0.232	5.220	0.188
0.230	0.636	1.230	0.708	2.230	0.440	3.230	0.303	4.230	0.232	5.230	0.187
0.240	0.651	1.240	0.708	2.240	0.438	3.240	0.302	4.240	0.231	5.240	0.187
0.250	0.667	1.250	0.708	2.250	0.436	3.250	0.301	4.250	0.230	5.250	0.187
0.260	0.682	1.260	0.708	2.260	0.434	3.260	0.300	4.260	0.230	5.260	0.186
0.270	0.697	1.270	0.708	2.270	0.432	3.270	0.300	4.270	0.229	5.270	0.186
0.280	0.708	1.280	0.708	2.280	0.430	3.280	0.299	4.280	0.229	5.280	0.185
0.290	0.708	1.290	0.708	2.290	0.428	3.290	0.298	4.290	0.228	5.290	0.185
0.300	0.708	1.300	0.708	2.300	0.427	3.300	0.297	4.300	0.228	5.300	0.185
0.310	0.708	1.310	0.708	2.310	0.427	3.310	0.296	4.310	0.227	5.310	0.184
0.320	0.708	1.320	0.708	2.320	0.426	3.320	0.295	4.320	0.227	5.320	0.184
0.330	0.708	1.330	0.708	2.330	0.425	3.330	0.294	4.330	0.226	5.330	0.184
0.340	0.708	1.340	0.708	2.340	0.424	3.340	0.293	4.340	0.226	5.340	0.183
0.350	0.708	1.350	0.708	2.350	0.423	3.350	0.292	4.350	0.225	5.350	0.183
0.360	0.708	1.360	0.708	2.360	0.422	3.360	0.292	4.360	0.225	5.360	0.183
0.370	0.708	1.370	0.708	2.370	0.421	3.370	0.291	4.370	0.224	5.370	0.182
0.380	0.708	1.380	0.708	2.380	0.420	3.380	0.290	4.380	0.224	5.380	0.182
0.390	0.708	1.390	0.705	2.390	0.419	3.390	0.289	4.390	0.223	5.390	0.182
0.400	0.708	1.400	0.700	2.400	0.418	3.400	0.288	4.400	0.223	5.400	0.181

0.410	0.708	1.410	0.696	2.410	0.418	3.410	0.287	4.410	0.222	5.410	0.181
0.420	0.708	1.420	0.691	2.420	0.417	3.420	0.286	4.420	0.222	5.420	0.181
0.430	0.708	1.430	0.686	2.430	0.416	3.430	0.286	4.430	0.221	5.430	0.180
0.440	0.708	1.440	0.682	2.440	0.415	3.440	0.285	4.440	0.221	5.440	0.180
0.450	0.708	1.450	0.677	2.450	0.414	3.450	0.284	4.450	0.220	5.450	0.180
0.460	0.708	1.460	0.673	2.460	0.414	3.460	0.283	4.460	0.220	5.460	0.179
0.470	0.708	1.470	0.668	2.470	0.413	3.470	0.282	4.470	0.219	5.470	0.179
0.480	0.708	1.480	0.663	2.480	0.412	3.480	0.282	4.480	0.219	5.480	0.179
0.490	0.708	1.490	0.659	2.490	0.411	3.490	0.281	4.490	0.218	5.490	0.178
0.500	0.708	1.500	0.654	2.500	0.410	3.500	0.280	4.500	0.218	5.500	0.178
0.510	0.708	1.510	0.650	2.510	0.409	3.510	0.279	4.510	0.217	5.510	0.178
0.520	0.708	1.520	0.645	2.520	0.409	3.520	0.278	4.520	0.217	5.520	0.177
0.530	0.708	1.530	0.640	2.530	0.408	3.530	0.277	4.530	0.216	5.530	0.177
0.540	0.708	1.540	0.636	2.540	0.407	3.540	0.277	4.540	0.216	5.540	0.177
0.550	0.708	1.550	0.632	2.550	0.405	3.550	0.276	4.550	0.215	5.550	0.176
0.560	0.708	1.560	0.628	2.560	0.404	3.560	0.275	4.560	0.215	5.560	0.176
0.570	0.708	1.570	0.625	2.570	0.403	3.570	0.274	4.570	0.214	5.570	0.176
0.580	0.708	1.580	0.621	2.580	0.402	3.580	0.274	4.580	0.214	5.580	0.176
0.590	0.708	1.590	0.617	2.590	0.401	3.590	0.273	4.590	0.213	5.590	0.175
0.600	0.708	1.600	0.613	2.600	0.400	3.600	0.272	4.600	0.213	5.600	0.175
0.610	0.708	1.610	0.610	2.610	0.399	3.610	0.271	4.610	0.212	5.610	0.175
0.620	0.708	1.620	0.607	2.620	0.398	3.620	0.271	4.620	0.212	5.620	0.174
0.630	0.708	1.630	0.605	2.630	0.397	3.630	0.270	4.630	0.212	5.630	0.174
0.640	0.708	1.640	0.603	2.640	0.396	3.640	0.269	4.640	0.211	5.640	0.174
0.650	0.708	1.650	0.600	2.650	0.395	3.650	0.268	4.650	0.211	5.650	0.173
0.660	0.708	1.660	0.597	2.660	0.394	3.660	0.268	4.660	0.210	5.660	0.173
0.670	0.708	1.670	0.594	2.670	0.393	3.670	0.267	4.670	0.210	5.670	0.173
0.680	0.708	1.680	0.591	2.680	0.392	3.680	0.266	4.680	0.209	5.680	0.172
0.690	0.708	1.690	0.589	2.690	0.390	3.690	0.265	4.690	0.209	5.690	0.172
0.700	0.708	1.700	0.586	2.700	0.389	3.700	0.265	4.700	0.208	5.700	0.172
0.710	0.708	1.710	0.583	2.710	0.387	3.710	0.264	4.710	0.208	5.710	0.172
0.720	0.708	1.720	0.580	2.720	0.386	3.720	0.263	4.720	0.208	5.720	0.171
0.730	0.708	1.730	0.577	2.730	0.384	3.730	0.263	4.730	0.207	5.730	0.171
0.740	0.708	1.740	0.574	2.740	0.383	3.740	0.262	4.740	0.207	5.740	0.171
0.750	0.708	1.750	0.570	2.750	0.381	3.750	0.261	4.750	0.206	5.750	0.170
0.760	0.708	1.760	0.566	2.760	0.380	3.760	0.261	4.760	0.206	5.760	0.170
0.770	0.708	1.770	0.562	2.770	0.378	3.770	0.260	4.770	0.205	5.770	0.170
0.780	0.708	1.780	0.558	2.780	0.377	3.780	0.259	4.780	0.205	5.780	0.169
0.790	0.708	1.790	0.555	2.790	0.375	3.790	0.258	4.790	0.204	5.790	0.169
0.800	0.708	1.800	0.551	2.800	0.374	3.800	0.258	4.800	0.204	5.800	0.169
0.810	0.708	1.810	0.547	2.810	0.372	3.810	0.257	4.810	0.204	5.810	0.169
0.820	0.708	1.820	0.543	2.820	0.371	3.820	0.256	4.820	0.203	5.820	0.168
0.830	0.708	1.830	0.539	2.830	0.369	3.830	0.256	4.830	0.203	5.830	0.168

0.840	0.708	1.840	0.535	2.840	0.368	3.840	0.255	4.840	0.202	5.840	0.168
0.850	0.708	1.850	0.531	2.850	0.366	3.850	0.254	4.850	0.202	5.850	0.167
0.860	0.708	1.860	0.527	2.860	0.365	3.860	0.254	4.860	0.202	5.860	0.167
0.870	0.708	1.870	0.524	2.870	0.363	3.870	0.253	4.870	0.201	5.870	0.167
0.880	0.708	1.880	0.521	2.880	0.362	3.880	0.252	4.880	0.201	5.880	0.167
0.890	0.708	1.890	0.519	2.890	0.360	3.890	0.252	4.890	0.200	5.890	0.166
0.900	0.708	1.900	0.516	2.900	0.358	3.900	0.251	4.900	0.200	5.900	0.166
0.910	0.708	1.910	0.513	2.910	0.357	3.910	0.251	4.910	0.199	5.910	0.166
0.920	0.708	1.920	0.511	2.920	0.355	3.920	0.250	4.920	0.199	5.920	0.165
0.930	0.708	1.930	0.508	2.930	0.354	3.930	0.249	4.930	0.199	5.930	0.165
0.940	0.708	1.940	0.506	2.940	0.352	3.940	0.249	4.940	0.198	5.940	0.165
0.950	0.708	1.950	0.503	2.950	0.350	3.950	0.248	4.950	0.198	5.950	0.165
0.960	0.708	1.960	0.500	2.960	0.349	3.960	0.247	4.960	0.197	5.960	0.164
0.970	0.708	1.970	0.498	2.970	0.347	3.970	0.247	4.970	0.197	5.970	0.164
0.980	0.708	1.980	0.495	2.980	0.345	3.980	0.246	4.980	0.197	5.980	0.164
0.990	0.708	1.990	0.492	2.990	0.344	3.990	0.245	4.990	0.196	5.990	0.163
1.000	0.708	2.000	0.490	3.000	0.342	4.000	0.245	5.000	0.196		

APPENDIX F

LIQUEFACTION ANALYSIS RESULTS – Cliq

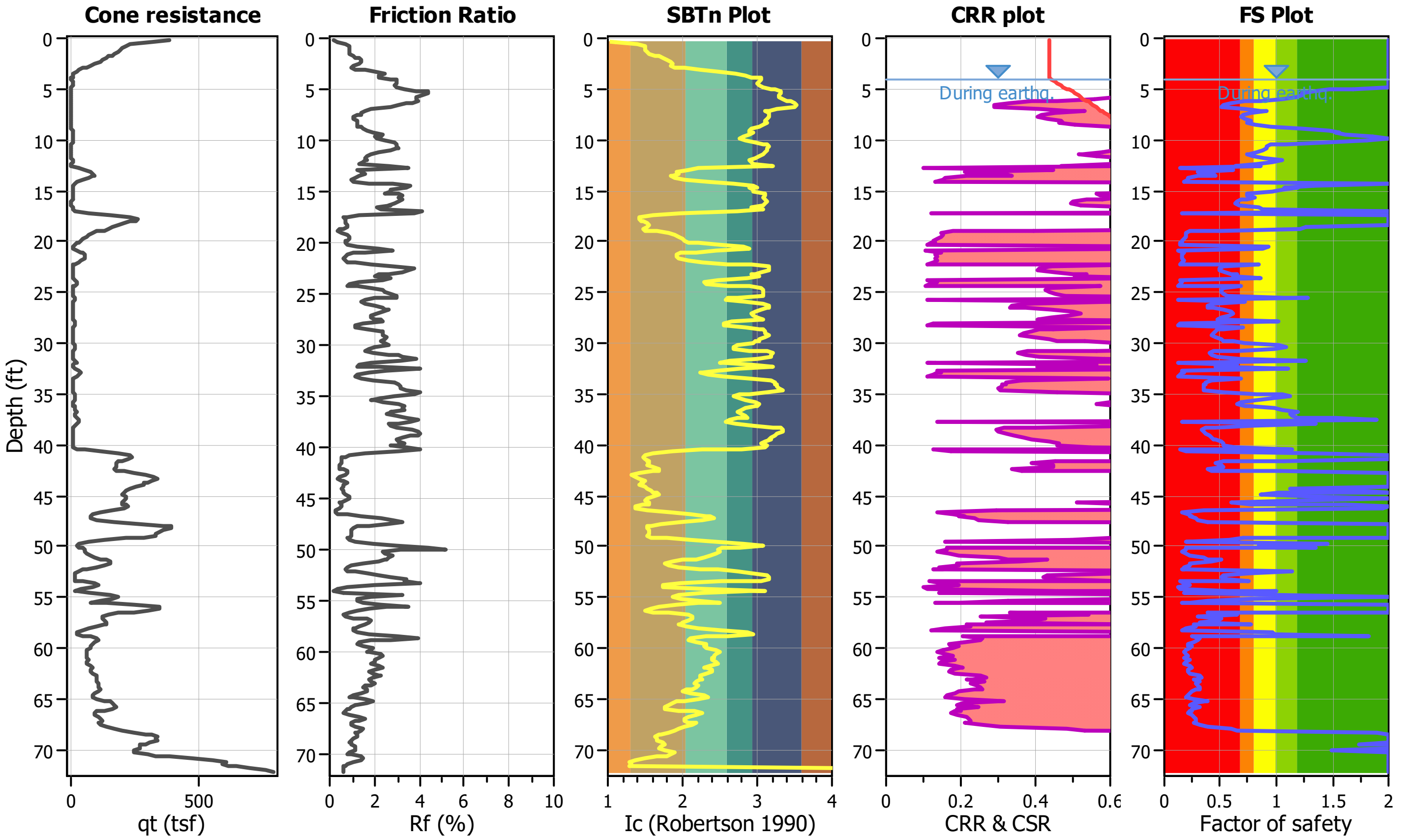
LIQUEFACTION ANALYSIS REPORT

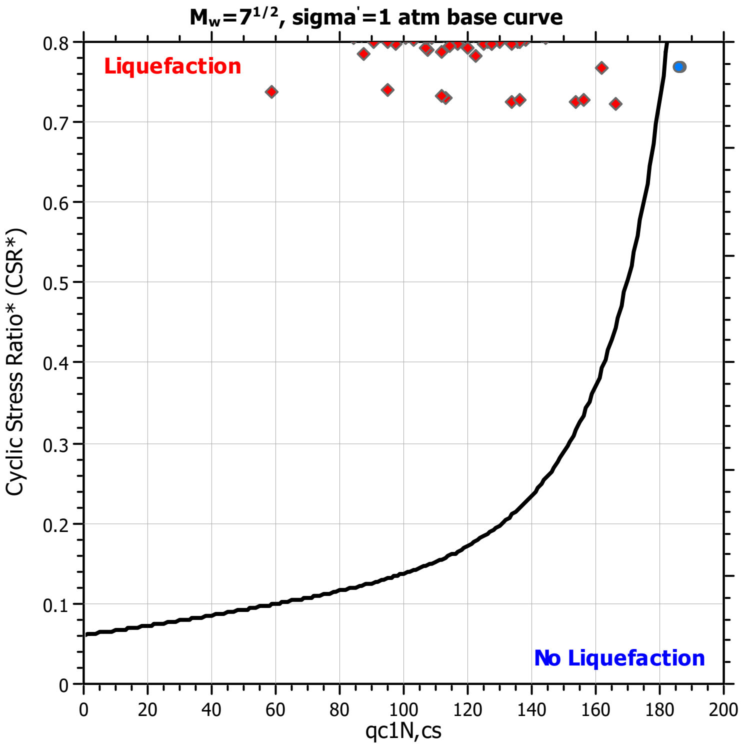
Project title : Issaquah TOD **Location : 1550 Newport Way Northwest, Issaquah, Washington**

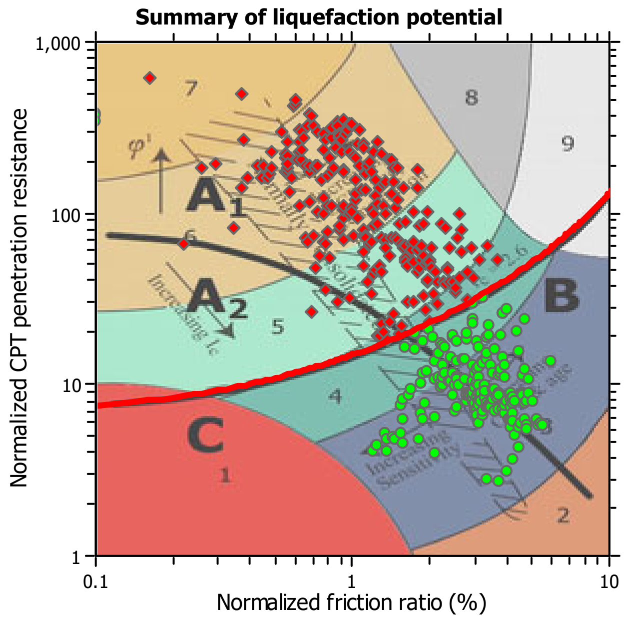
CPT file : CPT-01.pdf

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	4.50 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	4.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method
Peak ground acceleration:	0.74	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

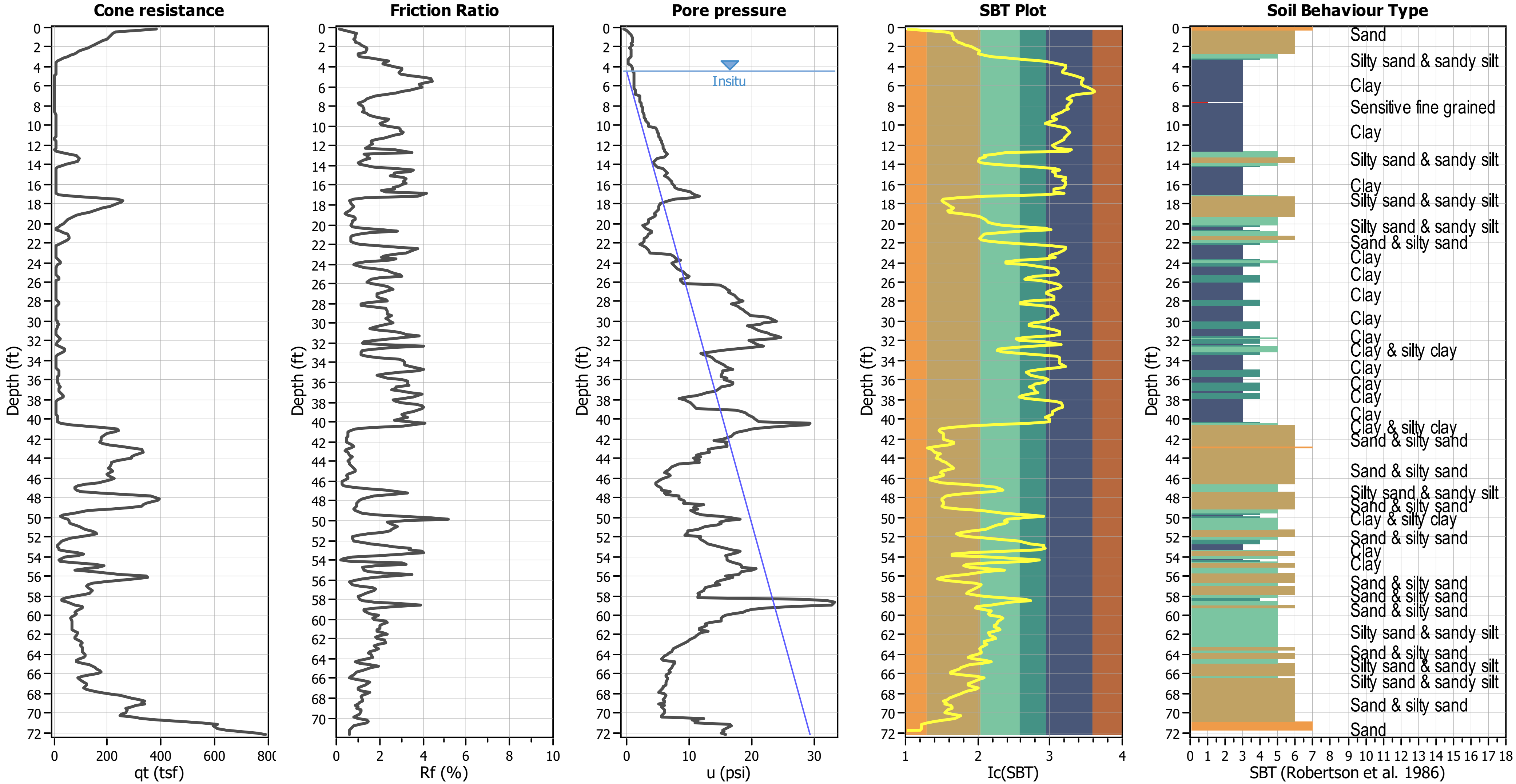






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



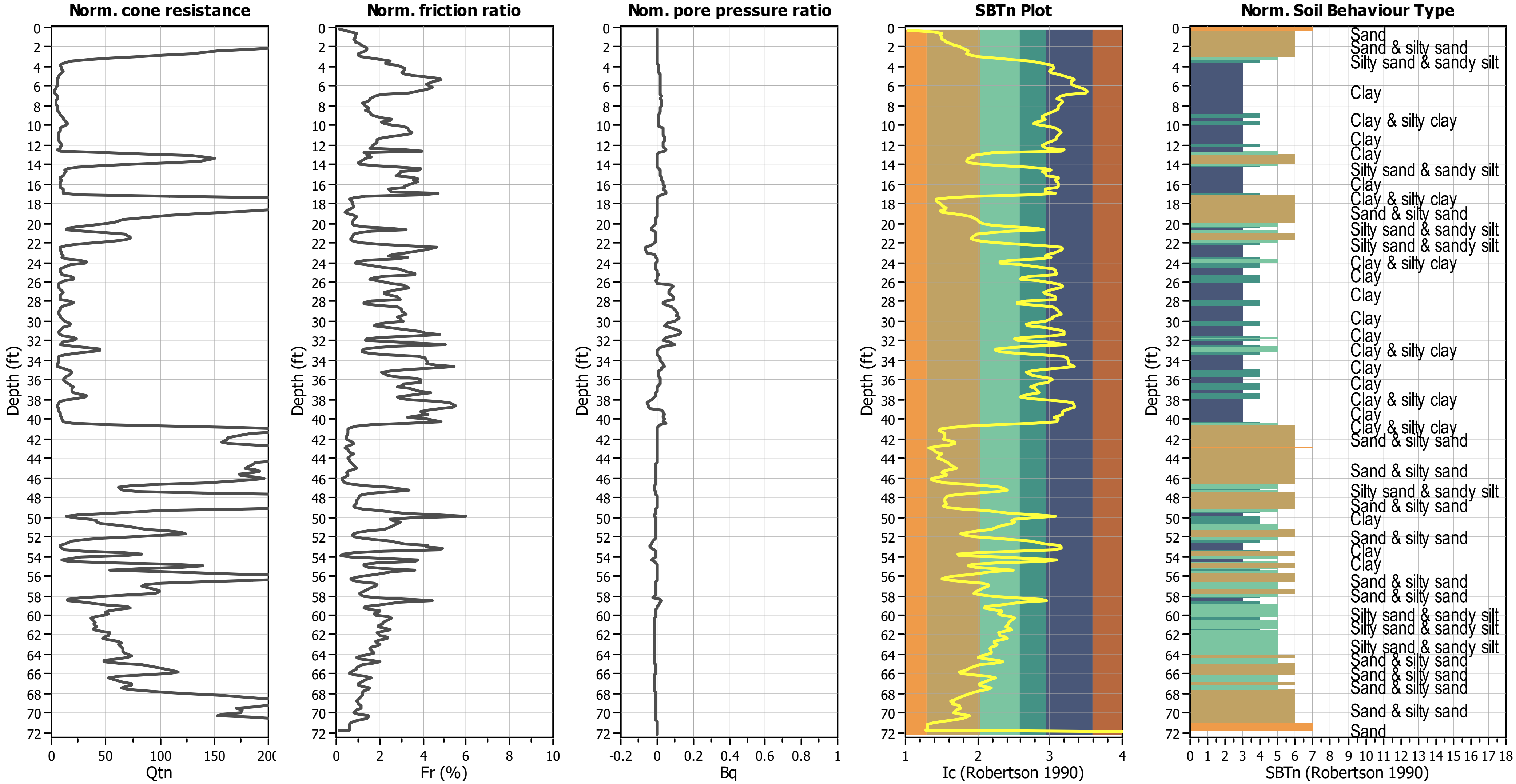
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.50 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

CPT basic interpretation plots (normalized)



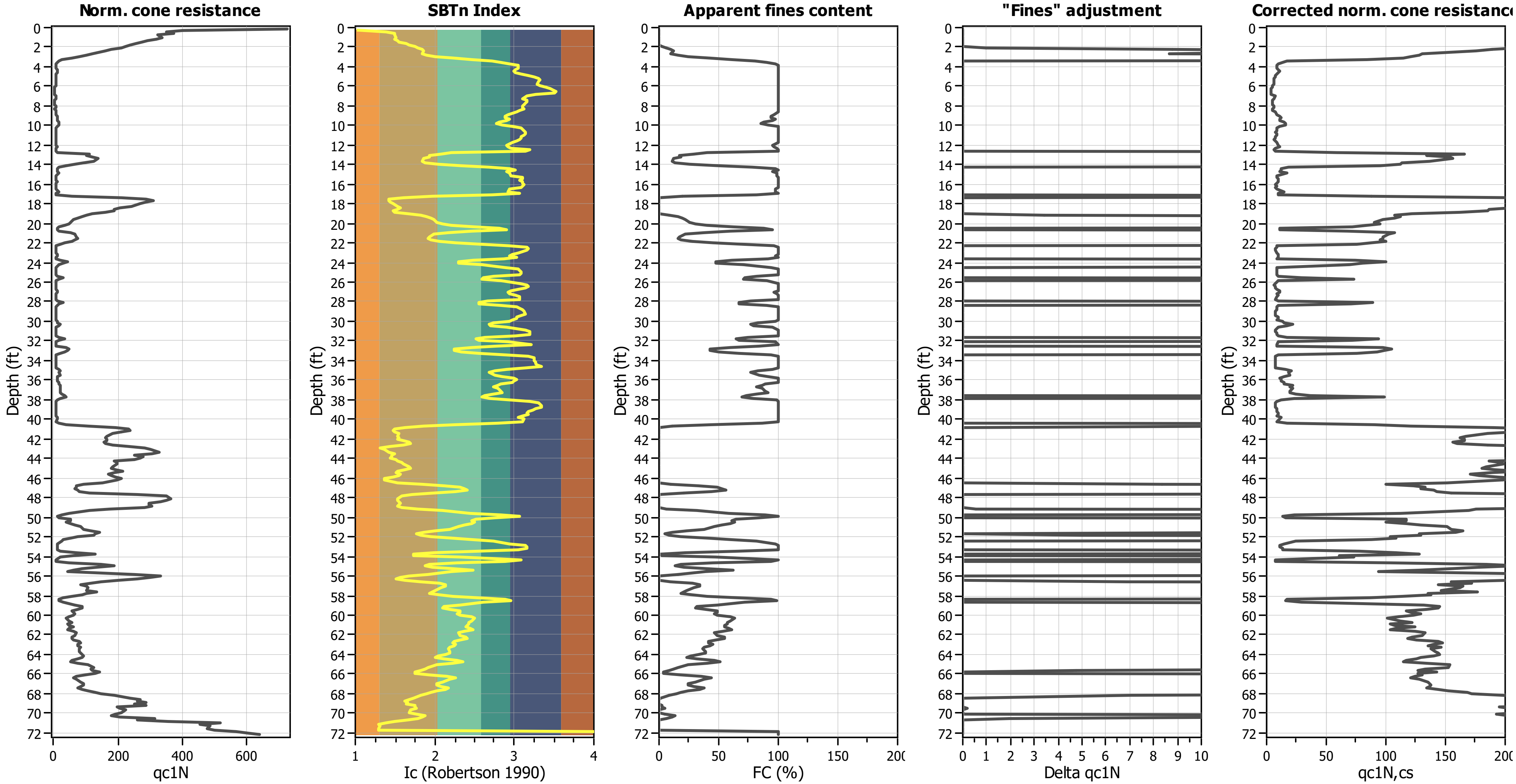
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.50 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

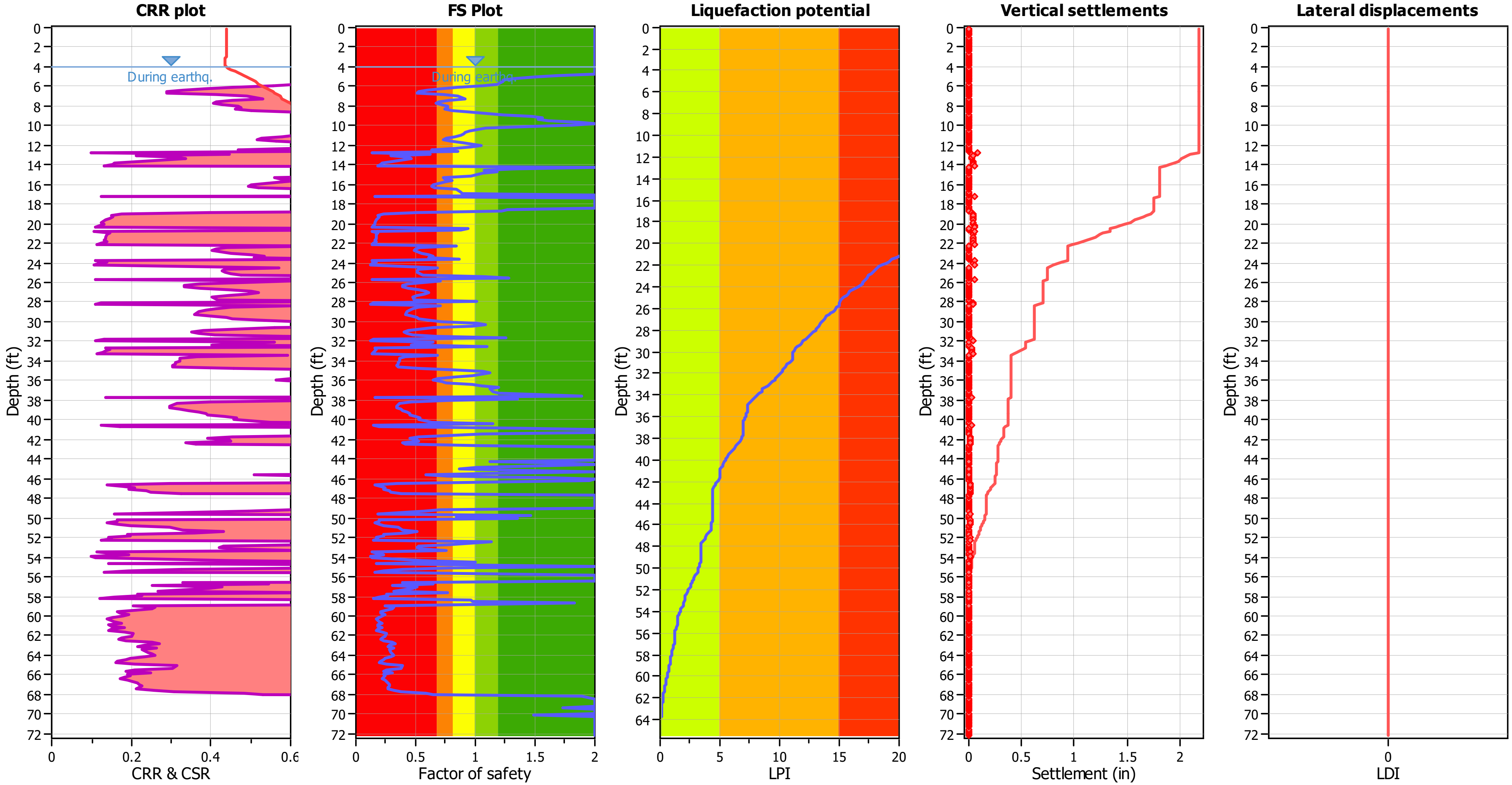
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.50 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.50 ft	Fill height:	N/A	Limit depth:	N/A

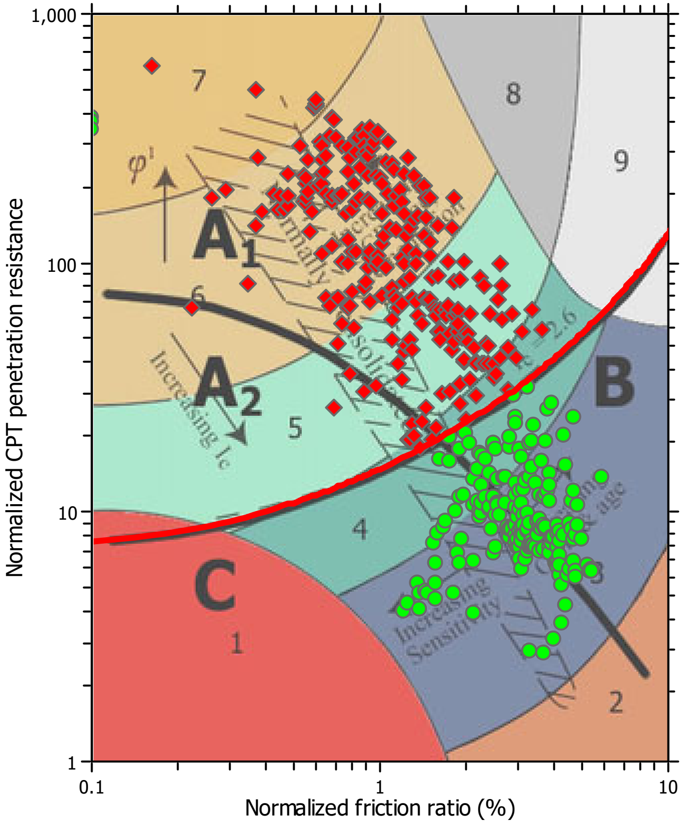
F.S. color scheme

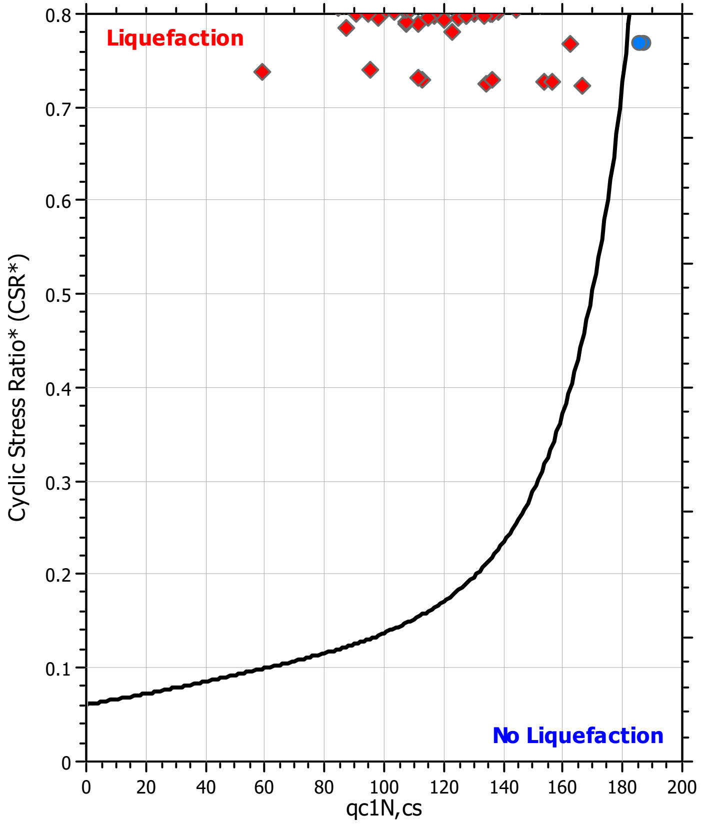
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

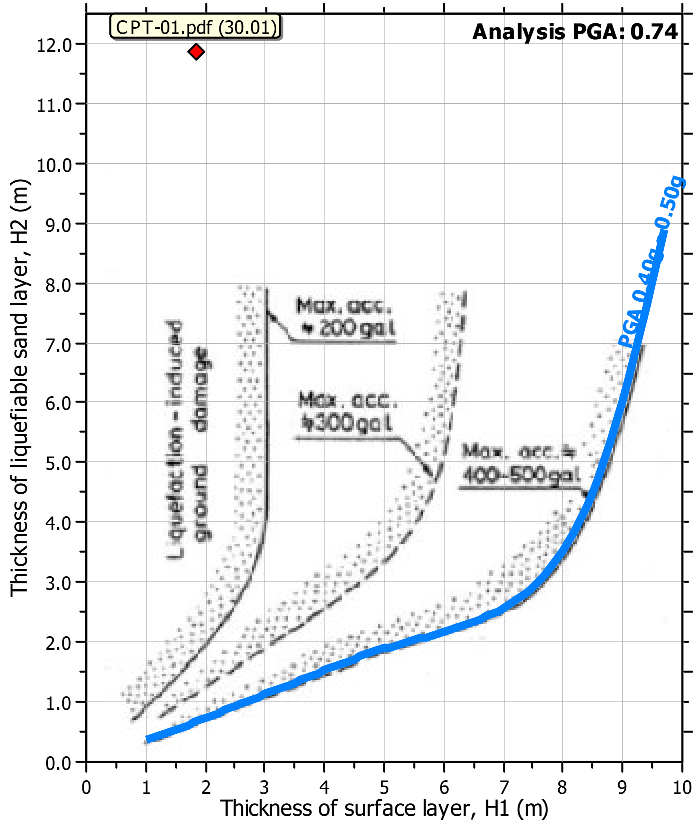
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



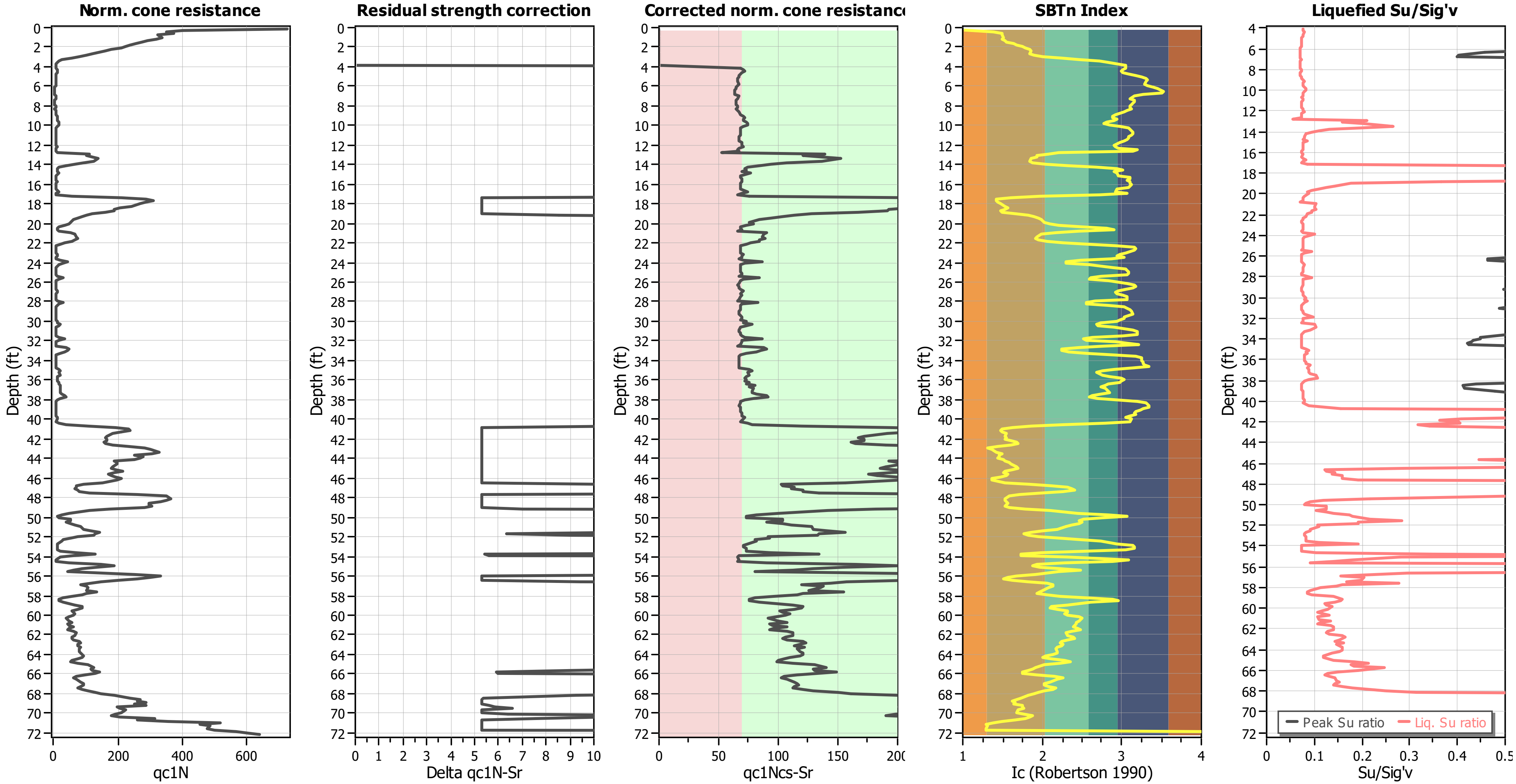




Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.50 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.50 ft	Fill height:	N/A	Limit depth:	N/A

PanGEO GEOTECHNICAL CONSULTANTS

PanGEO Inc.
3213 Eastlake Ave E, Ste B
Seattle, WA 98102
www.pangeoinc.com

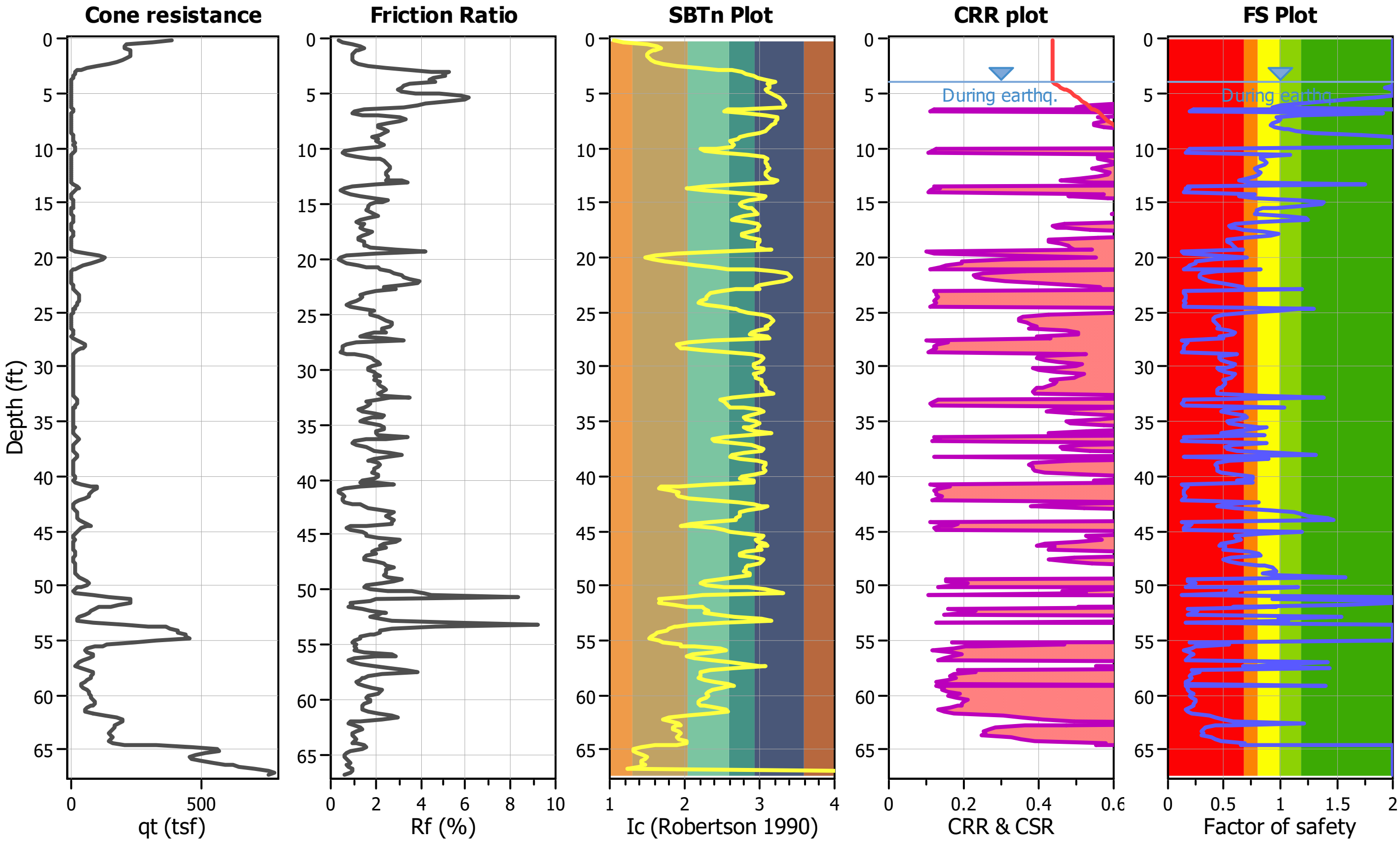
LIQUEFACTION ANALYSIS REPORT

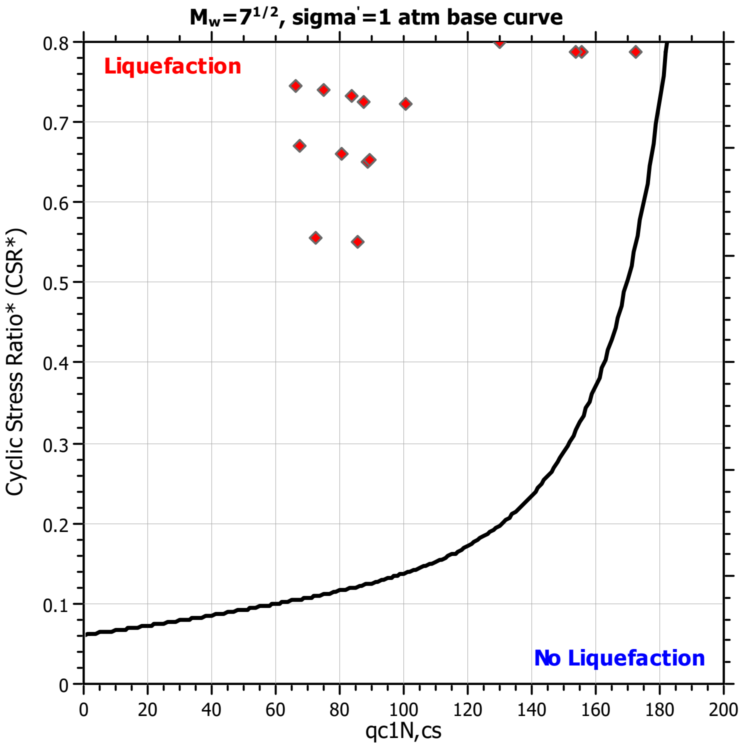
Project title : Issaquah TOD **Location : 1550 Newport Way Northwest, Issaquah, Washington**

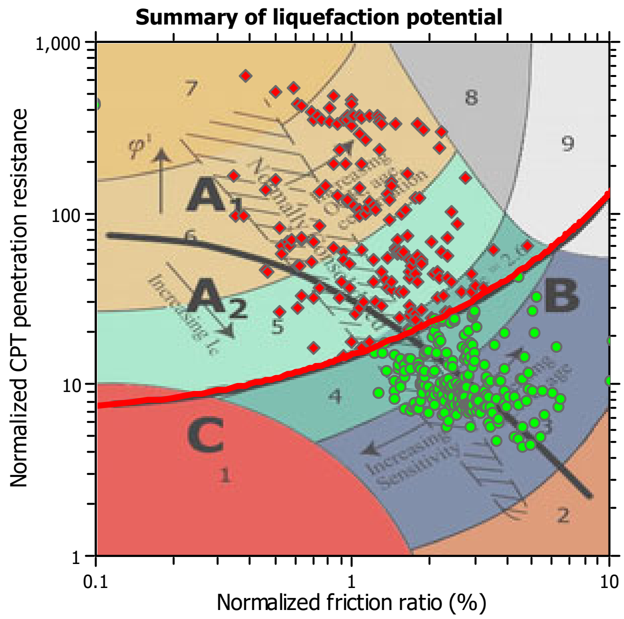
CPT file : CPT-02.pdf

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	4.30 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	4.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method
Peak ground acceleration:	0.74	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

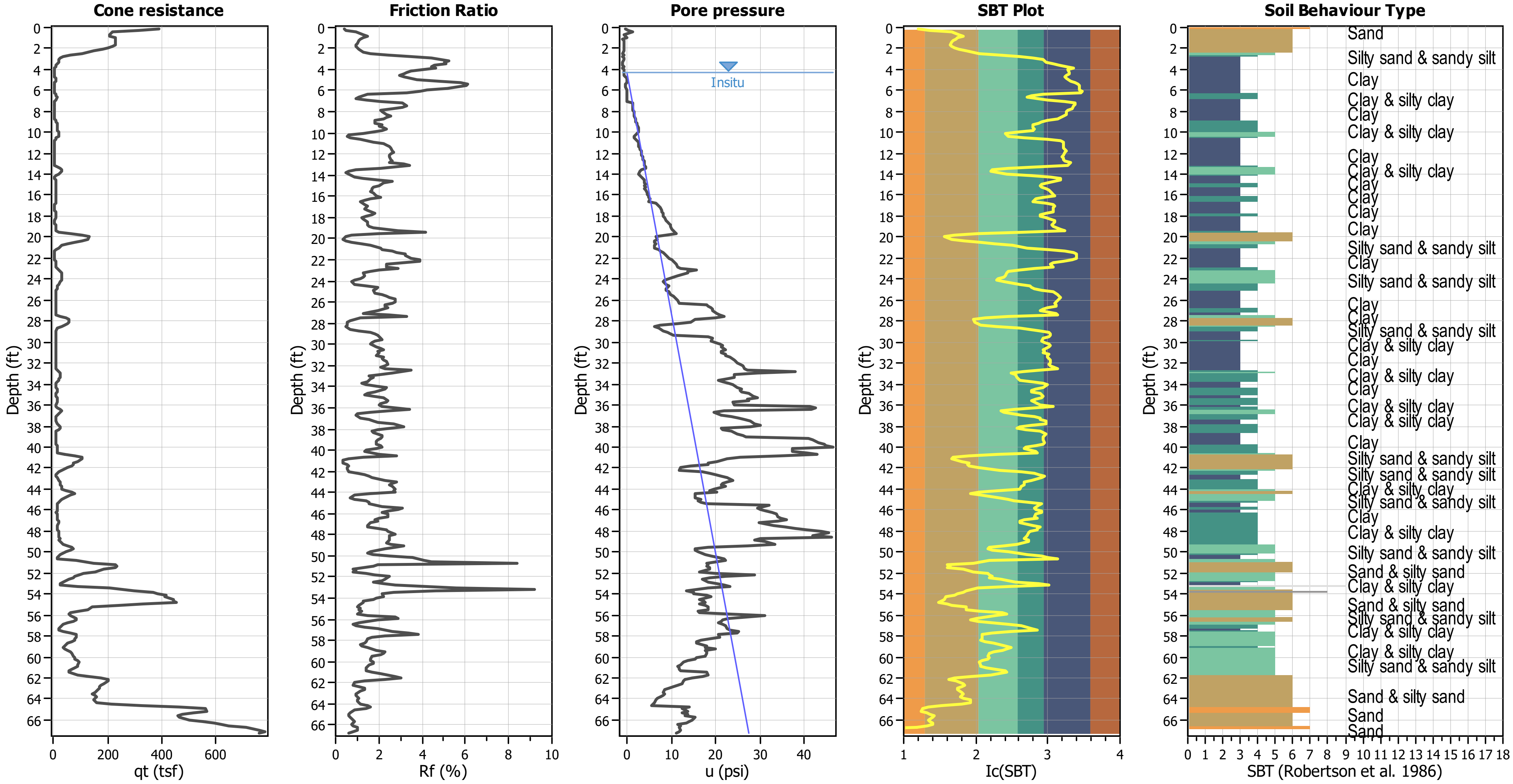






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



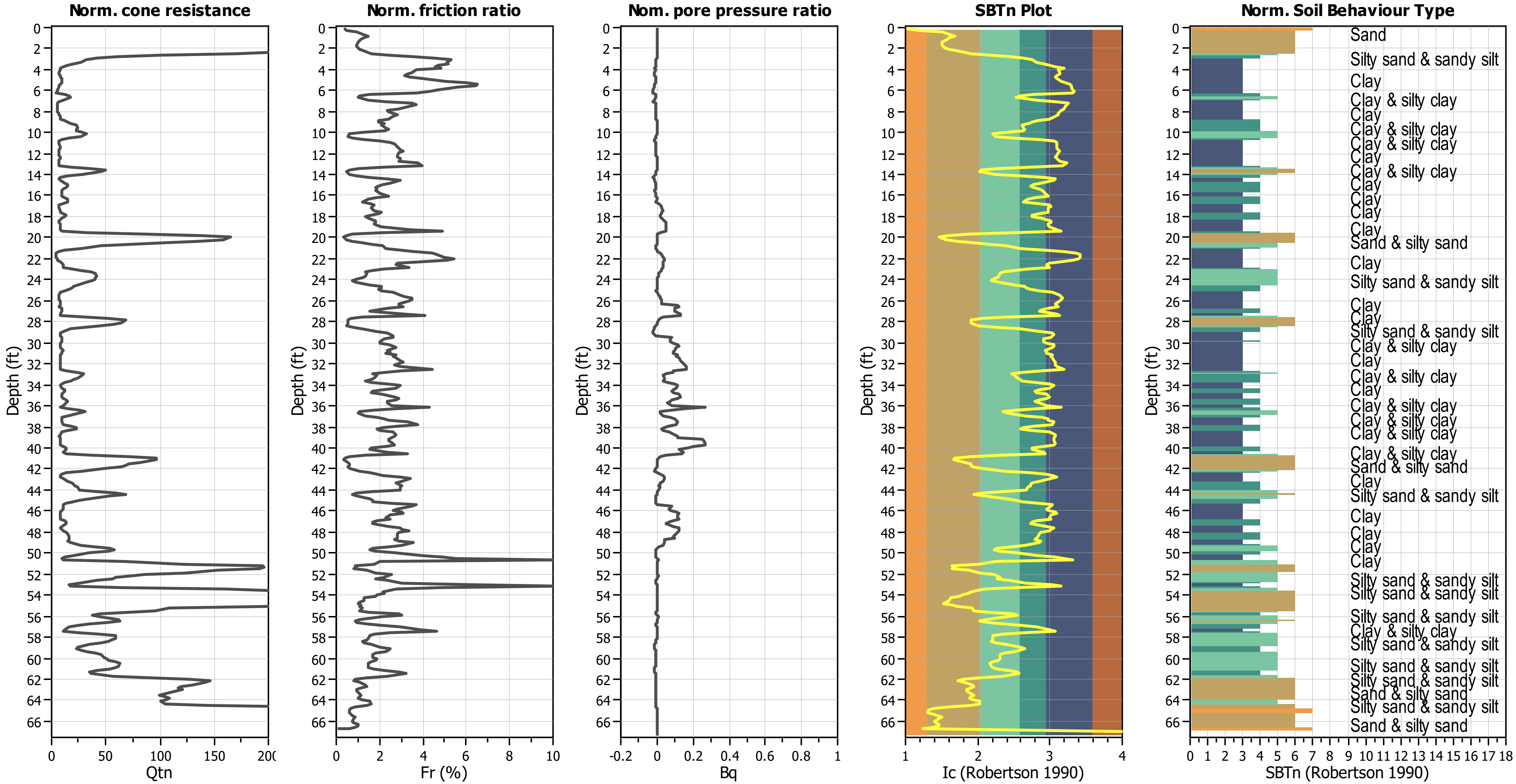
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

CPT basic interpretation plots (normalized)



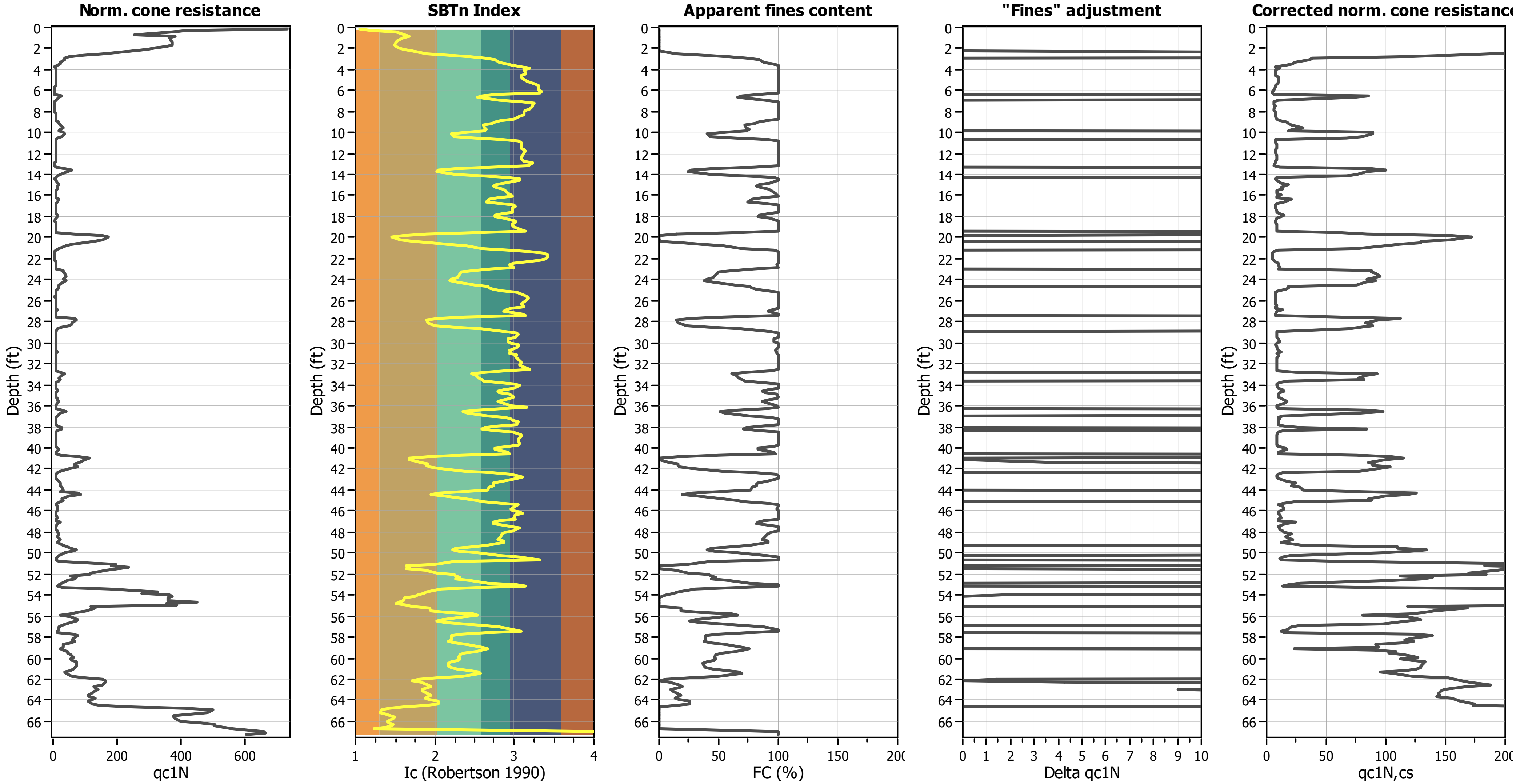
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

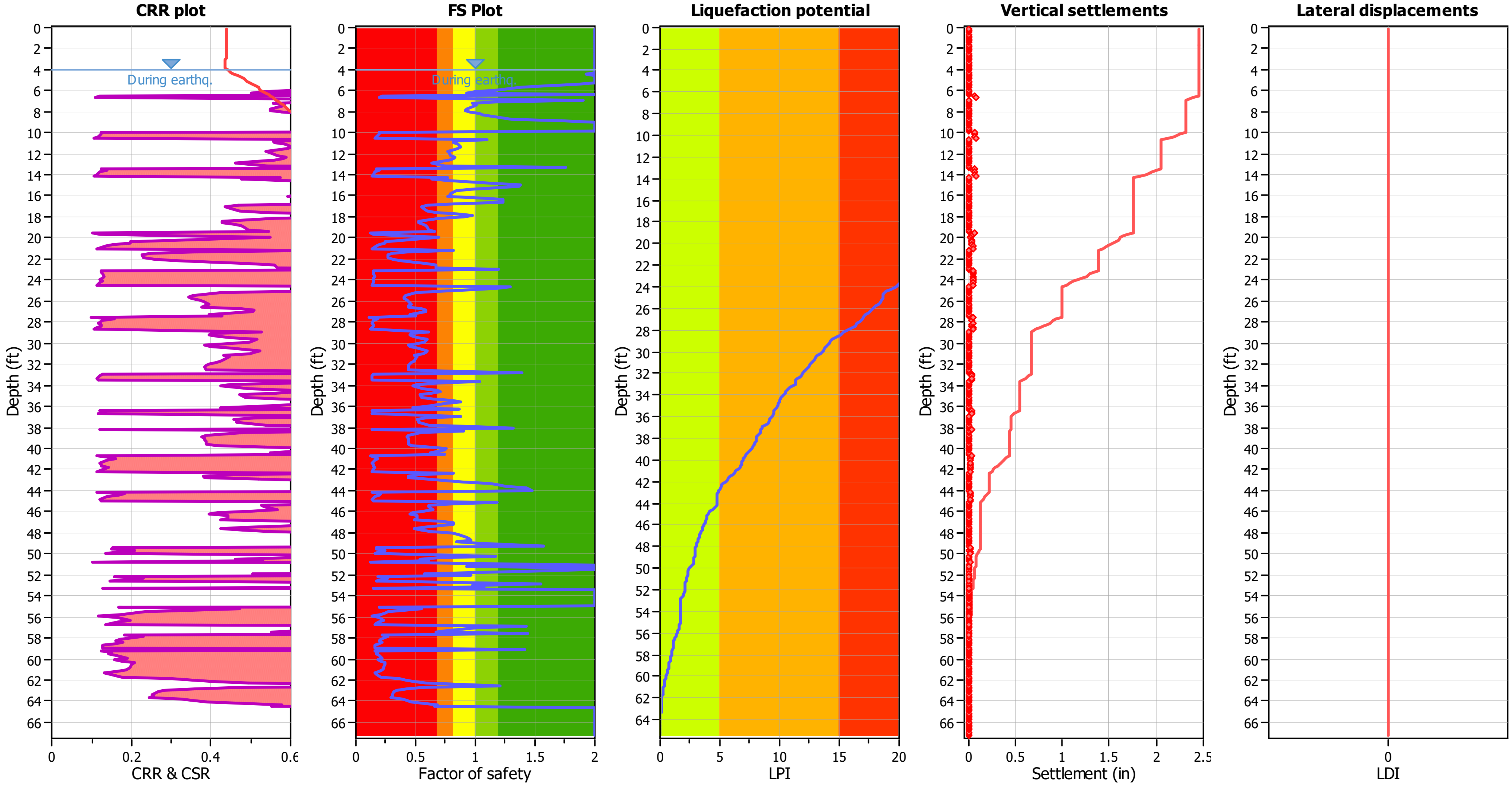
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)
Fines correction method:	B&I (2014)
Points to test:	Based on Ic value
Earthquake magnitude M_w:	7.50
Peak ground acceleration:	0.74
Depth to water table (insitu):	4.30 ft
Depth to GWT (erthq.):	4.00 ft
Average results interval:	3
Ic cut-off value:	2.60
Unit weight calculation:	Based on SBT
Use fill:	No
Fill height:	N/A
Fill weight:	N/A
Transition detect. applied:	No
K_σ applied:	Yes
Clay like behavior applied:	Sand & Clay
Limit depth applied:	No
Limit depth:	N/A

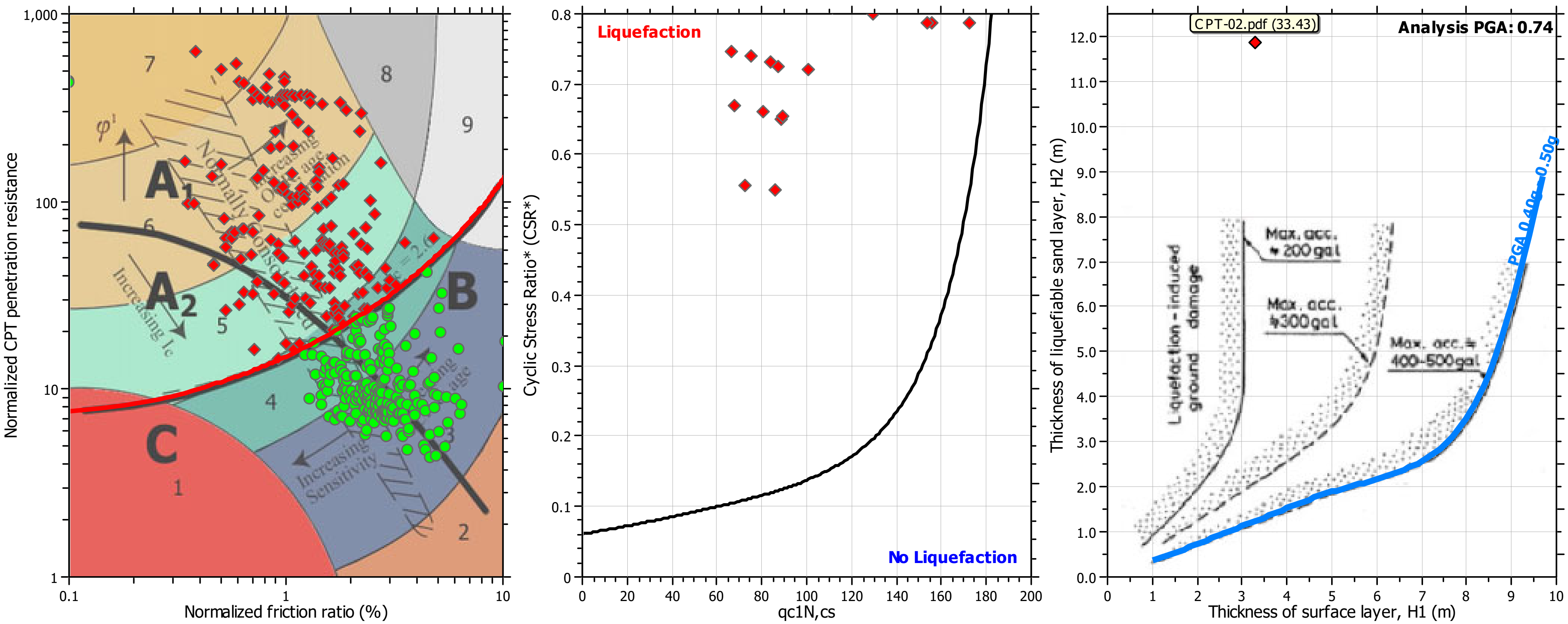
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

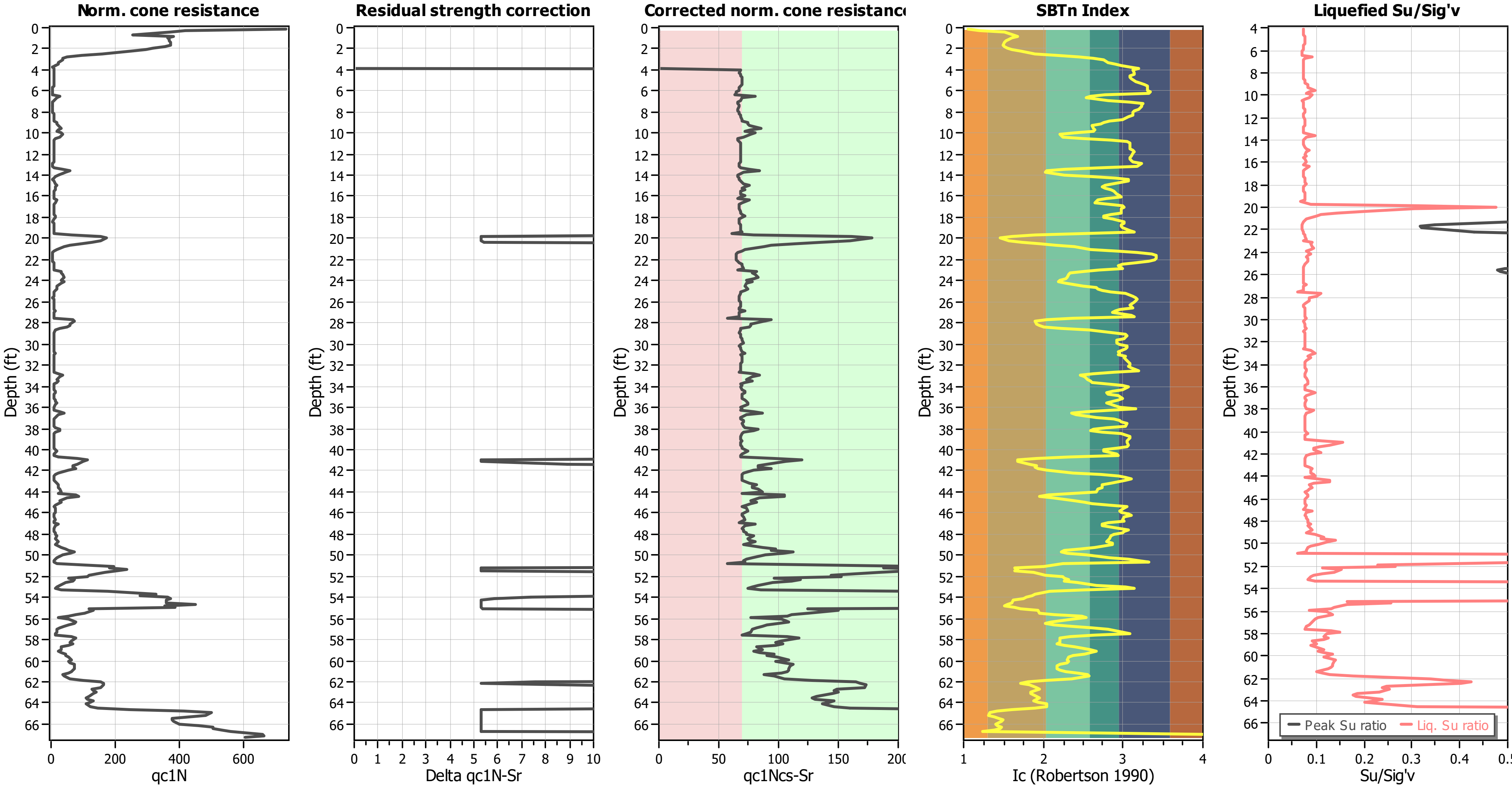
Liquefaction analysis summary plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 ft	Fill height:	N/A	Limit depth:	N/A

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3213 Eastlake Ave E, Ste B
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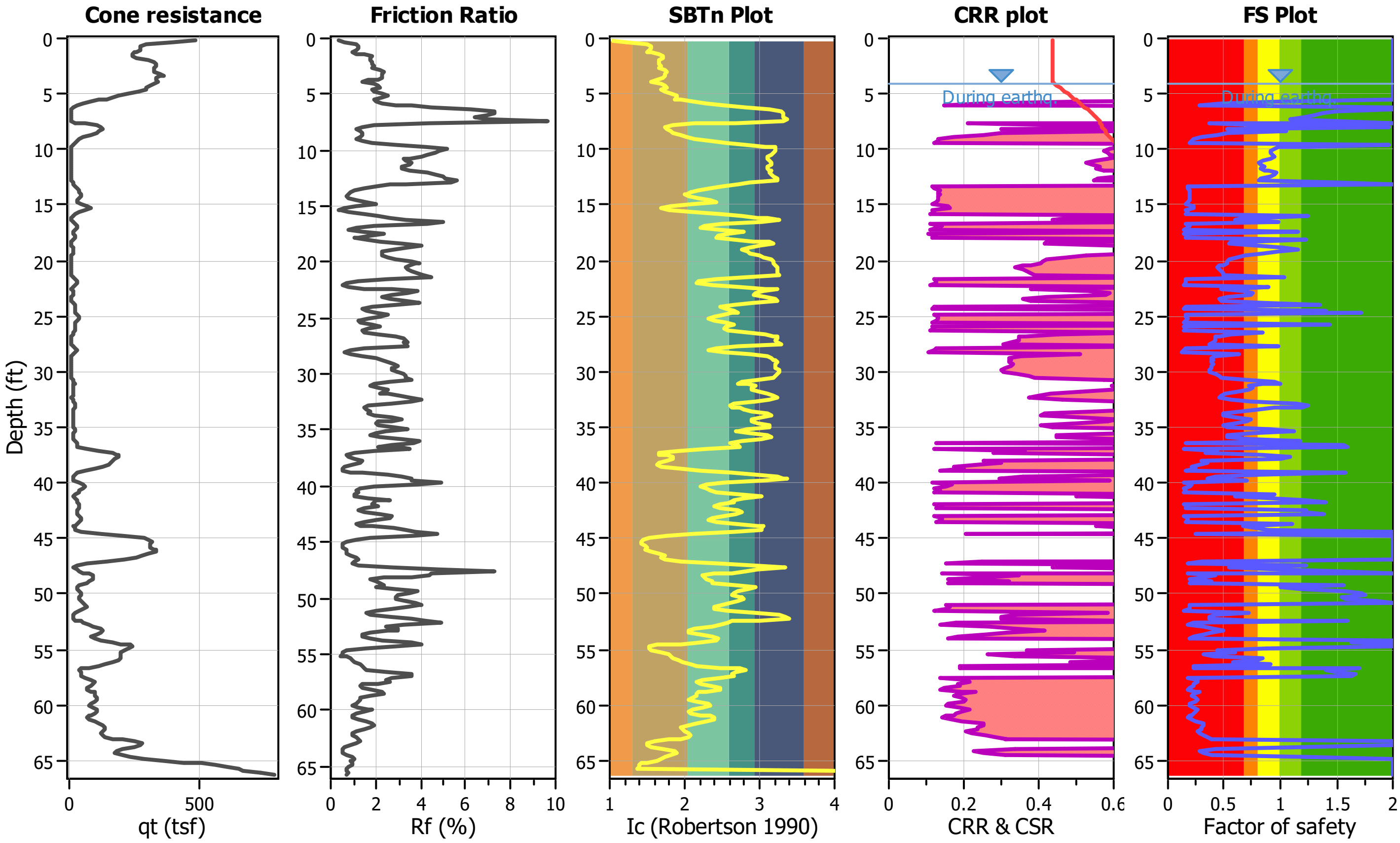
LIQUEFACTION ANALYSIS REPORT

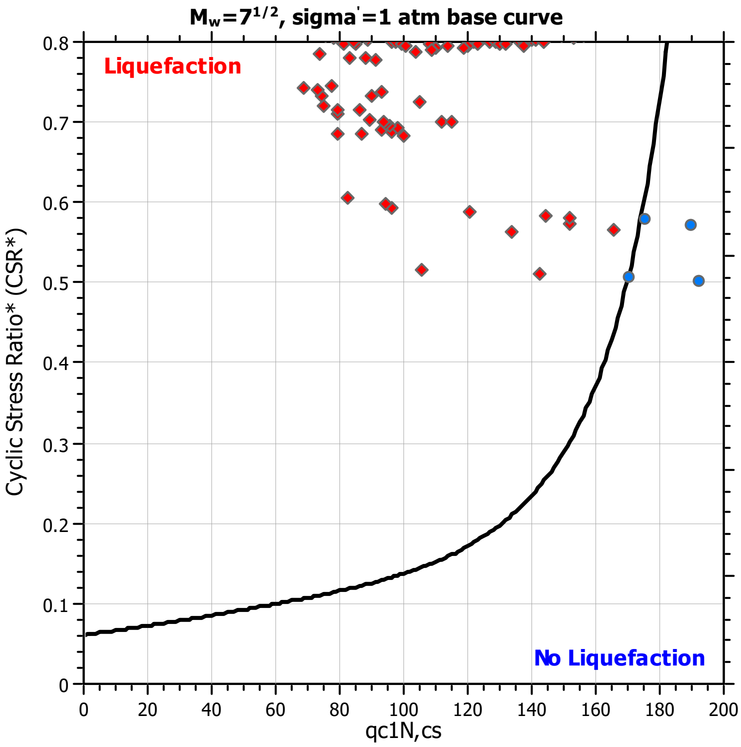
Project title : Issaquah TOD **Location : 1550 Newport Way Northwest, Issaquah, Washington**

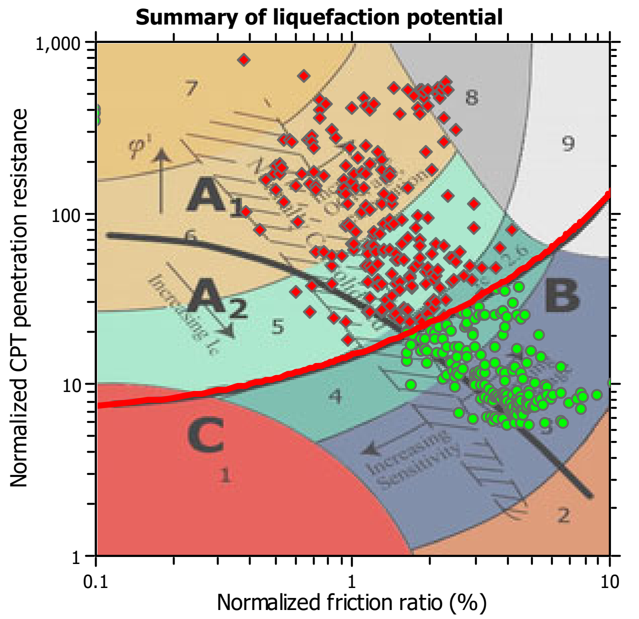
CPT file : CPT-03.pdf

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	4.10 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	4.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method
Peak ground acceleration:	0.74	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

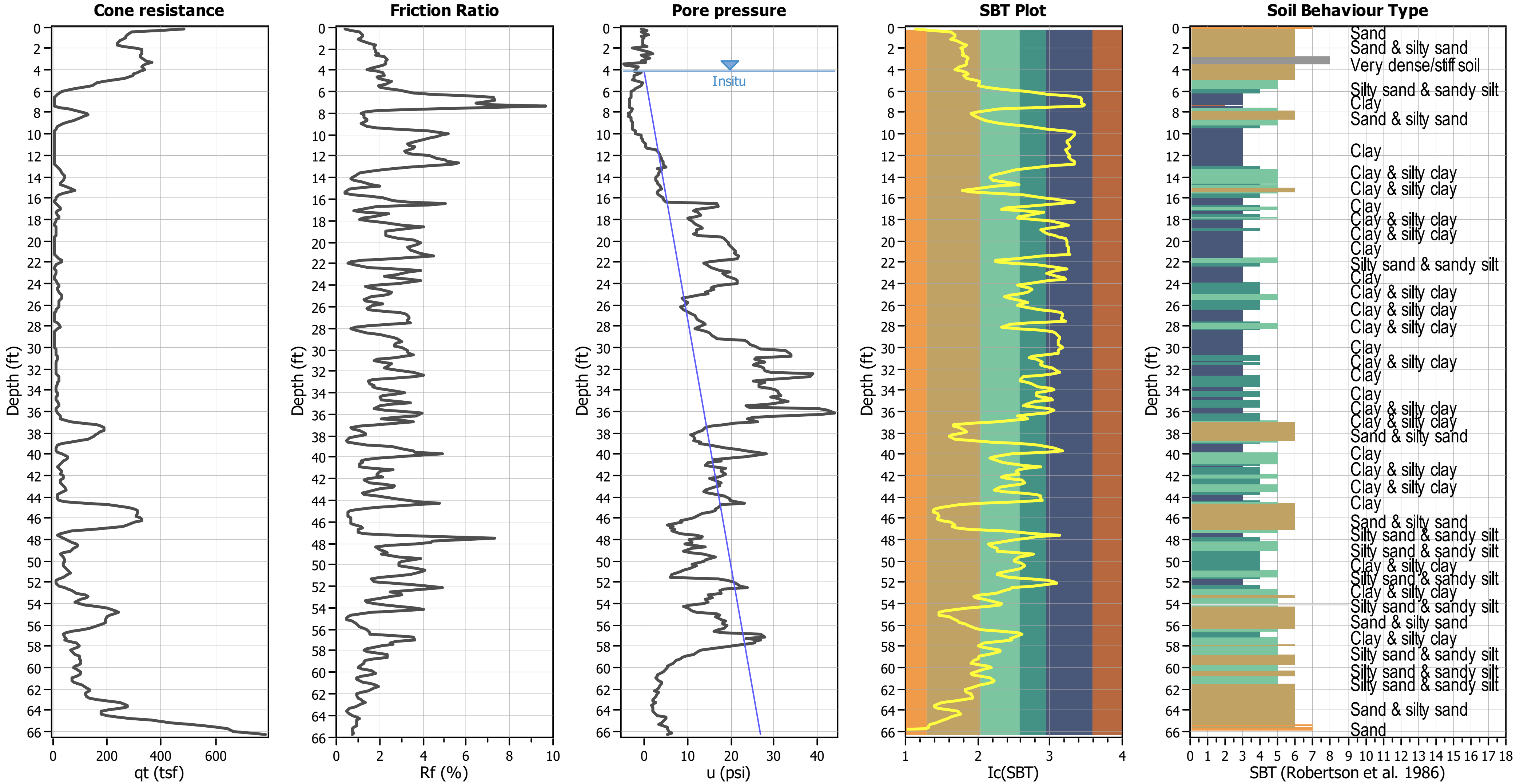






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



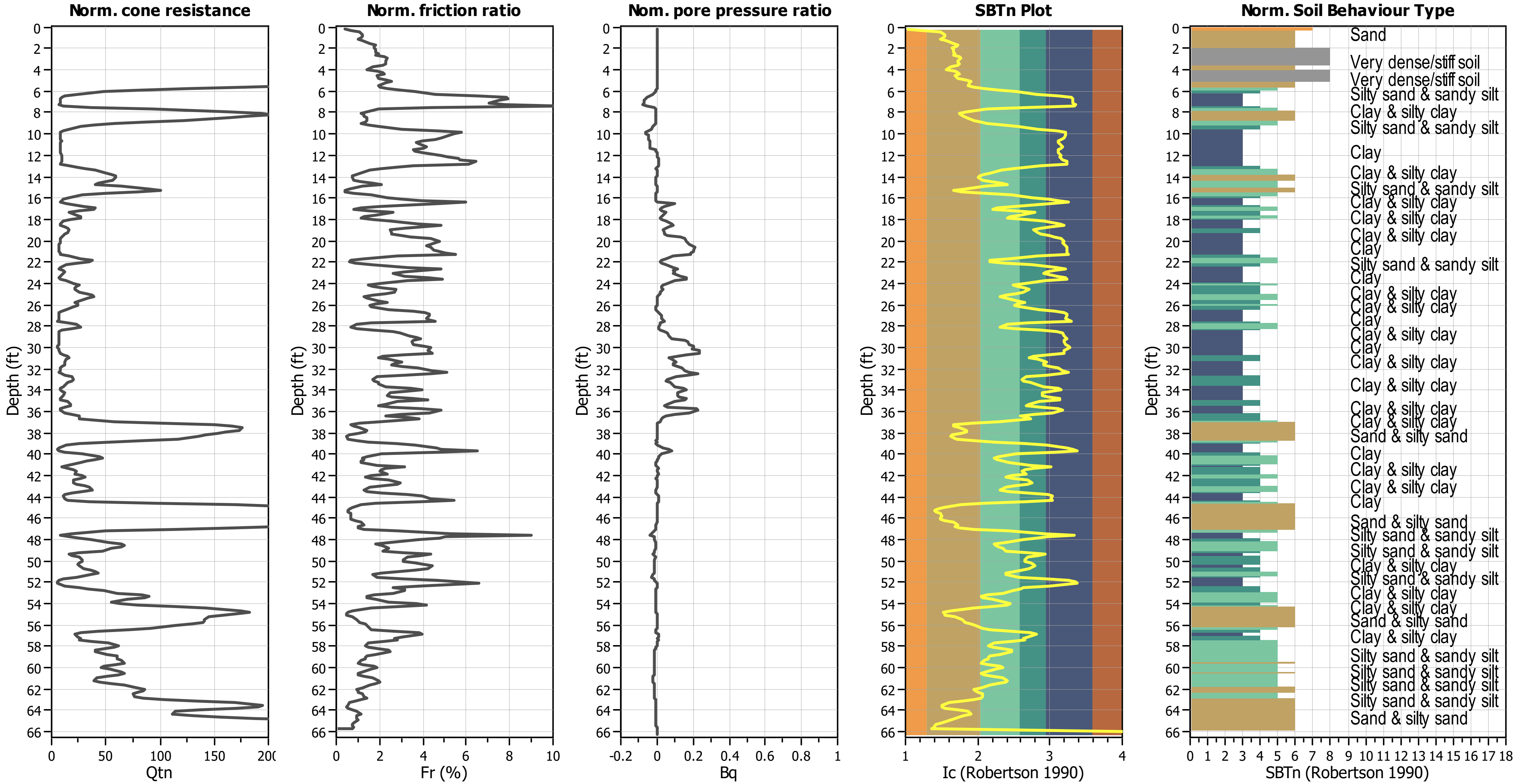
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.10 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

CPT basic interpretation plots (normalized)



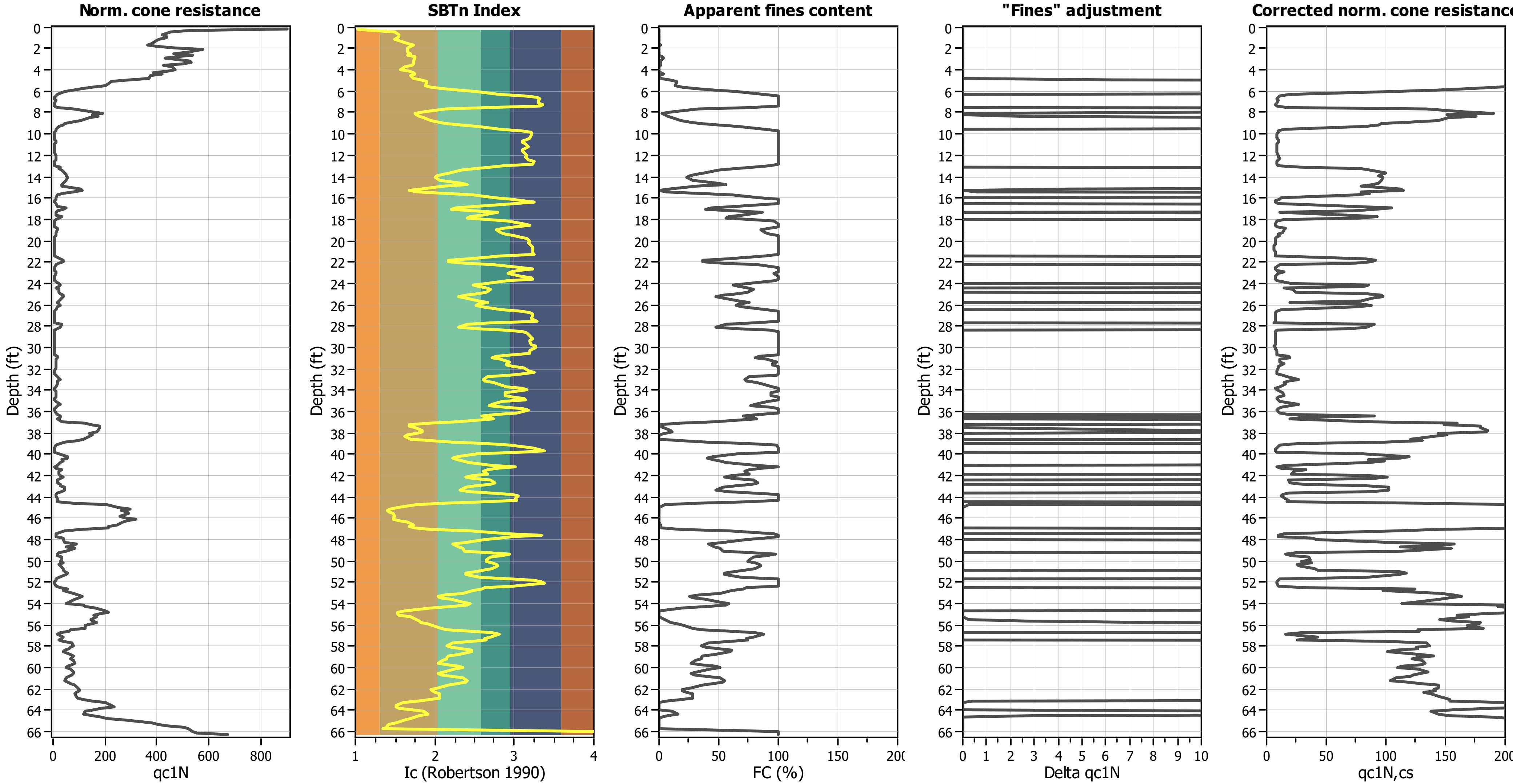
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.10 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

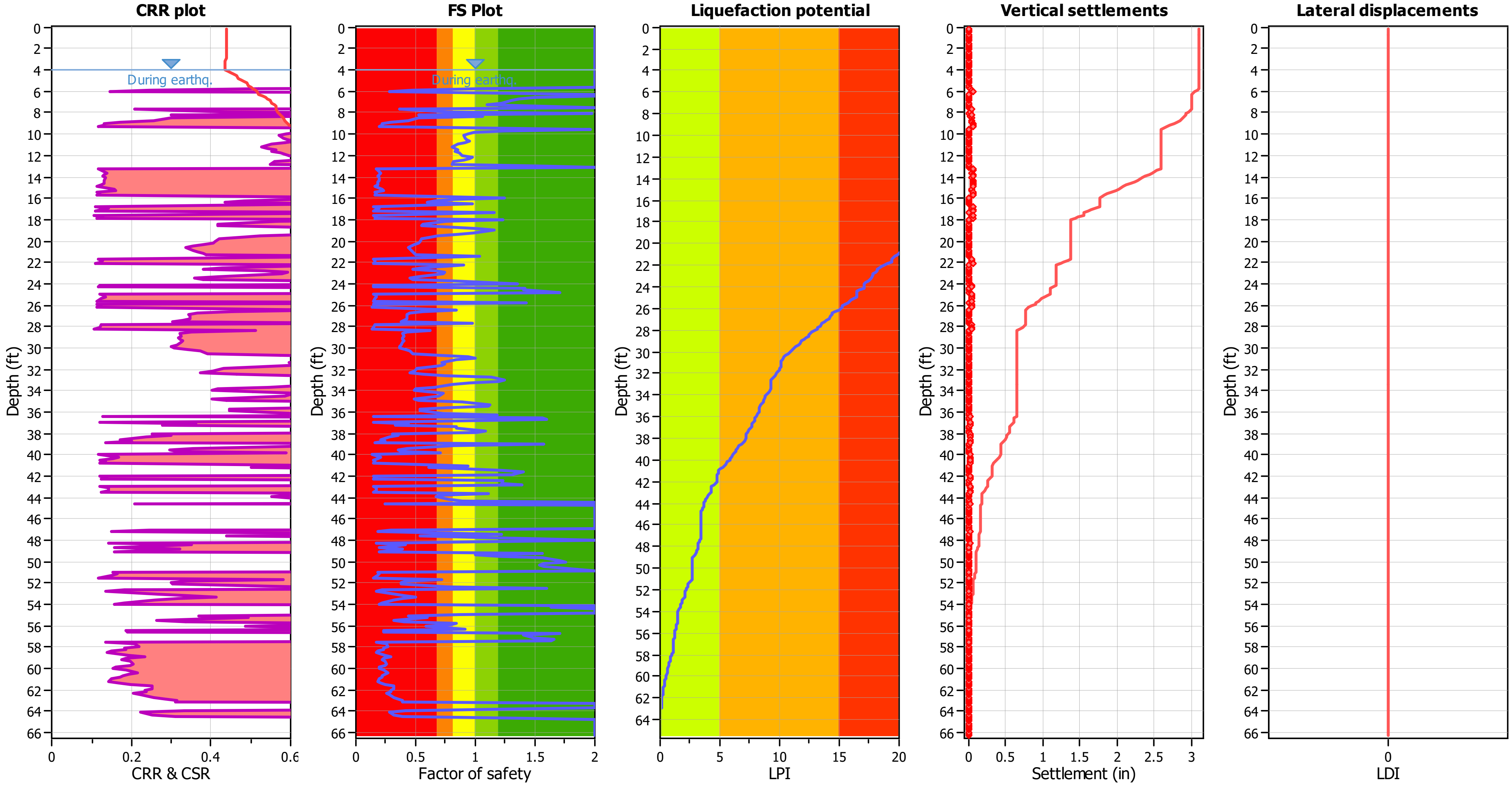
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.10 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.10 ft	Fill height:	N/A	Limit depth:	N/A

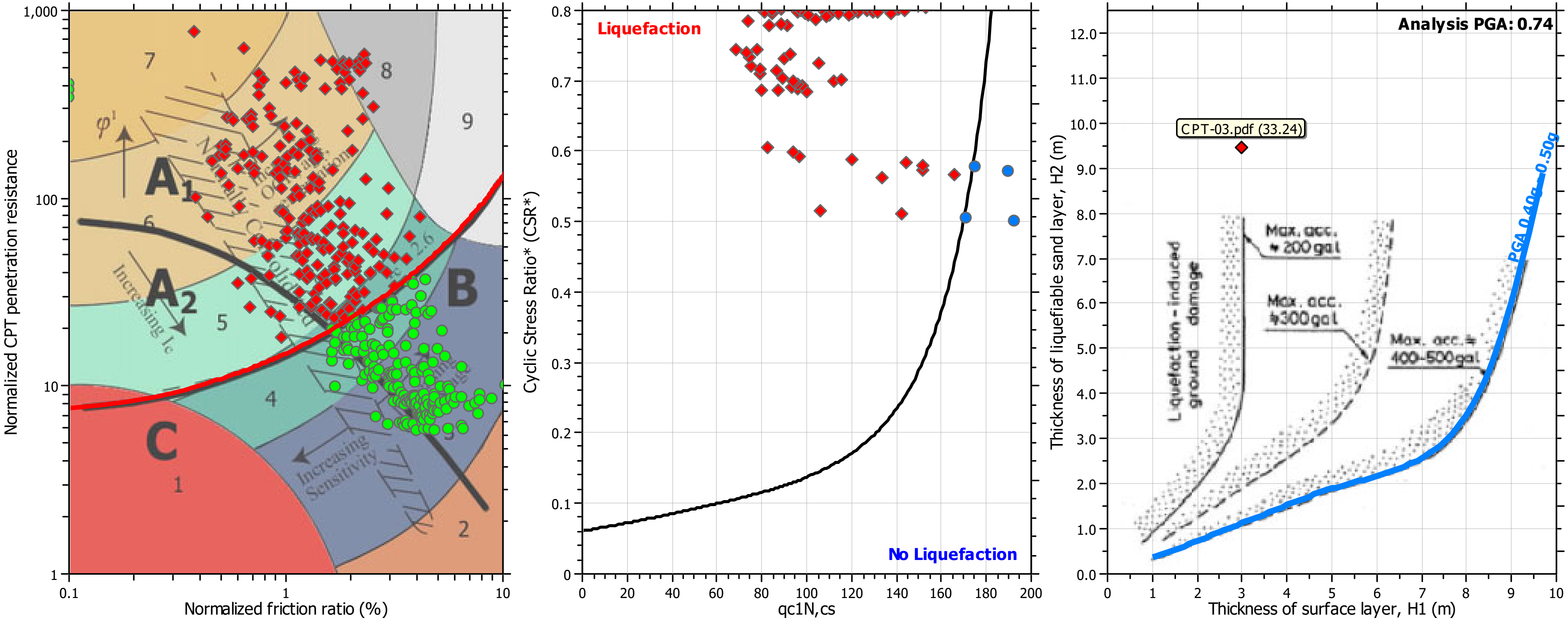
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

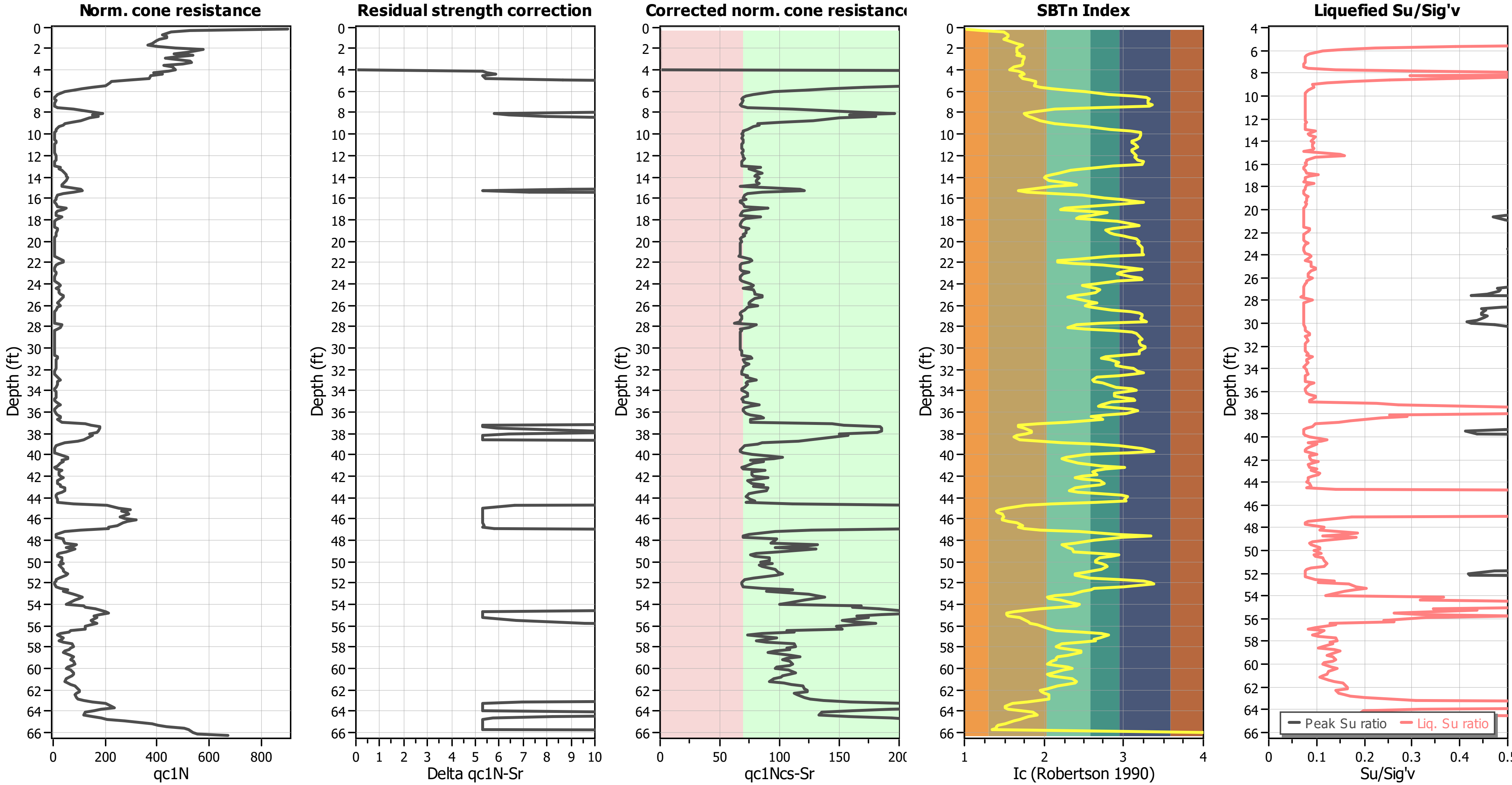
Liquefaction analysis summary plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.10 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.10 ft	Fill height:	N/A	Limit depth:	N/A

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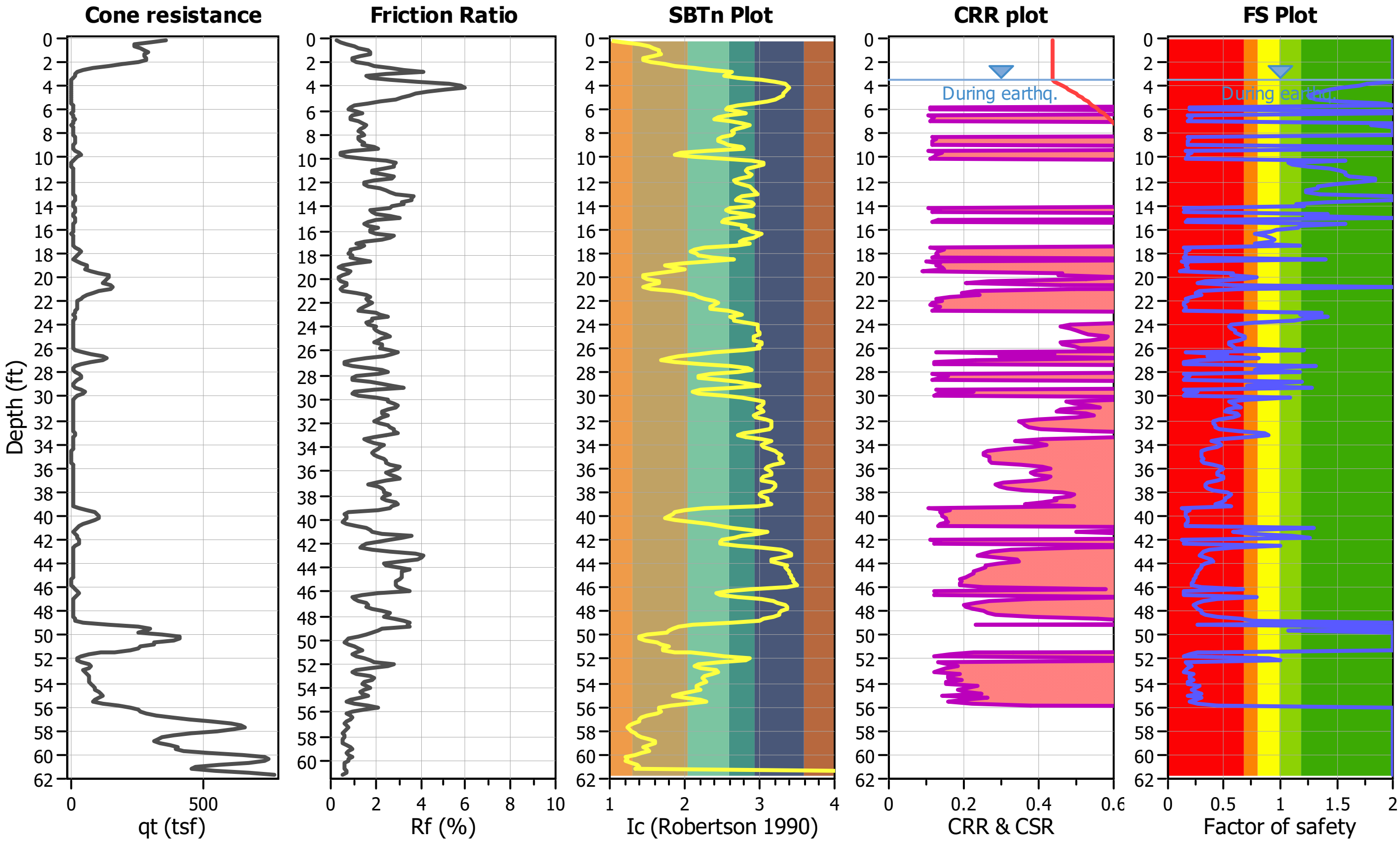
LIQUEFACTION ANALYSIS REPORT

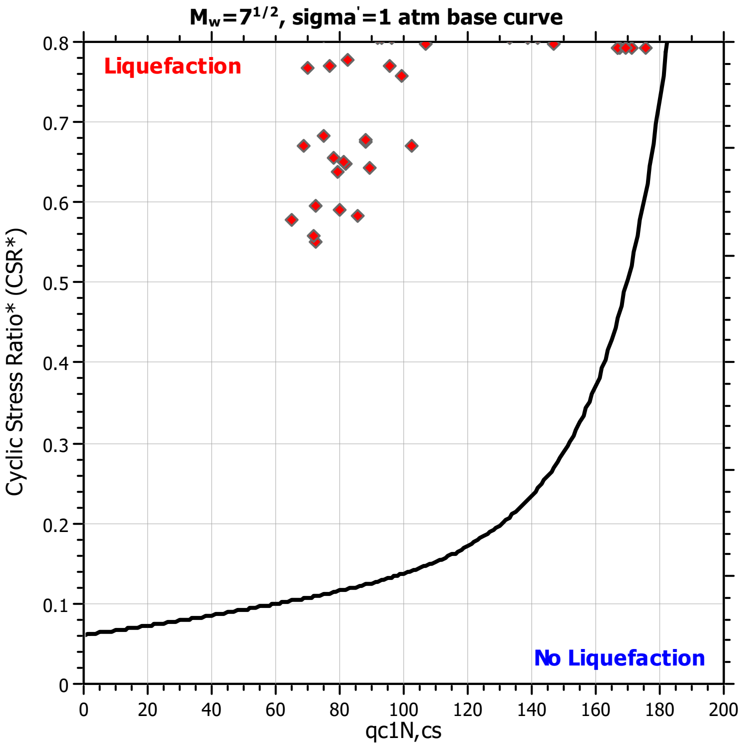
Project title : Issaquah TOD **Location : 1550 Newport Way Northwest, Issaquah, Washington**

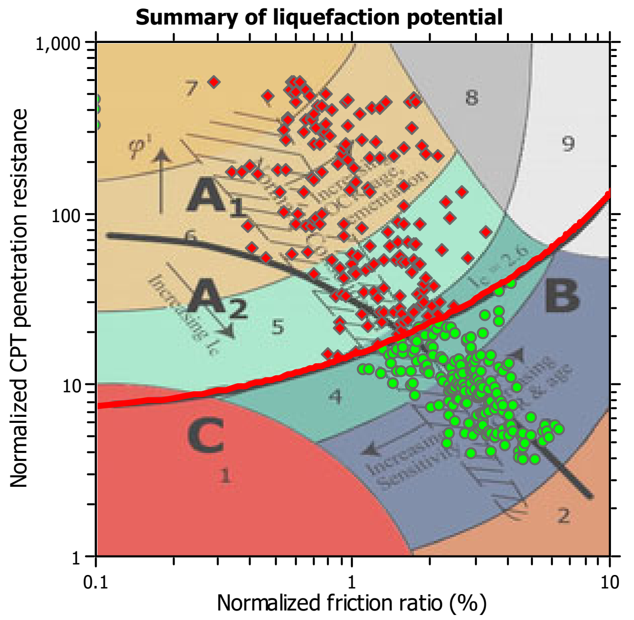
CPT file : CPT-04.pdf

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	3.80 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	3.50 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method
Peak ground acceleration:	0.74	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

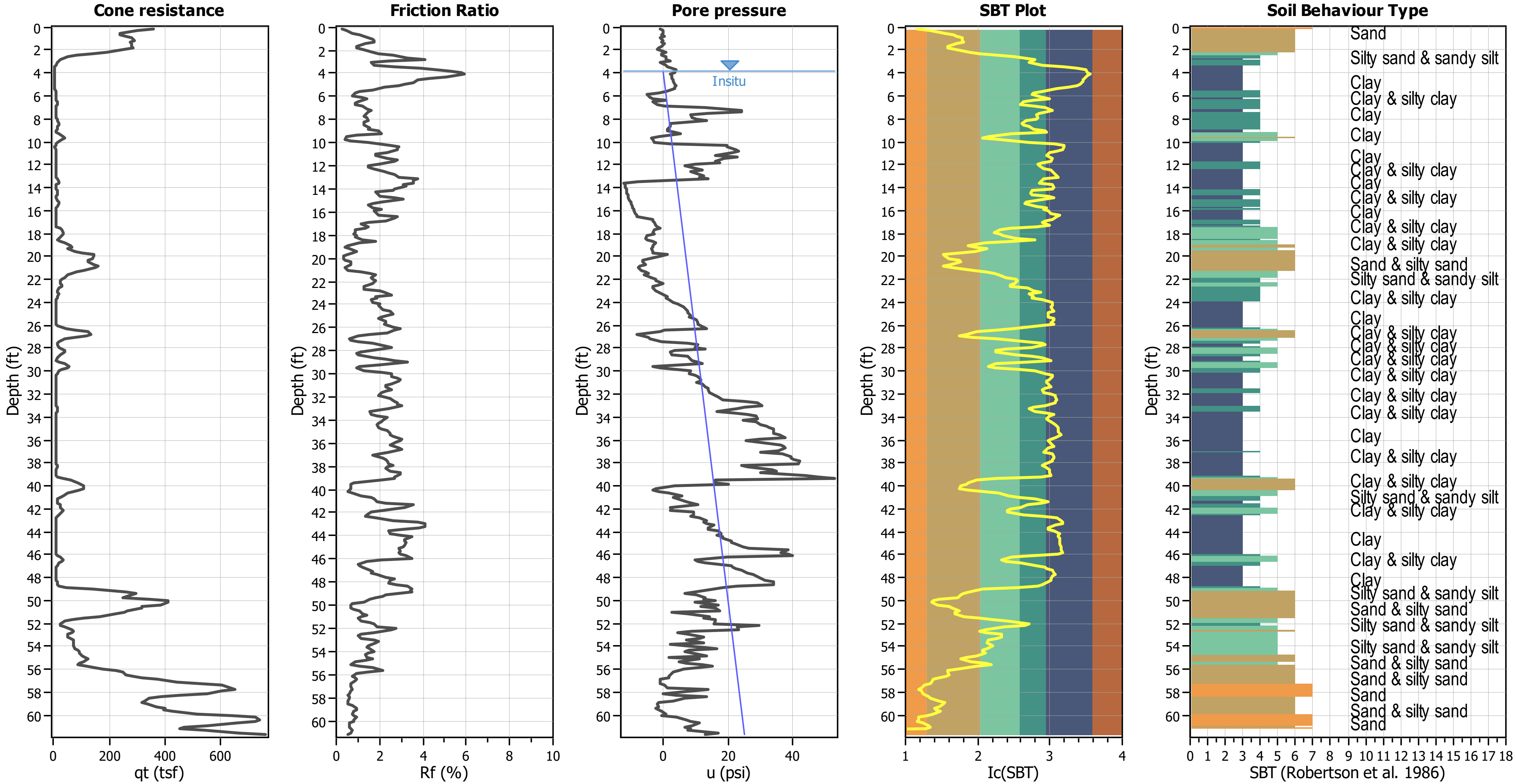






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



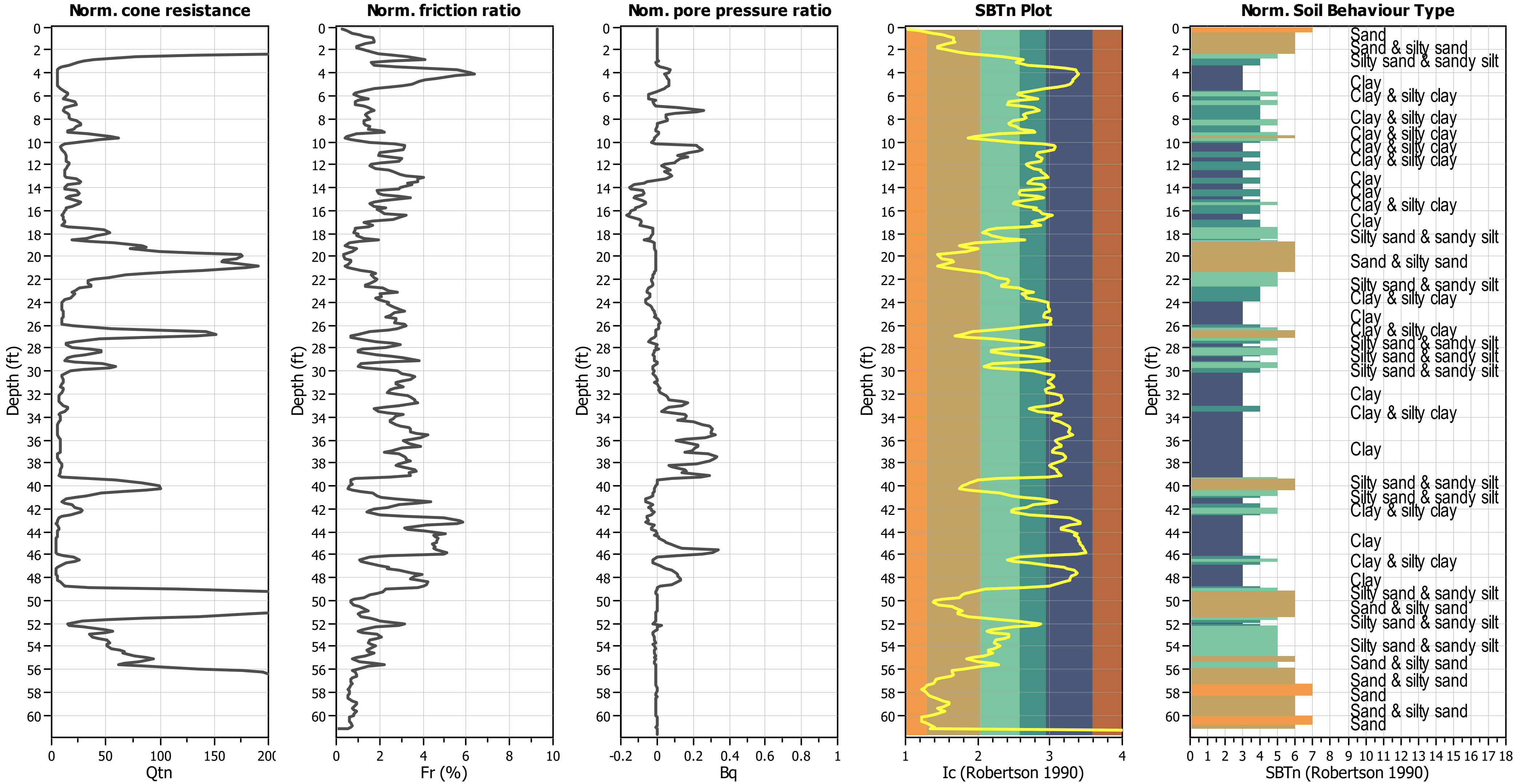
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	3.50 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.80 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained

CPT basic interpretation plots (normalized)



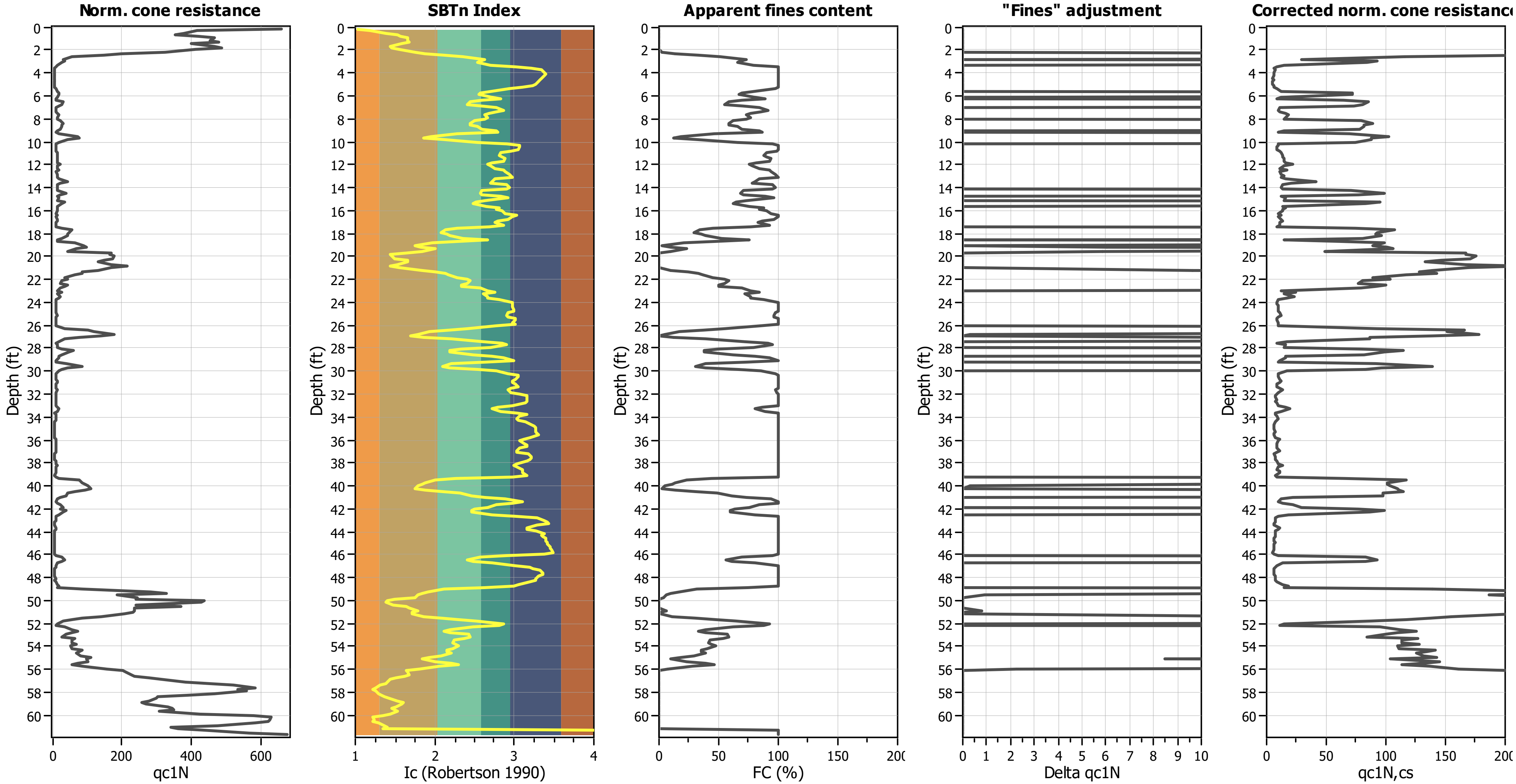
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	3.50 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.80 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained

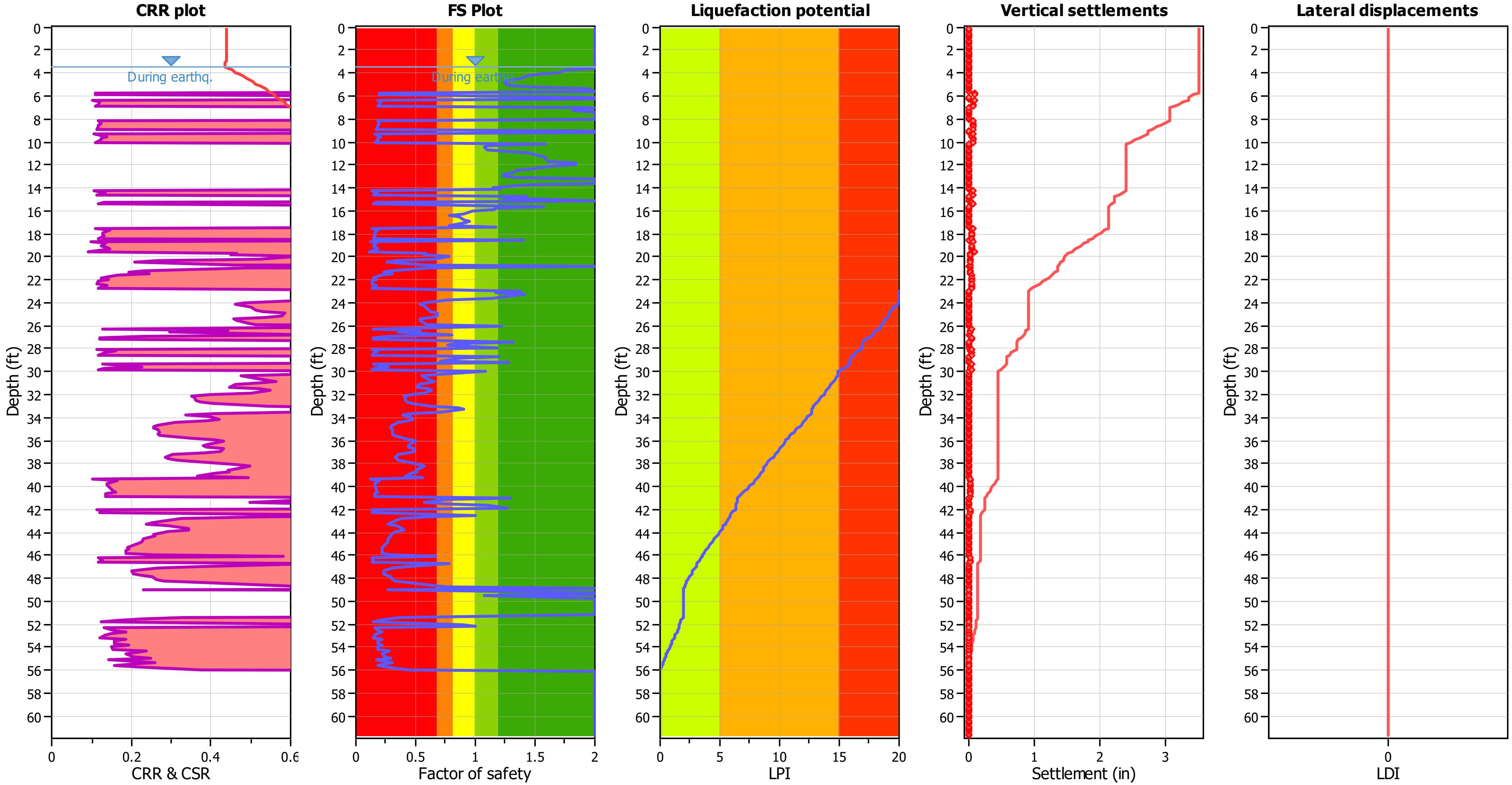
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	3.50 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.80 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	3.50 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.80 ft	Fill height:	N/A	Limit depth:	N/A

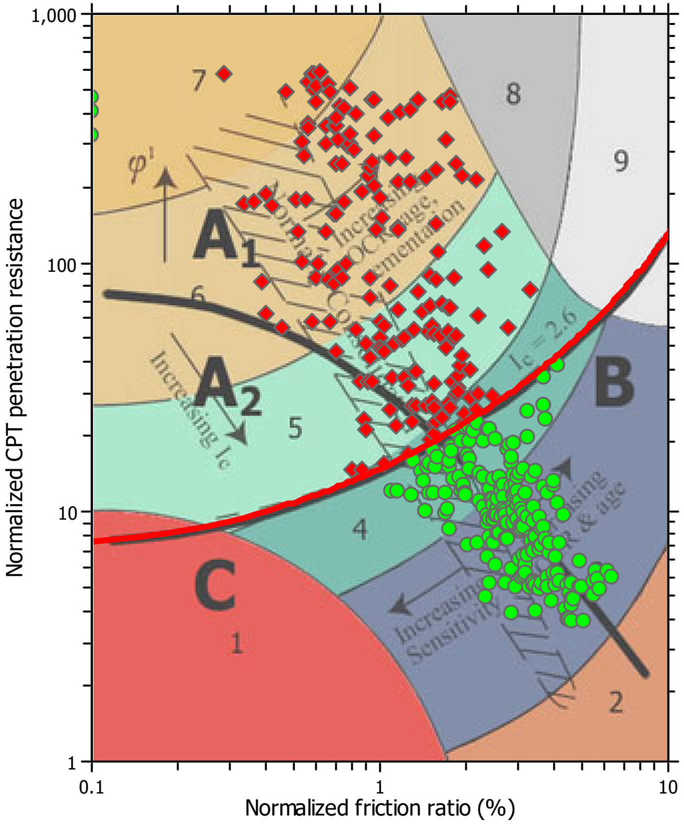
F.S. color scheme

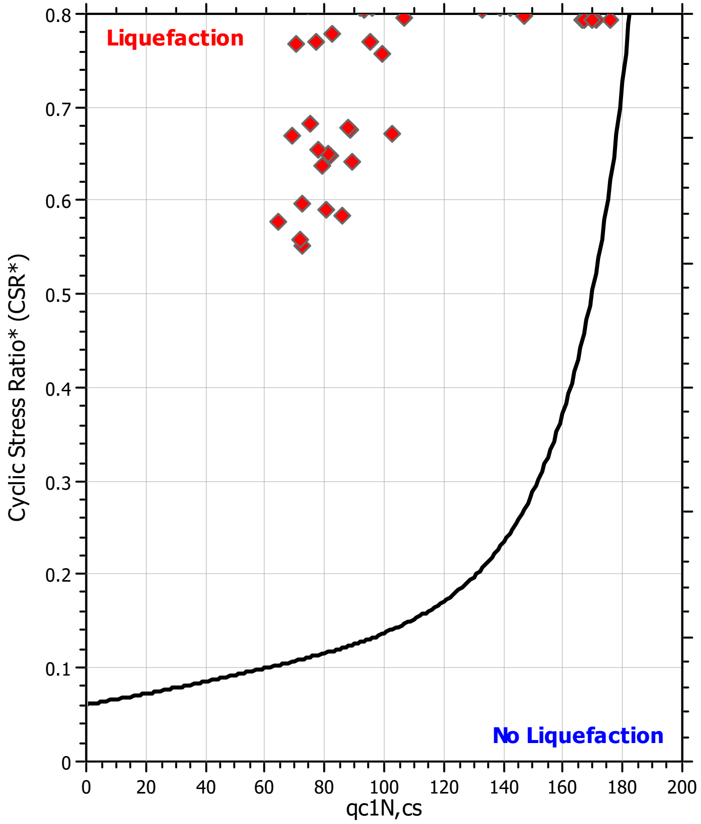
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

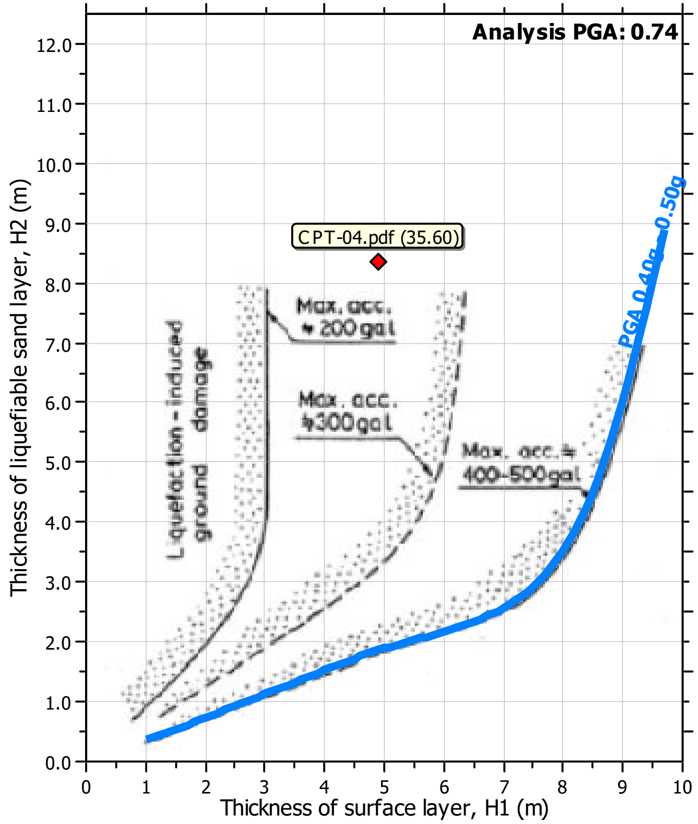
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



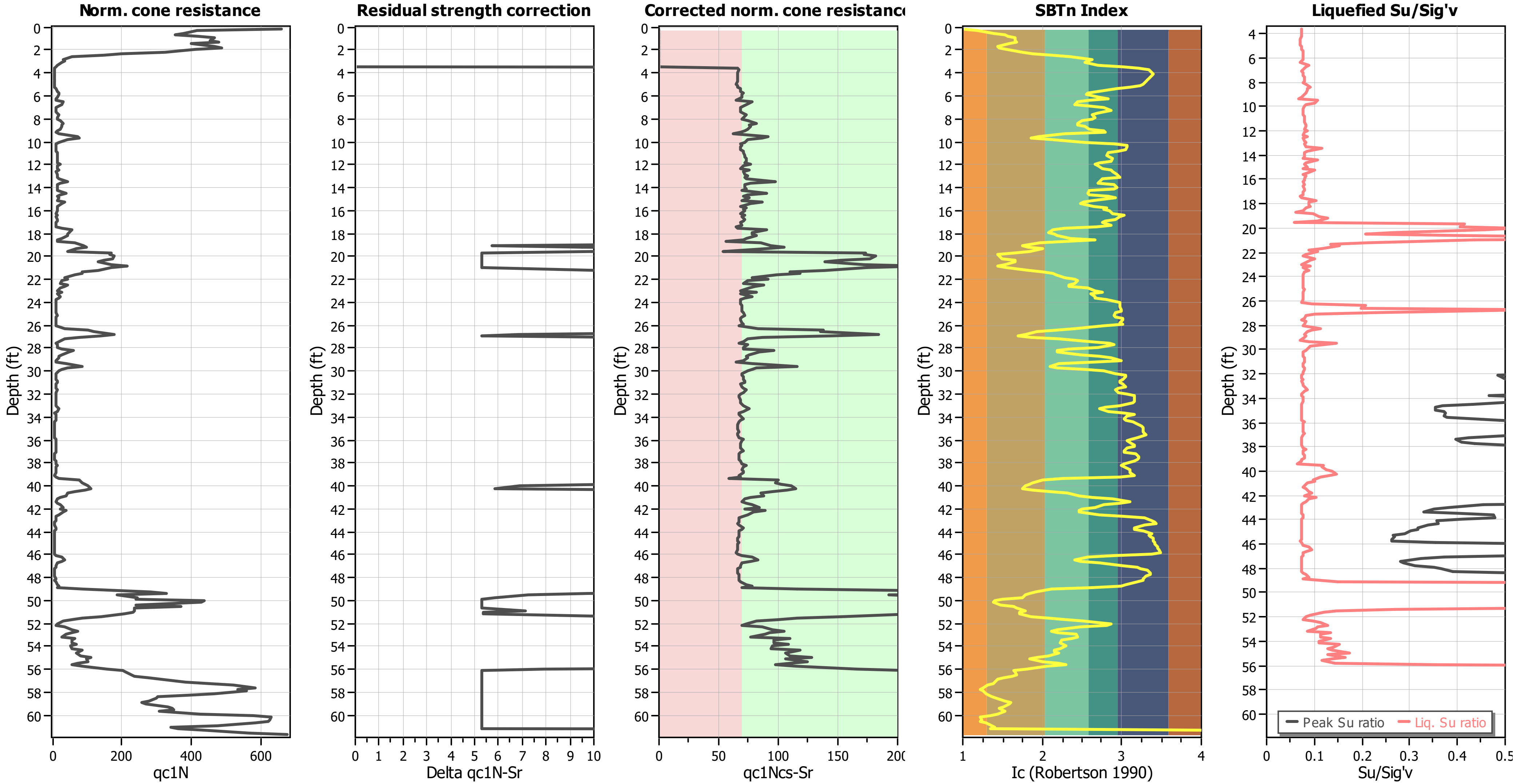




Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	3.50 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.80 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	3.50 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.80 ft	Fill height:	N/A	Limit depth:	N/A

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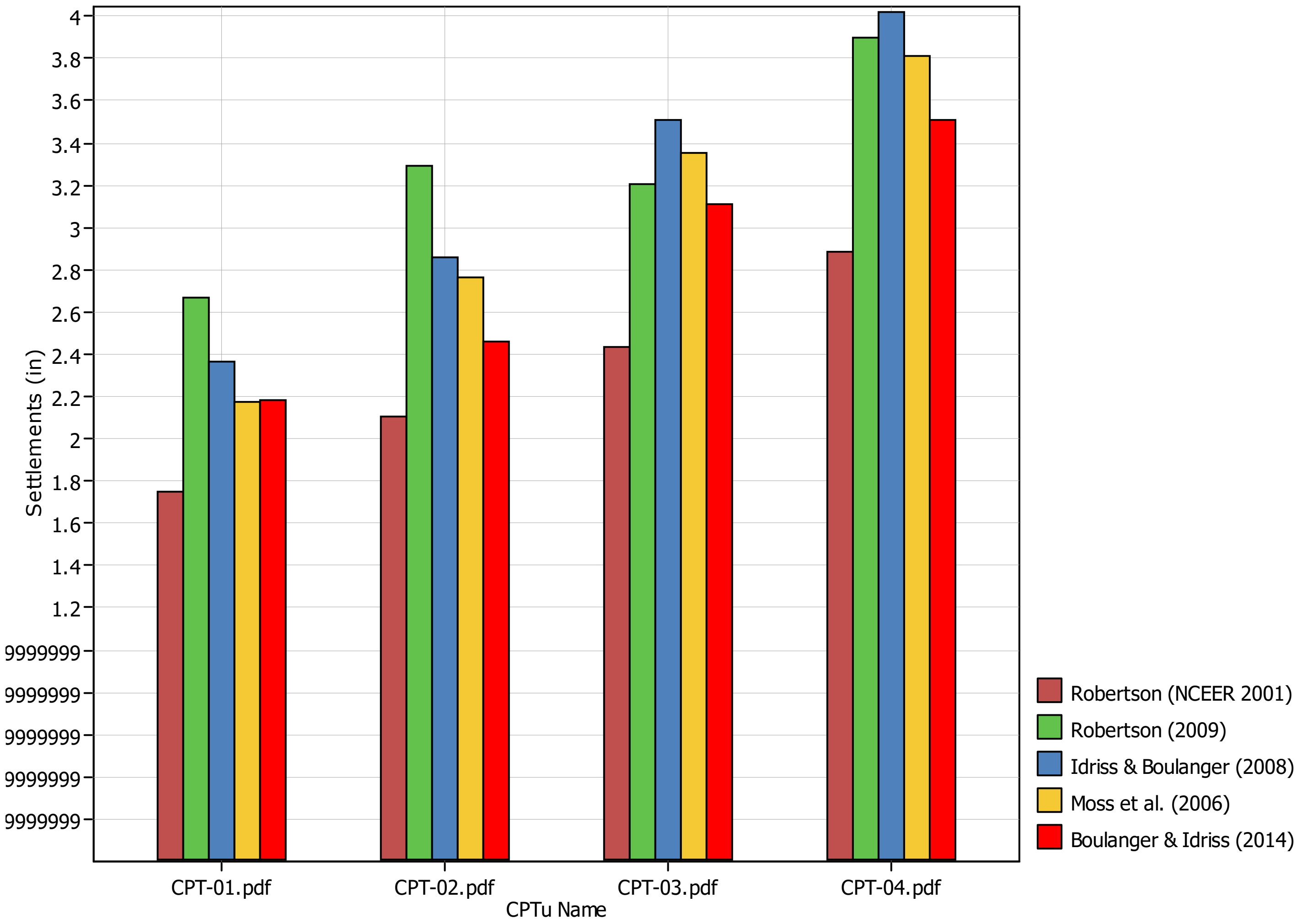
Overall Parametric Assessment Method



:: CPT main liquefaction parameters details ::

CPT Name	Earthquake Mag.	Earthquake Accel.	GWT in situ (ft)	GWT earthq. (ft)
CPT-01.pdf	7.50	0.74	4.50	4.00
CPT-02.pdf	7.50	0.74	4.30	4.00
CPT-03.pdf	7.50	0.74	4.10	4.00
CPT-04.pdf	7.50	0.74	3.80	3.50

APPENDIX G

LIQUEFACTION ANALYSIS RESULTS - LiqSV

PanGEO, Inc
3213 Eastlake Ave E, Ste B
Seattle, WA 98102
206.262.0370

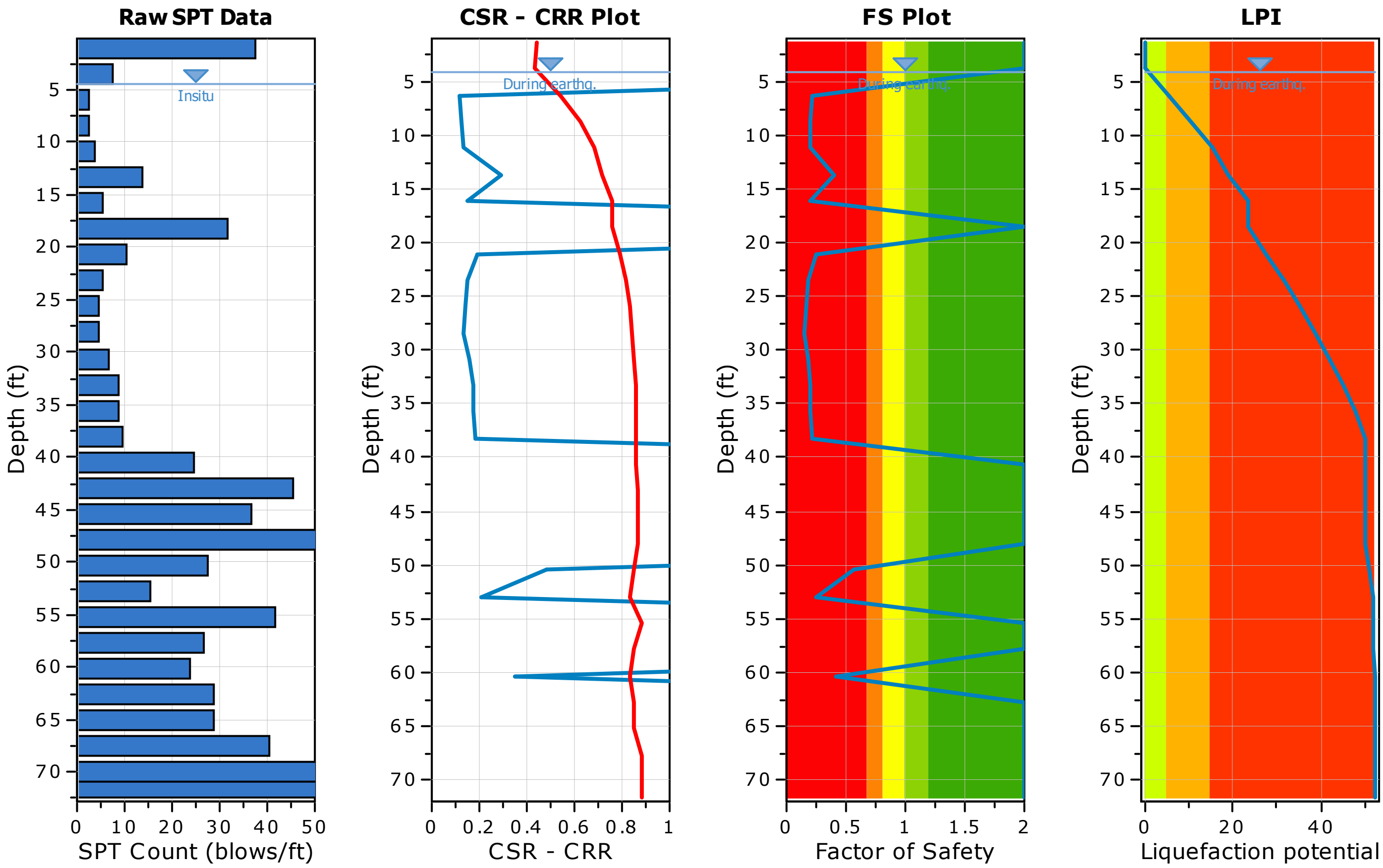
SPT BASED LIQUEFACTION ANALYSIS REPORT

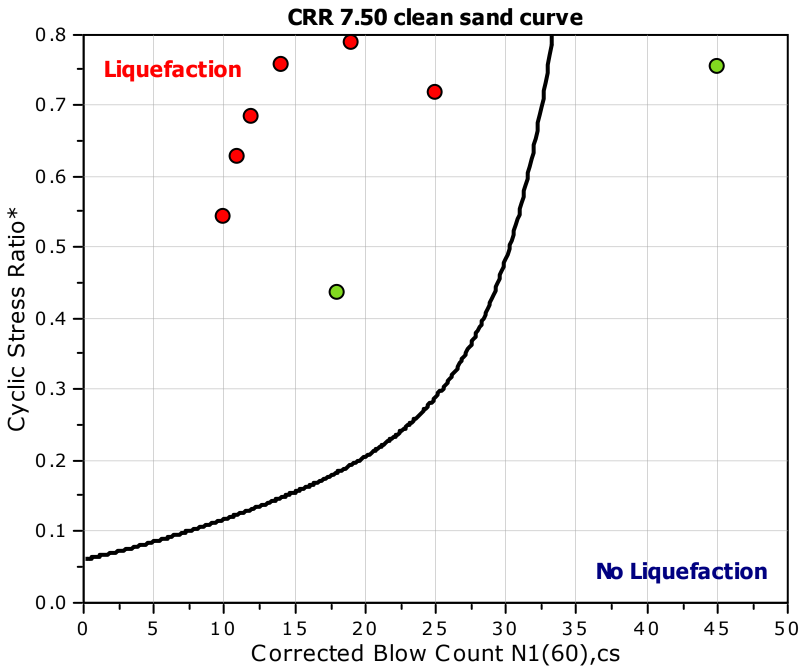
Project title : Issaquah TOD, 24-484 **SPT Name: SPT_CPT_01**

Location : 1550 Newport Way Northwest, Issaquah, Washington

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	4.50 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	4.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	7.50
Borehole diameter:	200mm	Peak ground acceleration:	0.74 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.00		





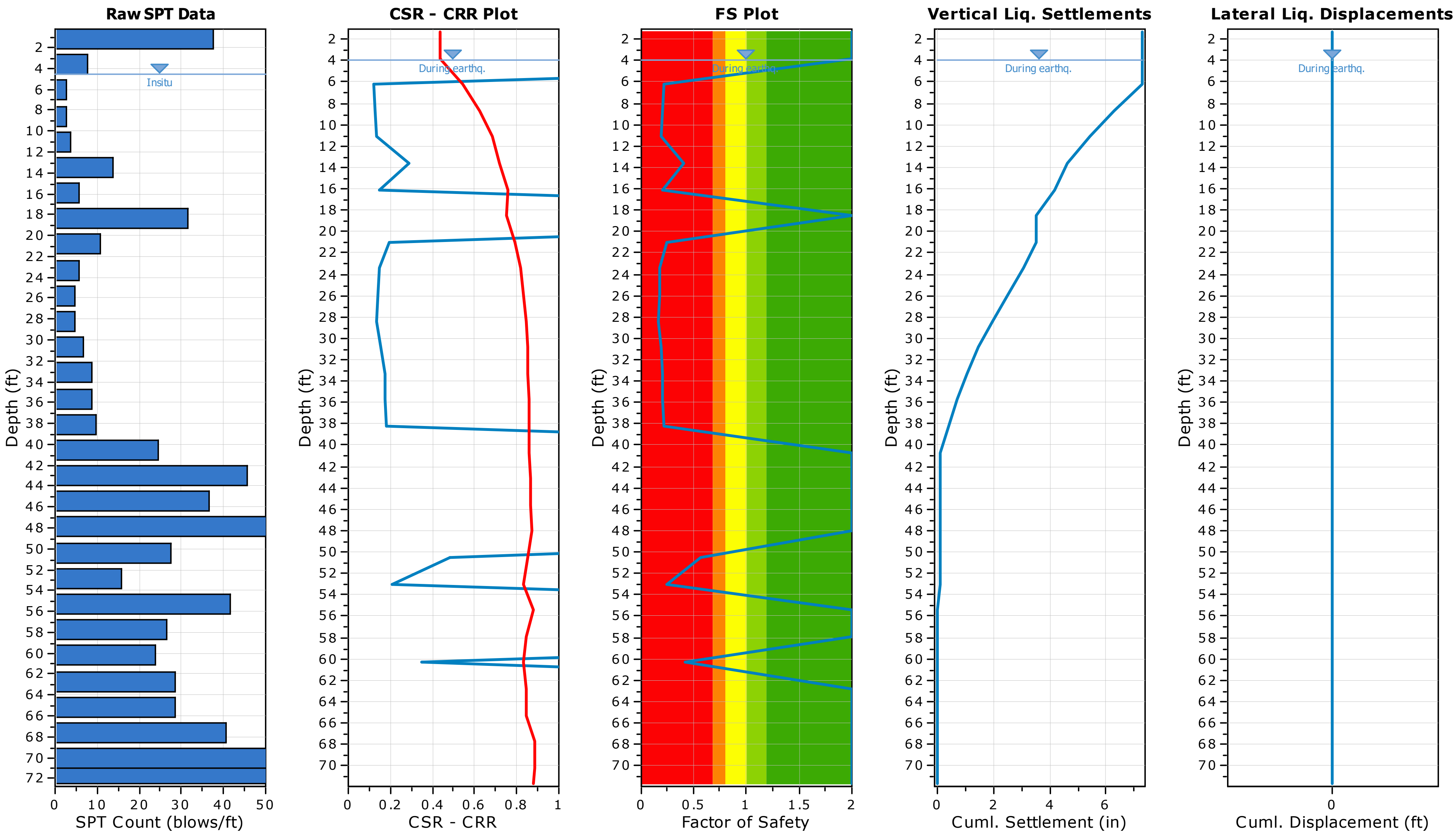
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
1.31	38	20.00	124.00	2.54	No
3.77	8	50.00	108.00	2.46	Yes
6.23	3	59.00	103.00	2.46	Yes
8.69	3	60.00	103.00	2.46	Yes
11.16	4	66.00	104.00	2.46	Yes
13.62	14	35.00	112.00	2.46	Yes
16.08	6	60.00	106.00	2.46	Yes
18.54	32	20.00	121.00	2.46	Yes
21.00	11	20.00	110.50	2.46	Yes
23.46	6	50.00	106.00	2.46	Yes
25.92	5	50.00	105.00	2.46	Yes
28.38	5	50.00	105.00	2.46	Yes
30.84	7	50.00	107.00	2.46	Yes
33.30	9	50.00	109.00	2.46	Yes
35.76	9	50.00	109.00	2.46	Yes
38.22	10	50.00	110.00	2.46	Yes
40.68	25	50.00	117.50	2.46	Yes
43.14	46	10.00	128.00	2.46	Yes
45.60	37	10.00	123.50	2.46	Yes
48.06	54	10.00	130.60	2.46	Yes
50.53	28	10.00	119.00	2.46	Yes
52.99	16	20.00	113.00	2.46	Yes
55.45	42	20.00	126.00	2.46	Yes
57.91	27	20.00	118.50	2.46	Yes
60.37	24	20.00	117.00	2.46	Yes
62.83	29	20.00	119.50	2.46	Yes
65.29	29	20.00	119.50	2.46	Yes
67.75	41	15.00	125.50	2.46	Yes
70.21	67	15.00	132.55	1.97	Yes
71.69	100	15.00	135.00	0.98	Yes

Abbreviations

Depth:	Depth at which test was performed (ft)
SPT Field Value:	Number of blows per foot
Fines Content:	Fines content at test depth (%)
Unit Weight:	Unit weight at test depth (pcf)
Infl. Thickness:	Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy:	User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
1.31	38	124.00	0.08	0.00	0.08	0.26	1.70	1.00	1.15	0.75	1.00	56	20.00	4.48	60	4.000
3.77	8	108.00	0.21	0.00	0.21	0.43	1.70	1.00	1.15	0.75	1.00	12	50.00	5.61	18	4.000
6.23	3	103.00	0.34	0.05	0.29	0.50	1.70	1.00	1.15	0.75	1.00	4	59.00	5.60	10	0.118
8.69	3	103.00	0.47	0.13	0.34	0.49	1.70	1.00	1.15	0.80	1.00	5	60.00	5.60	11	0.125
11.16	4	104.00	0.60	0.21	0.39	0.48	1.62	1.00	1.15	0.85	1.00	6	66.00	5.59	12	0.132
13.62	14	112.00	0.73	0.28	0.45	0.39	1.39	1.00	1.15	0.85	1.00	19	35.00	5.51	25	0.290
16.08	6	106.00	0.86	0.36	0.50	0.47	1.42	1.00	1.15	0.85	1.00	8	60.00	5.60	14	0.148

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
18.54	32	121.00	1.01	0.44	0.57	0.26	1.17	1.00	1.15	0.95	1.00	41	20.00	4.48	45	4.000
21.00	11	110.50	1.15	0.51	0.63	0.44	1.25	1.00	1.15	0.95	1.00	15	20.00	4.48	19	0.194
23.46	6	106.00	1.28	0.59	0.69	0.49	1.23	1.00	1.15	0.95	1.00	8	50.00	5.61	14	0.148
25.92	5	105.00	1.41	0.67	0.74	0.50	1.20	1.00	1.15	0.95	1.00	7	50.00	5.61	13	0.140
28.38	5	105.00	1.54	0.75	0.79	0.51	1.16	1.00	1.15	0.95	1.00	6	50.00	5.61	12	0.132
30.84	7	107.00	1.67	0.82	0.85	0.48	1.11	1.00	1.15	1.00	1.00	9	50.00	5.61	15	0.156
33.30	9	109.00	1.80	0.90	0.90	0.47	1.08	1.00	1.15	1.00	1.00	11	50.00	5.61	17	0.174
35.76	9	109.00	1.94	0.98	0.96	0.47	1.05	1.00	1.15	1.00	1.00	11	50.00	5.61	17	0.174
38.22	10	110.00	2.07	1.05	1.02	0.46	1.02	1.00	1.15	1.00	1.00	12	50.00	5.61	18	0.184
40.68	25	117.50	2.22	1.13	1.09	0.34	0.99	1.00	1.15	1.00	1.00	28	50.00	5.61	34	4.000
43.14	46	128.00	2.37	1.21	1.17	0.26	0.97	1.00	1.15	1.00	1.00	52	10.00	1.15	53	4.000
45.60	37	123.50	2.53	1.28	1.24	0.29	0.95	1.00	1.15	1.00	1.00	41	10.00	1.15	42	4.000
48.06	54	130.60	2.69	1.36	1.33	0.26	0.94	1.00	1.15	1.00	1.00	58	10.00	1.15	59	4.000
50.53	28	119.00	2.83	1.44	1.40	0.36	0.90	1.00	1.15	1.00	1.00	29	10.00	1.15	30	0.485
52.99	16	113.00	2.97	1.51	1.46	0.44	0.87	1.00	1.15	1.00	1.00	16	20.00	4.48	20	0.206
55.45	42	126.00	3.13	1.59	1.54	0.26	0.91	1.00	1.15	1.00	1.00	44	20.00	4.48	48	4.000
57.91	27	118.50	3.27	1.67	1.61	0.36	0.86	1.00	1.15	1.00	1.00	27	20.00	4.48	31	4.000
60.37	24	117.00	3.42	1.74	1.67	0.39	0.84	1.00	1.15	1.00	1.00	23	20.00	4.48	27	0.347
62.83	29	119.50	3.56	1.82	1.74	0.35	0.84	1.00	1.15	1.00	1.00	28	20.00	4.48	32	4.000
65.29	29	119.50	3.71	1.90	1.81	0.35	0.83	1.00	1.15	1.00	1.00	28	20.00	4.48	32	4.000
67.75	41	125.50	3.87	1.97	1.89	0.28	0.85	1.00	1.15	1.00	1.00	40	15.00	3.26	43	4.000
70.21	67	132.55	4.03	2.05	1.98	0.26	0.85	1.00	1.15	1.00	1.00	65	15.00	3.26	68	4.000
71.69	100	135.00	4.13	2.10	2.03	0.26	0.84	1.00	1.15	1.00	1.00	97	15.00	3.26	100	4.000

Abbreviations

σ_v: Total stress during SPT test (tsf)
u_o: Water pore pressure during SPT test (tsf)
σ'_{vo}: Effective overburden pressure during SPT test (tsf)
m: Stress exponent normalization factor
C_N: Overburden corretion factor
C_E: Energy correction factor
C_B: Borehole diameter correction factor
C_R: Rod length correction factor
C_S: Liner correction factor
$N_{1(60)}$: Corrected N_{SPT} to a 60% energy ratio
$\Delta(N_1)_{60}$ Equivalent clean sand adjustment
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$: Cyclic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
1.31	124.00	0.08	0.00	0.08	1.00	1.00	0.483	2.20	60	1.00	0.483	1.10	0.439	2.000
3.77	108.00	0.21	0.00	0.21	1.00	1.00	0.480	1.42	18	1.00	0.480	1.10	0.436	2.000
6.23	103.00	0.34	0.07	0.27	0.99	1.00	0.600	1.19	10	1.00	0.600	1.10	0.545	0.217
8.69	103.00	0.47	0.15	0.32	0.99	1.00	0.690	1.21	11	1.00	0.690	1.10	0.627	0.200
11.16	104.00	0.60	0.22	0.37	0.98	1.00	0.752	1.24	12	1.00	0.752	1.10	0.684	0.194
13.62	112.00	0.73	0.30	0.43	0.97	1.00	0.790	1.72	25	1.00	0.790	1.10	0.718	0.404
16.08	106.00	0.86	0.38	0.49	0.96	1.00	0.821	1.29	14	1.00	0.821	1.08	0.758	0.195
18.54	121.00	1.01	0.45	0.56	0.95	1.00	0.831	2.20	45	1.00	0.831	1.10	0.755	2.000
21.00	110.50	1.15	0.53	0.62	0.94	1.00	0.844	1.45	19	1.00	0.844	1.07	0.789	0.246

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
23.46	106.00	1.28	0.61	0.67	0.93	1.00	0.856	1.29	14	1.00	0.856	1.05	0.816	0.181
25.92	105.00	1.41	0.68	0.72	0.92	1.00	0.865	1.26	13	1.00	0.865	1.04	0.832	0.168
28.38	105.00	1.54	0.76	0.78	0.91	1.00	0.871	1.24	12	1.00	0.871	1.03	0.845	0.157
30.84	107.00	1.67	0.84	0.83	0.90	1.00	0.873	1.32	15	1.00	0.873	1.03	0.851	0.184
33.30	109.00	1.80	0.91	0.89	0.89	1.00	0.872	1.38	17	1.00	0.872	1.02	0.854	0.204
35.76	109.00	1.94	0.99	0.95	0.88	1.00	0.870	1.38	17	1.00	0.870	1.01	0.858	0.203
38.22	110.00	2.07	1.07	1.00	0.87	1.00	0.865	1.42	18	1.00	0.865	1.01	0.860	0.214
40.68	117.50	2.22	1.14	1.07	0.86	1.00	0.856	2.20	34	1.00	0.856	1.00	0.859	2.000
43.14	128.00	2.37	1.22	1.15	0.85	1.00	0.842	2.20	53	1.00	0.842	0.97	0.864	2.000
45.60	123.50	2.53	1.30	1.23	0.84	1.00	0.830	2.20	42	1.00	0.830	0.96	0.868	2.000
48.06	130.60	2.69	1.37	1.31	0.83	1.00	0.815	2.20	59	1.00	0.815	0.94	0.870	2.000
50.53	119.00	2.83	1.45	1.38	0.82	1.00	0.805	2.00	30	1.00	0.805	0.95	0.851	0.570
52.99	113.00	2.97	1.53	1.44	0.81	1.00	0.797	1.49	20	1.00	0.797	0.96	0.832	0.247
55.45	126.00	3.13	1.61	1.52	0.79	1.00	0.785	2.20	48	1.00	0.785	0.89	0.879	2.000
57.91	118.50	3.27	1.68	1.59	0.78	1.00	0.775	2.06	31	1.00	0.775	0.91	0.849	2.000
60.37	117.00	3.42	1.76	1.66	0.77	1.00	0.766	1.82	27	1.00	0.766	0.92	0.832	0.417
62.83	119.50	3.56	1.84	1.73	0.76	1.00	0.756	2.12	32	1.00	0.756	0.89	0.849	2.000
65.29	119.50	3.71	1.91	1.80	0.75	1.00	0.746	2.12	32	1.00	0.746	0.88	0.846	2.000
67.75	125.50	3.87	1.99	1.88	0.74	1.00	0.734	2.20	43	1.00	0.734	0.83	0.884	2.000
70.21	132.55	4.03	2.07	1.96	0.73	1.00	0.722	2.20	68	1.00	0.722	0.82	0.883	2.000
71.69	135.00	4.13	2.11	2.02	0.73	1.00	0.714	2.20	100	1.00	0.714	0.81	0.882	2.000

Abbreviations

$\sigma_{v,eq}$: Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$: Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$: Effective overburden pressure, during earthquake (tsf)
r_d: Nonlinear shear mass factor
α: Improvement factor due to stone columns
CSR : Cyclic Stress Ratio
MSF : Magnitude Scaling Factor
$CSR_{eq,M=7.5}$: CSR adjusted for M=7.5
K_{sigma}: Effective overburden stress factor
CSR^*: CSR fully adjusted (user FS applied)***
FS: Calculated factor of safety against soil liquefaction

*** User FS: 1.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
1.31	2.000	0.00	9.80	2.46	0.00
3.77	2.000	0.00	9.43	2.46	0.00
6.23	0.217	0.78	9.05	2.46	5.32
8.69	0.200	0.80	8.68	2.46	5.21
11.16	0.194	0.81	8.30	2.47	5.04
13.62	0.404	0.60	7.92	2.46	3.54
16.08	0.195	0.80	7.55	2.46	4.56
18.54	2.000	0.00	7.17	2.46	0.00
21.00	0.246	0.75	6.80	2.46	3.84
23.46	0.181	0.82	6.42	2.46	3.94
25.92	0.168	0.83	6.05	2.46	3.77
28.38	0.157	0.84	5.67	2.46	3.59

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
30.84	0.184	0.82	5.30	2.46	3.24
33.30	0.204	0.80	4.93	2.46	2.94
35.76	0.203	0.80	4.55	2.46	2.72
38.22	0.214	0.79	4.18	2.46	2.46
40.68	2.000	0.00	3.80	2.46	0.00
43.14	2.000	0.00	3.43	2.46	0.00
45.60	2.000	0.00	3.05	2.46	0.00
48.06	2.000	0.00	2.68	2.46	0.00
50.53	0.570	0.43	2.30	2.47	0.74
52.99	0.247	0.75	1.92	2.46	1.09
55.45	2.000	0.00	1.55	2.46	0.00
57.91	2.000	0.00	1.17	2.46	0.00
60.37	0.417	0.58	0.80	2.46	0.35
62.83	2.000	0.00	0.42	2.46	0.00
65.29	2.000	0.00	0.05	2.46	0.00
67.75	2.000	0.00	0.00	0.00	0.00
70.21	2.000	0.00	0.00	0.00	0.00
71.69	2.000	0.00	0.00	0.00	0.00

Overall potential I_L : 52.36

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} weight factor	ε_{Nc} (%)	Δh (ft)	ΔS (in)
1.31	56	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.54	0.000
3.77	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.94	0.00	2.46	0.000

Cumulative settlemetns: 0.000

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus (tsf)
α, b: Shear strain formula variables
γ: Average shear strain
ε_{15}: Volumetric strain after 15 cycles
N_c: Number of cycles
ε_{Nc}: Volumetric strain for number of cycles N_c (%)
Δh: Thickness of soil layer (in)
ΔS: Settlement of soil layer (in)

:: Vertical & Lateral displ.acements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60cs}$	γ_{lim} (%)	F_α	FS_{liq}	γ_{max} (%)	e_v weight factor	e_v (%)	dz (ft)	S_{v-1D} (in)	LDI (ft)
6.23	10	47.32	0.91	0.217	47.32	0.90	3.35	2.46	0.988	0.00
8.69	11	42.40	0.89	0.200	42.40	0.86	3.02	2.46	0.891	0.00
11.16	12	38.03	0.86	0.194	38.03	0.81	2.72	2.46	0.803	0.00

:: Vertical & Lateral displ.acements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60cs}$	γ_{lim} (%)	F_α	FS_{liq}	γ_{max} (%)	e_v weight factor	e_v (%)	dz (ft)	S_{v-1D} (in)	LDI (ft)
13.62	25	8.88	0.23	0.404	8.88	0.77	1.47	2.46	0.433	0.00
16.08	14	30.65	0.79	0.195	30.65	0.73	2.21	2.46	0.652	0.00
18.54	45	0.25	-1.19	2.000	0.00	0.69	0.00	2.46	0.000	0.00
21.00	19	17.78	0.57	0.246	17.78	0.65	1.56	2.46	0.461	0.00
23.46	14	30.65	0.79	0.181	30.65	0.61	1.84	2.46	0.542	0.00
25.92	13	34.14	0.83	0.168	34.14	0.57	1.80	2.46	0.532	0.00
28.38	12	38.03	0.86	0.157	38.03	0.53	1.76	2.46	0.520	0.00
30.84	15	27.51	0.75	0.184	27.51	0.49	1.40	2.46	0.412	0.00
33.30	17	22.15	0.67	0.204	22.15	0.44	1.17	2.46	0.344	0.00
35.76	17	22.15	0.67	0.203	22.15	0.40	1.06	2.46	0.313	0.00
38.22	18	19.85	0.62	0.214	19.85	0.36	0.91	2.46	0.269	0.00
40.68	34	2.58	-0.36	2.000	0.00	0.32	0.00	2.46	0.000	0.00
43.14	53	0.00	-1.83	2.000	0.00	0.28	0.00	2.46	0.000	0.00
45.60	42	0.56	-0.96	2.000	0.00	0.24	0.00	2.46	0.000	0.00
48.06	59	0.00	-2.34	2.000	0.00	0.20	0.00	2.46	0.000	0.00
50.53	30	4.65	-0.09	0.570	4.65	0.16	0.15	2.46	0.043	0.00
52.99	20	15.90	0.52	0.247	15.90	0.12	0.27	2.46	0.079	0.00
55.45	48	0.09	-1.43	2.000	0.00	0.08	0.00	2.46	0.000	0.00
57.91	31	4.04	-0.16	2.000	0.00	0.03	0.00	2.46	0.000	0.00
60.37	27	6.92	0.11	0.417	6.92	0.00	0.00	2.46	0.000	0.00
62.83	32	3.50	-0.22	2.000	0.00	0.00	0.00	2.46	0.000	0.00
65.29	32	3.50	-0.22	2.000	0.00	0.00	0.00	2.46	0.000	0.00
67.75	43	0.44	-1.03	2.000	0.00	0.00	0.00	2.46	0.000	0.00
70.21	68	0.00	-3.12	2.000	0.00	0.00	0.00	1.97	0.000	0.00
71.69	100	0.00	-6.07	2.000	0.00	0.00	0.00	0.98	0.000	0.00

Cumulative settlements: 7.283 0.00

Abbreviations

γ_{lim}: Limiting shear strain (%)
F_α/N: Maximun shear strain factor
γ_{max}: Maximum shear strain (%)
e_v:: Post liquefaction volumetric strain (%)
S_{v-1D}: Estimated vertical settlement (in)
LDI: Estimated lateral displacement (ft)

PanGEO, Inc
3213 Eastlake Ave E, Ste B
Seattle, WA 98102
206.262.0370

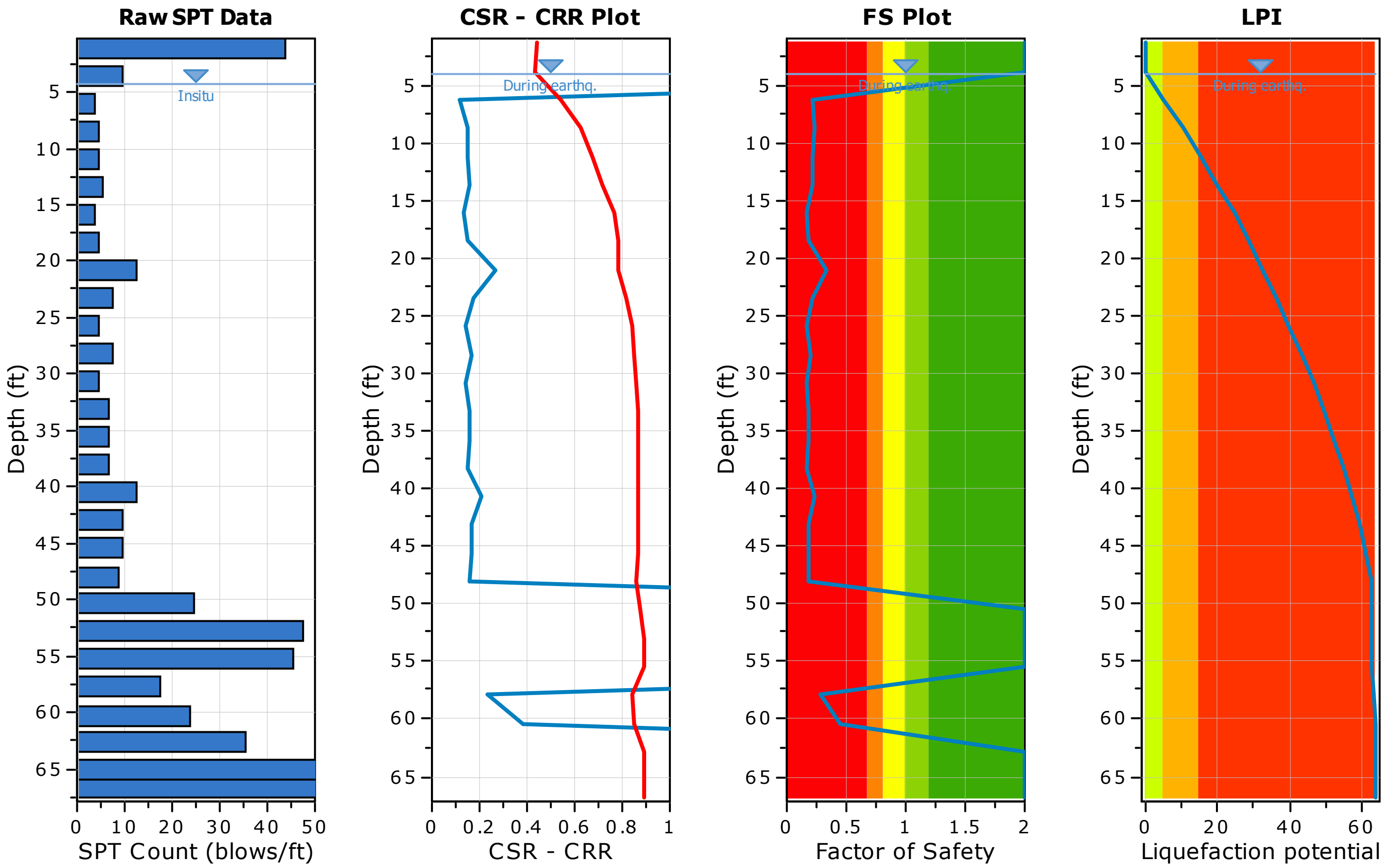
SPT BASED LIQUEFACTION ANALYSIS REPORT

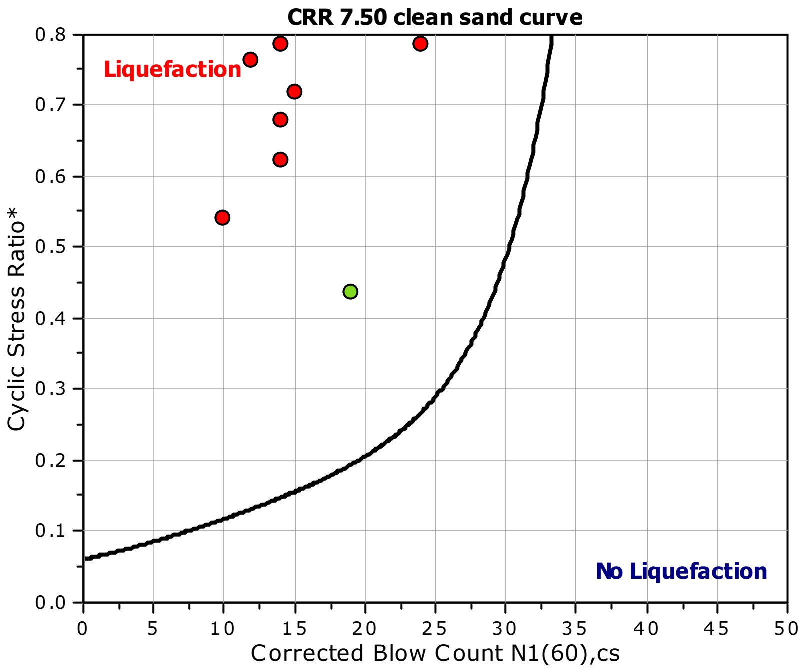
Project title : Issaquah TOD, 24-484 **SPT Name: SPT_CPT_02**

Location : 1550 Newport Way Northwest, Issaquah, Washington

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	4.30 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	4.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	7.50
Borehole diameter:	200mm	Peak ground acceleration:	0.74 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.00		





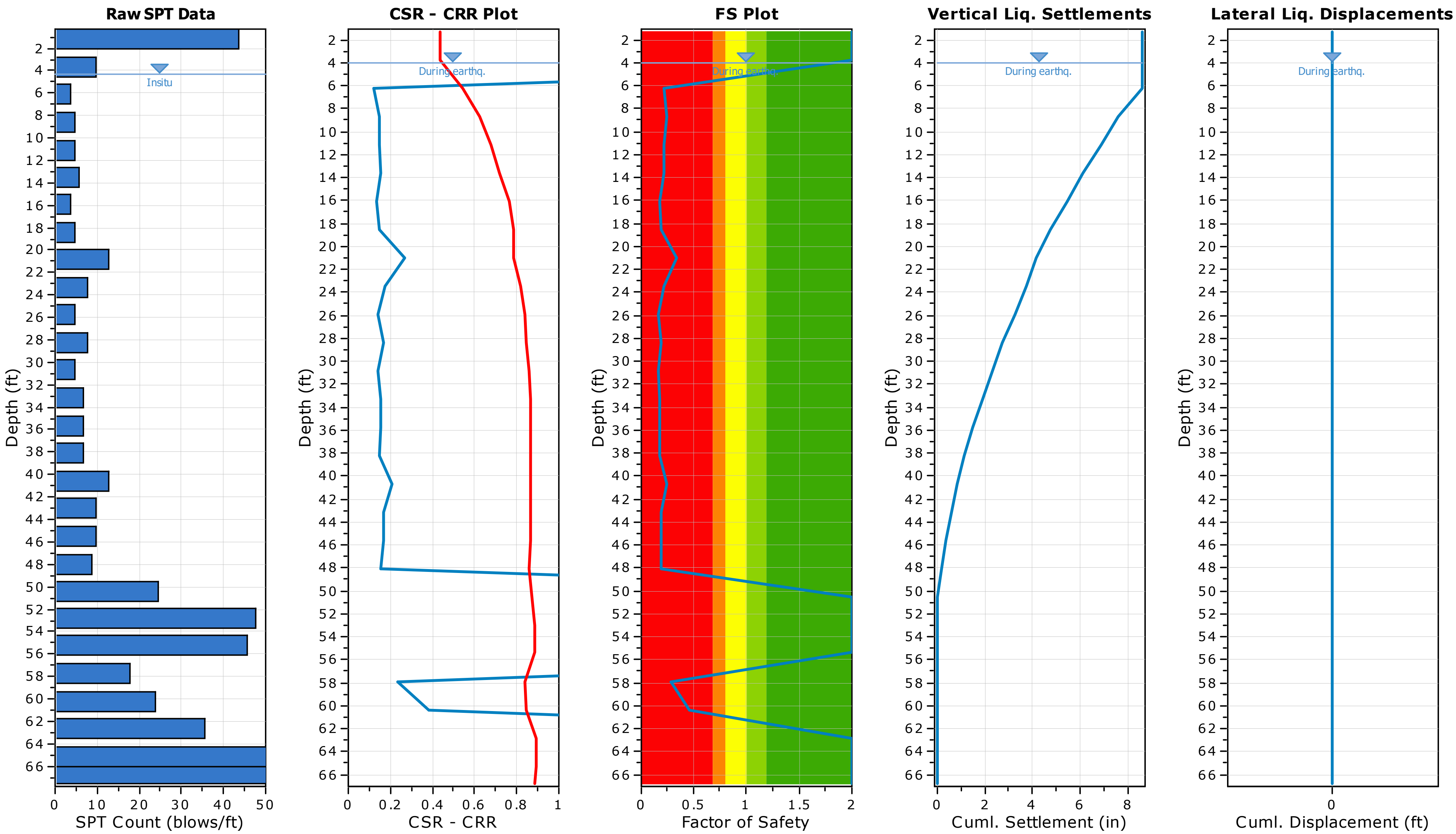
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
1.30	44	20.00	127.00	2.54	No
3.80	10	20.00	110.00	2.46	Yes
6.20	4	20.00	104.00	2.46	Yes
8.70	5	35.00	105.00	2.46	Yes
11.20	5	47.00	105.00	2.46	Yes
13.60	6	47.00	106.00	2.46	Yes
16.10	4	47.00	104.00	2.46	Yes
18.50	5	47.00	105.00	2.46	Yes
21.00	13	35.00	111.50	2.46	Yes
23.50	8	35.00	108.00	2.46	Yes
25.90	5	35.00	105.00	2.46	Yes
28.40	8	35.00	108.00	2.46	Yes
30.80	5	35.00	105.00	2.46	Yes
33.30	7	35.00	107.00	2.46	Yes
35.80	7	35.00	107.00	2.46	Yes
38.20	7	35.00	107.00	2.46	Yes
40.70	13	30.00	111.50	2.46	Yes
43.10	10	30.00	110.00	2.46	Yes
45.60	10	30.00	110.00	2.46	Yes
48.10	9	25.00	109.00	2.46	Yes
50.50	25	25.00	117.50	2.46	Yes
53.00	48	25.00	129.00	2.46	Yes
55.40	46	20.00	128.00	2.46	Yes
57.90	18	20.00	114.00	2.46	Yes
60.40	24	20.00	117.00	2.46	Yes
62.80	36	15.00	123.00	2.46	Yes
65.30	77	15.00	134.05	1.97	Yes
66.80	100	15.00	135.00	0.98	Yes

Abbreviations

Depth: Depth at which test was performed (ft)
SPT Field Value: Number of blows per foot
Fines Content: Fines content at test depth (%)
Unit Weight: Unit weight at test depth (pcf)
Infl. Thickness: Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy: User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
1.30	44	127.00	0.08	0.00	0.08	0.26	1.70	1.00	1.15	0.75	1.00	65	20.00	4.48	69	4.000
3.80	10	110.00	0.22	0.00	0.22	0.42	1.70	1.00	1.15	0.75	1.00	15	20.00	4.48	19	4.000
6.20	4	104.00	0.34	0.06	0.29	0.50	1.70	1.00	1.15	0.75	1.00	6	20.00	4.48	10	0.118
8.70	5	105.00	0.48	0.14	0.34	0.47	1.70	1.00	1.15	0.80	1.00	8	35.00	5.51	14	0.148
11.20	5	105.00	0.61	0.22	0.39	0.47	1.59	1.00	1.15	0.85	1.00	8	47.00	5.61	14	0.148
13.60	6	106.00	0.73	0.29	0.44	0.47	1.50	1.00	1.15	0.85	1.00	9	47.00	5.61	15	0.156
16.10	4	104.00	0.86	0.37	0.50	0.50	1.46	1.00	1.15	0.85	1.00	6	47.00	5.61	12	0.132
18.50	5	105.00	0.99	0.44	0.55	0.48	1.38	1.00	1.15	0.95	1.00	8	47.00	5.61	14	0.148
21.00	13	111.50	1.13	0.52	0.61	0.40	1.25	1.00	1.15	0.95	1.00	18	35.00	5.51	24	0.268

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
23.50	8	108.00	1.26	0.60	0.67	0.46	1.24	1.00	1.15	0.95	1.00	11	35.00	5.51	17	0.174
25.90	5	105.00	1.39	0.67	0.72	0.50	1.22	1.00	1.15	0.95	1.00	7	35.00	5.51	13	0.140
28.40	8	108.00	1.53	0.75	0.77	0.47	1.16	1.00	1.15	0.95	1.00	10	35.00	5.51	16	0.165
30.80	5	105.00	1.65	0.83	0.83	0.51	1.14	1.00	1.15	1.00	1.00	7	35.00	5.51	13	0.140
33.30	7	107.00	1.79	0.90	0.88	0.49	1.09	1.00	1.15	1.00	1.00	9	35.00	5.51	15	0.156
35.80	7	107.00	1.92	0.98	0.94	0.49	1.06	1.00	1.15	1.00	1.00	9	35.00	5.51	15	0.156
38.20	7	107.00	2.05	1.06	0.99	0.50	1.03	1.00	1.15	1.00	1.00	8	35.00	5.51	14	0.148
40.70	13	111.50	2.19	1.14	1.05	0.44	1.00	1.00	1.15	1.00	1.00	15	30.00	5.36	20	0.206
43.10	10	110.00	2.32	1.21	1.11	0.47	0.98	1.00	1.15	1.00	1.00	11	30.00	5.36	16	0.165
45.60	10	110.00	2.46	1.29	1.17	0.48	0.95	1.00	1.15	1.00	1.00	11	30.00	5.36	16	0.165
48.10	9	109.00	2.59	1.37	1.23	0.49	0.93	1.00	1.15	1.00	1.00	10	25.00	5.07	15	0.156
50.50	25	117.50	2.73	1.44	1.29	0.35	0.93	1.00	1.15	1.00	1.00	27	25.00	5.07	32	4.000
53.00	48	129.00	2.90	1.52	1.38	0.26	0.93	1.00	1.15	1.00	1.00	52	25.00	5.07	57	4.000
55.40	46	128.00	3.05	1.59	1.45	0.26	0.92	1.00	1.15	1.00	1.00	49	20.00	4.48	53	4.000
57.90	18	114.00	3.19	1.67	1.52	0.43	0.86	1.00	1.15	1.00	1.00	18	20.00	4.48	22	0.233
60.40	24	117.00	3.34	1.75	1.59	0.38	0.86	1.00	1.15	1.00	1.00	24	20.00	4.48	28	0.384
62.80	36	123.00	3.49	1.83	1.66	0.30	0.87	1.00	1.15	1.00	1.00	36	15.00	3.26	39	4.000
65.30	77	134.05	3.65	1.90	1.75	0.26	0.88	1.00	1.15	1.00	1.00	78	15.00	3.26	81	4.000
66.80	100	135.00	3.75	1.95	1.80	0.26	0.87	1.00	1.15	1.00	1.00	100	15.00	3.26	103	4.000

Abbreviations

σ_v: Total stress during SPT test (tsf)
u_o: Water pore pressure during SPT test (tsf)
σ'_{vo}: Effective overburden pressure during SPT test (tsf)
m: Stress exponent normalization factor
C_N: Overburden corretion factor
C_E: Energy correction factor
C_B: Borehole diameter correction factor
C_R: Rod length correction factor
C_S: Liner correction factor
$N_{1(60)}$: Corrected N_{SPT} to a 60% energy ratio
$\Delta(N_1)_{60}$ Equivalent clean sand adjustment
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$: Cyclic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
1.30	127.00	0.08	0.00	0.08	1.00	1.00	0.483	2.20	69	1.00	0.483	1.10	0.439	2.000
3.80	110.00	0.22	0.00	0.22	1.00	1.00	0.480	1.45	19	1.00	0.480	1.10	0.436	2.000
6.20	104.00	0.34	0.07	0.28	0.99	1.00	0.596	1.19	10	1.00	0.596	1.10	0.542	0.218
8.70	105.00	0.48	0.15	0.33	0.99	1.00	0.685	1.29	14	1.00	0.685	1.10	0.623	0.238
11.20	105.00	0.61	0.22	0.38	0.98	1.00	0.746	1.29	14	1.00	0.746	1.10	0.679	0.218
13.60	106.00	0.73	0.30	0.44	0.97	1.00	0.788	1.32	15	1.00	0.788	1.10	0.717	0.218
16.10	104.00	0.86	0.38	0.49	0.96	1.00	0.821	1.24	12	1.00	0.821	1.08	0.763	0.174
18.50	105.00	0.99	0.45	0.54	0.95	1.00	0.844	1.29	14	1.00	0.844	1.07	0.787	0.188
21.00	111.50	1.13	0.53	0.60	0.94	1.00	0.856	1.67	24	1.00	0.856	1.09	0.786	0.341
23.50	108.00	1.26	0.61	0.66	0.93	1.00	0.866	1.38	17	1.00	0.866	1.06	0.820	0.212
25.90	105.00	1.39	0.68	0.71	0.93	1.00	0.875	1.26	13	1.00	0.875	1.04	0.840	0.167
28.40	108.00	1.53	0.76	0.76	0.91	1.00	0.878	1.35	16	1.00	0.878	1.04	0.847	0.195
30.80	105.00	1.65	0.84	0.82	0.90	1.00	0.881	1.26	13	1.00	0.881	1.03	0.858	0.163

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
33.30	107.00	1.79	0.91	0.87	0.89	1.00	0.881	1.32	15	1.00	0.881	1.02	0.863	0.181
35.80	107.00	1.92	0.99	0.93	0.88	1.00	0.879	1.32	15	1.00	0.879	1.01	0.867	0.180
38.20	107.00	2.05	1.07	0.98	0.87	1.00	0.876	1.29	14	1.00	0.876	1.01	0.869	0.170
40.70	111.50	2.19	1.15	1.04	0.86	1.00	0.869	1.49	20	1.00	0.869	1.00	0.868	0.237
43.10	110.00	2.32	1.22	1.10	0.85	1.00	0.863	1.35	16	1.00	0.863	1.00	0.867	0.190
45.60	110.00	2.46	1.30	1.16	0.84	1.00	0.855	1.35	16	1.00	0.855	0.99	0.865	0.191
48.10	109.00	2.59	1.38	1.22	0.83	1.00	0.848	1.32	15	1.00	0.848	0.98	0.861	0.181
50.50	117.50	2.73	1.45	1.28	0.82	1.00	0.837	2.12	32	1.00	0.837	0.96	0.875	2.000
53.00	129.00	2.90	1.53	1.37	0.81	1.00	0.821	2.20	57	1.00	0.821	0.92	0.888	2.000
55.40	128.00	3.05	1.60	1.45	0.79	1.00	0.806	2.20	53	1.00	0.806	0.91	0.888	2.000
57.90	114.00	3.19	1.68	1.51	0.78	1.00	0.797	1.58	22	1.00	0.797	0.95	0.840	0.278
60.40	117.00	3.34	1.76	1.58	0.77	1.00	0.786	1.88	28	1.00	0.786	0.93	0.849	0.452
62.80	123.00	3.49	1.83	1.65	0.76	1.00	0.774	2.20	39	1.00	0.774	0.87	0.891	2.000
65.30	134.05	3.65	1.91	1.74	0.75	1.00	0.759	2.20	81	1.00	0.759	0.85	0.889	2.000
66.80	135.00	3.75	1.96	1.79	0.75	1.00	0.750	2.20	103	1.00	0.750	0.84	0.888	2.000

Abbreviations

$\sigma_{v,eq}$: Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$: Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$: Effective overburden pressure, during earthquake (tsf)
r_d: Nonlinear shear mass factor
α: Improvement factor due to stone columns
CSR : Cyclic Stress Ratio
MSF : Magnitude Scaling Factor
$CSR_{eq,M=7.5}$: CSR adjusted for M=7.5
K_{sigma}: Effective overburden stress factor
CSR^*: CSR fully adjusted (user FS applied)***
FS: Calculated factor of safety against soil liquefaction

*** User FS: 1.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
1.30	2.000	0.00	9.80	2.50	0.00
3.80	2.000	0.00	9.42	2.50	0.00
6.20	0.218	0.78	9.06	2.40	5.18
8.70	0.238	0.76	8.67	2.50	5.04
11.20	0.218	0.78	8.29	2.50	4.94
13.60	0.218	0.78	7.93	2.40	4.54
16.10	0.174	0.83	7.55	2.50	4.75
18.50	0.188	0.81	7.18	2.40	4.27
21.00	0.341	0.66	6.80	2.50	3.41
23.50	0.212	0.79	6.42	2.50	3.85
25.90	0.167	0.83	6.05	2.40	3.69
28.40	0.195	0.81	5.67	2.50	3.48
30.80	0.163	0.84	5.31	2.40	3.25
33.30	0.181	0.82	4.93	2.50	3.07
35.80	0.180	0.82	4.54	2.50	2.84
38.20	0.170	0.83	4.18	2.40	2.54
40.70	0.237	0.76	3.80	2.50	2.21
43.10	0.190	0.81	3.43	2.40	2.03

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
45.60	0.191	0.81	3.05	2.50	1.88
48.10	0.181	0.82	2.67	2.50	1.67
50.50	2.000	0.00	2.30	2.40	0.00
53.00	2.000	0.00	1.92	2.50	0.00
55.40	2.000	0.00	1.56	2.40	0.00
57.90	0.278	0.72	1.18	2.50	0.65
60.40	0.452	0.55	0.80	2.50	0.33
62.80	2.000	0.00	0.43	2.40	0.00
65.30	2.000	0.00	0.05	2.50	0.00
66.80	2.000	0.00	0.00	0.00	0.00

Overall potential I_L : 63.62

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} weight factor	ε_{Nc} (%)	Δh (ft)	ΔS (in)
1.30	65	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.54	0.000
3.80	15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.94	0.00	2.46	0.000

Cumulative settlemetns: 0.000

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus (tsf)
α, b: Shear strain formula variables
γ: Average shear strain
ε_{15}: Volumetric strain after 15 cycles
N_c: Number of cycles
ε_{Nc}: Volumetric strain for number of cycles N_c (%)
Δh: Thickness of soil layer (in)
ΔS: Settlement of soil layer (in)

:: Vertical & Lateral displ.acements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60cs}$	γ_{lim} (%)	F_α	FS_{liq}	γ_{max} (%)	e_v weight factor	e_v (%)	dz (ft)	S_{v-1D} (in)	LDI (ft)
6.20	10	47.32	0.91	0.218	47.32	0.90	3.35	2.46	0.989	0.00
8.70	14	30.65	0.79	0.238	30.65	0.86	2.58	2.46	0.761	0.00
11.20	14	30.65	0.79	0.218	30.65	0.81	2.45	2.46	0.724	0.00
13.60	15	27.51	0.75	0.218	27.51	0.77	2.22	2.46	0.656	0.00
16.10	12	38.03	0.86	0.174	38.03	0.73	2.45	2.46	0.722	0.00
18.50	14	30.65	0.79	0.188	30.65	0.69	2.09	2.46	0.616	0.00
21.00	24	10.02	0.29	0.341	10.02	0.65	1.28	2.46	0.378	0.00
23.50	17	22.15	0.67	0.212	22.15	0.61	1.59	2.46	0.471	0.00
25.90	13	34.14	0.83	0.167	34.14	0.57	1.80	2.46	0.532	0.00
28.40	16	24.69	0.71	0.195	24.69	0.53	1.44	2.46	0.426	0.00
30.80	13	34.14	0.83	0.163	34.14	0.49	1.54	2.46	0.456	0.00

:: Vertical & Lateral displ.acements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60cs}$	γ_{lim} (%)	F_α	FS_{liq}	γ_{max} (%)	e_v weight factor	e_v (%)	dz (ft)	S_{v-1D} (in)	LDI (ft)
33.30	15	27.51	0.75	0.181	27.51	0.44	1.28	2.46	0.378	0.00
35.80	15	27.51	0.75	0.180	27.51	0.40	1.16	2.46	0.342	0.00
38.20	14	30.65	0.79	0.170	30.65	0.36	1.10	2.46	0.324	0.00
40.70	20	15.90	0.52	0.237	15.90	0.32	0.74	2.46	0.219	0.00
43.10	16	24.69	0.71	0.190	24.69	0.28	0.77	2.46	0.228	0.00
45.60	16	24.69	0.71	0.191	24.69	0.24	0.66	2.46	0.194	0.00
48.10	15	27.51	0.75	0.181	27.51	0.20	0.57	2.46	0.168	0.00
50.50	32	3.50	-0.22	2.000	0.00	0.16	0.00	2.46	0.000	0.00
53.00	57	0.00	-2.17	2.000	0.00	0.12	0.00	2.46	0.000	0.00
55.40	53	0.00	-1.83	2.000	0.00	0.08	0.00	2.46	0.000	0.00
57.90	22	12.67	0.41	0.278	12.67	0.04	0.07	2.46	0.022	0.00
60.40	28	6.08	0.04	0.452	6.08	0.00	0.00	2.46	0.000	0.00
62.80	39	1.07	-0.73	2.000	0.00	0.00	0.00	2.46	0.000	0.00
65.30	81	0.00	-4.29	2.000	0.00	0.00	0.00	1.97	0.000	0.00
66.80	103	0.00	-6.36	2.000	0.00	0.00	0.00	0.98	0.000	0.00

Cumulative settlements: 8.606 0.00

Abbreviations

γ_{lim}: Limiting shear strain (%)
F_α/N: Maximun shear strain factor
γ_{max}: Maximum shear strain (%)
e_v:: Post liquefaction volumetric strain (%)
S_{v-1D}: Estimated vertical settlement (in)
LDI: Estimated lateral displacement (ft)

PanGEO, Inc
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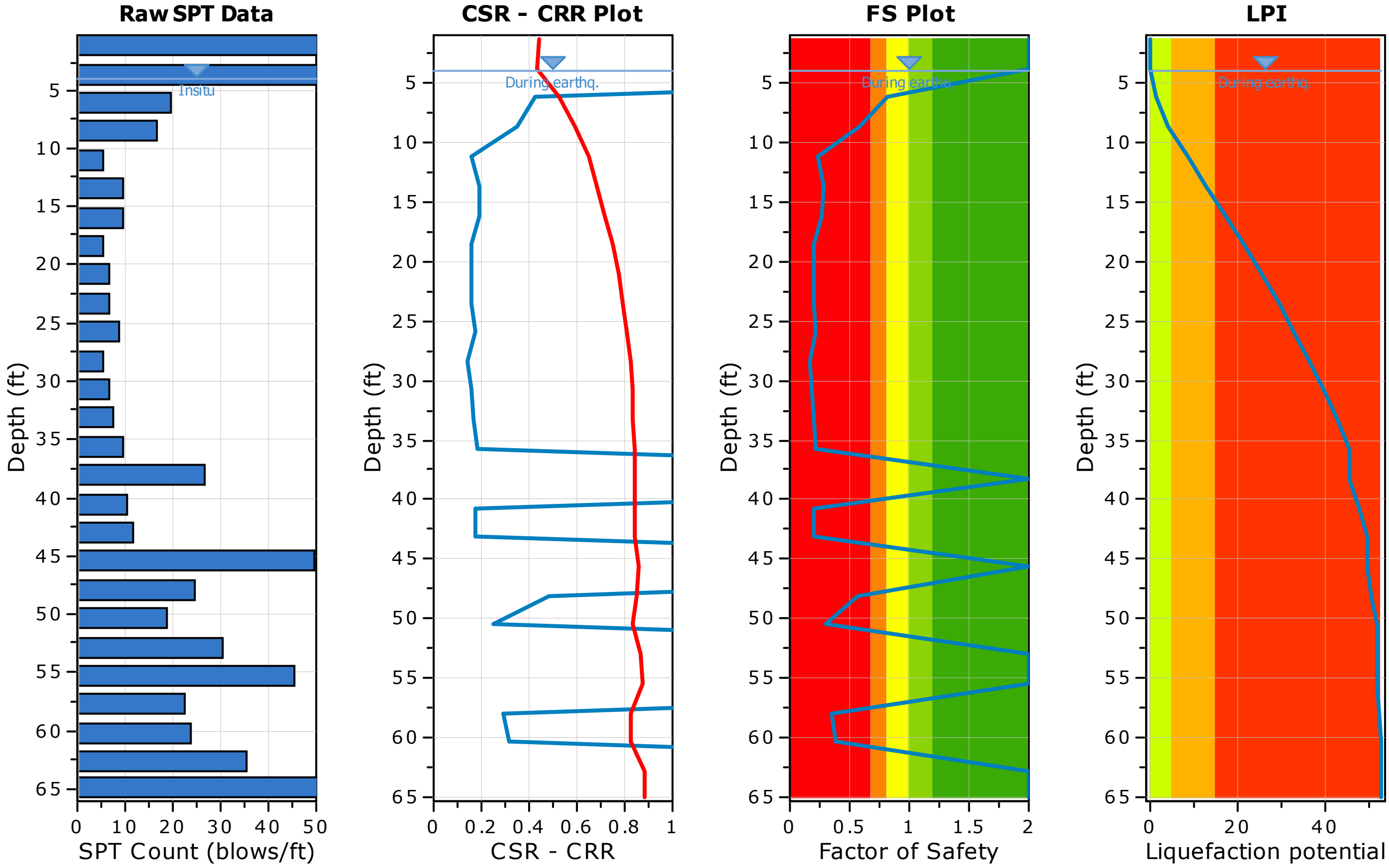
SPT BASED LIQUEFACTION ANALYSIS REPORT

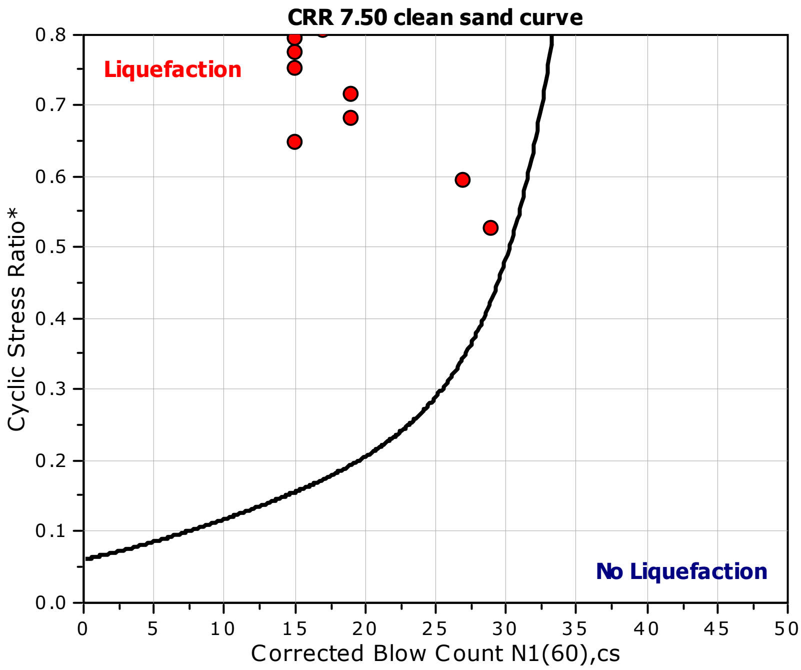
Project title : Issaquah TOD, 24-484 **SPT Name: SPT_CPT_03**

Location : 1550 Newport Way Northwest, Issaquah, Washington

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	4.10 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	4.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	7.50
Borehole diameter:	200mm	Peak ground acceleration:	0.74 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.00		

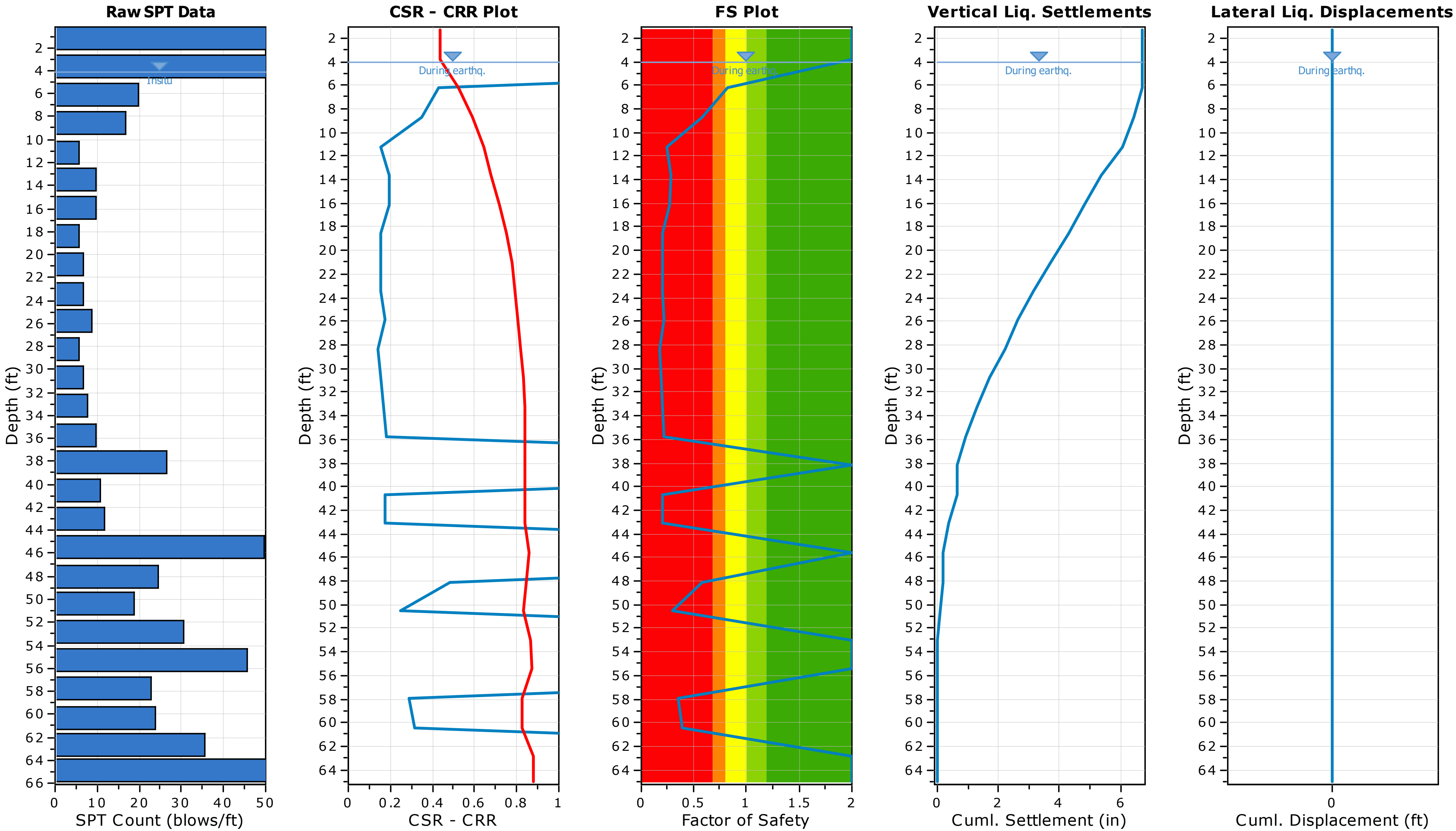




F.S. color scheme
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme
- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
1.30	62	15.00	131.80	2.54	No
3.80	90	15.00	135.00	2.46	Yes
6.20	20	15.00	115.00	2.46	Yes
8.70	17	25.00	113.50	2.46	Yes
11.20	6	40.00	106.00	2.46	Yes
13.60	10	40.00	110.00	2.46	Yes
16.10	10	40.00	110.00	2.46	Yes
18.50	6	40.00	106.00	2.46	Yes
21.00	7	40.00	107.00	2.46	Yes
23.50	7	40.00	107.00	2.46	Yes
25.90	9	40.00	109.00	2.46	Yes
28.40	6	40.00	106.00	2.46	Yes
30.80	7	40.00	107.00	2.46	Yes
33.30	8	40.00	108.00	2.46	Yes
35.80	10	40.00	110.00	2.46	Yes
38.20	27	25.00	118.50	2.46	Yes
40.70	11	25.00	110.50	2.46	Yes
43.10	12	20.00	111.00	2.46	Yes
45.60	50	20.00	130.00	2.46	Yes
48.10	25	20.00	117.50	2.46	Yes
50.50	19	20.00	114.50	2.46	Yes
53.00	31	15.00	120.50	2.46	Yes
55.40	46	15.00	128.00	2.46	Yes
57.90	23	15.00	116.50	2.46	Yes
60.40	24	15.00	117.00	2.46	Yes
62.80	36	15.00	123.00	2.34	Yes
65.00	69	15.00	132.85	2.46	Yes

Abbreviations

Depth:	Depth at which test was performed (ft)
SPT Field Value:	Number of blows per foot
Fines Content:	Fines content at test depth (%)
Unit Weight:	Unit weight at test depth (pcf)
Infl. Thickness:	Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy:	User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
1.30	62	131.80	0.09	0.00	0.09	0.26	1.70	1.00	1.15	0.75	1.00	91	15.00	3.26	94	4.000
3.80	90	135.00	0.25	0.00	0.25	0.26	1.46	1.00	1.15	0.75	1.00	113	15.00	3.26	116	4.000
6.20	20	115.00	0.39	0.07	0.33	0.36	1.52	1.00	1.15	0.75	1.00	26	15.00	3.26	29	0.429
8.70	17	113.50	0.53	0.14	0.39	0.36	1.44	1.00	1.15	0.80	1.00	22	25.00	5.07	27	0.347
11.20	6	106.00	0.67	0.22	0.45	0.47	1.50	1.00	1.15	0.85	1.00	9	40.00	5.58	15	0.156
13.60	10	110.00	0.80	0.30	0.50	0.43	1.38	1.00	1.15	0.85	1.00	13	40.00	5.58	19	0.194
16.10	10	110.00	0.94	0.37	0.56	0.44	1.32	1.00	1.15	0.85	1.00	13	40.00	5.58	19	0.194
18.50	6	106.00	1.06	0.45	0.61	0.48	1.30	1.00	1.15	0.95	1.00	9	40.00	5.58	15	0.156
21.00	7	107.00	1.20	0.53	0.67	0.47	1.24	1.00	1.15	0.95	1.00	9	40.00	5.58	15	0.156
23.50	7	107.00	1.33	0.61	0.73	0.48	1.20	1.00	1.15	0.95	1.00	9	40.00	5.58	15	0.156

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
25.90	9	109.00	1.46	0.68	0.78	0.46	1.15	1.00	1.15	0.95	1.00	11	40.00	5.58	17	0.174
28.40	6	106.00	1.59	0.76	0.84	0.50	1.13	1.00	1.15	0.95	1.00	7	40.00	5.58	13	0.140
30.80	7	107.00	1.72	0.83	0.89	0.49	1.09	1.00	1.15	1.00	1.00	9	40.00	5.58	15	0.156
33.30	8	108.00	1.86	0.91	0.95	0.48	1.05	1.00	1.15	1.00	1.00	10	40.00	5.58	16	0.165
35.80	10	110.00	2.00	0.99	1.01	0.46	1.02	1.00	1.15	1.00	1.00	12	40.00	5.58	18	0.184
38.20	27	118.50	2.14	1.06	1.07	0.32	1.00	1.00	1.15	1.00	1.00	31	25.00	5.07	36	4.000
40.70	11	110.50	2.28	1.14	1.13	0.47	0.97	1.00	1.15	1.00	1.00	12	25.00	5.07	17	0.174
43.10	12	111.00	2.41	1.22	1.19	0.46	0.95	1.00	1.15	1.00	1.00	13	20.00	4.48	17	0.174
45.60	50	130.00	2.57	1.29	1.28	0.26	0.95	1.00	1.15	1.00	1.00	55	20.00	4.48	59	4.000
48.10	25	117.50	2.72	1.37	1.35	0.36	0.92	1.00	1.15	1.00	1.00	26	20.00	4.48	30	0.485
50.50	19	114.50	2.86	1.45	1.41	0.41	0.89	1.00	1.15	1.00	1.00	19	20.00	4.48	23	0.249
53.00	31	120.50	3.01	1.53	1.48	0.33	0.89	1.00	1.15	1.00	1.00	32	15.00	3.26	35	4.000
55.40	46	128.00	3.16	1.60	1.56	0.26	0.90	1.00	1.15	1.00	1.00	48	15.00	3.26	51	4.000
57.90	23	116.50	3.31	1.68	1.63	0.40	0.84	1.00	1.15	1.00	1.00	22	15.00	3.26	25	0.290
60.40	24	117.00	3.45	1.76	1.70	0.40	0.83	1.00	1.15	1.00	1.00	23	15.00	3.26	26	0.316
62.80	36	123.00	3.60	1.83	1.77	0.31	0.85	1.00	1.15	1.00	1.00	35	15.00	3.26	38	4.000
65.00	69	132.85	3.75	1.90	1.85	0.26	0.86	1.00	1.15	1.00	1.00	69	15.00	3.26	72	4.000

Abbreviations

σ_v:	Total stress during SPT test (tsf)
u_o:	Water pore pressure during SPT test (tsf)
σ'_{vo}:	Effective overburden pressure during SPT test (tsf)
m:	Stress exponent normalization factor
C_N:	Overburden corretion factor
C_E:	Energy correction factor
C_B:	Borehole diameter correction factor
C_R:	Rod length correction factor
C_S:	Liner correction factor
$N_{1(60)}$:	Corrected N_{SPT} to a 60% energy ratio
$\Delta(N_1)_{60}$	Equivalent clean sand adjustment
$N_{1(60)cs}$:	Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$:	Cyclic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
1.30	131.80	0.09	0.00	0.09	1.00	1.00	0.483	2.20	94	1.00	0.483	1.10	0.439	2.000
3.80	135.00	0.25	0.00	0.25	1.00	1.00	0.480	2.20	116	1.00	0.480	1.10	0.436	2.000
6.20	115.00	0.39	0.07	0.32	0.99	1.00	0.578	1.94	29	1.00	0.578	1.10	0.526	0.816
8.70	113.50	0.53	0.15	0.39	0.99	1.00	0.653	1.82	27	1.00	0.653	1.10	0.594	0.584
11.20	106.00	0.67	0.22	0.44	0.98	1.00	0.709	1.32	15	1.00	0.709	1.10	0.647	0.241
13.60	110.00	0.80	0.30	0.50	0.97	1.00	0.747	1.45	19	1.00	0.747	1.10	0.681	0.285
16.10	110.00	0.94	0.38	0.56	0.96	1.00	0.775	1.45	19	1.00	0.775	1.08	0.717	0.271
18.50	106.00	1.06	0.45	0.61	0.95	1.00	0.798	1.32	15	1.00	0.798	1.06	0.752	0.208
21.00	107.00	1.20	0.53	0.67	0.94	1.00	0.815	1.32	15	1.00	0.815	1.05	0.776	0.201
23.50	107.00	1.33	0.61	0.72	0.93	1.00	0.828	1.32	15	1.00	0.828	1.04	0.794	0.197
25.90	109.00	1.46	0.68	0.78	0.93	1.00	0.835	1.38	17	1.00	0.835	1.04	0.806	0.216
28.40	106.00	1.59	0.76	0.83	0.91	1.00	0.842	1.26	13	1.00	0.842	1.02	0.822	0.170
30.80	107.00	1.72	0.84	0.89	0.90	1.00	0.846	1.32	15	1.00	0.846	1.02	0.829	0.188
33.30	108.00	1.86	0.91	0.94	0.89	1.00	0.847	1.35	16	1.00	0.847	1.01	0.836	0.197
35.80	110.00	2.00	0.99	1.00	0.88	1.00	0.845	1.42	18	1.00	0.845	1.01	0.839	0.219

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
38.20	118.50	2.14	1.07	1.07	0.87	1.00	0.838	2.20	36	1.00	0.838	1.00	0.841	2.000
40.70	110.50	2.28	1.15	1.13	0.86	1.00	0.834	1.38	17	1.00	0.834	0.99	0.840	0.207
43.10	111.00	2.41	1.22	1.19	0.85	1.00	0.829	1.38	17	1.00	0.829	0.99	0.840	0.207
45.60	130.00	2.57	1.30	1.27	0.84	1.00	0.815	2.20	59	1.00	0.815	0.95	0.862	2.000
48.10	117.50	2.72	1.38	1.34	0.83	1.00	0.806	2.00	30	1.00	0.806	0.95	0.847	0.573
50.50	114.50	2.86	1.45	1.40	0.82	1.00	0.798	1.62	23	1.00	0.798	0.96	0.834	0.299
53.00	120.50	3.01	1.53	1.48	0.81	1.00	0.788	2.20	35	1.00	0.788	0.91	0.864	2.000
55.40	128.00	3.16	1.60	1.56	0.79	1.00	0.776	2.20	51	1.00	0.776	0.89	0.876	2.000
57.90	116.50	3.31	1.68	1.62	0.78	1.00	0.767	1.72	25	1.00	0.767	0.93	0.825	0.352
60.40	117.00	3.45	1.76	1.69	0.77	1.00	0.758	1.77	26	1.00	0.758	0.92	0.824	0.383
62.80	123.00	3.60	1.83	1.76	0.76	1.00	0.748	2.20	38	1.00	0.748	0.85	0.881	2.000
65.00	132.85	3.75	1.90	1.84	0.75	1.00	0.736	2.20	72	1.00	0.736	0.84	0.880	2.000

Abbreviations

$\sigma_{v,eq}$: Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$: Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$: Effective overburden pressure, during earthquake (tsf)
r_d: Nonlinear shear mass factor
α: Improvement factor due to stone columns
CSR : Cyclic Stress Ratio
MSF : Magnitude Scaling Factor
$CSR_{eq,M=7.5}$: CSR adjusted for M=7.5
K_{sigma}: Effective overburden stress factor
CSR^*: CSR fully adjusted (user FS applied)***
FS: Calculated factor of safety against soil liquefaction

*** User FS: 1.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
1.30	2.000	0.00	9.80	2.50	0.00
3.80	2.000	0.00	9.42	2.50	0.00
6.20	0.816	0.18	9.06	2.40	1.22
8.70	0.584	0.42	8.67	2.50	2.75
11.20	0.241	0.76	8.29	2.50	4.79
13.60	0.285	0.71	7.93	2.40	4.14
16.10	0.271	0.73	7.55	2.50	4.19
18.50	0.208	0.79	7.18	2.40	4.16
21.00	0.201	0.80	6.80	2.50	4.14
23.50	0.197	0.80	6.42	2.50	3.93
25.90	0.216	0.78	6.05	2.40	3.47
28.40	0.170	0.83	5.67	2.50	3.59
30.80	0.188	0.81	5.31	2.40	3.15
33.30	0.197	0.80	4.93	2.50	3.01
35.80	0.219	0.78	4.54	2.50	2.70
38.20	2.000	0.00	4.18	2.40	0.00
40.70	0.207	0.79	3.80	2.50	2.29
43.10	0.207	0.79	3.43	2.40	1.99
45.60	2.000	0.00	3.05	2.50	0.00
48.10	0.573	0.43	2.67	2.50	0.87
50.50	0.299	0.70	2.30	2.40	1.18

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
53.00	2.000	0.00	1.92	2.50	0.00
55.40	2.000	0.00	1.56	2.40	0.00
57.90	0.352	0.65	1.18	2.50	0.58
60.40	0.383	0.62	0.80	2.50	0.37
62.80	2.000	0.00	0.43	2.40	0.00
65.00	2.000	0.00	0.09	2.20	0.00

Overall potential I_L : 52.55

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} weight factor	ε_{Nc} (%)	Δh (ft)	ΔS (in)
1.30	91	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.54	0.000
3.80	113	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.94	0.00	2.46	0.000

Cumulative settlemetns: 0.000

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus (tsf)
α, b: Shear strain formula variables
γ: Average shear strain
ε_{15}: Volumetric strain after 15 cycles
N_c: Number of cycles
ε_{Nc}: Volumetric strain for number of cycles N_c (%)
Δh: Thickness of soil layer (in)
ΔS: Settlement of soil layer (in)

:: Vertical & Lateral displ.acements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60cs}$	γ_{lim} (%)	F_α	FS_{liq}	γ_{max} (%)	e_v weight factor	e_v (%)	dz (ft)	S_{v-1D} (in)	LDI (ft)
6.20	29	5.33	-0.02	0.816	5.06	0.90	0.93	2.46	0.275	0.00
8.70	27	6.92	0.11	0.584	6.92	0.86	1.30	2.46	0.385	0.00
11.20	15	27.51	0.75	0.241	27.51	0.81	2.34	2.46	0.690	0.00
13.60	19	17.78	0.57	0.285	17.78	0.77	1.86	2.46	0.548	0.00
16.10	19	17.78	0.57	0.271	17.78	0.73	1.76	2.46	0.519	0.00
18.50	15	27.51	0.75	0.208	27.51	0.69	1.99	2.46	0.587	0.00
21.00	15	27.51	0.75	0.201	27.51	0.65	1.87	2.46	0.552	0.00
23.50	15	27.51	0.75	0.197	27.51	0.61	1.75	2.46	0.516	0.00
25.90	17	22.15	0.67	0.216	22.15	0.57	1.49	2.46	0.440	0.00
28.40	13	34.14	0.83	0.170	34.14	0.53	1.67	2.46	0.493	0.00
30.80	15	27.51	0.75	0.188	27.51	0.49	1.40	2.46	0.413	0.00
33.30	16	24.69	0.71	0.197	24.69	0.44	1.22	2.46	0.360	0.00
35.80	18	19.85	0.62	0.219	19.85	0.40	1.01	2.46	0.299	0.00
38.20	36	1.86	-0.51	2.000	0.00	0.36	0.00	2.46	0.000	0.00
40.70	17	22.15	0.67	0.207	22.15	0.32	0.84	2.46	0.249	0.00

:: Vertical & Lateral displ.acements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60cs}$	γ_{lim} (%)	F_α	FS_{liq}	γ_{max} (%)	e_v weight factor	e_v (%)	dz (ft)	S_{v-1D} (in)	LDI (ft)
43.10	17	22.15	0.67	0.207	22.15	0.28	0.74	2.46	0.218	0.00
45.60	59	0.00	-2.34	2.000	0.00	0.24	0.00	2.46	0.000	0.00
48.10	30	4.65	-0.09	0.573	4.65	0.20	0.18	2.46	0.054	0.00
50.50	23	11.27	0.35	0.299	11.27	0.16	0.32	2.46	0.096	0.00
53.00	35	2.20	-0.44	2.000	0.00	0.12	0.00	2.46	0.000	0.00
55.40	51	0.02	-1.67	2.000	0.00	0.08	0.00	2.46	0.000	0.00
57.90	25	8.88	0.23	0.352	8.88	0.04	0.07	2.46	0.020	0.00
60.40	26	7.85	0.17	0.383	7.85	0.00	0.00	2.46	0.000	0.00
62.80	38	1.30	-0.65	2.000	0.00	0.00	0.00	2.34	0.000	0.00
65.00	72	0.00	-3.47	2.000	0.00	0.00	0.00	2.46	0.000	0.00

Cumulative settlements: 6.713 0.00

Abbreviations

γ_{lim}: Limiting shear strain (%)
F_α/N: Maximun shear strain factor
γ_{max}: Maximum shear strain (%)
e_v:: Post liquefaction volumetric strain (%)
S_{v-1D}: Estimated vertical settlement (in)
LDI: Estimated lateral displacement (ft)

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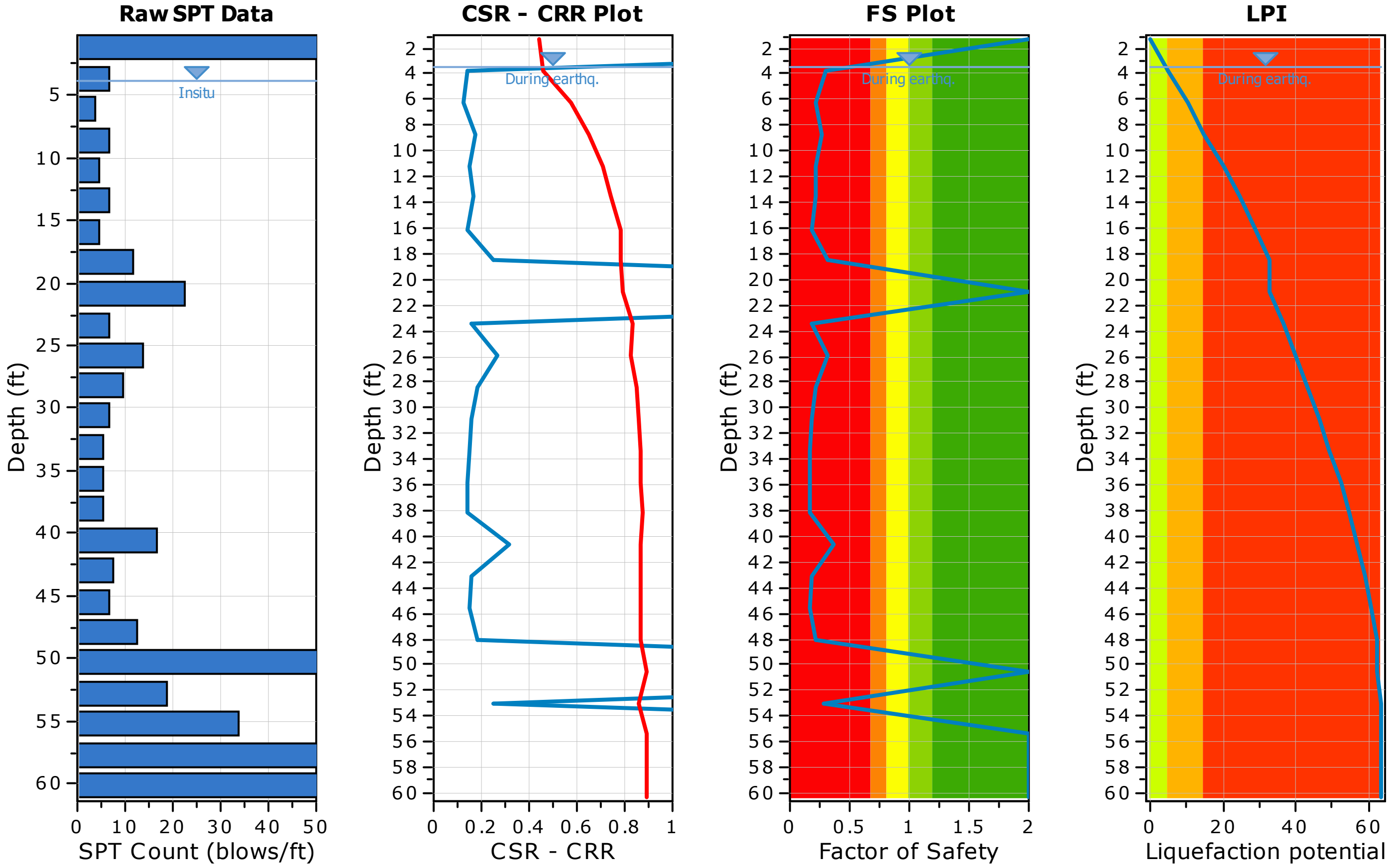
SPT BASED LIQUEFACTION ANALYSIS REPORT

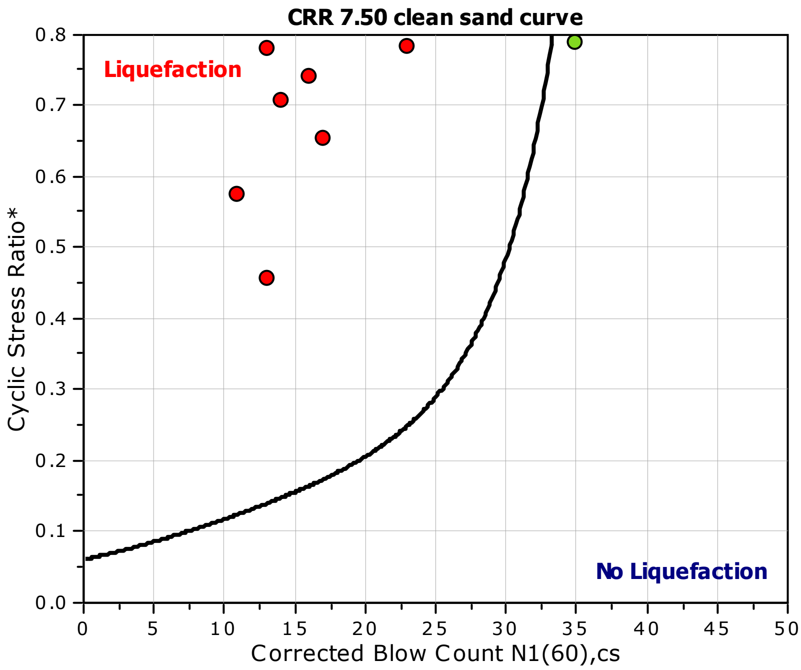
Project title : Issaquah TOD, 24-484 **SPT Name: SPT_CPT_04**

Location : 1550 Newport Way Northwest, Issaquah, Washington

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	3.80 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	3.50 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	7.50
Borehole diameter:	200mm	Peak ground acceleration:	0.74 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.00		

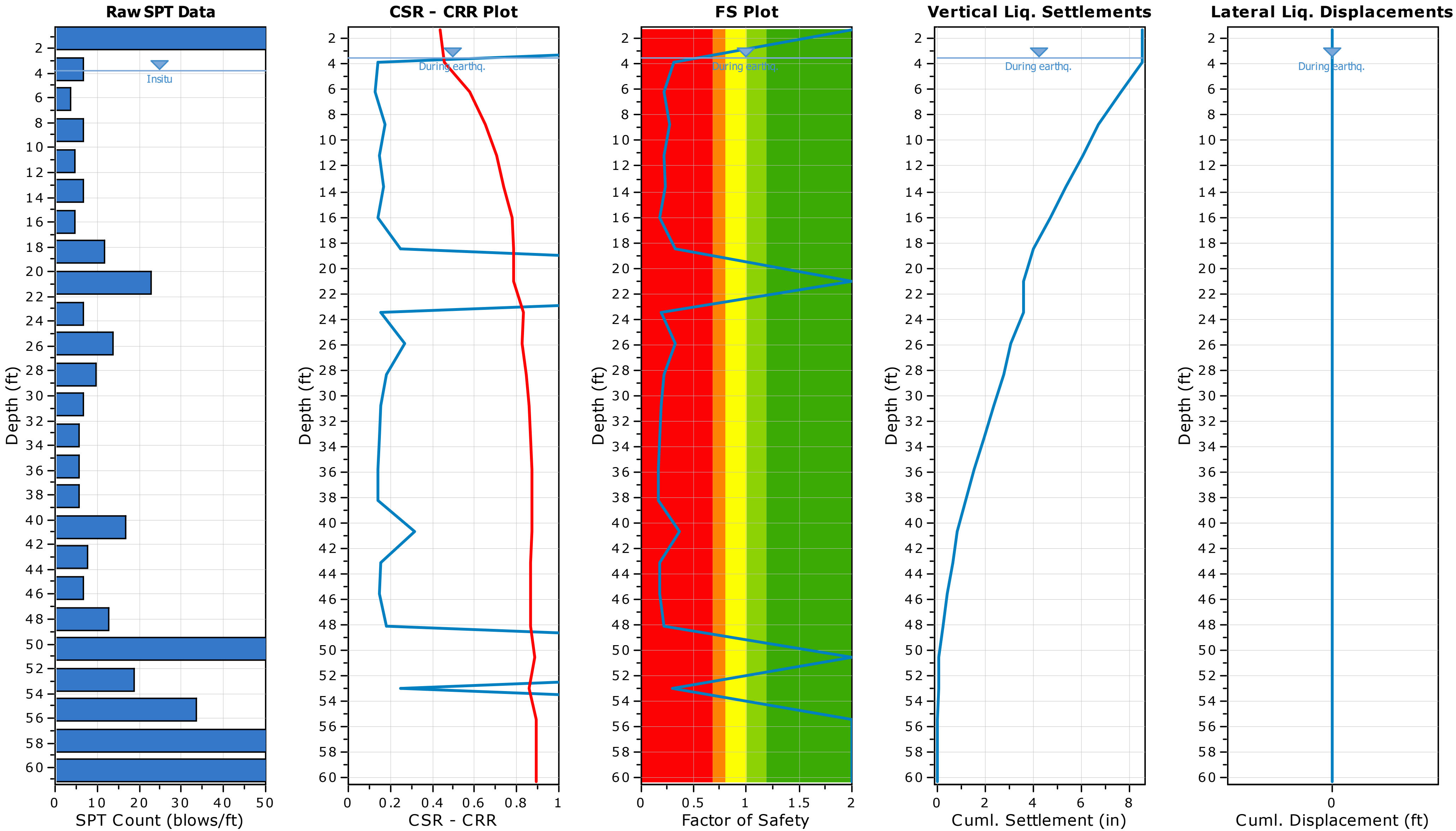




F.S. color scheme
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme
- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
1.30	55	15.00	130.75	2.54	No
3.80	7	15.00	107.00	2.46	Yes
6.20	4	25.00	104.00	2.46	Yes
8.70	7	36.00	107.00	2.46	Yes
11.20	5	60.00	105.00	2.46	Yes
13.60	7	60.00	107.00	2.46	Yes
16.10	5	60.00	105.00	2.46	Yes
18.50	12	60.00	111.00	2.46	Yes
21.00	23	30.00	116.50	2.46	Yes
23.50	7	30.00	107.00	2.46	Yes
25.90	14	50.00	112.00	2.46	Yes
28.40	10	50.00	110.00	2.46	Yes
30.80	7	50.00	107.00	2.46	Yes
33.30	6	50.00	106.00	2.46	Yes
35.80	6	50.00	106.00	2.46	Yes
38.20	6	50.00	106.00	2.46	Yes
40.70	17	35.00	113.50	2.46	Yes
43.10	8	35.00	108.00	2.46	Yes
45.60	7	35.00	107.00	2.46	Yes
48.10	13	20.00	111.50	2.46	Yes
50.50	55	20.00	130.75	2.46	Yes
53.00	19	15.00	114.50	2.46	Yes
55.40	34	15.00	122.00	2.46	Yes
57.90	75	15.00	133.75	2.42	Yes
60.30	90	15.00	135.00	2.46	Yes

Abbreviations

Depth: Depth at which test was performed (ft)
SPT Field Value: Number of blows per foot
Fines Content: Fines content at test depth (%)
Unit Weight: Unit weight at test depth (pcf)
Infl. Thickness: Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy: User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
1.30	55	130.75	0.08	0.00	0.08	0.26	1.70	1.00	1.15	0.75	1.00	81	15.00	3.26	84	4.000
3.80	7	107.00	0.22	0.00	0.22	0.48	1.70	1.00	1.15	0.75	1.00	10	15.00	3.26	13	0.140
6.20	4	104.00	0.34	0.07	0.27	0.49	1.70	1.00	1.15	0.75	1.00	6	25.00	5.07	11	0.125
8.70	7	107.00	0.48	0.15	0.32	0.44	1.68	1.00	1.15	0.80	1.00	11	36.00	5.52	17	0.174
11.20	5	105.00	0.61	0.23	0.38	0.47	1.62	1.00	1.15	0.85	1.00	8	60.00	5.60	14	0.148
13.60	7	107.00	0.74	0.31	0.43	0.45	1.50	1.00	1.15	0.85	1.00	10	60.00	5.60	16	0.165
16.10	5	105.00	0.87	0.38	0.48	0.48	1.46	1.00	1.15	0.85	1.00	7	60.00	5.60	13	0.140
18.50	12	111.00	1.00	0.46	0.54	0.40	1.31	1.00	1.15	0.95	1.00	17	60.00	5.60	23	0.249
21.00	23	116.50	1.15	0.54	0.61	0.32	1.19	1.00	1.15	0.95	1.00	30	30.00	5.36	35	4.000
23.50	7	107.00	1.28	0.61	0.67	0.47	1.25	1.00	1.15	0.95	1.00	10	30.00	5.36	15	0.156
25.90	14	112.00	1.42	0.69	0.73	0.40	1.16	1.00	1.15	0.95	1.00	18	50.00	5.61	24	0.268
28.40	10	110.00	1.55	0.77	0.79	0.45	1.14	1.00	1.15	0.95	1.00	12	50.00	5.61	18	0.184

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
30.80	7	107.00	1.68	0.84	0.84	0.48	1.12	1.00	1.15	1.00	1.00	9	50.00	5.61	15	0.156
33.30	6	106.00	1.81	0.92	0.89	0.50	1.09	1.00	1.15	1.00	1.00	8	50.00	5.61	14	0.148
35.80	6	106.00	1.95	1.00	0.95	0.50	1.06	1.00	1.15	1.00	1.00	7	50.00	5.61	13	0.140
38.20	6	106.00	2.07	1.07	1.00	0.51	1.03	1.00	1.15	1.00	1.00	7	50.00	5.61	13	0.140
40.70	17	113.50	2.22	1.15	1.06	0.40	1.00	1.00	1.15	1.00	1.00	20	35.00	5.51	26	0.316
43.10	8	108.00	2.34	1.23	1.12	0.49	0.97	1.00	1.15	1.00	1.00	9	35.00	5.51	15	0.156
45.60	7	107.00	2.48	1.30	1.17	0.51	0.95	1.00	1.15	1.00	1.00	8	35.00	5.51	14	0.148
48.10	13	111.50	2.62	1.38	1.24	0.46	0.93	1.00	1.15	1.00	1.00	14	20.00	4.48	18	0.184
50.50	55	130.75	2.77	1.46	1.32	0.26	0.94	1.00	1.15	1.00	1.00	60	20.00	4.48	64	4.000
53.00	19	114.50	2.92	1.54	1.38	0.42	0.89	1.00	1.15	1.00	1.00	20	15.00	3.26	23	0.249
55.40	34	122.00	3.06	1.61	1.45	0.31	0.91	1.00	1.15	1.00	1.00	35	15.00	3.26	38	4.000
57.90	75	133.75	3.23	1.69	1.54	0.26	0.91	1.00	1.15	1.00	1.00	78	15.00	3.26	81	4.000
60.30	90	135.00	3.39	1.76	1.63	0.26	0.89	1.00	1.15	1.00	1.00	92	15.00	3.26	95	4.000

Abbreviations

σ_v: Total stress during SPT test (tsf)
u_o: Water pore pressure during SPT test (tsf)
σ'_{vo}: Effective overburden pressure during SPT test (tsf)
m: Stress exponent normalization factor
C_N: Overburden corretion factor
C_E: Energy correction factor
C_B: Borehole diameter correction factor
C_R: Rod length correction factor
C_S: Liner correction factor
$N_{1(60)}$: Corrected N_{SPT} to a 60% energy ratio
$\Delta(N_1)_{60}$ Equivalent clean sand adjustment
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$: Cyclic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
1.30	130.75	0.08	0.00	0.08	1.00	1.00	0.483	2.20	84	1.00	0.483	1.10	0.439	2.000
3.80	107.00	0.22	0.01	0.21	1.00	1.00	0.501	1.26	13	1.00	0.501	1.10	0.456	0.307
6.20	104.00	0.34	0.08	0.26	0.99	1.00	0.632	1.21	11	1.00	0.632	1.10	0.575	0.218
8.70	107.00	0.48	0.16	0.32	0.99	1.00	0.718	1.38	17	1.00	0.718	1.10	0.653	0.266
11.20	105.00	0.61	0.24	0.37	0.98	1.00	0.777	1.29	14	1.00	0.777	1.10	0.706	0.209
13.60	107.00	0.74	0.32	0.42	0.97	1.00	0.815	1.35	16	1.00	0.815	1.10	0.741	0.222
16.10	105.00	0.87	0.39	0.48	0.96	1.00	0.846	1.26	13	1.00	0.846	1.08	0.781	0.179
18.50	111.00	1.00	0.47	0.53	0.95	1.00	0.861	1.62	23	1.00	0.861	1.10	0.783	0.319
21.00	116.50	1.15	0.55	0.60	0.94	1.00	0.867	2.20	35	1.00	0.867	1.10	0.788	2.000
23.50	107.00	1.28	0.62	0.66	0.93	1.00	0.877	1.32	15	1.00	0.877	1.05	0.833	0.187
25.90	112.00	1.42	0.70	0.72	0.93	1.00	0.879	1.67	24	1.00	0.879	1.06	0.829	0.324
28.40	110.00	1.55	0.78	0.78	0.91	1.00	0.881	1.42	18	1.00	0.881	1.04	0.848	0.217
30.80	107.00	1.68	0.85	0.83	0.90	1.00	0.882	1.32	15	1.00	0.882	1.03	0.859	0.182
33.30	106.00	1.81	0.93	0.88	0.89	1.00	0.882	1.29	14	1.00	0.882	1.02	0.866	0.171
35.80	106.00	1.95	1.01	0.94	0.88	1.00	0.881	1.26	13	1.00	0.881	1.01	0.870	0.161
38.20	106.00	2.07	1.08	0.99	0.87	1.00	0.878	1.26	13	1.00	0.878	1.01	0.872	0.161
40.70	113.50	2.22	1.16	1.05	0.86	1.00	0.870	1.77	26	1.00	0.870	1.00	0.870	0.363
43.10	108.00	2.34	1.24	1.11	0.85	1.00	0.865	1.32	15	1.00	0.865	0.99	0.869	0.180
45.60	107.00	2.48	1.31	1.16	0.84	1.00	0.858	1.29	14	1.00	0.858	0.99	0.867	0.171

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS	
48.10	111.50	2.62	1.39	1.23	0.83	1.00	0.850	1.42	18	1.00	0.850	0.98	0.865	0.212	●
50.50	130.75	2.77	1.47	1.31	0.82	1.00	0.833	2.20	64	1.00	0.833	0.94	0.889	2.000	●
53.00	114.50	2.92	1.54	1.37	0.81	1.00	0.823	1.62	23	1.00	0.823	0.96	0.856	0.291	●
55.40	122.00	3.06	1.62	1.45	0.79	1.00	0.810	2.20	38	1.00	0.810	0.91	0.892	2.000	●
57.90	133.75	3.23	1.70	1.53	0.78	1.00	0.794	2.20	81	1.00	0.794	0.89	0.891	2.000	●
60.30	135.00	3.39	1.77	1.62	0.77	1.00	0.778	2.20	95	1.00	0.778	0.87	0.890	2.000	●

Abbreviations

$\sigma_{v,eq}$: Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$: Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$: Effective overburden pressure, during earthquake (tsf)
r_d: Nonlinear shear mass factor
α: Improvement factor due to stone columns
CSR : Cyclic Stress Ratio
MSF : Magnitude Scaling Factor
$CSR_{eq,M=7.5}$: CSR adjusted for M=7.5
K_{sigma}: Effective overburden stress factor
CSR^*: CSR fully adjusted (user FS applied)***
FS: Calculated factor of safety against soil liquefaction

*** User FS: 1.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
1.30	2.000	0.00	9.80	2.50	0.00
3.80	0.307	0.69	9.42	2.50	4.97
6.20	0.218	0.78	9.06	2.40	5.18
8.70	0.266	0.73	8.67	2.50	4.85
11.20	0.209	0.79	8.29	2.50	5.00
13.60	0.222	0.78	7.93	2.40	4.51
16.10	0.179	0.82	7.55	2.50	4.72
18.50	0.319	0.68	7.18	2.40	3.58
21.00	2.000	0.00	6.80	2.50	0.00
23.50	0.187	0.81	6.42	2.50	3.97
25.90	0.324	0.68	6.05	2.40	2.99
28.40	0.217	0.78	5.67	2.50	3.39
30.80	0.182	0.82	5.31	2.40	3.18
33.30	0.171	0.83	4.93	2.50	3.11
35.80	0.161	0.84	4.54	2.50	2.91
38.20	0.161	0.84	4.18	2.40	2.57
40.70	0.363	0.64	3.80	2.50	1.84
43.10	0.180	0.82	3.43	2.40	2.06
45.60	0.171	0.83	3.05	2.50	1.93
48.10	0.212	0.79	2.67	2.50	1.60
50.50	2.000	0.00	2.30	2.40	0.00
53.00	0.291	0.71	1.92	2.50	1.04
55.40	2.000	0.00	1.56	2.40	0.00
57.90	2.000	0.00	1.18	2.50	0.00
60.30	2.000	0.00	0.81	2.40	0.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L

Overall potential I_L : 63.39

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} weight factor	ε_{Nc} (%)	Δh (ft)	ΔS (in)
1.30	81	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.54	0.000

Cumulative settlemetns: 0.000

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus (tsf)
α, b: Shear strain formula variables
γ: Average shear strain
ε_{15}: Volumetric strain after 15 cycles
N_c: Number of cycles
ε_{Nc}: Volumetric strain for number of cycles N_c (%)
Δh: Thickness of soil layer (in)
ΔS: Settlement of soil layer (in)

:: Vertical & Lateral displ.acements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60cs}$	γ_{lim} (%)	F_α	FS_{liq}	γ_{max} (%)	e_v weight factor	e_v (%)	dz (ft)	S_{v-1D} (in)	LDI (ft)
3.80	13	34.14	0.83	0.307	34.14	0.94	2.97	2.46	0.877	0.00
6.20	11	42.40	0.89	0.218	42.40	0.90	3.16	2.46	0.934	0.00
8.70	17	22.15	0.67	0.266	22.15	0.86	2.24	2.46	0.661	0.00
11.20	14	30.65	0.79	0.209	30.65	0.81	2.45	2.46	0.724	0.00
13.60	16	24.69	0.71	0.222	24.69	0.77	2.12	2.46	0.626	0.00
16.10	13	34.14	0.83	0.179	34.14	0.73	2.32	2.46	0.685	0.00
18.50	23	11.27	0.35	0.319	11.27	0.69	1.41	2.46	0.417	0.00
21.00	35	2.20	-0.44	2.000	0.00	0.65	0.00	2.46	0.000	0.00
23.50	15	27.51	0.75	0.187	27.51	0.61	1.75	2.46	0.516	0.00
25.90	24	10.02	0.29	0.324	10.02	0.57	1.12	2.46	0.330	0.00
28.40	18	19.85	0.62	0.217	19.85	0.53	1.32	2.46	0.390	0.00
30.80	15	27.51	0.75	0.182	27.51	0.49	1.40	2.46	0.413	0.00
33.30	14	30.65	0.79	0.171	30.65	0.44	1.34	2.46	0.396	0.00
35.80	13	34.14	0.83	0.161	34.14	0.40	1.28	2.46	0.378	0.00
38.20	13	34.14	0.83	0.161	34.14	0.36	1.15	2.46	0.340	0.00
40.70	26	7.85	0.17	0.363	7.85	0.32	0.58	2.46	0.170	0.00
43.10	15	27.51	0.75	0.180	27.51	0.28	0.81	2.46	0.239	0.00
45.60	14	30.65	0.79	0.171	30.65	0.24	0.72	2.46	0.214	0.00
48.10	18	19.85	0.62	0.212	19.85	0.20	0.50	2.46	0.147	0.00
50.50	64	0.00	-2.77	2.000	0.00	0.16	0.00	2.46	0.000	0.00
53.00	23	11.27	0.35	0.291	11.27	0.12	0.24	2.46	0.070	0.00
55.40	38	1.30	-0.65	2.000	0.00	0.08	0.00	2.46	0.000	0.00

:: Vertical & Lateral displ.acements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60cs}$	γ_{lim} (%)	F_α	FS_{liq}	γ_{max} (%)	e_v weight factor	e_v (%)	dz (ft)	S_{v-1D} (in)	LDI (ft)
57.90	81	0.00	-4.29	2.000	0.00	0.04	0.00	2.42	0.000	0.00
60.30	95	0.00	-5.59	2.000	0.00	0.00	0.00	2.46	0.000	0.00

Cumulative settlements: 8.530 0.00

Abbreviations

γ_{lim}: Limiting shear strain (%)
F_α/N: Maximun shear strain factor
γ_{max}: Maximum shear strain (%)
e_v:: Post liquefaction volumetric strain (%)
S_{v-1D}: Estimated vertical settlement (in)
LDI: Estimated lateral displacement (ft)

References

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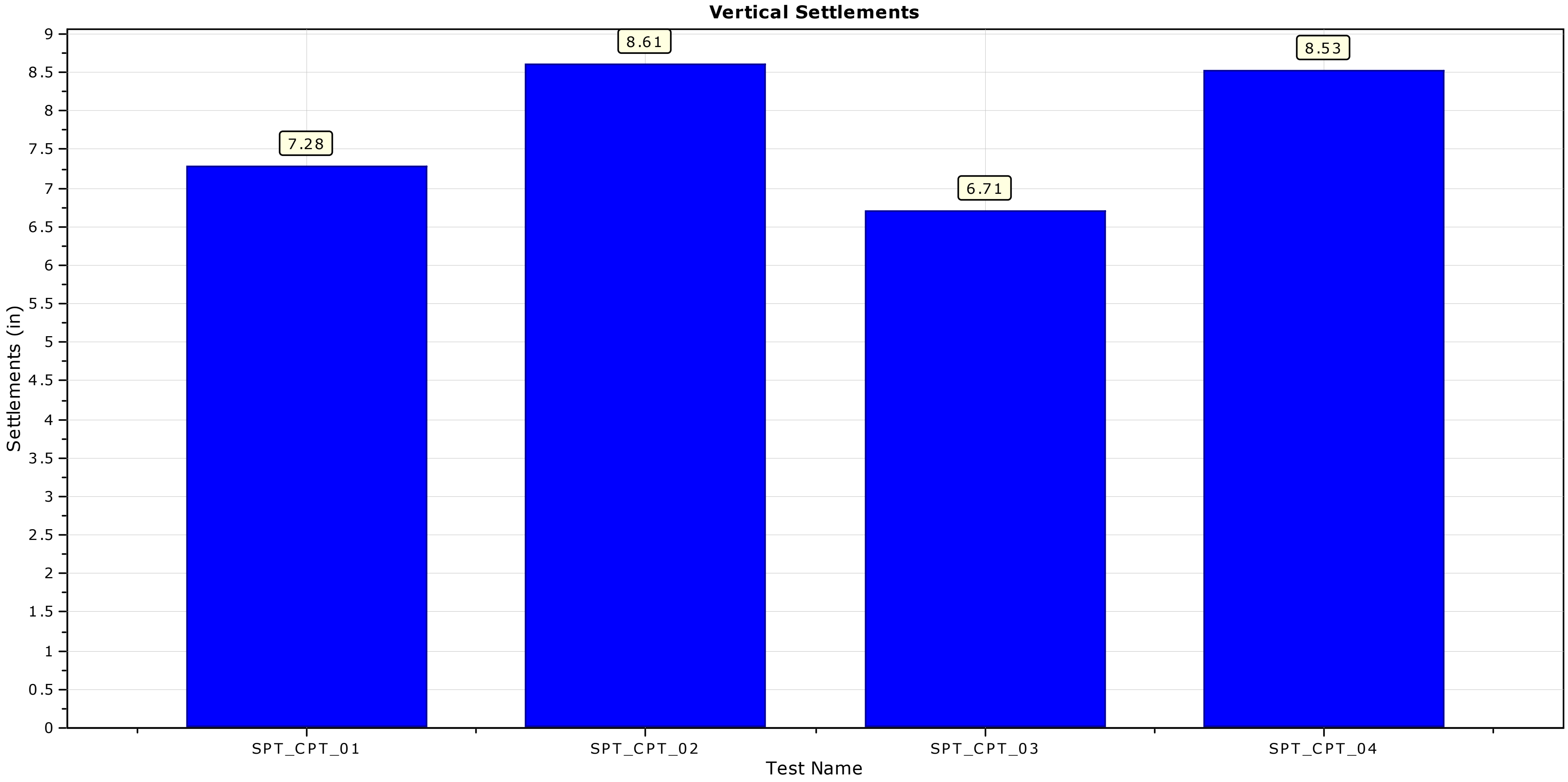
206.262.0370

SUMMARY CALCULATION REPORT

Project title : Issaquah TOD, 24-484

Location : 1550 Newport Way Northwest, Issaquah, Washington



TRAFFIC IMPACT ANALYSIS

Trailhead Apartments

Issaquah, Washington

March 13, 2025

1011 E Main Ave Suite 453 | Puyallup, WA | 98372
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TRAILHEAD APARTMENTS
TRAFFIC IMPACT ANALYSIS

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03/13/2025

TRAILHEAD APARTMENTS
TRAFFIC IMPACT ANALYSIS

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TRAILHEAD APARTMENTS
TRAFFIC IMPACT ANALYSIS

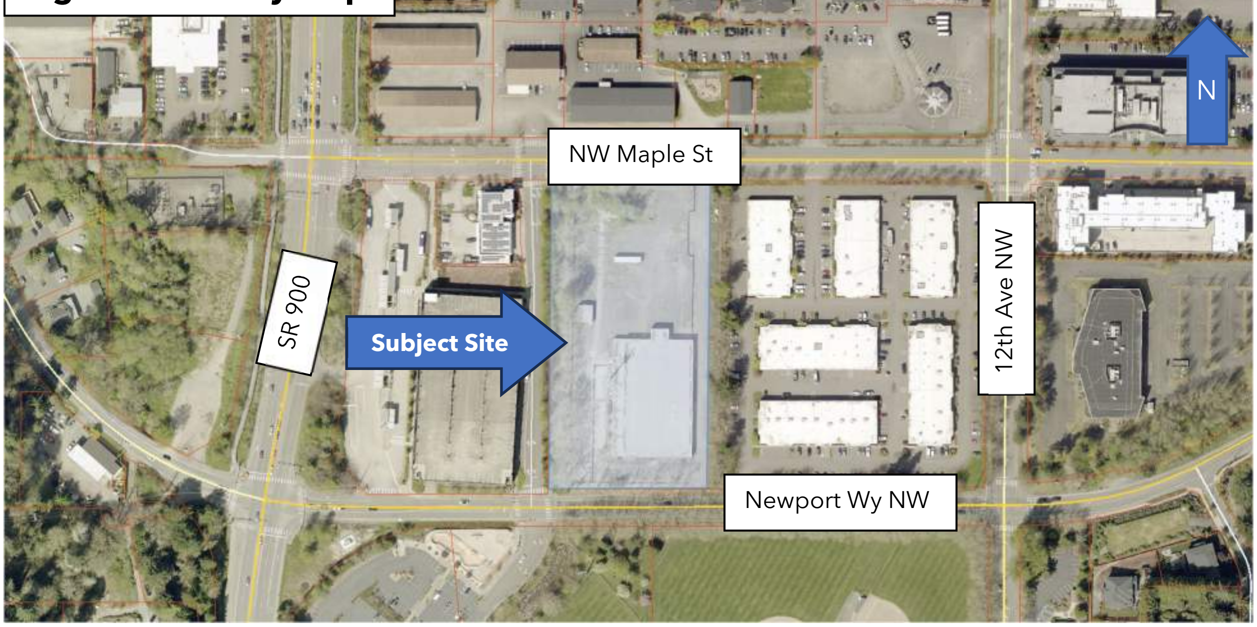
1. INTRODUCTION

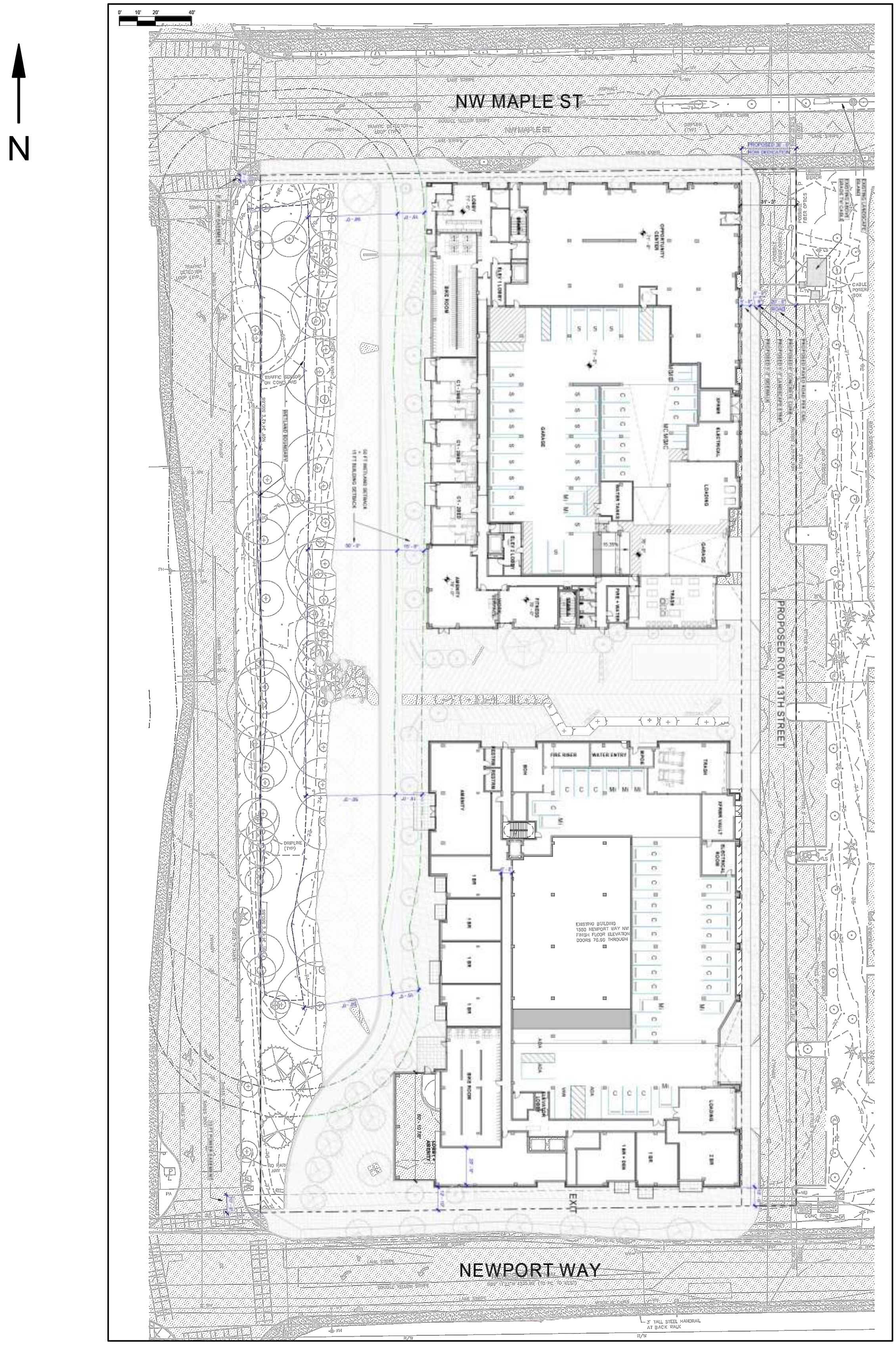
Heath & Associates has been engaged to prepare a Traffic Impact Analysis (TIA) for a proposed multifamily residential project, located in Issaquah, Washington. The study begins with evaluating current roadway conditions and establishing baseline volumes and traffic operations within a defined study area. The study area is then evaluated without and with the proposed development to determine whether adequate capacity, safety, and other conditions are met. The scope of this TIA is based on our approved scoping report (*Trailhead Apartments – Scoping Memo, February 2025)*.

2. PROJECT DESCRIPTION

Trailhead Apartments is a proposed residential development comprised of 373 total apartment units located within the City of Issaquah. The north building will consist of 158 affordable units and the southern building will consist of 215 market rate units. The subject site is bordered to the north via NW Maple Street and to the south via Newport Way NW situated on 4-acres within tax parcel number 2924069002. Existing on-site is a vacant 33,680 industrial building which would be demolished for new construction. Site ingress/egress is proposed via right-of-way dedication of 13th Street bordering the east side of the parcel. The newly constructed 13th Street will extend north to Maple Street and south to Newport Way. **Figure 1** provides a vicinity map of the surrounding street system. **Figure 2** on the following page provides a conceptual site plan.

Figure 1: Vicinity Map





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TRAILHEAD APARTMENTS

SITE PLAN
FIGURE 2

3. EXISTING CONDITIONS

3.1 Existing Street System

The major roadways in the study area are listed and described below.

Table 1: Roadway Network

Functional Classification	Roadway	Speed Limit (mph)	Lanes	Sidewalk	Bike Facilities
State Route	SR 900	40	2-5	Yes	No
Other Principal Arterial	Maple St	30	2-5	Yes	No
	Newport Way	25-30	2	Yes	No
Minor Arterial	12th Ave	25	2-3	Yes	Some
Local	Issaquah TC Rd	25*	2	Yes	No

*No observed posted speed limit, 25 mph is assumed.

SR 900

- Urban Other Principal Arterial
- Limited Access Partially Controlled
- 2023 ADT is 21,000 vehicles just north of Newport Way

3.2 Roadway Improvements

The City of Issaquah Six Year Transportation Improvement Program (2025-2030) was reviewed for potential improvements in the study area. See **Table 2** below for details.

Table 2: Transportation Improvement Projects

Name	Location	Improvement	Cost
Newport Improvements (ID# TR 022)	SR 900 to SE 54th St	Install intersection control and access management along Newport Way. This includes 1 lane in each direction with non-motorist facilities and a landscaped central median.	$14,800,000
Newport Way Improvements (ID# TR 023)	Maple to Sunset	This project will add a second southbound lane from Maple St to ~Holly St, construct roundabouts at Juniper St, Holly St, and Dogwood St, construct signal modifications at Sunset Way and Maple St, and construct bicycle and pedestrian access.	$15,400,000
Newport Way Bike and Ped Improvements (ID# TR 043)	SR 900 to 12th Ave	Constructs protected bike lanes for bicyclists and sidewalk for pedestrians along Newport Way between SR 900 and 12th Avenue NW.	$8,610,000

Multiple improvements are planned along Newport Way in the area enhancing the corridor with access management, intersection control, roundabouts, signal modifications, and improved pedestrian and bicycle facilities, including protected bike lanes and sidewalks

3.3 Transit Service

The subject site is located next to the Issaquah Transit Center (TC), with multiple transit routes accessible within walking distance of the proposed development. **Table 3** below provides service descriptions for each route.

Table 3: Bus Routes

Route	Description	Weekday Service	Weekend Service	Nearest Stop
		Sound Transit Routes		
554	Issaquah-Seattle	4:19 AM to 11:44 PM	6:25 AM to 11:03 PM	Issaquah TC
556	Issaquah-University District	5:12 AM to 7:28 PM[1]	N/A	Issaquah TC
		King County Routes		
208	North Bend-Snoqualmie-Issaquah	5:04 AM to 9:21 PM	6:59 AM to 10:20 PM	Issaquah TC
269	Issaquah-Overlake	6:02 AM to 7:58 PM	N/A	Issaquah TC
271	Issaquah-Bellevue-University District	5:38 AM to 11:35 PM	6:32 AM to 11:30	Issaquah TC

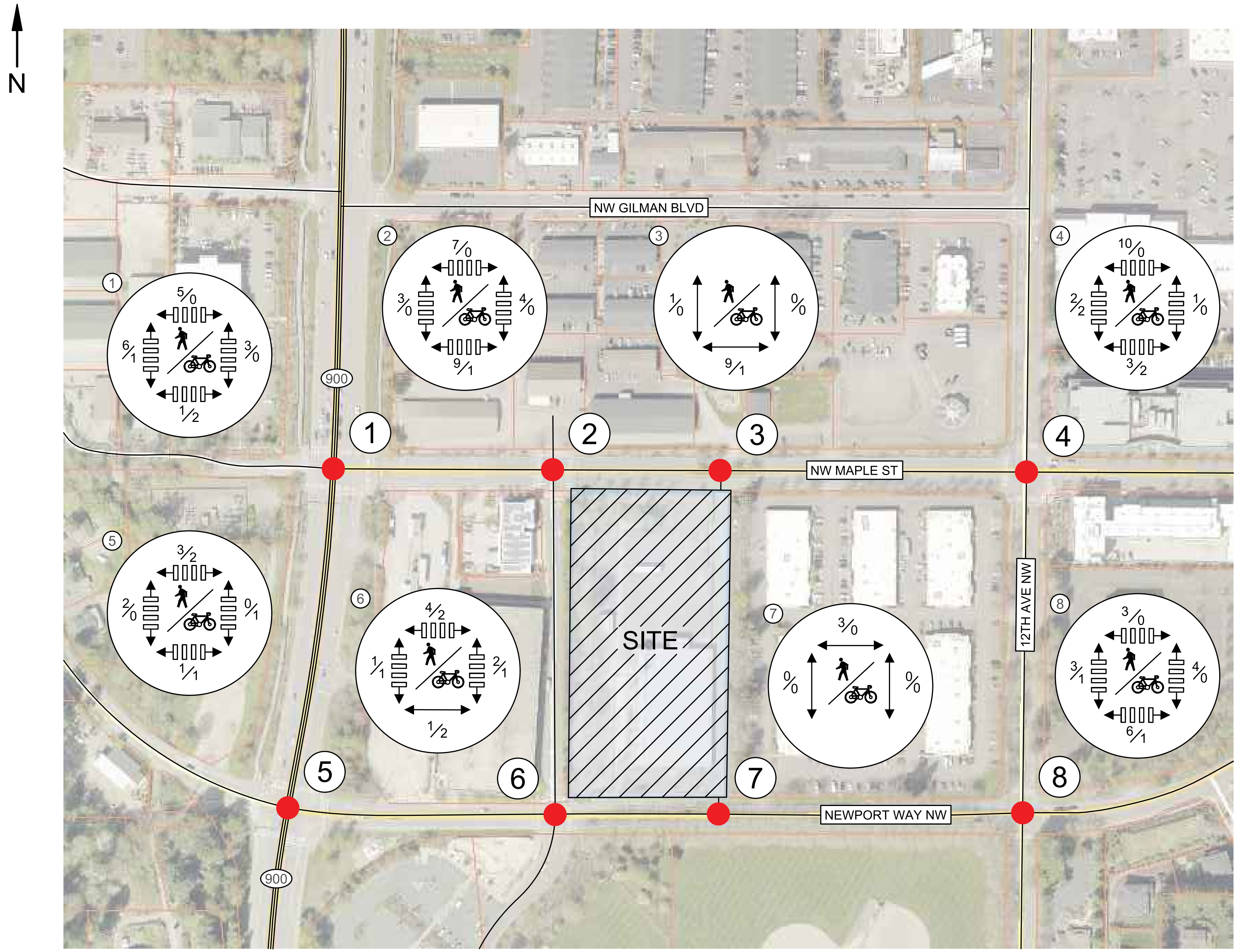
3.4 Existing Peak Hour Volumes and Travel Patterns

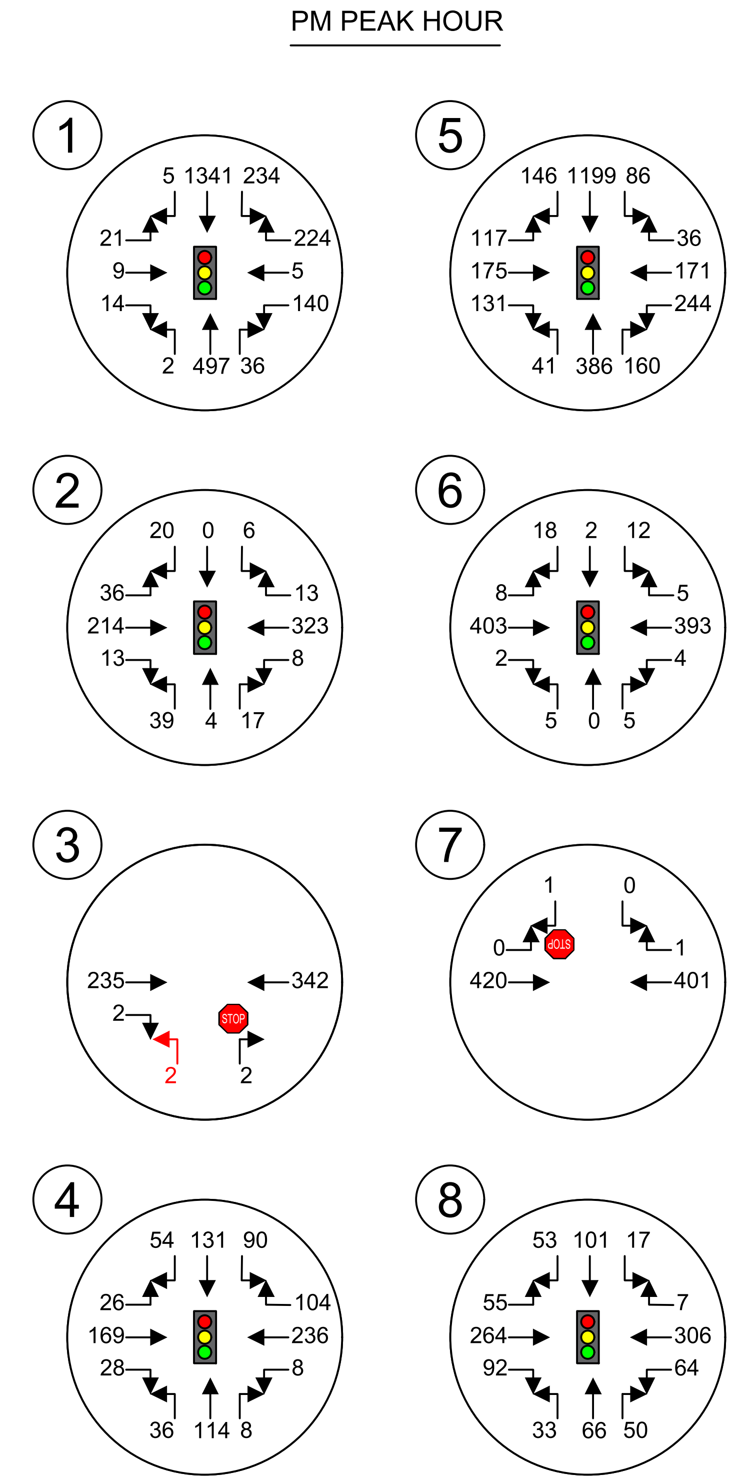
Traffic counts were collected in February of 2025 at the following eight intersections that were established through the scoping process:

1. SR 900 & NW Maple Street
2. NW Maple Street & Issaquah TC Road
3. NW Maple Street & 13th Street
4. NW Maple Street & 12th Avenue NW
5. SR 900 & Newport Way NW
6. Newport Way NW & Issaquah TC Road
7. Newport Way NW & 13th Street
8. Newport Way NW & 12th Avenue NW

The PM peak hour was targeted for capacity evaluation. Counts were therefore collected between 4:00 PM - 6:00 PM. The one-hour which reflects the highest volumes from each field count, known as the peak hour, is then used for analysis to identify operations at peak congestion. Existing vehicular PM peak hour volumes are illustrated in **Figure 3**. Count sheets are attached in the appendix.

[1] Bus route stops running from 10:12 AM to 3:00 PM.





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TRAILHEAD APARTMENTS

EXISTING PM PEAK HOUR VOLUMES
FIGURE 3

3.5 Non-Motorist Activity and Infrastructure

Non-motorist activity was recorded during the PM peak hour at each study intersection as shown in **Figure 3** on the previous page. There is generally sidewalk on at least one side of roadways within the study area with crossing opportunities at the signals. School-aged children residing in the future Trailhead Apartments development would likely attend either Issaquah Valley Elementary School or Issaquah Middle School—both of which are around one-mile or less walking distance from the site.

Figure 4 below illustrates the existing sidewalk/walking routes in the study area. A contiguous sidewalk is available to the middle school from the subject site. However, there are a few missing sidewalk segments to the elementary school along Newport Way. Note that the City's TR 023 Improvement Project (refer to Table 2) intends to upgrade non-motorist infrastructure along Newport Way from Maple to Sunset, which would fill these gaps and establish a continuous walking route from the subject site. Complete sidewalks/waking paths are available to/from Tibbetts Park, the Issaquah Transit Center, Cougar Mountain Middle School and commercial development to the north/east further encouraging multi-modal transport from Trailhead Apartments.

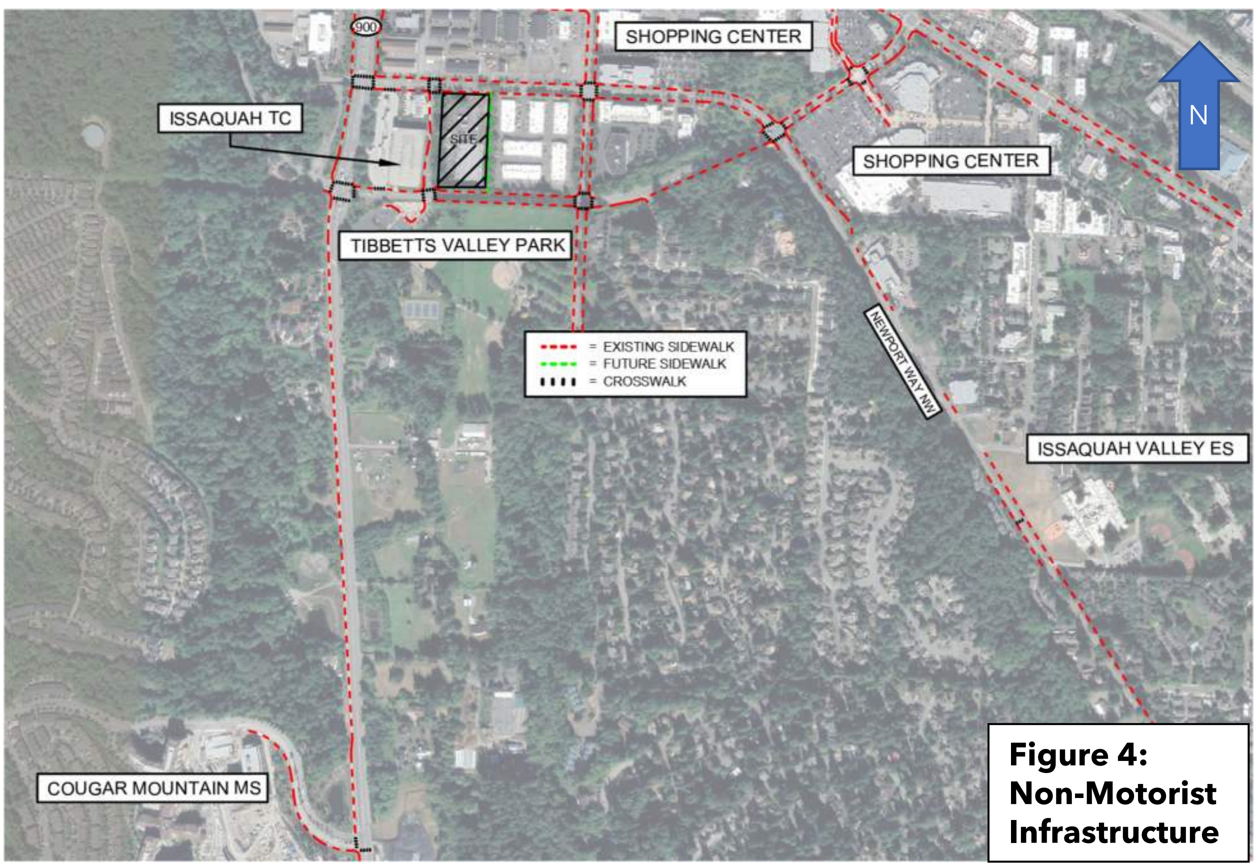

Figure 4: Non-Motorist Infrastructure

3.6 Existing Level of Service

Level of Service (LOS) rates[2] the quality of traffic flow and user experience, typically on a scale from A to F, where:

- **LOS A** represents free-flowing traffic with minimal delays and low congestion.
- **LOS B** indicates stable traffic flow with some minor delays.
- **LOS C** shows moderate traffic flow with noticeable delays at peak times.
- **LOS D** is high-density traffic flow with more frequent and longer delays.
- **LOS E** is near-capacity conditions with significant delays and congestion.
- **LOS F** denotes over-capacity conditions, where traffic flow breaks down, resulting in severe congestion and delays.

LOS calculations were performed using Synchro 12. Signalized intersections report the overall LOS, while stop-controlled intersections report the worst approach LOS. **Table 4** below summarizes the PM peak hour LOS results. Signal timing was received from the City of Issaquah and WSDOT.

Table 4: Existing Weekday PM Peak Hour Level of Service

Delays given in seconds per vehicle

Ref. #	Intersection	Control	LOS	Delay
1	SR 900 & NW Maple Street	Signal	C	28.9
2	NW Maple Street & Issaquah TC Road	Signal	A	9.7
3	NW Maple Street & 13th Street	Stop	A	9.2
4	NW Maple Street & 12th Avenue NW	Signal	B	14.1
5	SR 900 & Newport Way NW	Signal	E	61.9
6	Newport Way NW & Issaquah TC Road	Signal	A	8.5
7	Newport Way NW & 13th Street	Stop	B	10.8
8	Newport Way NW & 12th Avenue NW	Signal	B	11.6

WSDOT Level of Service Standards[3]: LOS E Mitigated or better (SR 900).

---

2 *Signalized Intersections - Level of Service*

Level of Service	Control Delay per Vehicle (sec)
A	≤10
B	>10 and ≤20
C	>20 and ≤35
D	>35 and ≤55
E	>55 and ≤80
F	>80

Stop Controlled Intersections – Level of Service

Level of Service	Control Delay per Vehicle (sec)
A	≤10
B	>10 and ≤15
C	>15 and ≤25
D	>25 and ≤35
E	>35 and ≤50
F	>50

Highway Capacity Manual (HCM), 7th Edition

3 WSDOT Level of Service Standards - ArcGIS

City of Issaquah Level of Service Standards[4]: LOS D or better.

With the exception of SR 900 & Newport Way NW operating at LOS E, all other intersections operate with LOS C or better conditions. All intersections currently meet LOS standards.

3.7 Collision History Analysis

Collision History Analysis
A list of the recorded incident history for the five most recent full years (beginning of 2019 through end of 2023) for each study intersection was requested from WSDOT. **Table 5** below outlines yearly incidents.

Table 5: Collision History Overview

Intersection/Roadway Segment	2019	2020	2021	2022	2023	Avg/ Yr
1. SR 900 & NW Maple Street	2	4	3	2	2	2.6
2. NW Maple Street & Issaquah TC Road	0	0	0	0	0	0.0
3. NW Maple Street & 13th Street	0	0	0	0	0	0.0
4. NW Maple Street & 12th Avenue NW	2	2	1	0	1	1.2
5. SR 900 & Newport Way NW	7	3	6	2	3	4.2
6. Newport Way NW & Issaquah TC Road	0	0	0	0	0	0.0
7. Newport Way NW & 13th Street	0	0	0	0	0	0.0
8. Newport Way NW & 12th Avenue NW	1	1	1	1	1	1.0

A total of 45 collisions were recorded in the study area. The following sections will summarize the collision types, injury severity, and contributing factors.

Collision Type Analysis
Summaries of collision types that occurred at and/or related to each study intersection are provided in **Table 6** below.

Table 6: Collision History Crash Types

Crash Type	Number of Crashes (2019-2023)			
	I/S #1	#4	#5	#8
Rear-end	1	1	7	2
Entering at angle	2	4	10	0
Vehicle turning left hits ped	0	0	0	**1**
From opposite direction	10	1	2	2
From same direction	0	0	2	0

[4] Issaquah Comprehensive Plan – Issaquah 2044.

1. SR 900 & NW Maple Street: 13 collisions were recorded over the 5-year period, resulting in an average of 2.6 incidents per year. The collision types were listed as "rear-end" (1/13), "entering at angle" (2/13), and "from opposite direction" (10/13).

4. NW Maple Street & 12th Avenue NW: 6 collisions were recorded over the 5-year period, resulting in an average of 1.2 incidents per year. The collision types were listed as "rear-end" (1/6), "entering at angle" (4/6), and "from opposite direction" (1/6).

5. SR 900 & Newport Way NW: 21 collisions were recorded over the 5-year period resulting in 4.2 accidents per year. The collision types were listed as "rear-end" (7/21), "entering at angle" (10/21), "from opposite direction" (2/21), and "from same direction" (2/21).

8. Newport Way NW & 12th Avenue NW: 5 collisions were recorded over the 5-year period, resulting in an average of 1.0 incidents per year. The collision types were listed as "rear-end" (2/5), "from opposite direction" (2/5), and "vehicle turning left hits pedestrian" (1/5).

Collision Severity, Contributing Factor Analysis & Trends

A collision severity summary associated with each study intersection is provided below in **Table 7**. No fatalities were recorded at any of the study intersections.

Table 7: Collision History Severity

Number of Crashes (2019-2023)				
Crash Type	**#1**	**#4**	**#5**	**#8**
Fatal (K)	0	0	0	0
Incapacitating Injury (A)	**1**	0	0	0
Non-incapacitating Injury (B)	3	0	3	0
Possible Injury (C)	1	1	35	2
Property Damage Only (PDO)/Unknown	8	5	14	3

In review of overall trends, collisions were primarily property damage only and largely contributed to driver error/inattention. However, a single non-motorist collision occurred. An incapacitating injury was also recorded. Each pedestrian (1) and serious injury collision (1) are described in detail on the following page.

Collisions Involving Serious Injury

- A serious injury collision occurred at the intersection of SR 900 & Maple Street in September of 2020 at around 5:45 PM. A southbound vehicle collided with a stopped vehicle (rear-end). The southbound driver that collided with the stopped vehicle was under the influence of drugs and also disregarded traffic signs and signals. The weather is listed as clear with dry roadway conditions, the collision resulted in suspected serious injury.

Collisions Involving Non-Motorist

- The non-motorist collision occurred at the intersection of Newport Way & 12th Avenue in November of 2021 at around 5:25 PM when a left turning vehicle struck a pedestrian. Weather conditions are listed as raining with a wet roadway surface. There is no contributing driver/pedestrian circumstance.

4. FORECAST TRAFFIC DEMAND & ANALYSIS

4.1 Project Trip Generation

Trip generation is defined as the number of vehicle movements that enter or exit the prospective project site during a designated time period such as the PM peak hour or an entire day. Trip estimates were derived from the Institute of Transportation Engineers (ITE) publication, *Trip Generation,* 11th Edition. Both the former and proposed uses are described below. ITE's rates or equations were used in accordance with the Trip Generation Handbook.

Previous: The former use on-site was a dispatch/maintenance center for Lumen service trucks. The building size is 33,680 square feet and is to be demolished for new construction. In review of ITE, Land Use Code (LUC) 110 – General Light Industrial was assigned as a comparable use to estimate previous trip generation from the site.

Proposed: ITE has limited trip generation data for affordable housing. Therefore, all units were assigned under LUC 221 – Multifamily Housing (Mid-Rise) which is for buildings between 4-10 floors. Moreover, the setting/location of "Dense Multi-Use Urban" was selected given the proximity to the Issaquah Transit Center, Tibbetts Valley Park, and other commercial amenities to the north.

Refer to **Table 8** below for the estimated project trip generation.

Table 8: Project Trip Generation

Land Use	Variable	AWDT	AM Peak-Hour Trips			PM Peak-Hour Trips		
			In	Out	Total	In	Out	Total
Former Use: Light Industrial[5]	33.68 ksf	-177	-24	-3	-27	-3	-15	-18
Proposed Use: Multi-Family[6]	373 units	1,093	14	85	99	75	27	102
Net New Trips		**916**	-10	82	**72**	72	12	**84**

Based on ITE data, the project is estimated to generate 916 net new average weekday daily trips with 72 net new AM peak hour trips (-10 inbound / 82 outbound) and 84 net new PM peak hour trips (72 inbound / 12 outbound). However, for analysis purposes, the total project trips are applied.

5 ITE Equations
6 ITE Rates

4.2 Distribution and Assignment

Trip distribution describes the process by which project generated trips are dispersed on the roadway network surrounding the site. No trip reductions from the previous user were considered in the trip distribution.

Trip distribution assignments are illustrated in **Figure 5** which are based on a review of the adjacent street network, proximity to major routes (SR 900, I-90, etc.), and engineering judgement. All site-generated traffic was assigned from the improved 13th Street. Currently, there is a raised vegetated median along NW Maple Street at the 13th Street intersection. However, per City direction, the trip distribution assumes this intersection would allow lefts in and out. The trip distribution therefore reflects this modification.

4.3 Future Peak Hour Volumes

A 3-year horizon (2028) was used for future traffic delay analysis, assuming full buildout and occupancy of the Trailhead Apartments development. Forecast background traffic volumes were determined by applying a 2.0%[7] compound annual growth rate to existing PM peak hour volumes shown in Figure 3.

Forecast 2028 PM peak hour background volumes are illustrated in **Figure 6**. **Figure 7** illustrates forecast 2028 PM peak hour volumes with the project.

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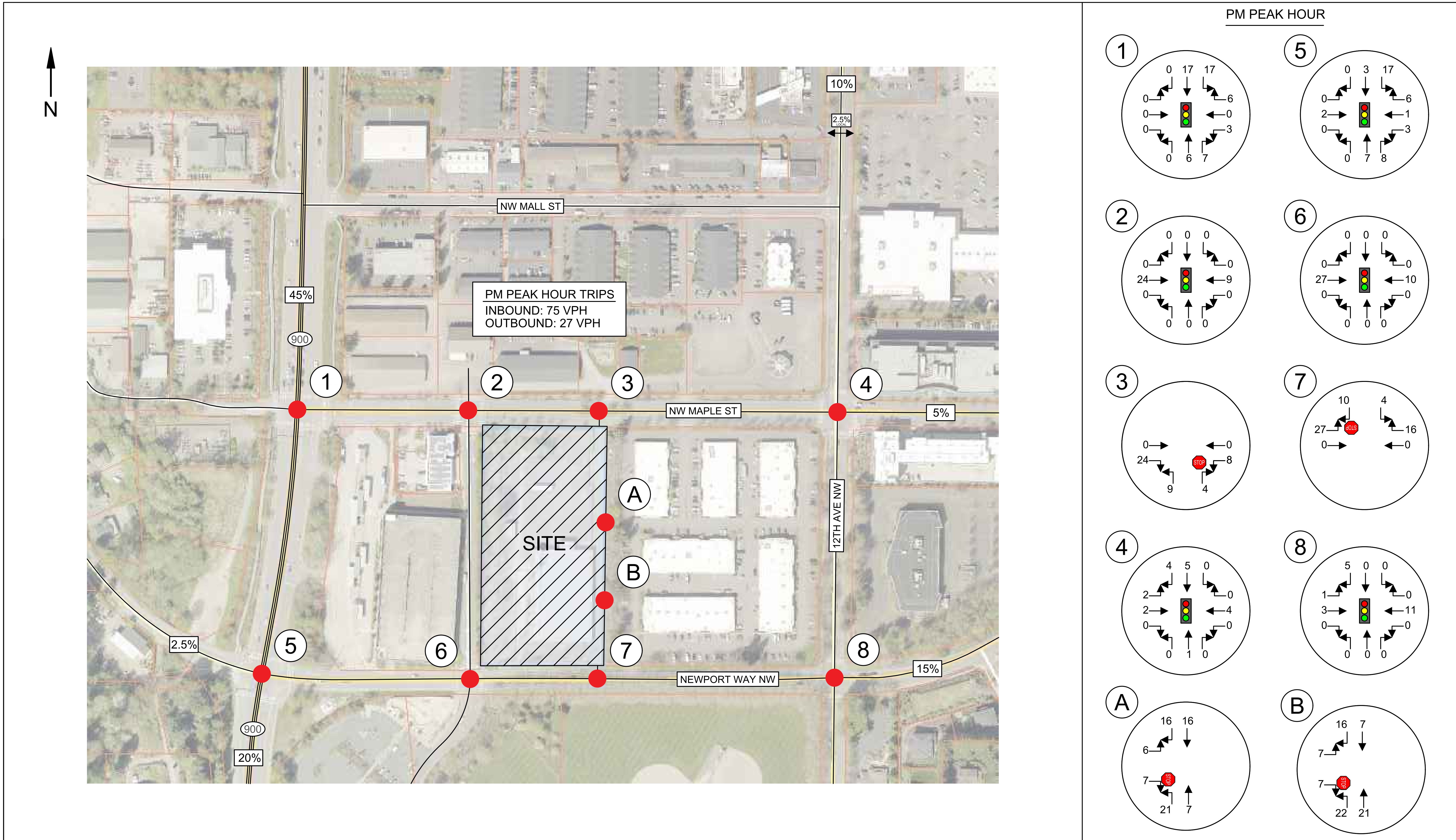
[7] Per City scoping comments.



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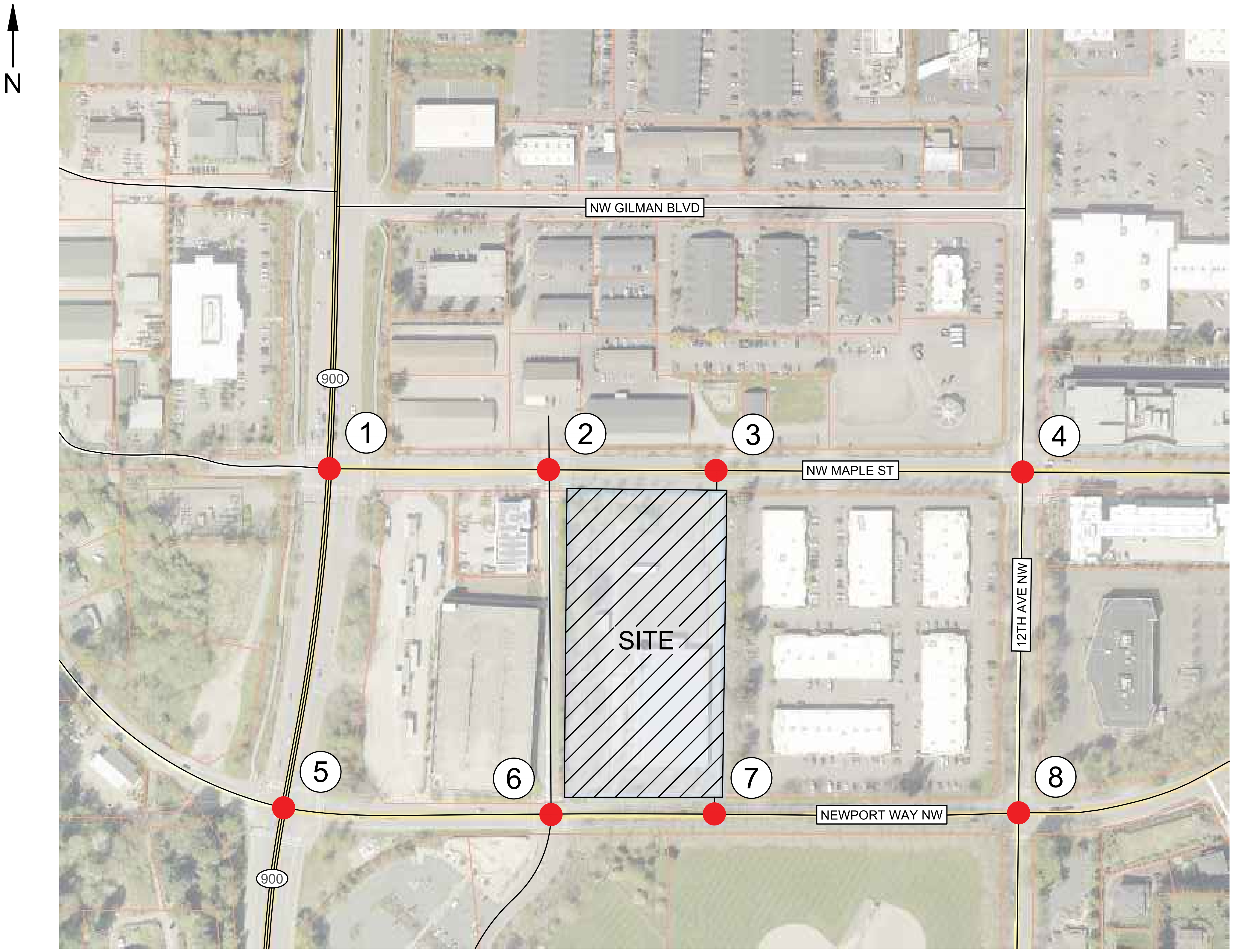
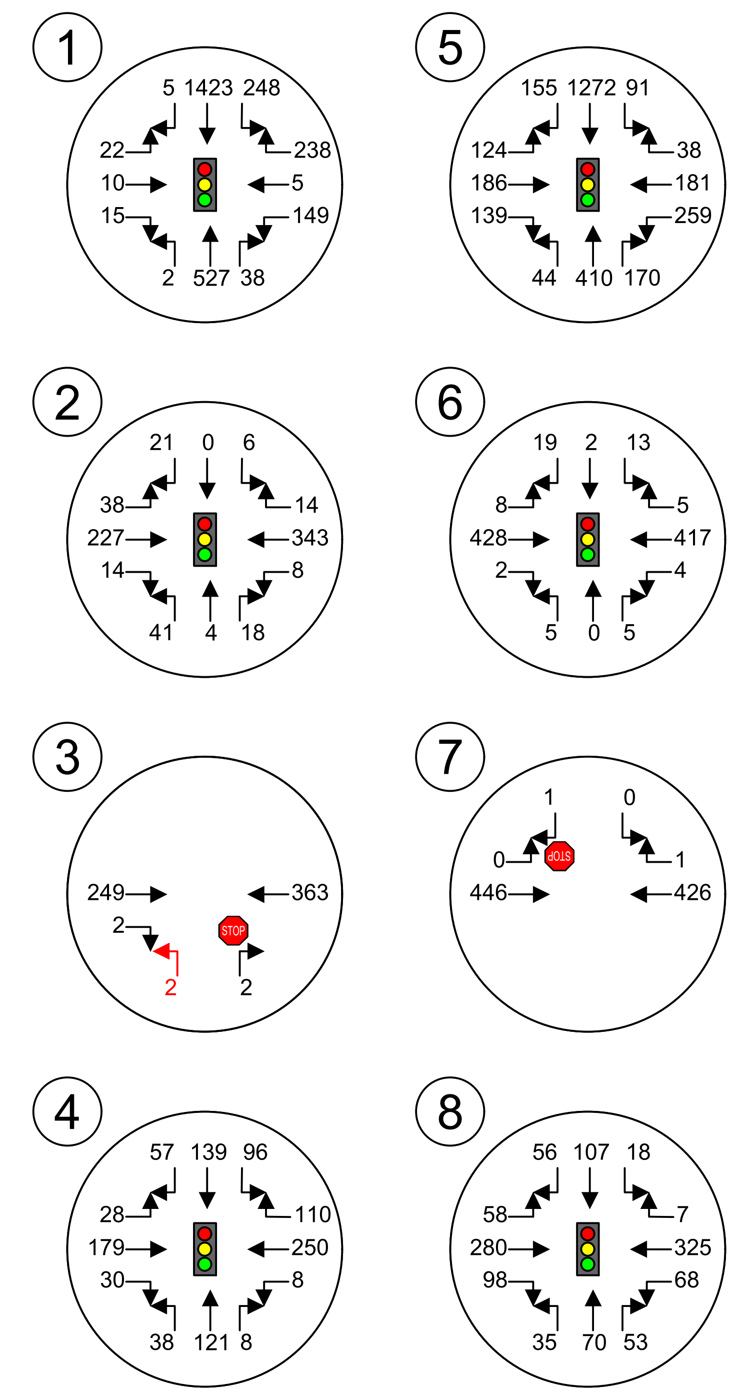
TRAILHEAD APARTMENTS
PM PEAK HOUR TRIP DISTRIBUTION & ASSIGNMENT
FIGURE 5



PM PEAK HOUR



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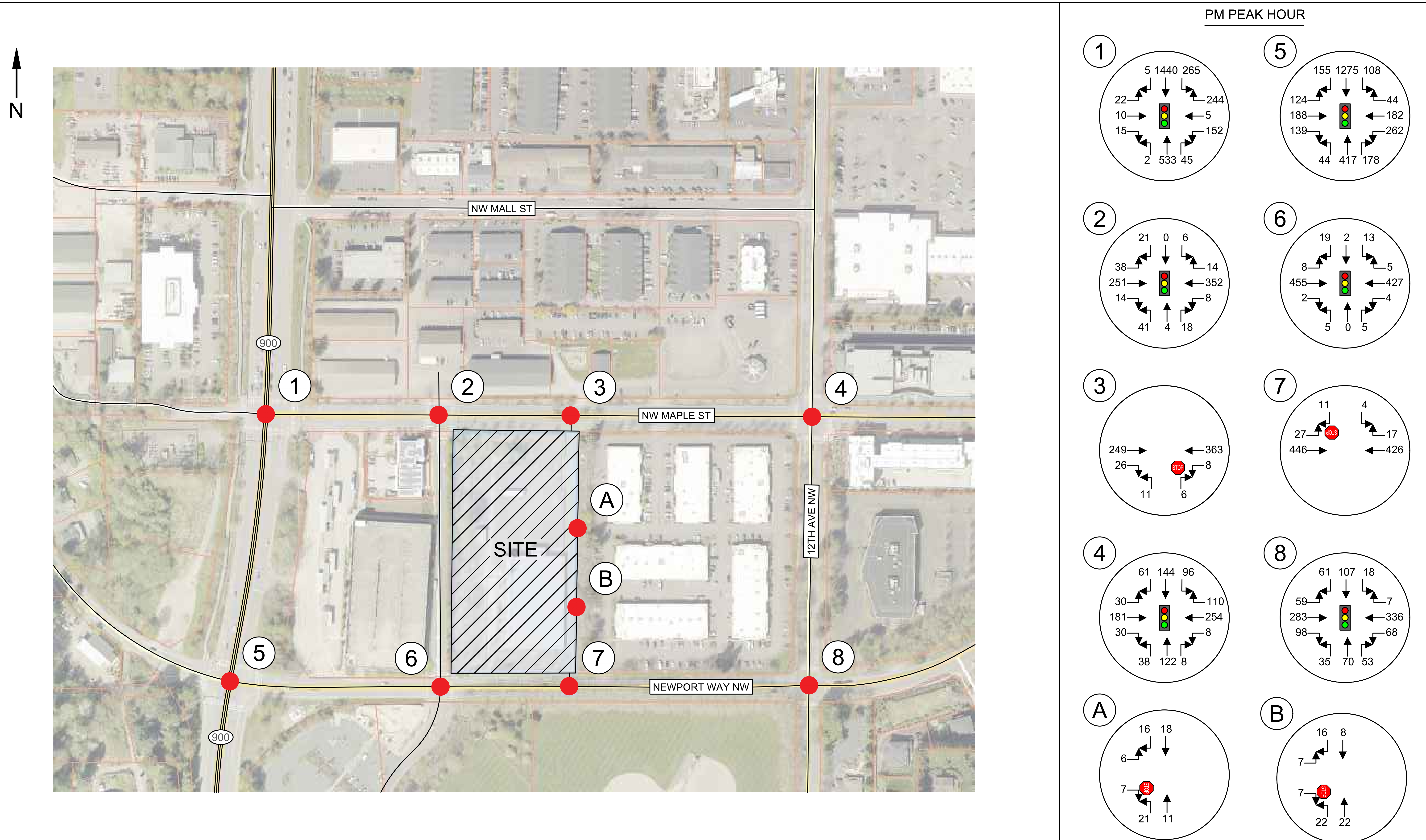
TRAILHEAD APARTMENTS

FORECAST 2028 PM PEAK HOUR VOLUMES WITHOUT PROJECT
FIGURE 6



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TRAILHEAD APARTMENTS

FORECAST 2028 PM PEAK HOUR VOLUMES WITH PROJECT

FIGURE 7

4.4 Future Level of Service & Queuing

Level of Service

A level of service analysis was conducted for future PM peak hour volumes, both without (background) and with project-generated trips. Consistent with WSDOT protocol, all WSDOT intersections include a future peak hour factor of 1.0. Level of service outputs are provided in **Table 9** below.

Table 9: Forecast 2028 Weekday PM Peak Hour Level of Service

Delays given in seconds per vehicle

		Without Project		With Project	
Intersection	**Control**	**LOS**	**Delay**	**LOS**	**Delay**
1. SR 900 & NW Maple Street	Signal	C	28.5	C	29.0
2. NW Maple Street & Issaquah TC Rd	Signal	A	9.8	A	9.9
3. NW Maple Street & 13th Street	Stop	A	9.3	B	10.9
4. NW Maple Street & 12th Ave NW	Signal	B	14.3	B	14.5
5. SR 900 & Newport Way NW	Signal	E	62.3	E	62.7
6. Newport Way NW & Issaquah TC Rd	Signal	A	8.8	A	8.9
7. Newport Way NW & 13th Street	Stop	B	11.0	B	13.6
8. Newport Way NW & 12th Ave NW	Signal	B	11.9	B	11.9
A. 13th Street & North Access	Stop	--	--	A	8.7
B. 13th Street & South Access	Stop	--	--	A	8.8

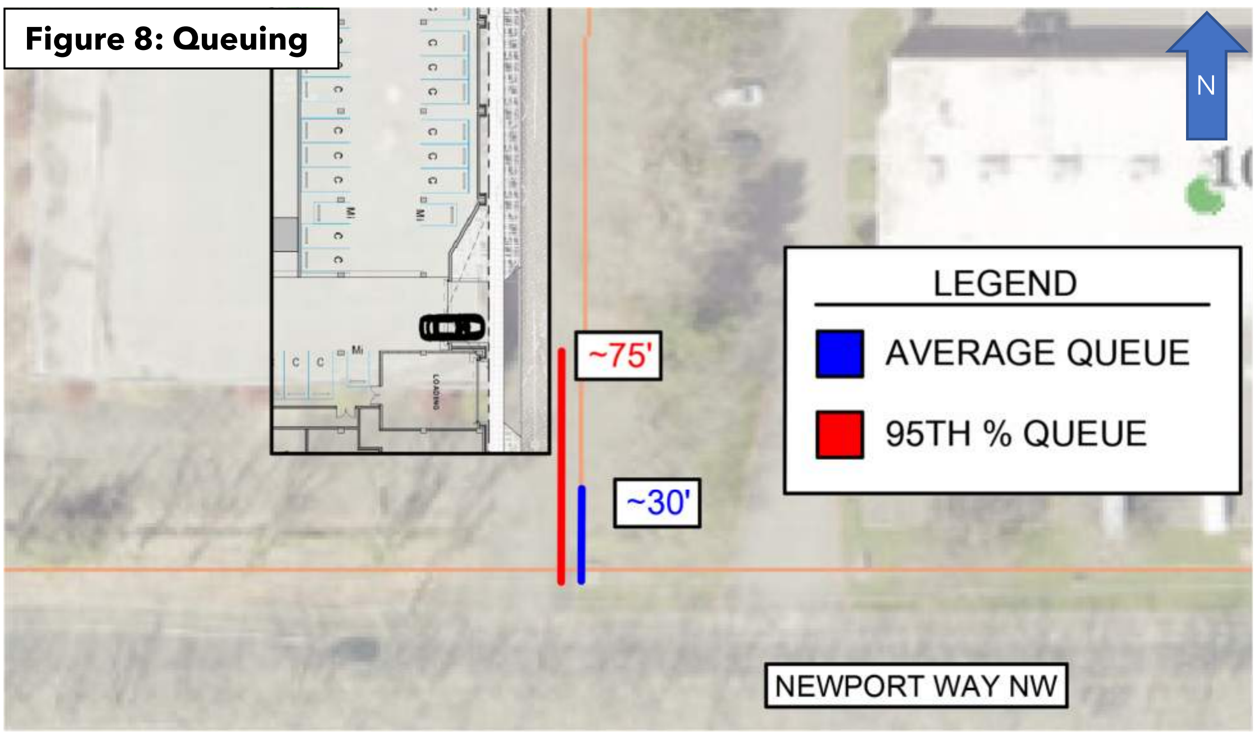
As illustrated in **Table 9**, all study intersections are projected to continue to meet level of service standards. The intersection of SR 900 & Newport Way is estimated to continue operating with LOS E conditions with or without the project traffic, meeting WSDOT standards (LOS E Mitigated).

<u>Queuing</u>
Per City scoping comments, a queuing analysis was conducted for the intersection of Newport Way & 13th Street to evaluate whether the site's south driveway would be blocked during the PM peak hour. Queues were estimated using SimTraffic by running five peak hour simulations. The southern edge of the site driveway is approximately 85 feet from Newport Way. Based on the forecasted 2028 queuing results:

- Average southbound queue: 30 feet
- 95th percentile queue: 75 feet

The analysis indicates that the southern access point would remain unblocked during the PM peak hour. In the rare event that the queue exceeds three vehicles and momentarily blocks the driveway, the queue would quickly dissipate, allowing site ingress. Refer to **Figure 8** below.

Figure 8: Queuing



4.5 Left Turn Lane Guidelines

Left turn lanes provide necessary storage for vehicles turning left at intersections. To determine storage requirements at the proposed Newport Way & 13th Street intersection, procedures from WSDOT Design Manual Exhibit 1310-9 were applied. Based on forecasted 2028 PM peak hour volumes with project traffic, a left turn lane does not meet the minimum warrant thresholds. Refer to the appendix for the left turn warrant nomograph.

Based on the low number of through volumes anticipated along 13th Street, both the north and south access points would also not meet WSDOT's left turn threshold.

4.6 Access Sight Distance & Spacing

Sight Distance

Site ingress/egress is proposed via two new access points which are both to extend west via 13th Street. Sight lines would need to meet the minimum distances outlined in the City's Roadway Standards[8]. However, given the flat grades and no horizontal curvature, no sight distance deficiencies are identified. Both vehicle and pedestrian sight lines should be verified with the final civil plans.

Spacing

The City's design standards states that local accesses should be offset from center-to-center by 200-feet. **Figure 9** shows the spacing between the proposed driveways and from NW Maple Street and Newport Way NW. The southern driveway would be around 105-feet north from Newport Way NE. However, per the queuing analysis, this driveway is anticipated to remain unblocked during the peak hour. Moreover, the volumes along 13th Street are projected to be low with little to no other curb cuts along the street thereby reducing the potential conflicts.

[8] City of Issaquah Street Standards, June 8, 2023.

TRAILHEAD APARTMENTS

DRIVEWAY SPACING
FIGURE 9

HEATH&ASSOCIATES
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5. CONCLUSIONS & MITIGATION

Trailhead Apartments is a proposed 373-unit multifamily development located in the City of Issaquah, situated on a 4-acre parcel south of Maple Street and north of Newport Way. The site currently contains a vacant industrial building, which will be demolished for new construction. Access will be provided via two access points extending west from an improved 13th Street.

A total of eight intersections were analyzed for level of service (LOS). All currently operate at LOS E or better, meeting agency standards, which vary by location. Collision history, summarized in Tables 5-7, shows the highest incident rates at SR 900 & Newport Way (4.2 incidents per year) and SR 900 & Maple Street (2.6 incidents per year), though no fatal crashes were reported at these locations within the past five years.

Based on ITE data, the Trailhead Apartments development is projected to generate 916 net new average weekday daily trips, with 72 net new trips during the AM peak hour and 84 net new trips during the PM peak hour. The existing infrastructure, along with planned improvements in the City's Six-Year Transportation Improvement Plan, will provide continuous walking routes to the local elementary and middle schools.

A three-year buildout horizon (2028) was used for the forecast PM peak hour analysis. By 2028, all study intersections are expected to continue meeting LOS standards. Based on WSDOT left turn guidelines, a left turn lane was found not warranted at the intersection of 13th Street & Newport. While the site's southern access does not meet the 200-foot spacing requirement from Newport Way NW, southbound queuing on 13th Street is not expected to extend beyond the site driveway.

The following mitigation measures are identified for Trailhead Apartments:

1. Frontage improvement requirements shall be coordinated with the City of Issaquah including potential removal or modification of the vegetated median along NW Maple Street to allow left-turn movements in and out of the 13th Street intersection.

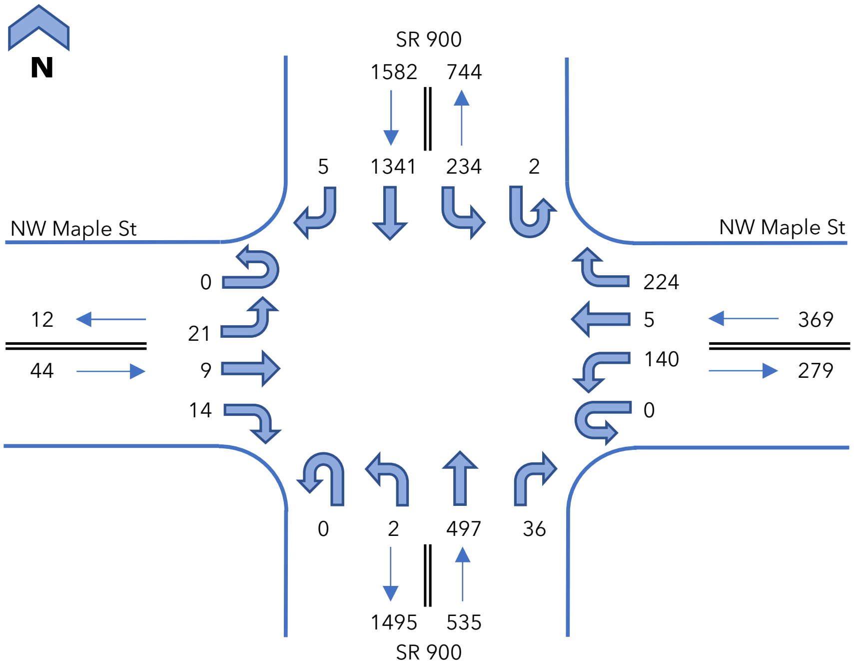
2. Per Issaquah Municipal Code Chapter 3.71, Traffic Impact Fees (TIF) are required for new development. The City will determine the applicable fees after reviewing the Traffic Impact Analysis. Note than Section 3.71.040 provides exemptions for affordable housing, with eligibility based on the project's compliance with the City's affordable housing criteria. The applicability of this exemption should be further coordinated between the Applicant and the City.

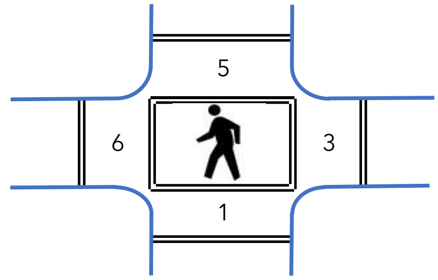
TRAILHEAD APARTMENTS
TRAFFIC IMPACT ANALYSIS

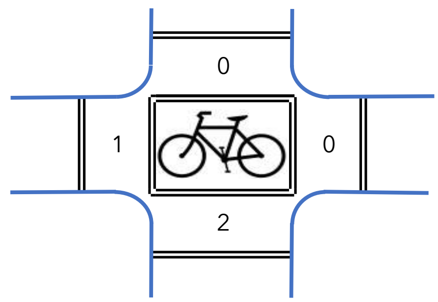
APPENDIX

Date Collected:	2/25/2025
Day of the Week:	Tuesday
Time Period:	4:00 PM to 6:00 PM
Peak Hour:	4:00 PM to 5:00 PM

NW Maple St & SR 900







Interval Start Time	NW Maple St				NW Maple St				SR 900				SR 900				15 Minute Totals	Hourly Totals
	Eastbound				Westbound				Northbound				Southbound					
	UT	LT	T	RT	UT	LT	T	RT	UT	LT	T	RT	UT	LT	T	RT		
4:00 PM	0	5	4	1	0	40	0	53	0	2	117	16	0	72	385	2	697	
4:15 PM	0	6	0	3	0	36	3	51	0	0	117	5	0	57	295	1	574	
4:30 PM	0	8	4	7	0	33	2	64	0	0	122	8	2	55	356	0	661	
4:45 PM	0	2	1	3	0	31	0	56	0	0	141	7	3	50	305	2	601	2533
5:00 PM	0	8	3	5	0	38	0	54	0	0	130	7	4	56	340	1	646	2482
5:15 PM	0	7	2	4	0	33	0	56	0	1	97	7	1	40	319	1	568	2476
5:30 PM	0	5	3	4	0	21	1	38	0	0	134	12	2	50	265	0	535	2350
5:45 PM	0	1	2	5	0	27	0	39	0	0	117	2	1	30	282	0	506	2255
Count Total	0	42	19	32	0	259	6	411	0	3	975	64	13	410	2547	7	4788	--
Peak Hour Total	0	21	9	14	0	140	5	224	0	2	497	36	5	234	1341	5	2533	--
PHF	0.58				0.93				0.90				0.86				0.91	--
Heavy Vehicles	0	0	0	0	0	4	0	2	0	1	23	0	0	4	35	0	69	--
HV %	0.0%	0.0%	0.0%	0.0%	0.0%	2.9%	0.0%	0.9%	0.0%	50.0%	4.6%	0.0%	0.0%	1.7%	2.6%	0.0%	2.7%	--

Interval Start Time	Heavy Vehicles				
	EB	WB	NB	SB	Total
4:00 PM	0	4	6	18	28
4:15 PM	0	1	3	8	12
4:30 PM	0	1	9	4	14
4:45 PM	0	0	6	9	15
5:00 PM	0	1	4	8	13
5:15 PM	0	0	4	3	7
5:30 PM	0	1	3	8	12
5:45 PM	0	0	2	10	12
Count Total	0	8	37	68	113
Peak Hour Total	0	6	24	39	69
Peak Hour HV%	0.0%	1.6%	4.5%	2.5%	2.7%

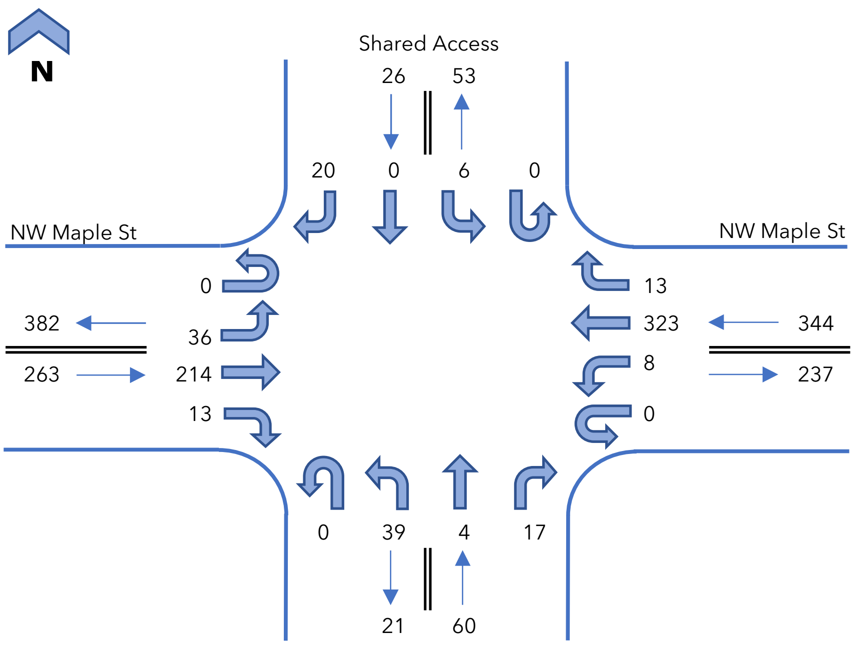
Pedestrians (Leg)				
E	W	N	S	Total
0	1	0	1	2
1	2	0	0	3
1	2	2	0	5
1	1	3	0	5
1	2	0	0	3
1	0	1	0	2
0	0	0	0	0
0	0	0	0	0
5	8	6	1	20
3	6	5	1	15

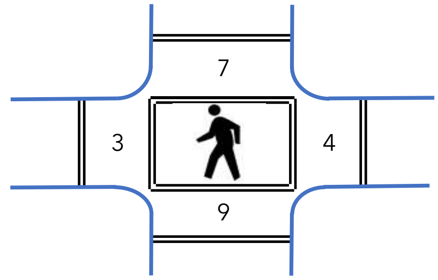
Bicycles (Leg)				
E	W	N	S	Total
0	0	0	0	0
0	0	0	0	0
0	0	0	0	0
0	1	0	2	3
0	0	0	1	1
0	0	0	0	0
0	3	2	1	6
0	0	0	0	0
0	4	2	4	10
0	1	0	2	3

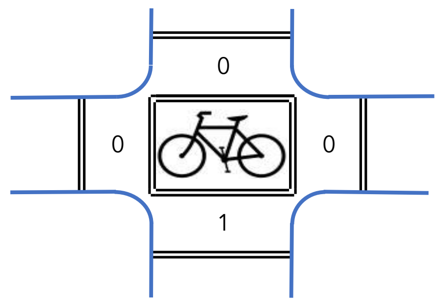
HEATH & ASSOCIATES
Transportation Planning & Engineering

Date Collected:	2/25/2025		
Day of the Week:	Tuesday		
Time Period:	4:00 PM	to	6:00 PM
Peak Hour:	4:00 PM	to	5:00 PM

NW Maple St & Shared Accesses







Interval Start Time	NW Maple St				NW Maple St				Bus/Fire Station Access				Shared Access				15 Minute Totals	Hourly Totals
	Eastbound				Westbound				Northbound				Southbound					
	UT	LT	T	RT	UT	LT	T	RT	UT	LT	T	RT	UT	LT	T	RT		
4:00 PM	0	12	64	4	0	2	92	5	0	8	2	3	0	1	0	5	198	
4:15 PM	0	11	49	0	0	1	72	4	0	7	1	2	0	1	0	9	157	
4:30 PM	0	9	52	4	0	3	89	3	0	14	1	7	0	3	0	4	189	
4:45 PM	0	4	49	5	0	2	70	1	0	10	0	5	0	1	0	2	149	693
5:00 PM	0	15	54	1	0	3	88	1	0	3	0	2	0	2	0	4	173	668
5:15 PM	0	8	40	1	0	0	72	3	0	9	0	5	0	2	2	10	152	663
5:30 PM	0	10	51	4	0	0	55	4	0	7	0	1	0	0	0	0	132	606
5:45 PM	0	2	31	4	0	0	54	3	0	4	0	4	0	4	1	9	116	573
Count Total	0	71	390	23	0	11	592	24	0	62	4	29	0	14	3	43	1266	--
Peak Hour Total	0	36	214	13	0	8	323	13	0	39	4	17	0	6	0	20	693	--
PHF	0.82				0.87				0.68				0.65				0.88	--
Heavy Vehicles	0	0	5	2	0	0	6	0	0	1	0	0	0	0	0	0	14	--
HV %	0.0%	0.0%	2.3%	15.4%	0.0%	0.0%	1.9%	0.0%	0.0%	2.6%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	2.0%	--

Interval Start Time	Heavy Vehicles				
	EB	WB	NB	SB	Total
4:00 PM	3	5	1	0	9
4:15 PM	1	0	0	0	1
4:30 PM	2	1	0	0	3
4:45 PM	1	0	0	0	1
5:00 PM	2	2	0	0	4
5:15 PM	1	0	0	0	1
5:30 PM	2	1	1	0	4
5:45 PM	4	0	0	0	4
Count Total	16	9	2	0	27
Peak Hour Total	7	6	1	0	14
Peak Hour HV%	2.7%	1.7%	1.7%	0.0%	2.0%

Pedestrians (Leg)				
E	W	N	S	Total
0	2	1	0	3
1	1	0	2	4
1	0	2	2	5
2	0	4	5	11
0	2	0	3	5
0	4	3	2	9
0	0	0	0	0
0	1	1	2	4
4	10	11	16	41
4	3	7	9	23

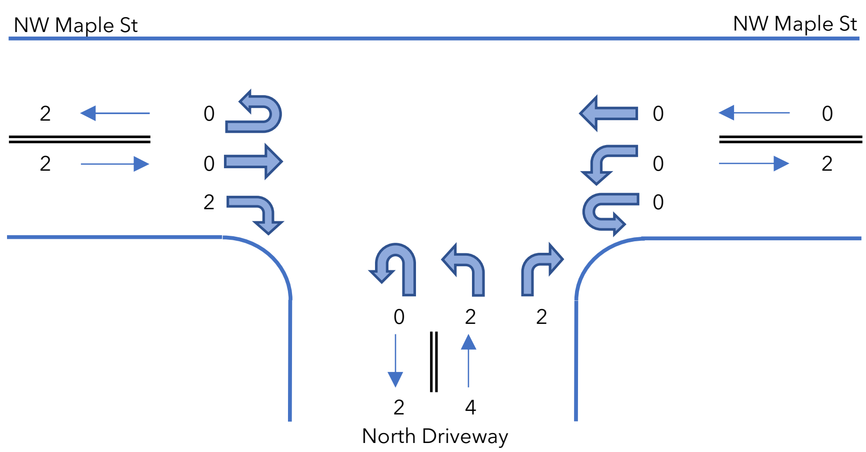
Bicycles (Leg)				
E	W	N	S	Total
0	0	0	0	0
0	0	0	0	0
0	0	0	0	0
0	0	0	1	1
0	0	0	0	0
0	0	0	0	0
0	0	2	0	2
0	0	0	0	0
0	0	2	1	3
0	0	0	1	1

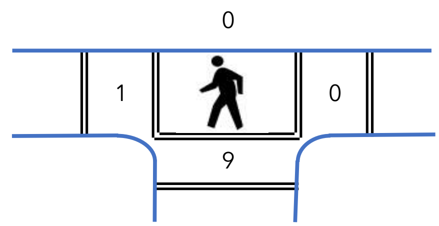
HEATH & ASSOCIATES
Transportation Planning & Engineering

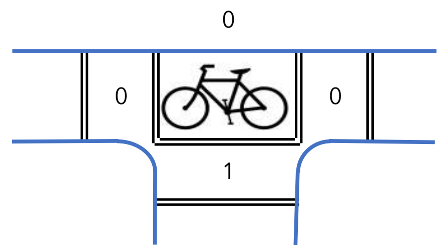
Date Collected: 2/25/2025
Day of the Week: Tuesday
Time Period: 4:00 PM to 6:00 PM
Peak Hour: 4:15 PM to 5:15 PM

NW Maple St & North Driveway

N







Interval Start Time	NW Maple St Eastbound				NW Maple St Westbound				North Driveway Northbound								15 Minute Totals	Hourly Totals
	UT	LT	T	RT	UT	LT	T	RT	UT	LT	T	RT						
4:00 PM	0	--	0	0	0	0	0	--	0	0	--	0	--	--	--	--	0	
4:15 PM	0	--	0	1	0	0	0	--	0	0	--	2	--	--	--	--	3	
4:30 PM	0	--	0	0	0	0	0	--	0	0	--	0	--	--	--	--	0	
4:45 PM	0	--	0	0	0	0	0	--	0	0	--	0	--	--	--	--	0	3
5:00 PM	0	--	0	1	0	0	0	--	0	2	--	0	--	--	--	--	3	6
5:15 PM	0	--	0	0	0	0	0	--	0	0	--	0	--	--	--	--	0	3
5:30 PM	0	--	0	0	0	0	0	--	0	0	--	0	--	--	--	--	0	3
5:45 PM	0	--	0	0	0	0	0	--	0	0	--	0	--	--	--	--	0	3
Count Total	0	--	0	2	0	0	0	--	0	2	--	2	--	--	--	--	6	--
Peak Hour Total	0	--	0	2	0	0	0	--	0	2	--	2	--	--	--	--	6	--
PHF	0.50				#DIV/0!				0.50				--				0.50	--
Heavy Vehicles	0	--	0	0	0	0	0	--	0	0	--	0	--	--	--	--	0	--
HV %	0.0%	--	0.0%	0.0%	0.0%	0.0%	0.0%	--	0.0%	0.0%	--	0.0%	--	--	--	--	0.0%	--

Interval Start Time	Heavy Vehicles				
	EB	WB	NB	SB	Total
4:00 PM	0	0	0	--	0
4:15 PM	0	0	0	--	0
4:30 PM	0	0	0	--	0
4:45 PM	0	0	0	--	0
5:00 PM	0	0	0	--	0
5:15 PM	0	0	0	--	0
5:30 PM	0	0	0	--	0
5:45 PM	0	0	0	--	0
Count Total	0	0	0	--	0
Peak Hour Total	0	0	0	--	0
Peak Hour HV%	0.0%	0.0%	0.0%	--	0.0%

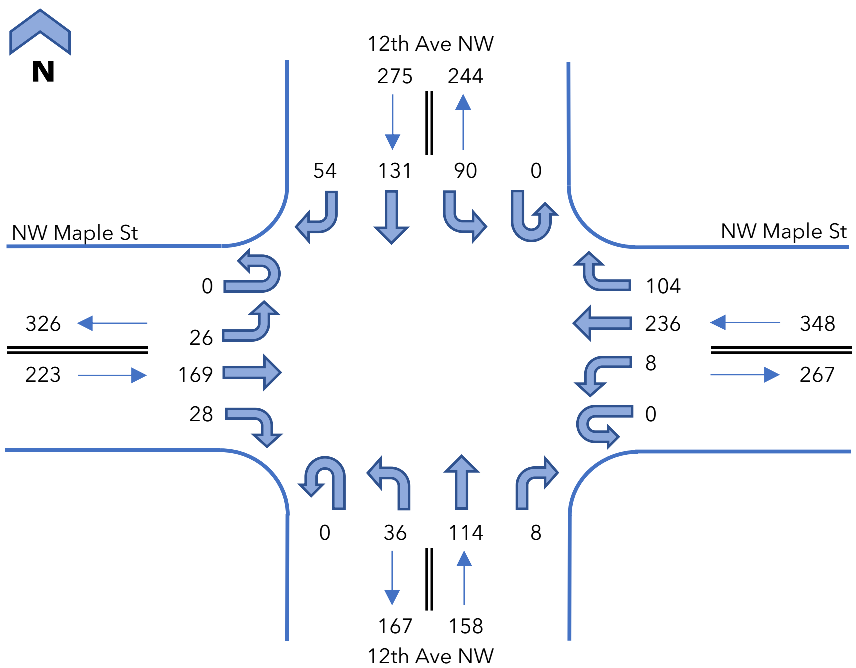
Pedestrians (Leg)				
E	W	N	S	Total
0	0	0	0	0
0	0	0	1	1
0	0	0	1	1
0	1	0	6	7
0	0	0	1	1
0	0	0	2	2
0	0	0	0	0
0	0	0	1	1
0	1	0	12	13
0	1	0	9	10

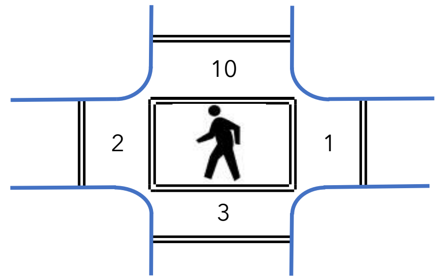
Bicycles (Leg)				
E	W	N	S	Total
0	0	0	0	0
0	0	0	0	0
0	0	0	0	0
0	0	0	1	1
0	0	0	0	0
0	0	0	0	0
0	0	0	0	0
0	0	0	0	0
0	0	0	1	1
0	0	0	1	1

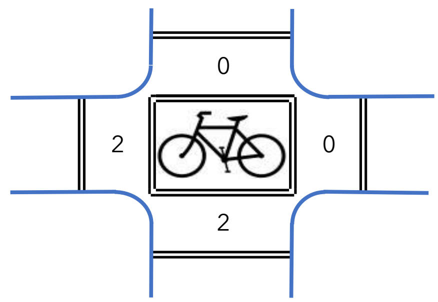
HEATH & ASSOCIATES
Transportation Planning & Engineering

Date Collected: 2/25/2025
Day of the Week: Tuesday
Time Period: 4:00 PM to 6:00 PM
Peak Hour: 4:00 PM to 5:00 PM

NW Maple St & 12th Ave NW







Interval Start Time	NW Maple St				NW Maple St				12th Ave NW				12th Ave NW				15 Minute Totals	Hourly Totals
	Eastbound				Westbound				Northbound				Southbound					
	UT	LT	T	RT	UT	LT	T	RT	UT	LT	T	RT	UT	LT	T	RT		
4:00 PM	0	8	50	10	0	2	74	27	0	8	25	2	0	29	37	6	278	
4:15 PM	0	5	43	3	0	4	48	28	0	9	30	2	0	20	46	16	254	
4:30 PM	0	8	36	7	0	1	69	20	0	7	29	3	0	24	20	17	241	
4:45 PM	0	5	40	8	0	1	45	29	0	12	30	1	0	17	28	15	231	1004
5:00 PM	0	7	35	6	0	2	53	27	0	7	19	0	0	31	33	17	237	963
5:15 PM	0	7	42	7	0	2	55	27	0	6	34	2	0	23	42	10	257	966
5:30 PM	0	7	42	3	0	4	40	19	0	3	23	3	0	26	32	13	215	940
5:45 PM	0	4	32	2	0	1	37	18	0	5	26	1	0	24	32	17	199	908
Count Total	0	51	320	46	0	17	421	195	0	57	216	14	0	194	270	111	1912	--
Peak Hour Total	0	26	169	28	0	8	236	104	0	36	114	8	0	90	131	54	1004	--
PHF	0.82				0.84				0.92				0.84				0.90	--
Heavy Vehicles	0	1	6	0	0	0	6	1	0	0	0	1	0	1	1	0	17	--
HV %	0.0%	3.8%	3.6%	0.0%	0.0%	0.0%	2.5%	1.0%	0.0%	0.0%	0.0%	12.5%	0.0%	1.1%	0.8%	0.0%	1.7%	--

Interval Start Time	Heavy Vehicles				
	EB	WB	NB	SB	Total
4:00 PM	4	5	0	1	10
4:15 PM	1	1	1	0	3
4:30 PM	1	1	0	1	3
4:45 PM	1	0	0	0	1
5:00 PM	3	2	0	0	5
5:15 PM	1	0	0	2	3
5:30 PM	2	2	0	0	4
5:45 PM	3	0	0	0	3
Count Total	16	11	1	4	32
Peak Hour Total	7	7	1	2	17
Peak Hour HV%	3.1%	2.0%	0.6%	0.7%	1.7%

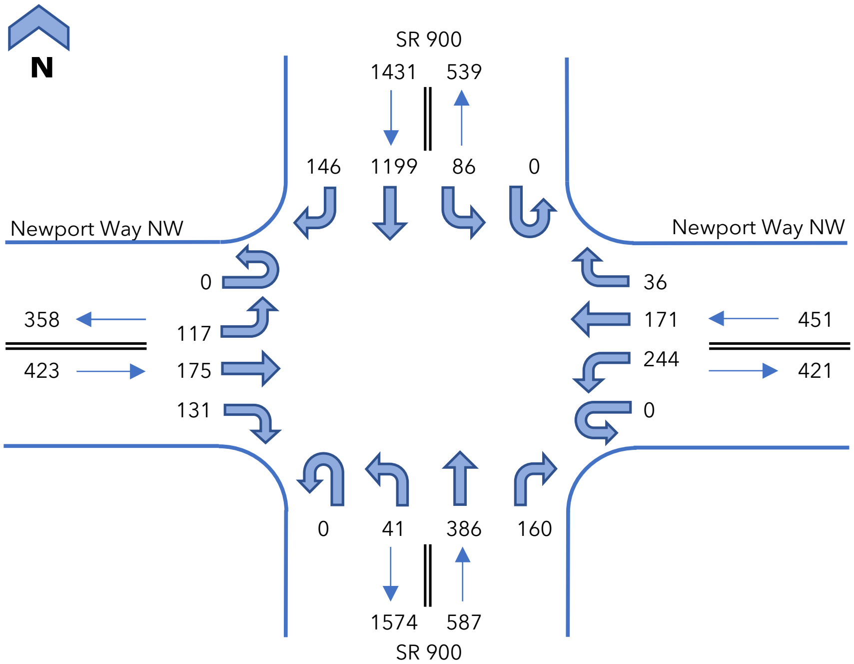
Pedestrians (Leg)				
E	W	N	S	Total
0	0	2	1	3
1	0	1	0	2
0	0	3	1	4
0	2	4	1	7
1	1	1	1	4
3	0	2	2	7
0	0	0	0	0
1	0	3	1	5
6	3	16	7	32
1	2	10	3	16

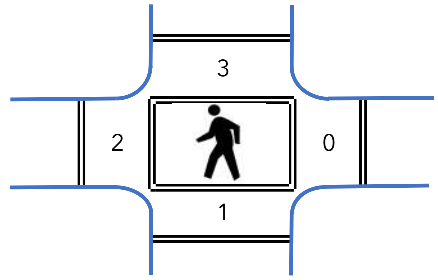
Bicycles (Leg)				
E	W	N	S	Total
0	0	0	0	0
0	0	0	0	0
0	0	0	0	0
0	2	0	2	4
0	0	0	0	0
0	0	0	0	0
0	0	2	0	2
0	0	0	0	0
0	2	2	2	6
0	2	0	2	4

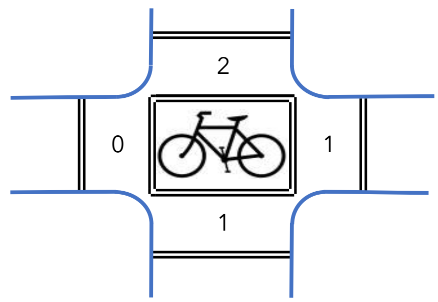
HEATH & ASSOCIATES
Transportation Planning & Engineering

Date Collected: 2/25/2025
Day of the Week: Tuesday
Time Period: 4:00 PM to 6:00 PM
Peak Hour: 4:00 PM to 5:00 PM

Newport Way NW & SR 900







Interval Start Time	Newport Way NW				Newport Way NW				SR 900				SR 900				15 Minute Totals	Hourly Totals
	Eastbound				Westbound				Northbound				Southbound					
	UT	LT	T	RT	UT	LT	T	RT	UT	LT	T	RT	UT	LT	T	RT		
4:00 PM	0	26	54	36	0	61	35	10	0	6	88	29	0	19	360	35	759	
4:15 PM	0	31	47	33	0	71	41	3	0	9	97	47	1	22	260	32	694	
4:30 PM	0	27	38	35	0	59	39	9	0	14	94	37	0	25	310	40	727	
4:45 PM	0	33	36	27	0	53	56	14	0	12	107	47	0	20	269	39	713	2893
5:00 PM	0	21	44	36	0	47	37	11	0	8	97	38	0	19	291	44	693	2827
5:15 PM	0	20	42	27	0	60	32	4	0	9	82	44	0	31	288	46	685	2818
5:30 PM	0	21	46	23	0	68	40	14	0	4	117	46	0	16	236	32	663	2754
5:45 PM	0	17	36	23	0	43	45	8	0	12	80	27	0	27	280	39	637	2678
Count Total	0	196	343	240	0	462	325	73	0	74	762	315	1	179	2294	307	5571	--
Peak Hour Total	0	117	175	131	0	244	171	36	0	41	386	160	1	86	1199	146	2893	--
PHF	0.91				0.92				0.88				0.86				0.95	--
Heavy Vehicles	0	2	7	3	0	1	3	2	0	1	12	9	0	0	25	5	70	--
HV %	0.0%	1.7%	4.0%	2.3%	0.0%	0.4%	1.8%	5.6%	0.0%	2.4%	3.1%	5.6%	0.0%	0.0%	2.1%	3.4%	2.4%	--

Interval Start Time	Heavy Vehicles				
	EB	WB	NB	SB	Total
4:00 PM	4	1	4	17	26
4:15 PM	5	2	6	6	19
4:30 PM	1	3	10	2	16
4:45 PM	2	0	2	5	9
5:00 PM	1	0	2	5	8
5:15 PM	1	0	2	1	4
5:30 PM	2	0	1	5	8
5:45 PM	0	0	0	4	4
Count Total	16	6	27	45	94
Peak Hour Total	12	6	22	30	70
Peak Hour HV%	2.8%	1.3%	3.7%	2.1%	2.4%

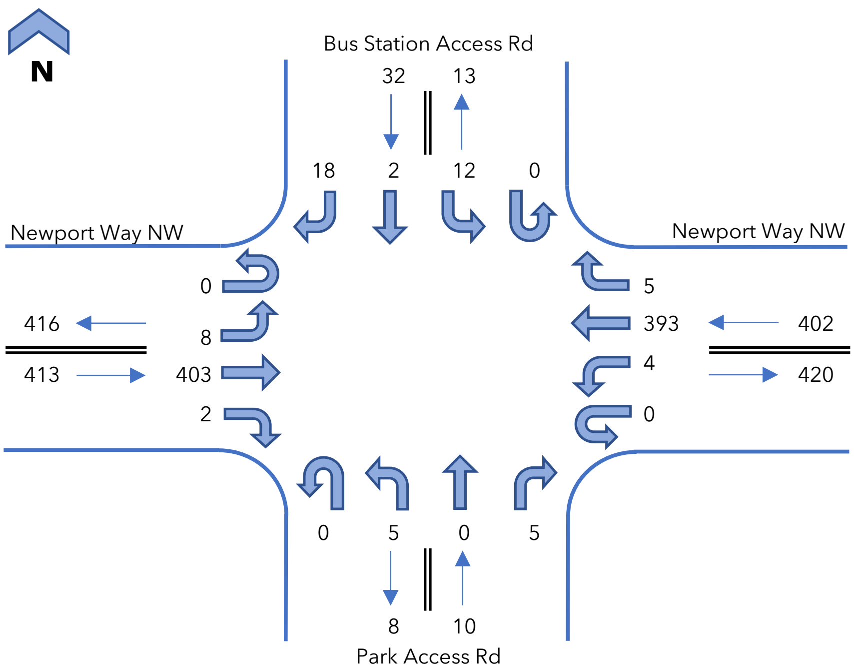
Pedestrians (Leg)				
E	W	N	S	Total
0	0	1	0	1
0	0	1	0	1
0	1	0	0	1
0	1	1	1	3
0	1	2	0	3
0	1	0	0	1
0	0	2	0	2
0	0	0	0	0
0	4	7	1	12
0	2	3	1	6

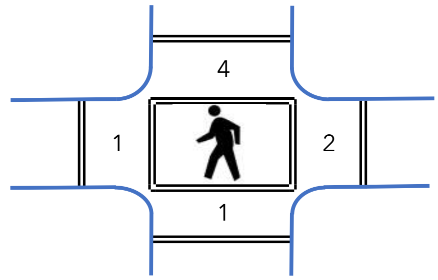
Bicycles (Leg)				
E	W	N	S	Total
0	0	0	0	0
0	0	2	0	2
0	0	0	0	0
1	0	0	1	2
0	0	0	0	0
0	0	0	0	0
0	0	0	0	0
0	0	0	0	0
1	0	2	1	4
1	0	2	1	4

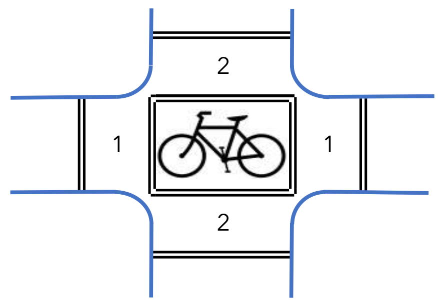
HEATH & ASSOCIATES
Transportation Planning & Engineering

Date Collected:	2/25/2025
Day of the Week:	Tuesday
Time Period:	4:00 PM to 6:00 PM
Peak Hour:	4:00 PM to 5:00 PM

Newport Way NW & Access Roads







Interval Start Time	Newport Way NW Eastbound				Newport Way NW Westbound				Park Access Rd Northbound				Bus Station Access Rd Southbound				15 Minute Totals	Hourly Totals
	UT	LT	T	RT	UT	LT	T	RT	UT	LT	T	RT	UT	LT	T	RT		
4:00 PM	0	3	87	0	0	0	105	1	0	0	0	2	0	3	0	5	206	
4:15 PM	0	3	111	1	0	3	94	0	0	2	0	0	0	1	1	3	219	
4:30 PM	0	0	105	0	0	0	100	2	0	2	0	0	0	2	0	4	215	
4:45 PM	0	2	100	1	0	1	94	2	0	1	0	3	0	6	1	6	217	857
5:00 PM	0	1	103	0	0	1	88	1	0	0	1	1	0	1	0	2	199	850
5:15 PM	0	2	109	0	0	2	92	0	0	0	0	2	0	5	0	3	215	846
5:30 PM	0	1	105	1	0	0	99	1	0	1	1	0	0	2	0	1	212	843
5:45 PM	0	4	85	3	0	1	89	0	0	1	0	3	0	4	0	4	194	820
Count Total	0	16	805	6	0	8	761	7	0	7	2	11	0	24	2	28	1677	--
Peak Hour Total	0	8	403	2	0	4	393	5	0	5	0	5	0	12	2	18	857	--
PHF	0.90				0.95				0.63				0.62				0.98	--
Heavy Vehicles	0	1	15	0	0	0	5	0	0	0	0	1	0	1	0	2	25	--
HV %	0.0%	12.5%	3.7%	0.0%	0.0%	0.0%	1.3%	0.0%	0.0%	0.0%	0.0%	20.0%	0.0%	8.3%	0.0%	11.1%	2.9%	--

Interval Start Time	Heavy Vehicles				
	EB	WB	NB	SB	Total
4:00 PM	2	0	1	2	5
4:15 PM	7	2	0	0	9
4:30 PM	5	3	0	1	9
4:45 PM	2	0	0	0	2
5:00 PM	1	0	0	0	1
5:15 PM	0	0	0	0	0
5:30 PM	2	0	0	0	2
5:45 PM	1	0	0	0	1
Count Total	20	5	1	3	29
Peak Hour Total	16	5	1	3	25
Peak Hour HV%	3.9%	1.2%	10.0%	9.4%	2.9%

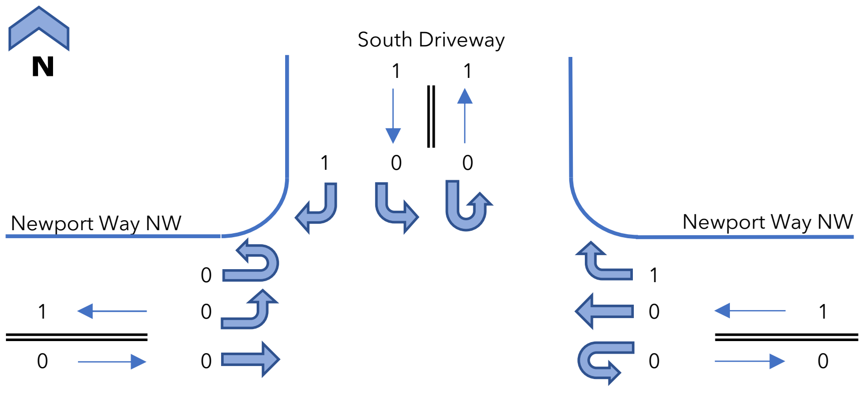
Pedestrians (Leg)				
E	W	N	S	Total
0	0	0	0	0
0	0	0	0	0
0	0	1	1	2
2	1	3	0	6
0	1	0	0	1
0	1	0	0	1
0	3	0	0	3
0	0	0	0	0
2	6	4	1	13
2	1	4	1	8

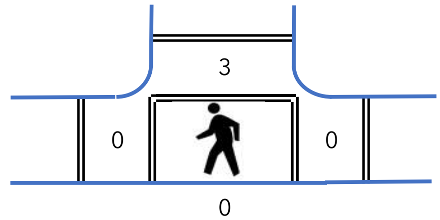
Bicycles (Leg)				
E	W	N	S	Total
1	0	1	0	2
0	0	1	0	1
0	0	0	1	1
0	1	0	1	2
0	0	0	0	0
0	0	0	0	0
0	0	0	0	0
0	0	0	0	0
1	1	2	2	6
1	1	2	2	6

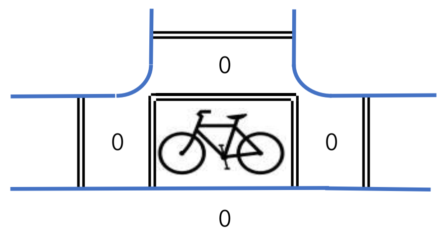
HEATH & ASSOCIATES
Transportation Planning & Engineering

Date Collected: 2/25/2025
Day of the Week: Tuesday
Time Period: 4:00 PM to 6:00 PM
Peak Hour: 4:00 PM to 5:00 PM

Newport Way NW & South Driveway







Interval Start Time	Newport Way NW				Newport Way NW								South Driveway				15 Minute Totals	Hourly Totals
	Eastbound				Westbound								Southbound					
	UT	LT	T	RT	UT	LT	T	RT					UT	LT	T	RT		
4:00 PM	0	0	0	--	0	--	0	1	--	--	--	--	0	0	--	0	1	
4:15 PM	0	0	0	--	0	--	0	0	--	--	--	--	0	0	--	0	0	
4:30 PM	0	0	0	--	0	--	0	0	--	--	--	--	0	0	--	1	1	
4:45 PM	0	0	0	--	0	--	0	0	--	--	--	--	0	0	--	0	0	2
5:00 PM	0	0	0	--	0	--	0	0	--	--	--	--	0	0	--	0	0	1
5:15 PM	0	0	0	--	0	--	0	1	--	--	--	--	0	0	--	0	1	2
5:30 PM	0	0	0	--	0	--	0	0	--	--	--	--	0	0	--	1	1	2
5:45 PM	0	0	0	--	0	--	0	0	--	--	--	--	0	0	--	0	0	2
Count Total	0	0	0	--	0	--	0	2	--	--	--	--	0	0	--	2	4	--
Peak Hour Total	0	0	0	--	0	--	0	1	--	--	--	--	0	0	--	1	2	--
PHF	#DIV/0!				0.25				--				0.25				0.50	--
Heavy Vehicles	0	0	0	--	0	--	0	0	--	--	--	--	0	0	--	0	0	--
HV %	0.0%	0.0%	0.0%	--	0.0%	--	0.0%	0.0%	--	--	--	--	0.0%	0.0%	--	0.0%	0.0%	--

Interval Start Time	Heavy Vehicles				
	EB	WB	NB	SB	Total
4:00 PM	0	0	--	0	0
4:15 PM	0	0	--	0	0
4:30 PM	0	0	--	0	0
4:45 PM	0	0	--	0	0
5:00 PM	0	0	--	0	0
5:15 PM	0	0	--	0	0
5:30 PM	0	0	--	0	0
5:45 PM	0	0	--	0	0
Count Total	0	0	--	0	0
Peak Hour Total	0	0	--	0	0
Peak Hour HV%	0.0%	0.0%	--	0.0%	0.0%

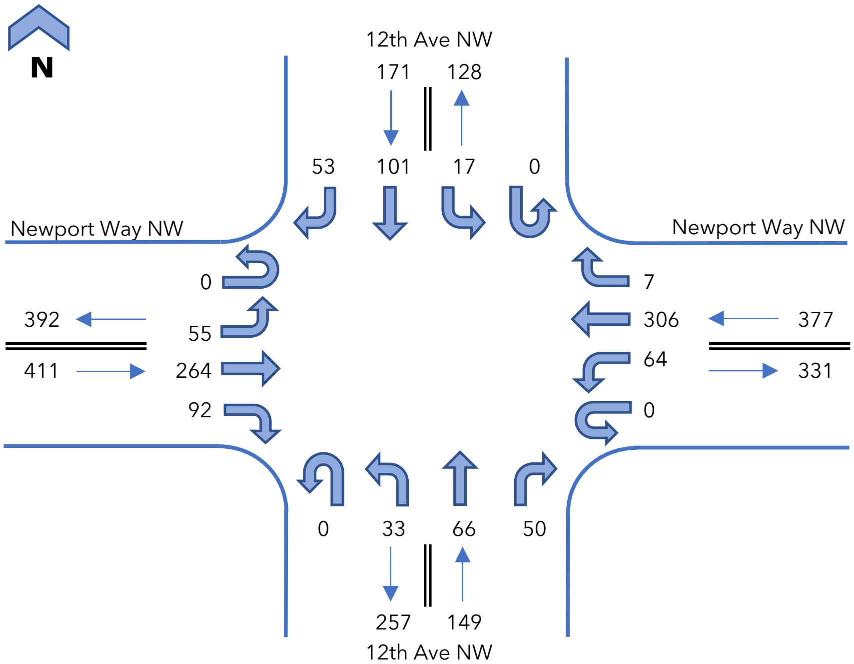
Pedestrians (Leg)				
E	W	N	S	Total
0	0	0	0	0
0	0	0	0	0
0	0	1	0	1
0	0	2	0	2
0	0	0	0	0
0	0	0	0	0
0	0	0	0	0
0	0	0	0	0
0	0	3	0	3
0	0	3	0	3

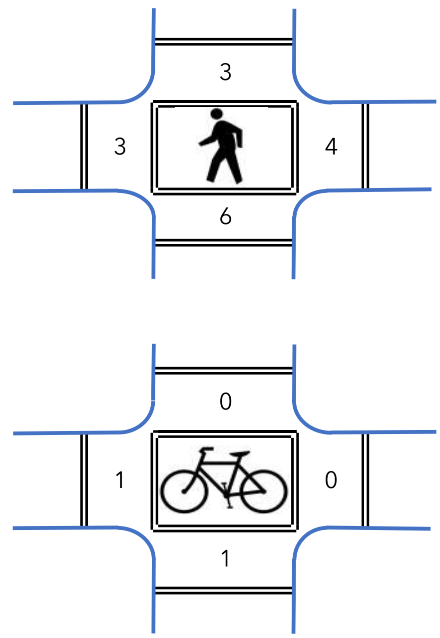
Bicycles (Leg)				
E	W	N	S	Total
0	0	0	0	0
0	0	0	0	0
0	0	0	0	0
0	0	0	0	0
0	0	0	0	0
0	0	0	0	0
0	0	0	0	0
0	0	0	0	0
0	0	0	0	0
0	0	0	0	0

HEATH & ASSOCIATES
Transportation Planning & Engineering

Date Collected:	2/25/2025
Day of the Week:	Tuesday
Time Period:	4:00 PM to 6:00 PM
Peak Hour:	4:00 PM to 5:00 PM

Newport Way NW & 12th Ave NW





Interval Start Time	Newport Way NW				Newport Way NW				12th Ave NW				12th Ave NW				15 Minute Totals	Hourly Totals
	Eastbound				Westbound				Northbound				Southbound					
	UT	LT	T	RT	UT	LT	T	RT	UT	LT	T	RT	UT	LT	T	RT		
4:00 PM	0	13	60	20	0	11	94	4	0	6	10	7	0	6	38	10	279	
4:15 PM	0	16	72	22	0	14	68	1	0	5	17	17	0	3	32	18	285	
4:30 PM	0	13	71	26	0	23	81	2	0	8	15	17	0	2	13	12	283	
4:45 PM	0	13	61	24	0	16	63	0	0	14	24	9	0	6	18	13	261	1108
5:00 PM	0	12	73	21	0	16	59	1	0	10	12	15	0	6	23	13	261	1090
5:15 PM	0	11	70	27	0	21	77	2	0	3	20	11	0	4	30	14	290	1095
5:30 PM	0	14	60	26	0	15	82	4	0	10	12	5	0	2	26	8	264	1076
5:45 PM	0	13	61	27	0	14	72	1	0	5	17	6	0	3	18	11	248	1063
Count Total	0	105	528	193	0	130	596	15	0	61	127	87	0	32	198	99	2171	--
Peak Hour Total	0	55	264	92	0	64	306	7	0	33	66	50	0	17	101	53	1108	--
PHF	0.93				0.86				0.79				0.79				0.97	--
Heavy Vehicles	0	0	20	2	0	0	5	0	0	0	1	1	0	0	2	1	32	--
HV %	0.0%	0.0%	7.6%	2.2%	0.0%	0.0%	1.6%	0.0%	0.0%	0.0%	1.5%	2.0%	0.0%	0.0%	2.0%	1.9%	2.9%	--

Interval Start Time	Heavy Vehicles				
	EB	WB	NB	SB	Total
4:00 PM	4	0	0	1	5
4:15 PM	8	2	1	0	11
4:30 PM	7	3	1	2	13
4:45 PM	3	0	0	0	3
5:00 PM	1	0	0	0	1
5:15 PM	1	0	0	0	1
5:30 PM	2	0	0	1	3
5:45 PM	3	1	0	0	4
Count Total	29	6	2	4	41
Peak Hour Total	22	5	2	3	32
Peak Hour HV%	5.4%	1.3%	1.3%	1.8%	2.9%

Interval Start Time	Pedestrians (Leg)				
	E	W	N	S	Total
4:00 PM	1	0	0	0	1
4:15 PM	2	1	1	2	6
4:30 PM	0	0	0	0	0
4:45 PM	1	2	2	4	9
5:00 PM	0	2	2	3	7
5:15 PM	0	0	0	0	0
5:30 PM	0	0	0	1	1
5:45 PM	0	0	0	0	0
Count Total	4	5	5	10	24
Peak Hour Total	4	3	3	6	16

Interval Start Time	Bicycles (Leg)				
	E	W	N	S	Total
4:00 PM	0	0	0	0	0
4:15 PM	0	0	0	0	0
4:30 PM	0	0	0	0	0
4:45 PM	0	1	0	1	2
5:00 PM	0	1	0	0	1
5:15 PM	0	0	0	0	0
5:30 PM	0	0	0	0	0
5:45 PM	0	0	0	0	0
Count Total	0	2	0	1	3
Peak Hour Total	0	1	0	1	2

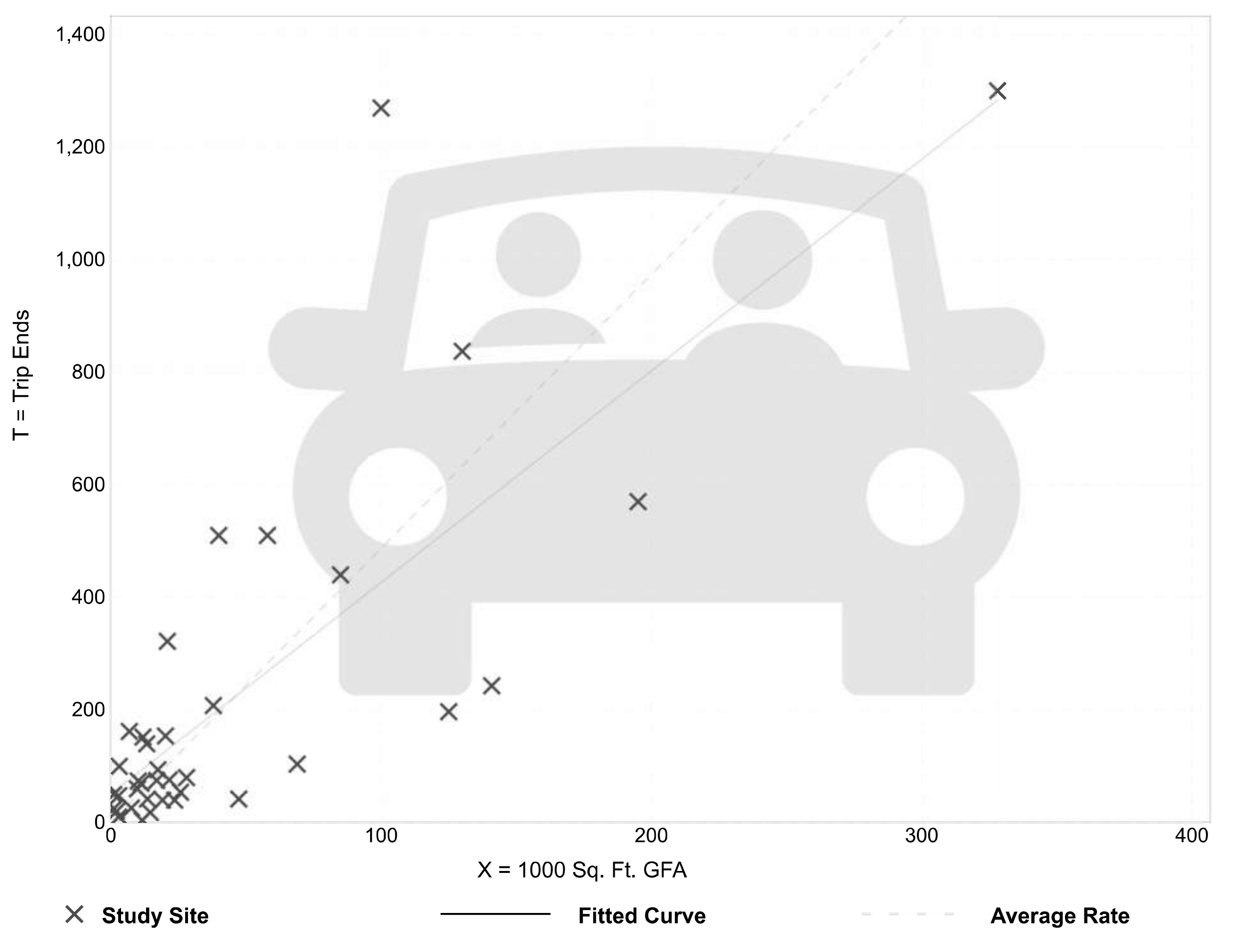
General Light Industrial
(110)

Vehicle Trip Ends vs:	**1000 Sq. Ft. GFA**
On a:	**Weekday**
Setting/Location:	**General Urban/Suburban**
Number of Studies:	37
Avg. 1000 Sq. Ft. GFA:	45
Directional Distribution:	50% entering, 50% exiting

Vehicle Trip Generation per 1000 Sq. Ft. GFA

Average Rate	Range of Rates	Standard Deviation
4.87	0.34 - 43.86	4.08

Data Plot and Equation



Fitted Curve Equation: $T = 3.76(X) + 50.47$ **$R^2 = 0.61$**

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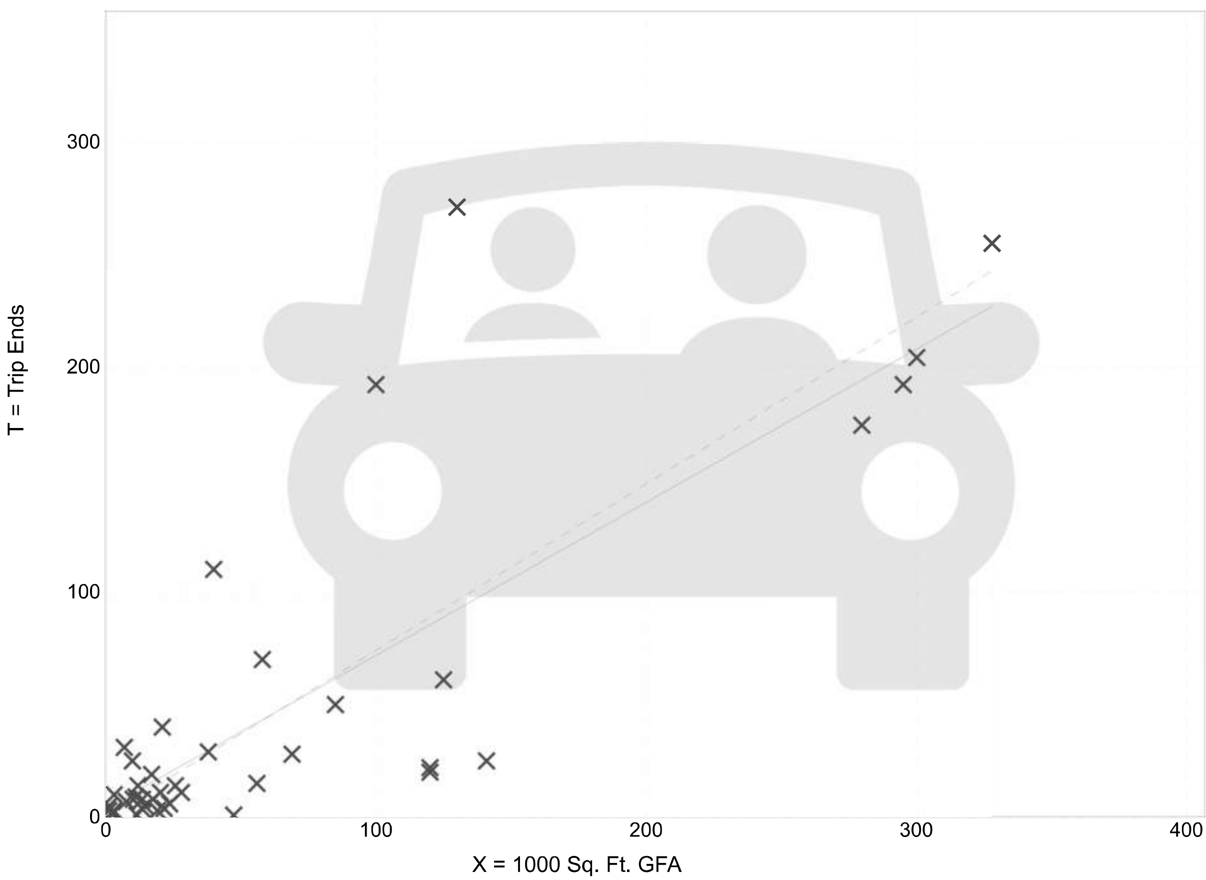
General Light Industrial
(110)

Vehicle Trip Ends vs:	**1000 Sq. Ft. GFA**
On a:	**Weekday, Peak Hour of Adjacent Street Traffic, One Hour Between 7 and 9 a.m.**
Setting/Location:	**General Urban/Suburban**
Number of Studies:	41
Avg. 1000 Sq. Ft. GFA:	65
Directional Distribution:	88% entering, 12% exiting

Vehicle Trip Generation per 1000 Sq. Ft. GFA

Average Rate	Range of Rates	Standard Deviation
0.74	0.02 - 4.46	0.61

Data Plot and Equation



× **Study Site** —— **Fitted Curve** - - - **Average Rate**

Fitted Curve Equation: T = 0.68(X) + 3.81 **R^2= 0.66**

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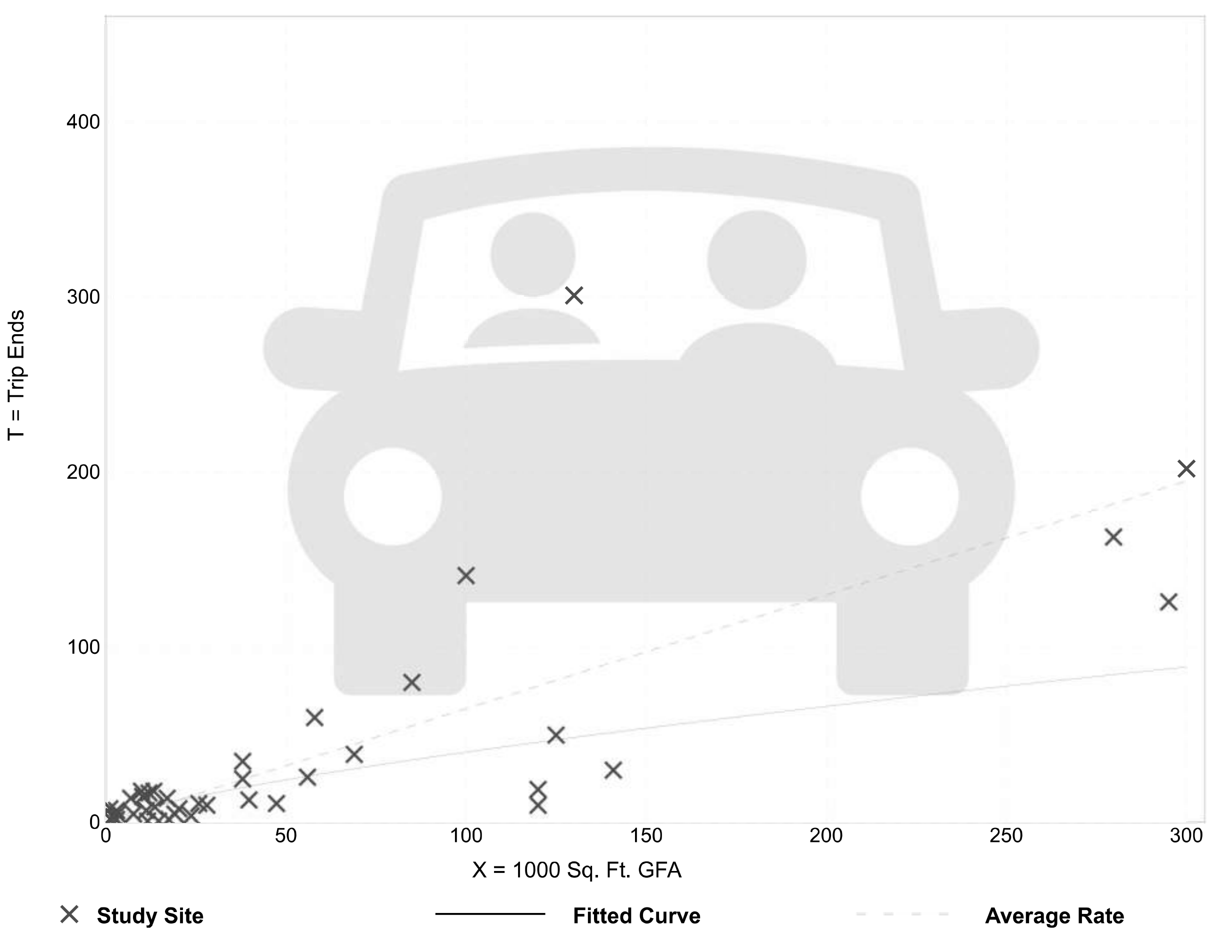
General Light Industrial
(110)

Vehicle Trip Ends vs:	**1000 Sq. Ft. GFA**
On a:	**Weekday, Peak Hour of Adjacent Street Traffic, One Hour Between 4 and 6 p.m.**
Setting/Location:	**General Urban/Suburban**
Number of Studies:	40
Avg. 1000 Sq. Ft. GFA:	58
Directional Distribution:	14% entering, 86% exiting

Vehicle Trip Generation per 1000 Sq. Ft. GFA

Average Rate	Range of Rates	Standard Deviation
0.65	0.07 - 7.02	0.56

Data Plot and Equation



Fitted Curve Equation: Ln(T) = 0.72 Ln(X) + 0.38 **R^2= 0.55**

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Multifamily Housing (Mid-Rise)
Not Close to Rail Transit (221)

Vehicle Trip Ends vs: **Dwelling Units**
On a: **Weekday**

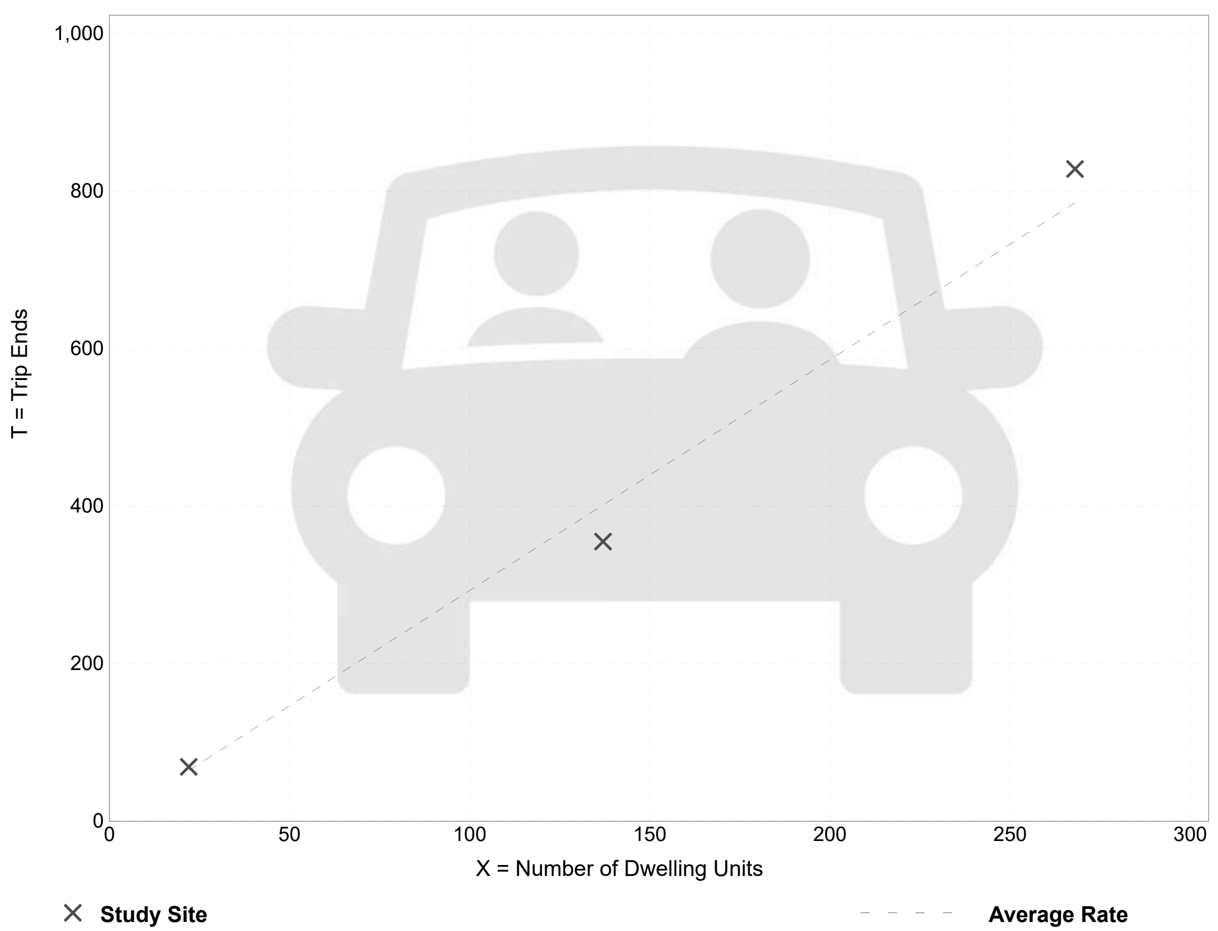
Setting/Location: **Dense Multi-Use Urban**
Number of Studies: 3
Avg. Num. of Dwelling Units: 142
Directional Distribution: 50% entering, 50% exiting

Vehicle Trip Generation per Dwelling Unit

Average Rate	Range of Rates	Standard Deviation
2.93	2.59 - 3.14	0.29

Data Plot and Equation

Caution – Small Sample Size



Fitted Curve Equation: Not Given

R^2= ****

Trip Gen Manual, 11th Edition ● Institute of Transportation Engineers

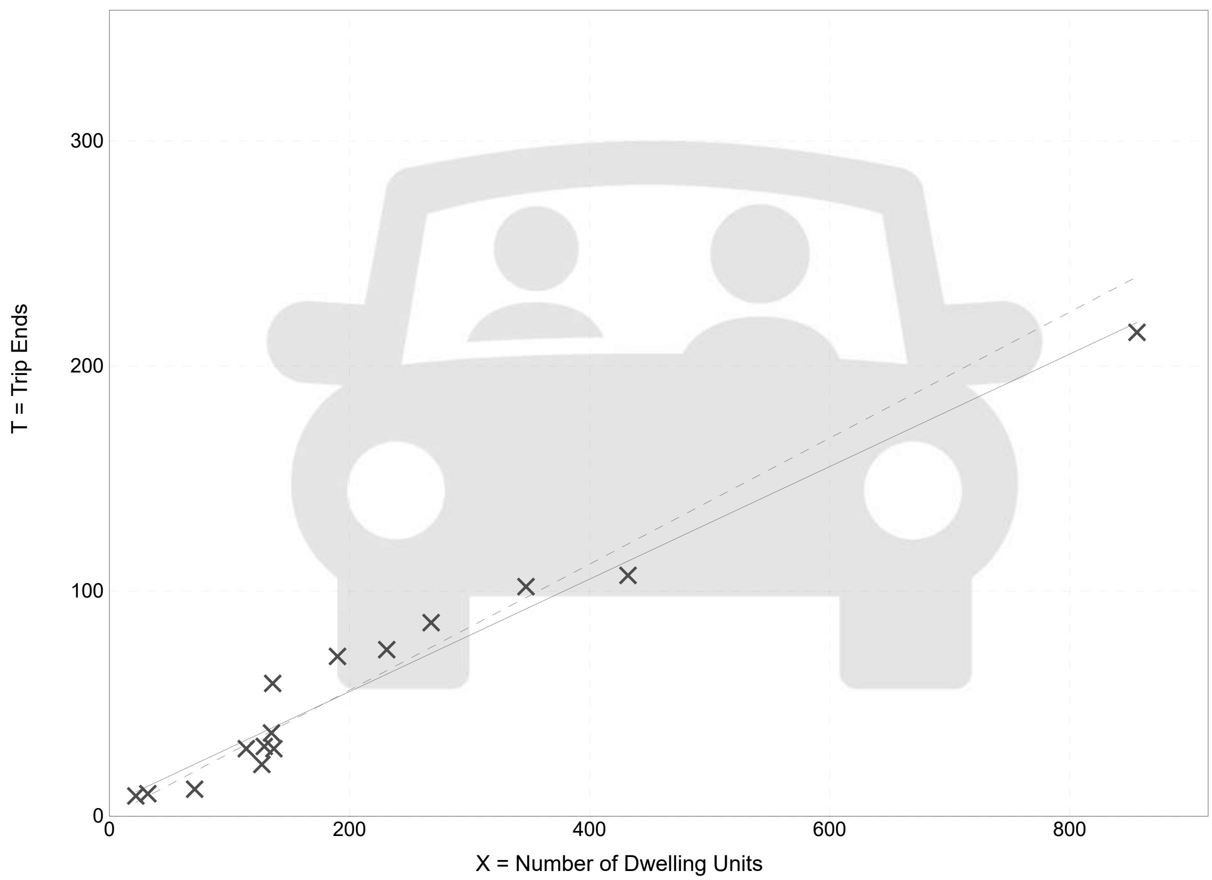
Multifamily Housing (Mid-Rise)
Not Close to Rail Transit (221)

Vehicle Trip Ends vs:	**Dwelling Units**
On a:	**Weekday,**
	Peak Hour of Adjacent Street Traffic,
	One Hour Between 7 and 9 a.m.
Setting/Location:	**Dense Multi-Use Urban**
Number of Studies:	15
Avg. Num. of Dwelling Units:	215
Directional Distribution:	14% entering, 86% exiting

Vehicle Trip Generation per Dwelling Unit

Average Rate	Range of Rates	Standard Deviation
0.28	0.17 - 0.43	0.06

Data Plot and Equation



× **Study Site** ——— **Fitted Curve** - - - **Average Rate**

Fitted Curve Equation: T = 0.25(X) + 5.35 **R^2= 0.96**

Trip Gen Manual, 11th Edition ● Institute of Transportation Engineers

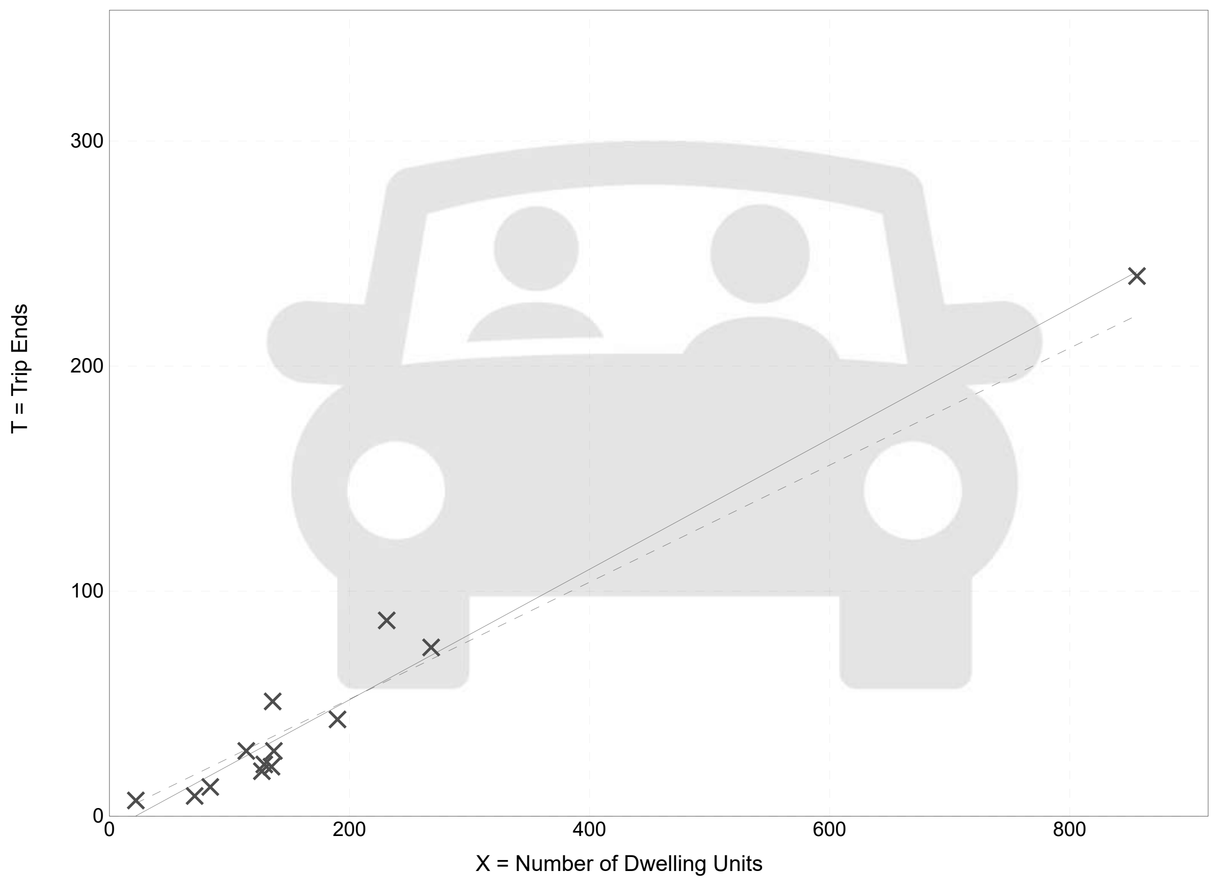
Multifamily Housing (Mid-Rise)
Not Close to Rail Transit (221)

Vehicle Trip Ends vs:	**Dwelling Units**
On a:	**Weekday,**
	Peak Hour of Adjacent Street Traffic,
	One Hour Between 4 and 6 p.m.
Setting/Location:	**Dense Multi-Use Urban**
Number of Studies:	13
Avg. Num. of Dwelling Units:	192
Directional Distribution:	74% entering, 26% exiting

Vehicle Trip Generation per Dwelling Unit

Average Rate	Range of Rates	Standard Deviation
0.26	0.13 - 0.38	0.07

Data Plot and Equation



Fitted Curve Equation: T = 0.29(X) - 6.26 **R^2= 0.97**

PM Peak Hour Forecast Intersection Volumes

Annual Growth Rate: 2 % 2028
# of Years to Horizon: 3

1. SR 900 & Maple St

	SBR	SBT	SBL	WBR	WBT	WBL	NBR	NBT	NBL	EBR	EBT	EBL
Existing	5	1341	234	224	5	140	36	497	2	14	9	21
Project Trips	0	17	17	6	0	3	7	6	0	0	0	0
Pipeline	0	0	0	0	0	0	0	0	0	0	0	0
Without	5	1,423	248	238	5	149	38	527	2	15	10	22
With	5	1,440	265	244	5	152	45	533	2	15	10	22

2. Maple St & TC Road

	SBR	SBT	SBL	WBR	WBT	WBL	NBR	NBT	NBL	EBR	EBT	EBL
Existing	20	0	6	13	323	8	17	4	39	13	214	36
Project Trips	0	0	0	0	9	0	0	0	0	0	24	0
Pipeline	0	0	0	0	0	0	0	0	0	0	0	0
Without	21	0	6	14	343	8	18	4	41	14	227	38
With	21	0	6	14	352	8	18	4	41	14	251	38

3. Maple St & 13th Ave

	SBR	SBT	SBL	WBR	WBT	WBL	NBR	NBT	NBL	EBR	EBT	EBL
Existing	0	0	0	0	342	0	2	0	2	2	235	0
Project Trips	0	0	0	0	0	8	4	0	9	24	0	0
Pipeline	0	0	0	0	0	0	0	0	0	0	0	0
Without	0	0	0	0	363	0	2	0	2	2	249	0
With	0	0	0	0	363	8	6	0	11	26	249	0

4. Maple St & 12th Ave

	SBR	SBT	SBL	WBR	WBT	WBL	NBR	NBT	NBL	EBR	EBT	EBL
Existing	54	131	90	104	236	8	8	114	36	28	169	26
Project Trips	4	5	0	0	4	0	0	1	0	0	2	2
Pipeline	0	0	0	0	0	0	0	0	0	0	0	0
Without	57	139	96	110	250	8	8	121	38	30	179	28
With	61	144	96	110	254	8	8	122	38	30	181	30

5. SR 900 & Newport

	SBR	SBT	SBL	WBR	WBT	WBL	NBR	NBT	NBL	EBR	EBT	EBL
Existing	146	1199	86	36	171	244	160	386	41	131	175	117
Project Trips	0	3	17	6	1	3	8	7	0	0	2	0
Pipeline	0	0	0	0	0	0	0	0	0	0	0	0
Without	155	1,272	91	38	181	259	170	410	44	139	186	124
With	155	1,275	108	44	182	262	178	417	44	139	188	124

6. Newport & TC Road

	SBR	SBT	SBL	WBR	WBT	WBL	NBR	NBT	NBL	EBR	EBT	EBL
Existing	18	2	12	5	393	4	5	0	5	2	403	8
Project Trips	0	0	0	0	10	0	0	0	0	0	27	0
Pipeline	0	0	0	0	0	0	0	0	0	0	0	0
Without	19	2	13	5	417	4	5	0	5	2	428	8
With	19	2	13	5	427	4	5	0	5	2	455	8

7. Newport & 13th St

	SBR	SBT	SBL	WBR	WBT	WBL	NBR	NBT	NBL	EBR	EBT	EBL
Existing	1	0	0	1	401	0	0	0	0	0	420	0
Project Trips	10	0	4	16	0	0	0	0	0	0	0	27
Pipeline	0	0	0	0	0	0	0	0	0	0	0	0
Without	1	0	0	1	426	0	0	0	0	0	446	0
With	11	0	4	17	426	0	0	0	0	0	446	27

8. Newport & 12th Ave

	SBR	SBT	SBL	WBR	WBT	WBL	NBR	NBT	NBL	EBR	EBT	EBL
Existing	53	101	17	7	306	64	50	66	33	92	264	55
Project Trips	5	0	0	0	11	0	0	0	0	0	3	1
Pipeline	0	0	0	0	0	0	0	0	0	0	0	0
Without	56	107	18	7	325	68	53	70	35	98	280	58
With	61	107	18	7	336	68	53	70	35	98	283	59

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	21	9	14	140	5	224	2	497	36	234	1341	5
Future Volume (veh/h)	21	9	14	140	5	224	2	497	36	234	1341	5
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Lane Width Adj.	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Ped-Bike Adj(A_pbT)	0.99		0.97	0.98		0.97	1.00		0.98	1.00		0.97
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1786	1786	1786	1758	1786	1786	1098	1730	1786	1772	1758	1786
Adj Flow Rate, veh/h	23	10	15	154	5	246	2	546	40	257	1474	5
Peak Hour Factor	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91
Percent Heavy Veh, %	1	1	1	3	1	1	50	5	1	2	3	1
Cap, veh/h	168	74	93	352	406	333	3	1814	817	318	3190	11
Arrive On Green	0.23	0.23	0.23	0.23	0.23	0.23	0.00	0.18	0.18	0.10	0.65	0.65
Sat Flow, veh/h	571	327	408	1279	1786	1462	1046	3287	1481	3274	4937	17
Grp Volume(v), veh/h	48	0	0	154	5	246	2	546	40	257	955	524
Grp Sat Flow(s),veh/h/ln	1307	0	0	1279	1786	1462	1046	1643	1481	1637	1600	1754
Q Serve(g_s), s	1.4	0.0	0.0	10.1	0.3	21.9	0.3	20.1	3.1	10.8	21.1	21.1
Cycle Q Clear(g_c), s	3.5	0.0	0.0	13.7	0.3	21.9	0.3	20.1	3.1	10.8	21.1	21.1
Prop In Lane	0.48		0.31	1.00		1.00	1.00		1.00	1.00		0.01
Lane Grp Cap(c), veh/h	335	0	0	352	406	333	3	1814	817	318	2068	1134
V/C Ratio(X)	0.14	0.00	0.00	0.44	0.01	0.74	0.72	0.30	0.05	0.81	0.46	0.46
Avail Cap(c_a), veh/h	433	0	0	449	542	444	56	1814	817	854	2068	1134
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	0.33	0.33	0.33	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	0.00	0.99	0.99	0.99	0.84	0.84	0.84	1.00	1.00	1.00
Uniform Delay (d), s/veh	43.1	0.0	0.0	46.7	41.9	50.2	69.9	33.9	26.9	61.9	12.5	12.5
Incr Delay (d2), s/veh	0.2	0.0	0.0	0.9	0.0	4.4	140.3	0.4	0.1	4.9	0.7	1.4
Initial Q Delay(d3), s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.4	0.0	0.0	4.8	0.1	8.4	0.2	9.0	1.1	4.6	7.3	8.2
Unsig. Movement Delay, s/veh												
LnGrp Delay(d), s/veh	43.2	0.0	0.0	47.6	41.9	54.7	210.2	34.2	27.0	66.8	13.2	13.8
LnGrp LOS	D			D	D	D	F	C	C	E	B	B
Approach Vol, veh/h		48			405			588			1736	
Approach Delay, s/veh		43.2			51.8			34.4			21.3	
Approach LOS		D			D			C			C	

Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	19.1	83.6		37.3	5.9	96.8		37.3				
Change Period (Y+Rc), s	5.5	6.3		5.5	5.5	6.3		5.5				
Max Green Setting (Gmax), s	36.5	43.7		42.5	7.5	72.7		42.5				
Max Q Clear Time (g_c+I1), s	12.8	22.1		5.5	2.3	23.1		23.9				
Green Ext Time (p_c), s	0.8	3.5		0.3	0.0	13.6		1.3				

Intersection Summary			
HCM 7th Control Delay, s/veh			28.9
HCM 7th LOS			C

Notes

User approved pedestrian interval to be less than phase max green.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	36	214	13	8	323	13	39	4	17	6	0	20
Future Volume (veh/h)	36	214	13	8	323	13	39	4	17	6	0	20
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Lane Width Adj.	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Ped-Bike Adj(A_pbT)	0.98		0.94	0.97		0.94	0.95		0.96	0.97		0.93
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1885	1870	1678	1885	1870	1885	1856	1885	1885	1885	1885	1885
Adj Flow Rate, veh/h	41	243	15	9	367	15	44	5	19	7	0	23
Peak Hour Factor	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88
Percent Heavy Veh, %	1	2	15	1	2	1	3	1	1	1	1	1
Cap, veh/h	485	1056	65	517	959	39	504	78	295	515	0	348
Arrive On Green	0.05	0.31	0.31	0.01	0.28	0.28	0.23	0.23	0.23	0.23	0.00	0.23
Sat Flow, veh/h	1795	3388	207	1795	3469	141	1314	332	1261	1359	0	1486
Grp Volume(v), veh/h	41	127	131	9	187	195	44	0	24	7	0	23
Grp Sat Flow(s),veh/h/ln	1795	1777	1818	1795	1777	1834	1314	0	1593	1359	0	1486
Q Serve(g_s), s	0.5	1.8	1.8	0.1	2.9	2.9	0.9	0.0	0.4	0.1	0.0	0.4
Cycle Q Clear(g_c), s	0.5	1.8	1.8	0.1	2.9	2.9	1.3	0.0	0.4	0.5	0.0	0.4
Prop In Lane	1.00		0.11	1.00		0.08	1.00		0.79	1.00		1.00
Lane Grp Cap(c), veh/h	485	554	567	517	491	507	504	0	373	515	0	348
V/C Ratio(X)	0.08	0.23	0.23	0.02	0.38	0.38	0.09	0.00	0.06	0.01	0.00	0.07
Avail Cap(c_a), veh/h	877	1100	1125	972	1100	1135	1164	0	1174	1198	0	1095
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	8.1	8.7	8.7	8.6	9.9	9.9	10.6	0.0	10.1	10.3	0.0	10.1
Incr Delay (d2), s/veh	0.0	0.2	0.2	0.0	0.4	0.4	0.0	0.0	0.0	0.0	0.0	0.0
Initial Q Delay(d3), s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.1	0.5	0.5	0.0	0.9	0.9	0.2	0.0	0.1	0.0	0.0	0.1
Unsig. Movement Delay, s/veh												
LnGrp Delay(d), s/veh	8.1	8.8	8.8	8.6	10.3	10.3	10.6	0.0	10.1	10.3	0.0	10.1
LnGrp LOS	A	A	A	A	B	B	B		B	B		B
Approach Vol, veh/h		299			391			68			30	
Approach Delay, s/veh		8.7			10.3			10.5			10.2	
Approach LOS		A			B			B			B	

Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	6.6	14.4		12.9	5.4	15.6		12.9				
Change Period (Y+Rc), s	5.0	5.0		5.0	5.0	5.0		5.0				
Max Green Setting (Gmax), s	9.0	21.0		25.0	9.0	21.0		25.0				
Max Q Clear Time (g_c+I1), s	2.5	4.9		3.3	2.1	3.8		2.5				
Green Ext Time (p_c), s	0.0	1.6		0.1	0.0	1.1		0.1				

Intersection Summary			
HCM 7th Control Delay, s/veh			9.7
HCM 7th LOS			A

Intersection	
Int Delay, s/veh	0

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Lane Configurations						
Traffic Vol, veh/h	235	2	0	342	0	2
Future Vol, veh/h	235	2	0	342	0	2
Conflicting Peds, #/hr	0	10	10	0	10	10
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	-	-	-	-	0
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	255	2	0	372	0	2

Major/Minor	Major1		Major2		Minor1	
Conflicting Flow All	0	0	-	-	-	149
Stage 1	-	-	-	-	-	-
Stage 2	-	-	-	-	-	-
Critical Hdwy	-	-	-	-	-	6.94
Critical Hdwy Stg 1	-	-	-	-	-	-
Critical Hdwy Stg 2	-	-	-	-	-	-
Follow-up Hdwy	-	-	-	-	-	3.32
Pot Cap-1 Maneuver	-	-	0	-	0	871
Stage 1	-	-	0	-	0	-
Stage 2	-	-	0	-	0	-
Platoon blocked, %	-	-		-		
Mov Cap-1 Maneuver	-	-	-	-	-	855
Mov Cap-2 Maneuver	-	-	-	-	-	-
Stage 1	-	-	-	-	-	-
Stage 2	-	-	-	-	-	-

Approach	EB	WB	NB
HCM Ctrl Dly, s/v	0	0	9.22
HCM LOS			A

Minor Lane/Major Mvmt	NBLn1	EBT	EBR	WBT
Capacity (veh/h)	855	-	-	-
HCM Lane V/C Ratio	0.003	-	-	-
HCM Ctrl Dly (s/v)	9.2	-	-	-
HCM Lane LOS	A	-	-	-
HCM 95th %tile Q(veh)	0	-	-	-

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations	↰	↑↱		↰	↑↱		↰	↱		↰	↱	
Traffic Volume (veh/h)	26	169	28	8	236	104	36	114	8	90	131	54
Future Volume (veh/h)	26	169	28	8	236	104	36	114	8	90	131	54
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Lane Width Adj.	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Ped-Bike Adj(A_pbT)	0.98		0.94	0.98		0.94	0.98		0.96	0.98		0.96
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1841	1841	1885	1885	1856	1885	1885	1885	1707	1885	1885	1885
Adj Flow Rate, veh/h	29	188	31	9	262	116	40	127	9	100	146	60
Peak Hour Factor	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90
Percent Heavy Veh, %	4	4	1	1	3	1	1	1	13	1	1	1
Cap, veh/h	348	730	118	412	523	223	393	324	23	470	281	116
Arrive On Green	0.03	0.24	0.24	0.01	0.22	0.22	0.04	0.19	0.19	0.08	0.22	0.22
Sat Flow, veh/h	1753	2987	481	1795	2357	1003	1795	1734	123	1795	1254	515
Grp Volume(v), veh/h	29	108	111	9	193	185	40	0	136	100	0	206
Grp Sat Flow(s),veh/h/ln	1753	1749	1719	1795	1763	1597	1795	0	1857	1795	0	1769
Q Serve(g_s), s	0.5	2.1	2.2	0.2	4.0	4.3	0.7	0.0	2.7	1.8	0.0	4.3
Cycle Q Clear(g_c), s	0.5	2.1	2.2	0.2	4.0	4.3	0.7	0.0	2.7	1.8	0.0	4.3
Prop In Lane	1.00		0.28	1.00		0.63	1.00		0.07	1.00		0.29
Lane Grp Cap(c), veh/h	348	427	420	412	391	355	393	0	347	470	0	397
V/C Ratio(X)	0.08	0.25	0.26	0.02	0.49	0.52	0.10	0.00	0.39	0.21	0.00	0.52
Avail Cap(c_a), veh/h	746	873	858	775	796	721	569	0	971	664	0	1009
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	12.1	12.8	12.8	12.4	14.3	14.4	12.8	0.0	15.0	12.0	0.0	14.3
Incr Delay (d2), s/veh	0.0	0.1	0.1	0.0	0.4	0.4	0.0	0.0	0.3	0.1	0.0	0.4
Initial Q Delay(d3), s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.2	0.7	0.7	0.1	1.4	1.3	0.3	0.0	1.0	0.6	0.0	1.5
Unsig. Movement Delay, s/veh												
LnGrp Delay(d), s/veh	12.1	12.9	13.0	12.4	14.7	14.8	12.9	0.0	15.3	12.1	0.0	14.7
LnGrp LOS	B	B	B	B	B	B	B		B	B		B
Approach Vol, veh/h		248			387			176			306	
Approach Delay, s/veh		12.8			14.7			14.7			13.9	
Approach LOS		B			B			B			B	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	6.4	14.3	8.4	12.9	5.5	15.3	6.9	14.4
Change Period (Y+Rc), s	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0
Max Green Setting (Gmax), s	11.0	19.0	8.0	22.0	9.0	21.0	6.0	24.0
Max Q Clear Time (g_c+I1), s	2.5	6.3	3.8	4.7	2.2	4.2	2.7	6.3
Green Ext Time (p_c), s	0.0	1.2	0.0	0.4	0.0	0.7	0.0	0.7

Intersection Summary		
HCM 7th Control Delay, s/veh		14.1
HCM 7th LOS		B

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	117	175	131	244	171	36	41	386	160	86	1199	146
Future Volume (veh/h)	117	175	131	244	171	36	41	386	160	86	1199	146
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Lane Width Adj.	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Ped-Bike Adj(A_pbT)	1.00		0.97	1.00		0.98	1.00		0.95	1.00		0.97
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1772	1744	1772	1786	1772	1716	1772	1758	1716	1786	1772	1758
Adj Flow Rate, veh/h	123	184	138	257	180	38	43	406	168	91	1262	154
Peak Hour Factor	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95
Percent Heavy Veh, %	2	4	2	1	2	6	2	3	6	1	2	3
Cap, veh/h	145	269	225	237	368	295	55	545	226	637	1721	740
Arrive On Green	0.09	0.15	0.15	0.14	0.21	0.21	0.03	0.16	0.16	0.12	0.17	0.17
Sat Flow, veh/h	1688	1744	1458	1701	1772	1418	1688	3340	1386	1701	3367	1447
Grp Volume(v), veh/h	123	184	138	257	180	38	43	406	168	91	1262	154
Grp Sat Flow(s),veh/h/ln	1688	1744	1458	1701	1772	1418	1688	1670	1386	1701	1683	1447
Q Serve(g_s), s	10.1	14.0	12.4	19.5	12.5	1.3	3.5	16.2	11.8	6.7	49.8	12.8
Cycle Q Clear(g_c), s	10.1	14.0	12.4	19.5	12.5	1.3	3.5	16.2	11.8	6.7	49.8	12.8
Prop In Lane	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	145	269	225	237	368	295	55	545	226	637	1721	740
V/C Ratio(X)	0.85	0.68	0.61	1.08	0.49	0.13	0.79	0.74	0.74	0.14	0.73	0.21
Avail Cap(c_a), veh/h	175	492	411	237	563	451	175	923	383	637	1721	740
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.33	0.33	0.33
Upstream Filter(I)	1.00	1.00	1.00	0.97	0.97	0.97	1.00	1.00	1.00	0.88	0.88	0.88
Uniform Delay (d), s/veh	63.1	56.0	55.3	60.3	48.9	8.0	67.3	55.8	29.9	41.3	49.1	33.8
Incr Delay (d2), s/veh	26.8	3.1	2.7	82.0	1.0	0.2	21.5	8.9	19.6	0.1	2.5	0.6
Initial Q Delay(d3), s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	5.4	6.4	4.8	13.9	5.7	1.1	1.8	7.4	5.3	2.9	23.1	5.1
Unsig. Movement Delay, s/veh												
LnGrp Delay(d), s/veh	89.9	59.0	58.0	142.3	49.9	8.2	88.8	64.7	49.5	41.4	51.6	34.3
LnGrp LOS	F	E	E	F	D	A	F	E	D	D	D	C
Approach Vol, veh/h		445			475			617			1507	
Approach Delay, s/veh		67.2			96.5			62.3			49.2	
Approach LOS		E			F			E			D	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	58.7	29.2	25.0	27.1	10.0	77.9	17.5	34.6
Change Period (Y+Rc), s	6.3	* 6.3	5.5	5.5	5.5	6.3	5.5	5.5
Max Green Setting (Gmax), s	19.5	* 39	19.5	39.5	14.5	43.7	14.5	44.5
Max Q Clear Time (g_c+I1), s	8.7	18.2	21.5	16.0	5.5	51.8	12.1	14.5
Green Ext Time (p_c), s	0.1	3.0	0.0	1.5	0.0	0.0	0.1	1.2

Intersection Summary	
HCM 7th Control Delay, s/veh	61.9
HCM 7th LOS	E

Notes

* HCM 7th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	8	403	2	4	393	5	5	0	5	12	2	18
Future Volume (veh/h)	8	403	2	4	393	5	5	0	5	12	2	18
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Lane Width Adj.	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Ped-Bike Adj(A_pbT)	1.00		0.96	1.00		0.96	0.97		0.95	0.96		0.95
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1707	1841	1885	1885	1885	1885	1885	1885	1604	1781	1885	1737
Adj Flow Rate, veh/h	8	411	2	4	401	5	5	0	5	12	2	18
Peak Hour Factor	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98
Percent Heavy Veh, %	13	4	1	1	1	1	1	1	20	8	1	11
Cap, veh/h	432	674	3	438	674	8	382	0	169	387	17	155
Arrive On Green	0.01	0.37	0.37	0.01	0.36	0.36	0.11	0.00	0.11	0.11	0.11	0.11
Sat Flow, veh/h	1626	1830	9	1795	1857	23	1357	0	1525	1297	155	1399
Grp Volume(v), veh/h	8	0	413	4	0	406	5	0	5	12	0	20
Grp Sat Flow(s),veh/h/ln	1626	0	1839	1795	0	1880	1357	0	1525	1297	0	1555
Q Serve(g_s), s	0.1	0.0	5.3	0.0	0.0	5.1	0.1	0.0	0.1	0.2	0.0	0.3
Cycle Q Clear(g_c), s	0.1	0.0	5.3	0.0	0.0	5.1	0.4	0.0	0.1	0.3	0.0	0.3
Prop In Lane	1.00		0.00	1.00		0.01	1.00		1.00	1.00		0.90
Lane Grp Cap(c), veh/h	432	0	678	438	0	683	382	0	169	387	0	172
V/C Ratio(X)	0.02	0.00	0.61	0.01	0.00	0.59	0.01	0.00	0.03	0.03	0.00	0.12
Avail Cap(c_a), veh/h	749	0	1389	798	0	1420	1257	0	1152	1223	0	1175
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	6.2	0.0	7.5	6.3	0.0	7.5	11.9	0.0	11.5	11.7	0.0	11.7
Incr Delay (d2), s/veh	0.0	0.0	0.9	0.0	0.0	0.8	0.0	0.0	0.0	0.0	0.0	0.1
Initial Q Delay(d3), s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.0	0.0	1.3	0.0	0.0	1.3	0.0	0.0	0.0	0.1	0.0	0.1
Unsig. Movement Delay, s/veh												
LnGrp Delay(d), s/veh	6.2	0.0	8.4	6.3	0.0	8.4	11.9	0.0	11.6	11.7	0.0	11.8
LnGrp LOS	A		A	A		A	B		B	B		B
Approach Vol, veh/h		421			410			10			32	
Approach Delay, s/veh		8.3			8.3			11.7			11.7	
Approach LOS		A			A			B			B	

Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	5.2	15.7		8.2	5.3	15.6		8.2				
Change Period (Y+Rc), s	5.0	5.0		5.0	5.0	5.0		5.0				
Max Green Setting (Gmax), s	6.0	22.0		22.0	6.0	22.0		22.0				
Max Q Clear Time (g_c+I1), s	2.0	7.3		2.3	2.1	7.1		2.4				
Green Ext Time (p_c), s	0.0	2.2		0.1	0.0	2.2		0.0				

Intersection Summary			
HCM 7th Control Delay, s/veh			8.5
HCM 7th LOS			A

Intersection						
Int Delay, s/veh	0					

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Vol, veh/h	0	420	401	1	0	1
Future Vol, veh/h	0	420	401	1	0	1
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	-	-	-	0	-
Veh in Median Storage, #	-	0	0	-	0	-
Grade, %	-	0	0	-	0	-
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	0	457	436	1	0	1

Major/Minor	Major1		Major2		Minor2	
Conflicting Flow All	437	0	-	0	893	436
Stage 1	-	-	-	-	436	-
Stage 2	-	-	-	-	457	-
Critical Hdwy	4.12	-	-	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	2.218	-	-	-	3.518	3.318
Pot Cap-1 Maneuver	1123	-	-	-	312	620
Stage 1	-	-	-	-	652	-
Stage 2	-	-	-	-	638	-
Platoon blocked, %		-	-	-		
Mov Cap-1 Maneuver	1123	-	-	-	312	620
Mov Cap-2 Maneuver	-	-	-	-	312	-
Stage 1	-	-	-	-	652	-
Stage 2	-	-	-	-	638	-

Approach	EB	WB	SB
HCM Ctrl Dly, s/v	0	0	10.82
HCM LOS			B

Minor Lane/Major Mvmt	EBL	EBT	WBT	WBR	SBLn1
Capacity (veh/h)	1123	-	-	-	620
HCM Lane V/C Ratio	-	-	-	-	0.002
HCM Ctrl Dly (s/v)	0	-	-	-	10.8
HCM Lane LOS	A	-	-	-	B
HCM 95th %tile Q(veh)	0	-	-	-	0

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	55	264	92	64	306	7	33	66	50	17	101	53
Future Volume (veh/h)	55	264	92	64	306	7	33	66	50	17	101	53
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Lane Width Adj.	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Ped-Bike Adj(A_pbT)	0.99		0.95	0.99		0.95	0.98		0.94	0.97		0.94
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1885	1781	1870	1885	1870	1885	1885	1870	1870	1885	1870	1870
Adj Flow Rate, veh/h	57	272	95	66	315	7	34	68	52	18	104	55
Peak Hour Factor	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97
Percent Heavy Veh, %	1	8	2	1	2	1	1	2	2	1	2	2
Cap, veh/h	487	399	140	435	596	13	366	219	167	397	258	136
Arrive On Green	0.06	0.32	0.32	0.07	0.33	0.33	0.23	0.23	0.23	0.23	0.23	0.23
Sat Flow, veh/h	1795	1244	434	1795	1820	40	1208	956	731	1249	1125	595
Grp Volume(v), veh/h	57	0	367	66	0	322	34	0	120	18	0	159
Grp Sat Flow(s),veh/h/ln	1795	0	1678	1795	0	1861	1208	0	1686	1249	0	1721
Q Serve(g_s), s	0.8	0.0	7.4	0.9	0.0	5.5	1.0	0.0	2.3	0.5	0.0	3.1
Cycle Q Clear(g_c), s	0.8	0.0	7.4	0.9	0.0	5.5	4.0	0.0	2.3	2.8	0.0	3.1
Prop In Lane	1.00		0.26	1.00		0.02	1.00		0.43	1.00		0.35
Lane Grp Cap(c), veh/h	487	0	539	435	0	610	366	0	386	397	0	394
V/C Ratio(X)	0.12	0.00	0.68	0.15	0.00	0.53	0.09	0.00	0.31	0.05	0.00	0.40
Avail Cap(c_a), veh/h	749	0	989	823	0	1239	739	0	907	782	0	925
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	8.1	0.0	11.5	8.4	0.0	10.7	14.5	0.0	12.5	13.6	0.0	12.8
Incr Delay (d2), s/veh	0.0	0.0	0.6	0.1	0.0	0.3	0.0	0.0	0.2	0.0	0.0	0.2
Initial Q Delay(d3), s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.2	0.0	2.2	0.3	0.0	1.8	0.2	0.0	0.8	0.1	0.0	1.0
Unsig. Movement Delay, s/veh												
LnGrp Delay(d), s/veh	8.2	0.0	12.1	8.4	0.0	10.9	14.5	0.0	12.7	13.7	0.0	13.0
LnGrp LOS	A		B	A		B	B		B	B		B
Approach Vol, veh/h		424			388			154			177	
Approach Delay, s/veh		11.6			10.5			13.1			13.1	
Approach LOS		B			B			B			B	

Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	7.3	17.8		13.9	7.6	17.5		13.9				
Change Period (Y+Rc), s	5.0	5.0		5.0	5.0	5.0		5.0				
Max Green Setting (Gmax), s	8.0	26.0		21.0	11.0	23.0		21.0				
Max Q Clear Time (g_c+I1), s	2.8	7.5		6.0	2.9	9.4		5.1				
Green Ext Time (p_c), s	0.0	1.2		0.4	0.0	1.3		0.6				

Intersection Summary			
HCM 7th Control Delay, s/veh			11.6
HCM 7th LOS			B

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	22	10	15	149	5	238	2	527	38	248	1423	5
Future Volume (veh/h)	22	10	15	149	5	238	2	527	38	248	1423	5
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Lane Width Adj.	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Ped-Bike Adj(A_pbT)	0.99		0.97	0.98		0.97	1.00		0.98	1.00		0.97
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1786	1786	1786	1758	1786	1786	1098	1730	1786	1772	1758	1786
Adj Flow Rate, veh/h	22	10	15	149	5	238	2	527	38	248	1423	5
Peak Hour Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Percent Heavy Veh, %	1	1	1	3	1	1	50	5	1	2	3	1
Cap, veh/h	163	76	94	347	401	328	3	1833	826	308	3205	11
Arrive On Green	0.22	0.22	0.22	0.22	0.22	0.22	0.00	0.18	0.18	0.09	0.65	0.65
Sat Flow, veh/h	560	337	421	1279	1786	1461	1046	3287	1482	3274	4936	17
Grp Volume(v), veh/h	47	0	0	149	5	238	2	527	38	248	922	506
Grp Sat Flow(s),veh/h/ln	1318	0	0	1279	1786	1461	1046	1643	1482	1637	1600	1754
Q Serve(g_s), s	1.1	0.0	0.0	9.8	0.3	21.1	0.3	19.3	3.0	10.4	19.9	19.9
Cycle Q Clear(g_c), s	3.4	0.0	0.0	13.2	0.3	21.1	0.3	19.3	3.0	10.4	19.9	19.9
Prop In Lane	0.47		0.32	1.00		1.00	1.00		1.00	1.00		0.01
Lane Grp Cap(c), veh/h	333	0	0	347	401	328	3	1833	826	308	2078	1139
V/C Ratio(X)	0.14	0.00	0.00	0.43	0.01	0.73	0.72	0.29	0.05	0.80	0.44	0.44
Avail Cap(c_a), veh/h	436	0	0	448	542	444	56	1833	826	854	2078	1139
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	0.33	0.33	0.33	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	0.00	0.99	0.99	0.99	0.84	0.84	0.84	1.00	1.00	1.00
Uniform Delay (d), s/veh	43.4	0.0	0.0	46.9	42.2	50.3	69.9	33.1	26.5	62.1	12.1	12.1
Incr Delay (d2), s/veh	0.2	0.0	0.0	0.8	0.0	3.8	140.3	0.3	0.1	4.9	0.7	1.3
Initial Q Delay(d3), s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.3	0.0	0.0	4.6	0.1	8.1	0.2	8.6	1.0	4.5	6.9	7.7
Unsig. Movement Delay, s/veh												
LnGrp Delay(d), s/veh	43.6	0.0	0.0	47.8	42.2	54.1	210.2	33.5	26.5	67.0	12.8	13.3
LnGrp LOS	D			D	D	D	F	C	C	E	B	B
Approach Vol, veh/h		47			392			567			1676	
Approach Delay, s/veh		43.6			51.6			33.6			21.0	
Approach LOS		D			D			C			C	

Timer - Assigned Phs	1	2		4	5	6		8
Phs Duration (G+Y+Rc), s	18.7	84.4		36.9	5.9	97.2		36.9
Change Period (Y+Rc), s	5.5	6.3		5.5	5.5	6.3		5.5
Max Green Setting (Gmax), s	36.5	43.7		42.5	7.5	72.7		42.5
Max Q Clear Time (g_c+I1), s	12.4	21.3		5.4	2.3	21.9		23.1
Green Ext Time (p_c), s	0.8	3.4		0.3	0.0	12.9		1.3

Intersection Summary	
HCM 7th Control Delay, s/veh	28.5
HCM 7th LOS	C

Notes
User approved pedestrian interval to be less than phase max green.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations	↰	↑↱		↰	↑↱		↰	↱		↰	↱	
Traffic Volume (veh/h)	38	227	14	8	343	14	41	4	18	6	0	21
Future Volume (veh/h)	38	227	14	8	343	14	41	4	18	6	0	21
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Lane Width Adj.	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Ped-Bike Adj(A_pbT)	0.98		0.94	0.97		0.94	0.95		0.96	0.97		0.93
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1885	1870	1678	1885	1870	1885	1856	1885	1885	1885	1885	1885
Adj Flow Rate, veh/h	43	258	16	9	390	16	47	5	20	7	0	24
Peak Hour Factor	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88
Percent Heavy Veh, %	1	2	15	1	2	1	3	1	1	1	1	1
Cap, veh/h	476	1057	65	508	954	39	505	75	301	516	0	352
Arrive On Green	0.05	0.31	0.31	0.01	0.28	0.28	0.24	0.24	0.24	0.24	0.00	0.24
Sat Flow, veh/h	1795	3386	208	1795	3469	142	1313	318	1273	1359	0	1487
Grp Volume(v), veh/h	43	134	140	9	199	207	47	0	25	7	0	24
Grp Sat Flow(s),veh/h/ln	1795	1777	1818	1795	1777	1834	1313	0	1591	1359	0	1487
Q Serve(g_s), s	0.6	1.9	2.0	0.1	3.1	3.1	1.0	0.0	0.4	0.1	0.0	0.4
Cycle Q Clear(g_c), s	0.6	1.9	2.0	0.1	3.1	3.1	1.4	0.0	0.4	0.6	0.0	0.4
Prop In Lane	1.00		0.11	1.00		0.08	1.00		0.80	1.00		1.00
Lane Grp Cap(c), veh/h	476	555	567	508	489	504	505	0	377	516	0	352
V/C Ratio(X)	0.09	0.24	0.25	0.02	0.41	0.41	0.09	0.00	0.07	0.01	0.00	0.07
Avail Cap(c_a), veh/h	861	1093	1118	960	1093	1128	1156	0	1165	1189	0	1089
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	8.1	8.7	8.8	8.7	10.1	10.1	10.7	0.0	10.1	10.3	0.0	10.1
Incr Delay (d2), s/veh	0.0	0.2	0.2	0.0	0.4	0.4	0.0	0.0	0.0	0.0	0.0	0.0
Initial Q Delay(d3), s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.2	0.6	0.6	0.0	1.0	1.0	0.2	0.0	0.1	0.0	0.0	0.1
Unsig. Movement Delay, s/veh												
LnGrp Delay(d), s/veh	8.2	8.9	8.9	8.7	10.5	10.5	10.7	0.0	10.1	10.3	0.0	10.1
LnGrp LOS	A	A	A	A	B	B	B		B	B		B
Approach Vol, veh/h		317			415			72			31	
Approach Delay, s/veh		8.8			10.5			10.5			10.2	
Approach LOS		A			B			B			B	
Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	6.7	14.4		13.1	5.4	15.7		13.1				
Change Period (Y+Rc), s	5.0	5.0		5.0	5.0	5.0		5.0				
Max Green Setting (Gmax), s	9.0	21.0		25.0	9.0	21.0		25.0				
Max Q Clear Time (g_c+I1), s	2.6	5.1		3.4	2.1	4.0		2.6				
Green Ext Time (p_c), s	0.0	1.7		0.1	0.0	1.1		0.1				
Intersection Summary												
HCM 7th Control Delay, s/veh			9.8									
HCM 7th LOS			A									

Intersection	
Int Delay, s/veh	0

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Lane Configurations						
Traffic Vol, veh/h	249	2	0	363	0	2
Future Vol, veh/h	249	2	0	363	0	2
Conflicting Peds, #/hr	0	10	10	0	10	10
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	-	-	-	-	0
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	271	2	0	395	0	2

Major/Minor	Major1		Major2		Minor1	
Conflicting Flow All	0	0	-	-	-	156
Stage 1	-	-	-	-	-	-
Stage 2	-	-	-	-	-	-
Critical Hdwy	-	-	-	-	-	6.94
Critical Hdwy Stg 1	-	-	-	-	-	-
Critical Hdwy Stg 2	-	-	-	-	-	-
Follow-up Hdwy	-	-	-	-	-	3.32
Pot Cap-1 Maneuver	-	-	0	-	0	861
Stage 1	-	-	0	-	0	-
Stage 2	-	-	0	-	0	-
Platoon blocked, %	-	-		-		
Mov Cap-1 Maneuver	-	-	-	-	-	845
Mov Cap-2 Maneuver	-	-	-	-	-	-
Stage 1	-	-	-	-	-	-
Stage 2	-	-	-	-	-	-

Approach	EB	WB	NB
HCM Ctrl Dly, s/v	0	0	9.27
HCM LOS			A

Minor Lane/Major Mvmt	NBLn1	EBT	EBR	WBT
Capacity (veh/h)	845	-	-	-
HCM Lane V/C Ratio	0.003	-	-	-
HCM Ctrl Dly (s/v)	9.3	-	-	-
HCM Lane LOS	A	-	-	-
HCM 95th %tile Q(veh)	0	-	-	-

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	28	179	30	8	250	110	38	121	8	96	139	57
Future Volume (veh/h)	28	179	30	8	250	110	38	121	8	96	139	57
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Lane Width Adj.	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Ped-Bike Adj(A_pbT)	0.98		0.94	0.98		0.94	0.98		0.96	0.98		0.96
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1841	1841	1885	1885	1856	1885	1885	1885	1707	1885	1885	1885
Adj Flow Rate, veh/h	31	199	33	9	278	122	42	134	9	107	154	63
Peak Hour Factor	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90
Percent Heavy Veh, %	4	4	1	1	3	1	1	1	13	1	1	1
Cap, veh/h	343	746	121	409	534	226	388	331	22	469	286	117
Arrive On Green	0.04	0.25	0.25	0.01	0.23	0.23	0.05	0.19	0.19	0.08	0.23	0.23
Sat Flow, veh/h	1753	2984	484	1795	2362	1000	1795	1741	117	1795	1256	514
Grp Volume(v), veh/h	31	115	117	9	205	195	42	0	143	107	0	217
Grp Sat Flow(s),veh/h/ln	1753	1749	1719	1795	1763	1599	1795	0	1858	1795	0	1770
Q Serve(g_s), s	0.6	2.3	2.4	0.2	4.4	4.6	0.8	0.0	2.9	2.0	0.0	4.6
Cycle Q Clear(g_c), s	0.6	2.3	2.4	0.2	4.4	4.6	0.8	0.0	2.9	2.0	0.0	4.6
Prop In Lane	1.00		0.28	1.00		0.63	1.00		0.06	1.00		0.29
Lane Grp Cap(c), veh/h	343	437	430	409	398	361	388	0	353	469	0	403
V/C Ratio(X)	0.09	0.26	0.27	0.02	0.51	0.54	0.11	0.00	0.41	0.23	0.00	0.54
Avail Cap(c_a), veh/h	728	853	838	763	778	705	556	0	949	652	0	986
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	12.2	13.0	13.0	12.6	14.6	14.7	13.0	0.0	15.3	12.2	0.0	14.6
Incr Delay (d2), s/veh	0.0	0.1	0.1	0.0	0.4	0.5	0.0	0.0	0.3	0.1	0.0	0.4
Initial Q Delay(d3), s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.2	0.8	0.8	0.1	1.5	1.5	0.3	0.0	1.1	0.7	0.0	1.7
Unsig. Movement Delay, s/veh												
LnGrp Delay(d), s/veh	12.2	13.1	13.1	12.6	15.0	15.2	13.0	0.0	15.6	12.3	0.0	15.0
LnGrp LOS	B	B	B	B	B	B	B		B	B		B
Approach Vol, veh/h		263			409			185			324	
Approach Delay, s/veh		13.0			15.0			15.0			14.1	
Approach LOS		B			B			B			B	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	6.5	14.7	8.6	13.2	5.5	15.8	7.0	14.8
Change Period (Y+Rc), s	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0
Max Green Setting (Gmax), s	11.0	19.0	8.0	22.0	9.0	21.0	6.0	24.0
Max Q Clear Time (g_c+I1), s	2.6	6.6	4.0	4.9	2.2	4.4	2.8	6.6
Green Ext Time (p_c), s	0.0	1.3	0.0	0.4	0.0	0.7	0.0	0.8

Intersection Summary	
HCM 7th Control Delay, s/veh	14.3
HCM 7th LOS	B

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	124	186	139	259	181	38	44	410	170	91	1272	155
Future Volume (veh/h)	124	186	139	259	181	38	44	410	170	91	1272	155
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Lane Width Adj.	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Ped-Bike Adj(A_pbT)	1.00		0.97	1.00		0.98	1.00		0.95	1.00		0.97
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1772	1744	1772	1786	1772	1716	1772	1758	1716	1786	1772	1758
Adj Flow Rate, veh/h	124	186	139	259	181	38	44	410	170	91	1272	155
Peak Hour Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Percent Heavy Veh, %	2	4	2	1	2	6	2	3	6	1	2	3
Cap, veh/h	146	271	226	237	369	295	56	549	228	634	1715	737
Arrive On Green	0.09	0.16	0.16	0.14	0.21	0.21	0.03	0.16	0.16	0.12	0.17	0.17
Sat Flow, veh/h	1688	1744	1458	1701	1772	1418	1688	3340	1386	1701	3367	1447
Grp Volume(v), veh/h	124	186	139	259	181	38	44	410	170	91	1272	155
Grp Sat Flow(s),veh/h/ln	1688	1744	1458	1701	1772	1418	1688	1670	1386	1701	1683	1447
Q Serve(g_s), s	10.1	14.1	12.5	19.5	12.6	1.3	3.6	16.4	12.0	6.7	50.3	12.9
Cycle Q Clear(g_c), s	10.1	14.1	12.5	19.5	12.6	1.3	3.6	16.4	12.0	6.7	50.3	12.9
Prop In Lane	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	146	271	226	237	369	295	56	549	228	634	1715	737
V/C Ratio(X)	0.85	0.69	0.61	1.09	0.49	0.13	0.79	0.75	0.75	0.14	0.74	0.21
Avail Cap(c_a), veh/h	175	492	411	237	563	451	175	923	383	634	1715	737
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.33	0.33	0.33
Upstream Filter(I)	1.00	1.00	1.00	0.97	0.97	0.97	1.00	1.00	1.00	0.89	0.89	0.89
Uniform Delay (d), s/veh	63.1	55.9	55.2	60.3	48.9	8.1	67.2	55.7	29.9	41.5	49.5	33.9
Incr Delay (d2), s/veh	27.1	3.1	2.7	84.7	1.0	0.2	21.1	9.0	19.8	0.1	2.6	0.6
Initial Q Delay(d3), s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	5.5	6.5	4.8	14.1	5.8	1.1	1.9	7.5	5.3	2.9	23.3	5.2
Unsig. Movement Delay, s/veh												
LnGrp Delay(d), s/veh	90.2	59.0	57.9	145.0	49.9	8.3	88.3	64.7	49.7	41.5	52.1	34.5
LnGrp LOS	F	E	E	F	D	A	F	E	D	D	D	C
Approach Vol, veh/h		449			478			624			1518	
Approach Delay, s/veh		67.3			98.1			62.3			49.7	
Approach LOS		E			F			E			D	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	58.5	29.3	25.0	27.2	10.1	77.6	17.6	34.6
Change Period (Y+Rc), s	6.3	* 6.3	5.5	5.5	5.5	6.3	5.5	5.5
Max Green Setting (Gmax), s	19.5	* 39	19.5	39.5	14.5	43.7	14.5	44.5
Max Q Clear Time (g_c+I1), s	8.7	18.4	21.5	16.1	5.6	52.3	12.1	14.6
Green Ext Time (p_c), s	0.1	3.0	0.0	1.5	0.0	0.0	0.1	1.2

Intersection Summary	
HCM 7th Control Delay, s/veh	62.3
HCM 7th LOS	E

Notes

* HCM 7th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	8	428	2	4	417	5	5	0	5	13	2	19
Future Volume (veh/h)	8	428	2	4	417	5	5	0	5	13	2	19
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Lane Width Adj.	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Ped-Bike Adj(A_pbT)	1.00		0.96	1.00		0.96	0.97		0.95	0.97		0.95
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1707	1841	1885	1885	1885	1885	1885	1885	1604	1781	1885	1737
Adj Flow Rate, veh/h	8	437	2	4	426	5	5	0	5	13	2	19
Peak Hour Factor	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98
Percent Heavy Veh, %	13	4	1	1	1	1	1	1	20	8	1	11
Cap, veh/h	414	673	3	418	673	8	384	0	173	390	17	159
Arrive On Green	0.01	0.37	0.37	0.01	0.36	0.36	0.11	0.00	0.11	0.11	0.11	0.11
Sat Flow, veh/h	1626	1830	8	1795	1858	22	1356	0	1525	1298	148	1405
Grp Volume(v), veh/h	8	0	439	4	0	431	5	0	5	13	0	21
Grp Sat Flow(s),veh/h/ln	1626	0	1839	1795	0	1880	1356	0	1525	1298	0	1553
Q Serve(g_s), s	0.1	0.0	5.8	0.0	0.0	5.5	0.1	0.0	0.1	0.3	0.0	0.4
Cycle Q Clear(g_c), s	0.1	0.0	5.8	0.0	0.0	5.5	0.5	0.0	0.1	0.3	0.0	0.4
Prop In Lane	1.00		0.00	1.00		0.01	1.00		1.00	1.00		0.90
Lane Grp Cap(c), veh/h	414	0	676	418	0	681	384	0	173	390	0	176
V/C Ratio(X)	0.02	0.00	0.65	0.01	0.00	0.63	0.01	0.00	0.03	0.03	0.00	0.12
Avail Cap(c_a), veh/h	731	0	1385	777	0	1417	1252	0	1149	1220	0	1170
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	6.3	0.0	7.7	6.4	0.0	7.7	11.8	0.0	11.5	11.7	0.0	11.6
Incr Delay (d2), s/veh	0.0	0.0	1.1	0.0	0.0	1.0	0.0	0.0	0.0	0.0	0.0	0.1
Initial Q Delay(d3), s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.0	0.0	1.5	0.0	0.0	1.5	0.0	0.0	0.0	0.1	0.0	0.1
Unsig. Movement Delay, s/veh												
LnGrp Delay(d), s/veh	6.3	0.0	8.7	6.4	0.0	8.7	11.8	0.0	11.5	11.7	0.0	11.7
LnGrp LOS	A		A	A		A	B		B	B		B
Approach Vol, veh/h		447			435			10			34	
Approach Delay, s/veh		8.7			8.7			11.7			11.7	
Approach LOS		A			A			B			B	
Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	5.2	15.7		8.3	5.3	15.6		8.3				
Change Period (Y+Rc), s	5.0	5.0		5.0	5.0	5.0		5.0				
Max Green Setting (Gmax), s	6.0	22.0		22.0	6.0	22.0		22.0				
Max Q Clear Time (g_c+I1), s	2.0	7.8		2.4	2.1	7.5		2.5				
Green Ext Time (p_c), s	0.0	2.3		0.1	0.0	2.3		0.0				
Intersection Summary												
HCM 7th Control Delay, s/veh			8.8									
HCM 7th LOS			A									

Intersection	
Int Delay, s/veh	0

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Vol, veh/h	0	446	426	1	0	1
Future Vol, veh/h	0	446	426	1	0	1
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	-	-	-	0	-
Veh in Median Storage, #	-	0	0	-	0	-
Grade, %	-	0	0	-	0	-
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	0	485	463	1	0	1

Major/Minor	Major1		Major2		Minor2	
Conflicting Flow All	464	0	-	0	948	464
Stage 1	-	-	-	-	464	-
Stage 2	-	-	-	-	485	-
Critical Hdwy	4.12	-	-	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	2.218	-	-	-	3.518	3.318
Pot Cap-1 Maneuver	1097	-	-	-	289	598
Stage 1	-	-	-	-	633	-
Stage 2	-	-	-	-	619	-
Platoon blocked, %		-	-	-		
Mov Cap-1 Maneuver	1097	-	-	-	289	598
Mov Cap-2 Maneuver	-	-	-	-	289	-
Stage 1	-	-	-	-	633	-
Stage 2	-	-	-	-	619	-

Approach	EB	WB	SB
HCM Ctrl Dly, s/v	0	0	11.03
HCM LOS			B

Minor Lane/Major Mvmt	EBL	EBT	WBT	WBR	SBLn1
Capacity (veh/h)	1097	-	-	-	598
HCM Lane V/C Ratio	-	-	-	-	0.002
HCM Ctrl Dly (s/v)	0	-	-	-	11
HCM Lane LOS	A	-	-	-	B
HCM 95th %tile Q(veh)	0	-	-	-	0

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations	↰	↱		↰	↱		↰	↱		↰	↱	
Traffic Volume (veh/h)	58	280	98	68	325	7	35	70	53	18	107	56
Future Volume (veh/h)	58	280	98	68	325	7	35	70	53	18	107	56
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Lane Width Adj.	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Ped-Bike Adj(A_pbT)	0.99		0.95	0.99		0.95	0.98		0.94	0.97		0.94
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1885	1781	1870	1885	1870	1885	1885	1870	1870	1885	1870	1870
Adj Flow Rate, veh/h	60	289	101	70	335	7	36	72	55	19	110	58
Peak Hour Factor	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97
Percent Heavy Veh, %	1	8	2	1	2	1	1	2	2	1	2	2
Cap, veh/h	484	411	144	429	615	13	351	216	165	383	255	134
Arrive On Green	0.06	0.33	0.33	0.07	0.34	0.34	0.23	0.23	0.23	0.23	0.23	0.23
Sat Flow, veh/h	1795	1244	435	1795	1823	38	1199	956	730	1241	1126	594
Grp Volume(v), veh/h	60	0	390	70	0	342	36	0	127	19	0	168
Grp Sat Flow(s),veh/h/ln	1795	0	1679	1795	0	1861	1199	0	1686	1241	0	1720
Q Serve(g_s), s	0.8	0.0	8.1	1.0	0.0	6.0	1.1	0.0	2.5	0.5	0.0	3.3
Cycle Q Clear(g_c), s	0.8	0.0	8.1	1.0	0.0	6.0	4.4	0.0	2.5	3.0	0.0	3.3
Prop In Lane	1.00		0.26	1.00		0.02	1.00		0.43	1.00		0.35
Lane Grp Cap(c), veh/h	484	0	555	429	0	628	351	0	381	383	0	389
V/C Ratio(X)	0.12	0.00	0.70	0.16	0.00	0.54	0.10	0.00	0.33	0.05	0.00	0.43
Avail Cap(c_a), veh/h	735	0	967	802	0	1212	710	0	887	755	0	905
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	8.1	0.0	11.7	8.4	0.0	10.7	15.1	0.0	12.9	14.2	0.0	13.2
Incr Delay (d2), s/veh	0.0	0.0	0.6	0.1	0.0	0.3	0.0	0.0	0.2	0.0	0.0	0.3
Initial Q Delay(d3), s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.2	0.0	2.4	0.3	0.0	1.9	0.3	0.0	0.8	0.1	0.0	1.1
Unsig. Movement Delay, s/veh												
LnGrp Delay(d), s/veh	8.1	0.0	12.3	8.5	0.0	11.0	15.2	0.0	13.1	14.2	0.0	13.5
LnGrp LOS	A		B	A		B	B		B	B		B
Approach Vol, veh/h		450			412			163			187	
Approach Delay, s/veh		11.7			10.6			13.6			13.6	
Approach LOS		B			B			B			B	
Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	7.4	18.5		14.0	7.7	18.2		14.0				
Change Period (Y+Rc), s	5.0	5.0		5.0	5.0	5.0		5.0				
Max Green Setting (Gmax), s	8.0	26.0		21.0	11.0	23.0		21.0				
Max Q Clear Time (g_c+I1), s	2.8	8.0		6.4	3.0	10.1		5.3				
Green Ext Time (p_c), s	0.0	1.2		0.5	0.0	1.4		0.6				
Intersection Summary												
HCM 7th Control Delay, s/veh			11.9									
HCM 7th LOS			B									

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	22	10	15	152	5	244	2	533	45	265	1440	5
Future Volume (veh/h)	22	10	15	152	5	244	2	533	45	265	1440	5
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Lane Width Adj.	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Ped-Bike Adj(A_pbT)	0.99		0.97	0.98		0.97	1.00		0.98	1.00		0.97
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1786	1786	1786	1758	1786	1786	1098	1730	1786	1772	1758	1786
Adj Flow Rate, veh/h	22	10	15	152	5	244	2	533	45	265	1440	5
Peak Hour Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Percent Heavy Veh, %	1	1	1	3	1	1	50	5	1	2	3	1
Cap, veh/h	165	76	95	350	405	331	3	1808	815	326	3194	11
Arrive On Green	0.23	0.23	0.23	0.23	0.23	0.23	0.00	0.18	0.18	0.10	0.65	0.65
Sat Flow, veh/h	559	335	419	1279	1786	1462	1046	3287	1481	3274	4936	17
Grp Volume(v), veh/h	47	0	0	152	5	244	2	533	45	265	933	512
Grp Sat Flow(s),veh/h/ln	1314	0	0	1279	1786	1462	1046	1643	1481	1637	1600	1754
Q Serve(g_s), s	1.1	0.0	0.0	10.1	0.3	21.7	0.3	19.6	3.5	11.1	20.4	20.4
Cycle Q Clear(g_c), s	3.4	0.0	0.0	13.5	0.3	21.7	0.3	19.6	3.5	11.1	20.4	20.4
Prop In Lane	0.47		0.32	1.00		1.00	1.00		1.00	1.00		0.01
Lane Grp Cap(c), veh/h	336	0	0	350	405	331	3	1808	815	326	2070	1135
V/C Ratio(X)	0.14	0.00	0.00	0.43	0.01	0.74	0.72	0.29	0.06	0.81	0.45	0.45
Avail Cap(c_a), veh/h	435	0	0	449	542	444	56	1808	815	854	2070	1135
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	0.33	0.33	0.33	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	0.00	0.99	0.99	0.99	0.84	0.84	0.84	1.00	1.00	1.00
Uniform Delay (d), s/veh	43.1	0.0	0.0	46.8	42.0	50.2	69.9	33.8	27.2	61.7	12.3	12.3
Incr Delay (d2), s/veh	0.2	0.0	0.0	0.8	0.0	4.3	140.3	0.3	0.1	4.9	0.7	1.3
Initial Q Delay(d3), s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.3	0.0	0.0	4.7	0.1	8.4	0.2	8.7	1.3	4.8	7.0	7.9
Unsig. Movement Delay, s/veh												
LnGrp Delay(d), s/veh	43.3	0.0	0.0	47.6	42.0	54.5	210.2	34.2	27.3	66.6	13.0	13.6
LnGrp LOS	D			D	D	D	F	C	C	E	B	B
Approach Vol, veh/h		47			401			580			1710	
Approach Delay, s/veh		43.3			51.7			34.2			21.5	
Approach LOS		D			D			C			C	

Timer - Assigned Phs	1	2		4	5	6		8
Phs Duration (G+Y+Rc), s	19.5	83.3		37.2	5.9	96.9		37.2
Change Period (Y+Rc), s	5.5	6.3		5.5	5.5	6.3		5.5
Max Green Setting (Gmax), s	36.5	43.7		42.5	7.5	72.7		42.5
Max Q Clear Time (g_c+I1), s	13.1	21.6		5.4	2.3	22.4		23.7
Green Ext Time (p_c), s	0.9	3.5		0.3	0.0	13.2		1.3

Intersection Summary	
HCM 7th Control Delay, s/veh	29.0
HCM 7th LOS	C

Notes
User approved pedestrian interval to be less than phase max green.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	38	251	14	8	352	14	41	4	18	6	0	21
Future Volume (veh/h)	38	251	14	8	352	14	41	4	18	6	0	21
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Lane Width Adj.	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Ped-Bike Adj(A_pbT)	0.98		0.94	0.97		0.94	0.95		0.96	0.97		0.93
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1885	1870	1678	1885	1870	1885	1856	1885	1885	1885	1885	1885
Adj Flow Rate, veh/h	43	285	16	9	400	16	47	5	20	7	0	24
Peak Hour Factor	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88
Percent Heavy Veh, %	1	2	15	1	2	1	3	1	1	1	1	1
Cap, veh/h	472	1065	59	496	956	38	505	75	301	516	0	352
Arrive On Green	0.05	0.31	0.31	0.01	0.28	0.28	0.24	0.24	0.24	0.24	0.00	0.24
Sat Flow, veh/h	1795	3409	190	1795	3473	138	1313	318	1273	1359	0	1487
Grp Volume(v), veh/h	43	148	153	9	204	212	47	0	25	7	0	24
Grp Sat Flow(s),veh/h/ln	1795	1777	1823	1795	1777	1835	1313	0	1591	1359	0	1487
Q Serve(g_s), s	0.6	2.1	2.2	0.1	3.2	3.2	1.0	0.0	0.4	0.1	0.0	0.4
Cycle Q Clear(g_c), s	0.6	2.1	2.2	0.1	3.2	3.2	1.4	0.0	0.4	0.6	0.0	0.4
Prop In Lane	1.00		0.10	1.00		0.08	1.00		0.80	1.00		1.00
Lane Grp Cap(c), veh/h	472	555	569	496	489	505	505	0	377	516	0	352
V/C Ratio(X)	0.09	0.27	0.27	0.02	0.42	0.42	0.09	0.00	0.07	0.01	0.00	0.07
Avail Cap(c_a), veh/h	857	1092	1120	947	1092	1128	1155	0	1164	1188	0	1088
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	8.2	8.8	8.8	8.7	10.1	10.1	10.7	0.0	10.1	10.3	0.0	10.1
Incr Delay (d2), s/veh	0.0	0.2	0.2	0.0	0.4	0.4	0.0	0.0	0.0	0.0	0.0	0.0
Initial Q Delay(d3), s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.2	0.6	0.6	0.0	1.0	1.0	0.2	0.0	0.1	0.0	0.0	0.1
Unsig. Movement Delay, s/veh												
LnGrp Delay(d), s/veh	8.2	9.0	9.0	8.7	10.6	10.6	10.7	0.0	10.1	10.3	0.0	10.1
LnGrp LOS	A	A	A	A	B	B	B		B	B		B
Approach Vol, veh/h		344			425			72			31	
Approach Delay, s/veh		8.9			10.5			10.5			10.2	
Approach LOS		A			B			B			B	
Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	6.7	14.4		13.1	5.4	15.7		13.1				
Change Period (Y+Rc), s	5.0	5.0		5.0	5.0	5.0		5.0				
Max Green Setting (Gmax), s	9.0	21.0		25.0	9.0	21.0		25.0				
Max Q Clear Time (g_c+I1), s	2.6	5.2		3.4	2.1	4.2		2.6				
Green Ext Time (p_c), s	0.0	1.8		0.1	0.0	1.3		0.1				
Intersection Summary												
HCM 7th Control Delay, s/veh			9.9									
HCM 7th LOS			A									

Intersection						
Int Delay, s/veh	0.4					

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Lane Configurations						
Traffic Vol, veh/h	249	26	8	363	11	6
Future Vol, veh/h	249	26	8	363	11	6
Conflicting Peds, #/hr	0	10	10	0	10	10
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	-	50	-	0	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	271	28	9	395	12	7

Major/Minor	Major1		Major2		Minor1	
Conflicting Flow All	0	0	309	0	519	169
Stage 1	-	-	-	-	295	-
Stage 2	-	-	-	-	225	-
Critical Hdwy	-	-	4.14	-	6.84	6.94
Critical Hdwy Stg 1	-	-	-	-	5.84	-
Critical Hdwy Stg 2	-	-	-	-	5.84	-
Follow-up Hdwy	-	-	2.22	-	3.52	3.32
Pot Cap-1 Maneuver	-	-	1248	-	486	845
Stage 1	-	-	-	-	730	-
Stage 2	-	-	-	-	791	-
Platoon blocked, %	-	-		-		
Mov Cap-1 Maneuver	-	-	1237	-	474	829
Mov Cap-2 Maneuver	-	-	-	-	557	-
Stage 1	-	-	-	-	723	-
Stage 2	-	-	-	-	778	-

Approach	EB	WB	NB
HCM Ctrl Dly, s/v	0	0.17	10.88
HCM LOS			B

Minor Lane/Major Mvmt	NBLn1	EBT	EBR	WBL	WBT
Capacity (veh/h)	630	-	-	1237	-
HCM Lane V/C Ratio	0.029	-	-	0.007	-
HCM Ctrl Dly (s/v)	10.9	-	-	7.9	-
HCM Lane LOS	B	-	-	A	-
HCM 95th %tile Q(veh)	0.1	-	-	0	-

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	30	181	30	8	254	110	38	122	8	96	144	61
Future Volume (veh/h)	30	181	30	8	254	110	38	122	8	96	144	61
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Lane Width Adj.	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Ped-Bike Adj(A_pbT)	0.98		0.94	0.98		0.94	0.99		0.96	0.98		0.96
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1841	1841	1885	1885	1856	1885	1885	1885	1707	1885	1885	1885
Adj Flow Rate, veh/h	33	201	33	9	282	122	42	136	9	107	160	68
Peak Hour Factor	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90
Percent Heavy Veh, %	4	4	1	1	3	1	1	1	13	1	1	1
Cap, veh/h	342	752	121	408	536	224	383	340	22	471	288	123
Arrive On Green	0.04	0.25	0.25	0.01	0.23	0.23	0.05	0.19	0.19	0.08	0.23	0.23
Sat Flow, veh/h	1753	2989	480	1795	2373	991	1795	1743	115	1795	1240	527
Grp Volume(v), veh/h	33	116	118	9	207	197	42	0	145	107	0	228
Grp Sat Flow(s),veh/h/ln	1753	1749	1720	1795	1763	1601	1795	0	1859	1795	0	1767
Q Serve(g_s), s	0.6	2.3	2.4	0.2	4.5	4.7	0.8	0.0	3.0	2.0	0.0	5.0
Cycle Q Clear(g_c), s	0.6	2.3	2.4	0.2	4.5	4.7	0.8	0.0	3.0	2.0	0.0	5.0
Prop In Lane	1.00		0.28	1.00		0.62	1.00		0.06	1.00		0.30
Lane Grp Cap(c), veh/h	342	440	433	408	398	361	383	0	362	471	0	411
V/C Ratio(X)	0.10	0.26	0.27	0.02	0.52	0.55	0.11	0.00	0.40	0.23	0.00	0.56
Avail Cap(c_a), veh/h	718	842	828	757	768	697	548	0	937	650	0	972
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	12.3	13.1	13.1	12.8	14.8	14.9	13.0	0.0	15.3	12.2	0.0	14.8
Incr Delay (d2), s/veh	0.0	0.1	0.1	0.0	0.4	0.5	0.0	0.0	0.3	0.1	0.0	0.4
Initial Q Delay(d3), s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.2	0.8	0.8	0.1	1.6	1.5	0.3	0.0	1.1	0.7	0.0	1.8
Unsig. Movement Delay, s/veh												
LnGrp Delay(d), s/veh	12.4	13.2	13.2	12.8	15.2	15.4	13.1	0.0	15.6	12.3	0.0	15.2
LnGrp LOS	B	B	B	B	B	B	B		B	B		B
Approach Vol, veh/h		267			413			187			335	
Approach Delay, s/veh		13.1			15.2			15.0			14.3	
Approach LOS		B			B			B			B	
Timer - Assigned Phs	1	2	3	4	5	6	7	8				
Phs Duration (G+Y+Rc), s	6.6	14.8	8.6	13.5	5.5	16.0	7.0	15.1				
Change Period (Y+Rc), s	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0				
Max Green Setting (Gmax), s	11.0	19.0	8.0	22.0	9.0	21.0	6.0	24.0				
Max Q Clear Time (g_c+I1), s	2.6	6.7	4.0	5.0	2.2	4.4	2.8	7.0				
Green Ext Time (p_c), s	0.0	1.3	0.0	0.4	0.0	0.8	0.0	0.8				
Intersection Summary												
HCM 7th Control Delay, s/veh			14.5									
HCM 7th LOS			B									

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations	↰	↑	↱	↰	↑	↱	↰	↑↑	↱	↰	↑↑	↱
Traffic Volume (veh/h)	124	188	139	262	182	44	44	417	178	108	1275	155
Future Volume (veh/h)	124	188	139	262	182	44	44	417	178	108	1275	155
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Lane Width Adj.	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Ped-Bike Adj(A_pbT)	1.00		0.97	1.00		0.98	1.00		0.95	1.00		0.97
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1772	1744	1772	1786	1772	1716	1772	1758	1716	1786	1772	1758
Adj Flow Rate, veh/h	124	188	139	262	182	44	44	417	178	108	1275	155
Peak Hour Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Percent Heavy Veh, %	2	4	2	1	2	6	2	3	6	1	2	3
Cap, veh/h	146	272	228	237	370	296	56	555	231	629	1712	736
Arrive On Green	0.09	0.16	0.16	0.14	0.21	0.21	0.03	0.17	0.17	0.12	0.17	0.17
Sat Flow, veh/h	1688	1744	1458	1701	1772	1418	1688	3340	1387	1701	3367	1447
Grp Volume(v), veh/h	124	188	139	262	182	44	44	417	178	108	1275	155
Grp Sat Flow(s),veh/h/ln	1688	1744	1458	1701	1772	1418	1688	1670	1387	1701	1683	1447
Q Serve(g_s), s	10.1	14.3	12.4	19.5	12.7	1.5	3.6	16.7	12.6	8.0	50.4	12.9
Cycle Q Clear(g_c), s	10.1	14.3	12.4	19.5	12.7	1.5	3.6	16.7	12.6	8.0	50.4	12.9
Prop In Lane	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	146	272	228	237	370	296	56	555	231	629	1712	736
V/C Ratio(X)	0.85	0.69	0.61	1.11	0.49	0.15	0.79	0.75	0.77	0.17	0.74	0.21
Avail Cap(c_a), veh/h	175	492	411	237	563	451	175	923	383	629	1712	736
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.33	0.33	0.33
Upstream Filter(I)	1.00	1.00	1.00	0.97	0.97	0.97	1.00	1.00	1.00	0.89	0.89	0.89
Uniform Delay (d), s/veh	63.1	55.9	55.1	60.3	48.8	8.2	67.2	55.6	29.9	42.2	49.6	34.0
Incr Delay (d2), s/veh	27.1	3.1	2.6	88.9	1.0	0.2	21.1	9.0	21.8	0.1	2.7	0.6
Initial Q Delay(d3), s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	5.5	6.5	4.8	14.3	5.8	1.3	1.9	7.6	5.7	3.5	23.4	5.2
Unsig. Movement Delay, s/veh												
LnGrp Delay(d), s/veh	90.2	59.0	57.7	149.2	49.8	8.4	88.3	64.6	51.7	42.4	52.3	34.6
LnGrp LOS	F	E	E	F	D	A	F	E	D	D	D	C
Approach Vol, veh/h		451			488			639			1538	
Approach Delay, s/veh		67.2			99.4			62.6			49.8	
Approach LOS		E			F			E			D	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	58.0	29.6	25.0	27.4	10.1	77.5	17.6	34.8
Change Period (Y+Rc), s	6.3	* 6.3	5.5	5.5	5.5	6.3	5.5	5.5
Max Green Setting (Gmax), s	19.5	* 39	19.5	39.5	14.5	43.7	14.5	44.5
Max Q Clear Time (g_c+I1), s	10.0	18.7	21.5	16.3	5.6	52.4	12.1	14.7
Green Ext Time (p_c), s	0.1	3.1	0.0	1.5	0.0	0.0	0.1	1.3

Intersection Summary	
HCM 7th Control Delay, s/veh	62.7
HCM 7th LOS	E

Notes

* HCM 7th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	8	455	2	4	427	5	5	0	5	13	2	19
Future Volume (veh/h)	8	455	2	4	427	5	5	0	5	13	2	19
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Lane Width Adj.	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Ped-Bike Adj(A_pbT)	1.00		0.96	1.00		0.96	0.97		0.95	0.97		0.95
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1707	1841	1885	1885	1885	1885	1885	1885	1604	1781	1885	1737
Adj Flow Rate, veh/h	8	464	2	4	436	5	5	0	5	13	2	19
Peak Hour Factor	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98
Percent Heavy Veh, %	13	4	1	1	1	1	1	1	20	8	1	11
Cap, veh/h	414	689	3	405	690	8	379	0	172	385	17	159
Arrive On Green	0.01	0.38	0.38	0.01	0.37	0.37	0.11	0.00	0.11	0.11	0.11	0.11
Sat Flow, veh/h	1626	1831	8	1795	1859	21	1356	0	1524	1297	148	1405
Grp Volume(v), veh/h	8	0	466	4	0	441	5	0	5	13	0	21
Grp Sat Flow(s),veh/h/ln	1626	0	1839	1795	0	1880	1356	0	1524	1297	0	1553
Q Serve(g_s), s	0.1	0.0	6.3	0.0	0.0	5.7	0.1	0.0	0.1	0.3	0.0	0.4
Cycle Q Clear(g_c), s	0.1	0.0	6.3	0.0	0.0	5.7	0.5	0.0	0.1	0.4	0.0	0.4
Prop In Lane	1.00		0.00	1.00		0.01	1.00		1.00	1.00		0.90
Lane Grp Cap(c), veh/h	414	0	692	405	0	698	379	0	172	385	0	176
V/C Ratio(X)	0.02	0.00	0.67	0.01	0.00	0.63	0.01	0.00	0.03	0.03	0.00	0.12
Avail Cap(c_a), veh/h	725	0	1362	758	0	1393	1231	0	1129	1200	0	1150
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	6.3	0.0	7.7	6.4	0.0	7.7	12.0	0.0	11.7	11.9	0.0	11.8
Incr Delay (d2), s/veh	0.0	0.0	1.1	0.0	0.0	1.0	0.0	0.0	0.0	0.0	0.0	0.1
Initial Q Delay(d3), s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.0	0.0	1.6	0.0	0.0	1.5	0.0	0.0	0.0	0.1	0.0	0.1
Unsig. Movement Delay, s/veh												
LnGrp Delay(d), s/veh	6.3	0.0	8.9	6.4	0.0	8.6	12.1	0.0	11.7	11.9	0.0	12.0
LnGrp LOS	A		A	A		A	B		B	B		B
Approach Vol, veh/h		474			445			10			34	
Approach Delay, s/veh		8.8			8.6			11.9			11.9	
Approach LOS		A			A			B			B	
Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	5.2	16.2		8.4	5.3	16.0		8.4				
Change Period (Y+Rc), s	5.0	5.0		5.0	5.0	5.0		5.0				
Max Green Setting (Gmax), s	6.0	22.0		22.0	6.0	22.0		22.0				
Max Q Clear Time (g_c+I1), s	2.0	8.3		2.4	2.1	7.7		2.5				
Green Ext Time (p_c), s	0.0	2.5		0.1	0.0	2.4		0.0				
Intersection Summary												
HCM 7th Control Delay, s/veh			8.9									
HCM 7th LOS			A									

Intersection	
Int Delay, s/veh	0.5

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Traffic Vol, veh/h	27	446	426	17	4	11
Future Vol, veh/h	27	446	426	17	4	11
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	-	-	-	0	-
Veh in Median Storage, #	-	0	0	-	0	-
Grade, %	-	0	0	-	0	-
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	29	485	463	18	4	12

Major/Minor	Major1		Major2		Minor2	
Conflicting Flow All	482	0	-	0	1016	472
Stage 1	-	-	-	-	472	-
Stage 2	-	-	-	-	543	-
Critical Hdwy	4.12	-	-	-	6.42	6.22
Critical Hdwy Stg 1	-	-	-	-	5.42	-
Critical Hdwy Stg 2	-	-	-	-	5.42	-
Follow-up Hdwy	2.218	-	-	-	3.518	3.318
Pot Cap-1 Maneuver	1081	-	-	-	264	592
Stage 1	-	-	-	-	627	-
Stage 2	-	-	-	-	582	-
Platoon blocked, %		-	-	-		
Mov Cap-1 Maneuver	1081	-	-	-	254	592
Mov Cap-2 Maneuver	-	-	-	-	254	-
Stage 1	-	-	-	-	604	-
Stage 2	-	-	-	-	582	-

Approach	EB		WB		SB	
HCM Ctrl Dly, s/v	0.48		0		13.56	
HCM LOS					B	

Minor Lane/Major Mvmt	EBL	EBT	WBT	WBR	SBLn1
Capacity (veh/h)	103	-	-	-	437
HCM Lane V/C Ratio	0.027	-	-	-	0.037
HCM Ctrl Dly (s/v)	8.4	0	-	-	13.6
HCM Lane LOS	A	A	-	-	B
HCM 95th %tile Q(veh)	0.1	-	-	-	0.1

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	59	283	98	68	336	7	35	70	53	18	107	61
Future Volume (veh/h)	59	283	98	68	336	7	35	70	53	18	107	61
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Lane Width Adj.	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Ped-Bike Adj(A_pbT)	0.99		0.95	0.99		0.95	0.98		0.94	0.97		0.94
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1885	1781	1870	1885	1870	1885	1885	1870	1870	1885	1870	1870
Adj Flow Rate, veh/h	61	292	101	70	346	7	36	72	55	19	110	63
Peak Hour Factor	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97
Percent Heavy Veh, %	1	8	2	1	2	1	1	2	2	1	2	2
Cap, veh/h	480	418	145	430	622	13	344	215	164	380	245	141
Arrive On Green	0.06	0.33	0.33	0.07	0.34	0.34	0.23	0.23	0.23	0.23	0.23	0.23
Sat Flow, veh/h	1795	1248	432	1795	1825	37	1194	956	730	1241	1089	624
Grp Volume(v), veh/h	61	0	393	70	0	353	36	0	127	19	0	173
Grp Sat Flow(s),veh/h/ln	1795	0	1679	1795	0	1862	1194	0	1686	1241	0	1713
Q Serve(g_s), s	0.9	0.0	8.2	1.0	0.0	6.2	1.1	0.0	2.5	0.5	0.0	3.5
Cycle Q Clear(g_c), s	0.9	0.0	8.2	1.0	0.0	6.2	4.6	0.0	2.5	3.1	0.0	3.5
Prop In Lane	1.00		0.26	1.00		0.02	1.00		0.43	1.00		0.36
Lane Grp Cap(c), veh/h	480	0	562	430	0	635	344	0	380	380	0	386
V/C Ratio(X)	0.13	0.00	0.70	0.16	0.00	0.56	0.10	0.00	0.33	0.05	0.00	0.45
Avail Cap(c_a), veh/h	593	0	1251	532	0	1387	668	0	837	717	0	850
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	8.1	0.0	11.6	8.4	0.0	10.8	15.4	0.0	13.1	14.4	0.0	13.4
Incr Delay (d2), s/veh	0.0	0.0	0.6	0.1	0.0	0.3	0.0	0.0	0.2	0.0	0.0	0.3
Initial Q Delay(d3), s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.2	0.0	2.4	0.3	0.0	2.0	0.3	0.0	0.8	0.1	0.0	1.2
Unsig. Movement Delay, s/veh												
LnGrp Delay(d), s/veh	8.1	0.0	12.2	8.4	0.0	11.1	15.5	0.0	13.3	14.4	0.0	13.7
LnGrp LOS	A		B	A		B	B		B	B		B
Approach Vol, veh/h		454			423			163			192	
Approach Delay, s/veh		11.7			10.6			13.8			13.8	
Approach LOS		B			B			B			B	

Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	7.5	18.7		14.1	7.7	18.5		14.1				
Change Period (Y+Rc), s	5.0	5.0		5.0	5.0	5.0		5.0				
Max Green Setting (Gmax), s	5.0	30.0		20.0	5.0	30.0		20.0				
Max Q Clear Time (g_c+I1), s	2.9	8.2		6.6	3.0	10.2		5.5				
Green Ext Time (p_c), s	0.0	1.3		0.4	0.0	1.6		0.6				

Intersection Summary			
HCM 7th Control Delay, s/veh			11.9
HCM 7th LOS			B

Intersection	
Int Delay, s/veh	3.4

Movement	EBL	EBR	NBL	NBT	SBT	SBR
Lane Configurations						
Traffic Vol, veh/h	6	7	21	11	18	16
Future Vol, veh/h	6	7	21	11	18	16
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Stop	Stop	Free	Free	Free	Free
RT Channelized	-	None	-	None	-	None
Storage Length	0	-	-	-	-	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	7	8	23	12	20	17

Major/Minor	Minor2		Major1		Major2	
Conflicting Flow All	86	28	37	0	-	0
Stage 1	28	-	-	-	-	-
Stage 2	58	-	-	-	-	-
Critical Hdwy	6.42	6.22	4.12	-	-	-
Critical Hdwy Stg 1	5.42	-	-	-	-	-
Critical Hdwy Stg 2	5.42	-	-	-	-	-
Follow-up Hdwy	3.518	3.318	2.218	-	-	-
Pot Cap-1 Maneuver	915	1047	1574	-	-	-
Stage 1	994	-	-	-	-	-
Stage 2	965	-	-	-	-	-
Platoon blocked, %				-	-	-
Mov Cap-1 Maneuver	902	1047	1574	-	-	-
Mov Cap-2 Maneuver	902	-	-	-	-	-
Stage 1	980	-	-	-	-	-
Stage 2	965	-	-	-	-	-

Approach	EB		NB		SB	
HCM Ctrl Dly, s/v	8.75		4.8		0	
HCM LOS	A					

Minor Lane/Major Mvmt	NBL	NBT	EBLn1	SBT	SBR
Capacity (veh/h)	1181	-	975	-	-
HCM Lane V/C Ratio	0.015	-	0.014	-	-
HCM Ctrl Dly (s/v)	7.3	0	8.7	-	-
HCM Lane LOS	A	A	A	-	-
HCM 95th %tile Q(veh)	0	-	0	-	-

Intersection	
Int Delay, s/veh	3.5

Movement	EBL	EBR	NBL	NBT	SBT	SBR
Lane Configurations						
Traffic Vol, veh/h	7	7	22	22	8	16
Future Vol, veh/h	7	7	22	22	8	16
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Stop	Stop	Free	Free	Free	Free
RT Channelized	-	None	-	None	-	None
Storage Length	0	-	-	-	-	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	92	92	92	92	92	92
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	8	8	24	24	9	17

Major/Minor	Minor2		Major1		Major2	
Conflicting Flow All	89	17	26	0	-	0
Stage 1	17	-	-	-	-	-
Stage 2	72	-	-	-	-	-
Critical Hdwy	6.42	6.22	4.12	-	-	-
Critical Hdwy Stg 1	5.42	-	-	-	-	-
Critical Hdwy Stg 2	5.42	-	-	-	-	-
Follow-up Hdwy	3.518	3.318	2.218	-	-	-
Pot Cap-1 Maneuver	911	1061	1588	-	-	-
Stage 1	1005	-	-	-	-	-
Stage 2	951	-	-	-	-	-
Platoon blocked, %				-	-	-
Mov Cap-1 Maneuver	898	1061	1588	-	-	-
Mov Cap-2 Maneuver	898	-	-	-	-	-
Stage 1	990	-	-	-	-	-
Stage 2	951	-	-	-	-	-

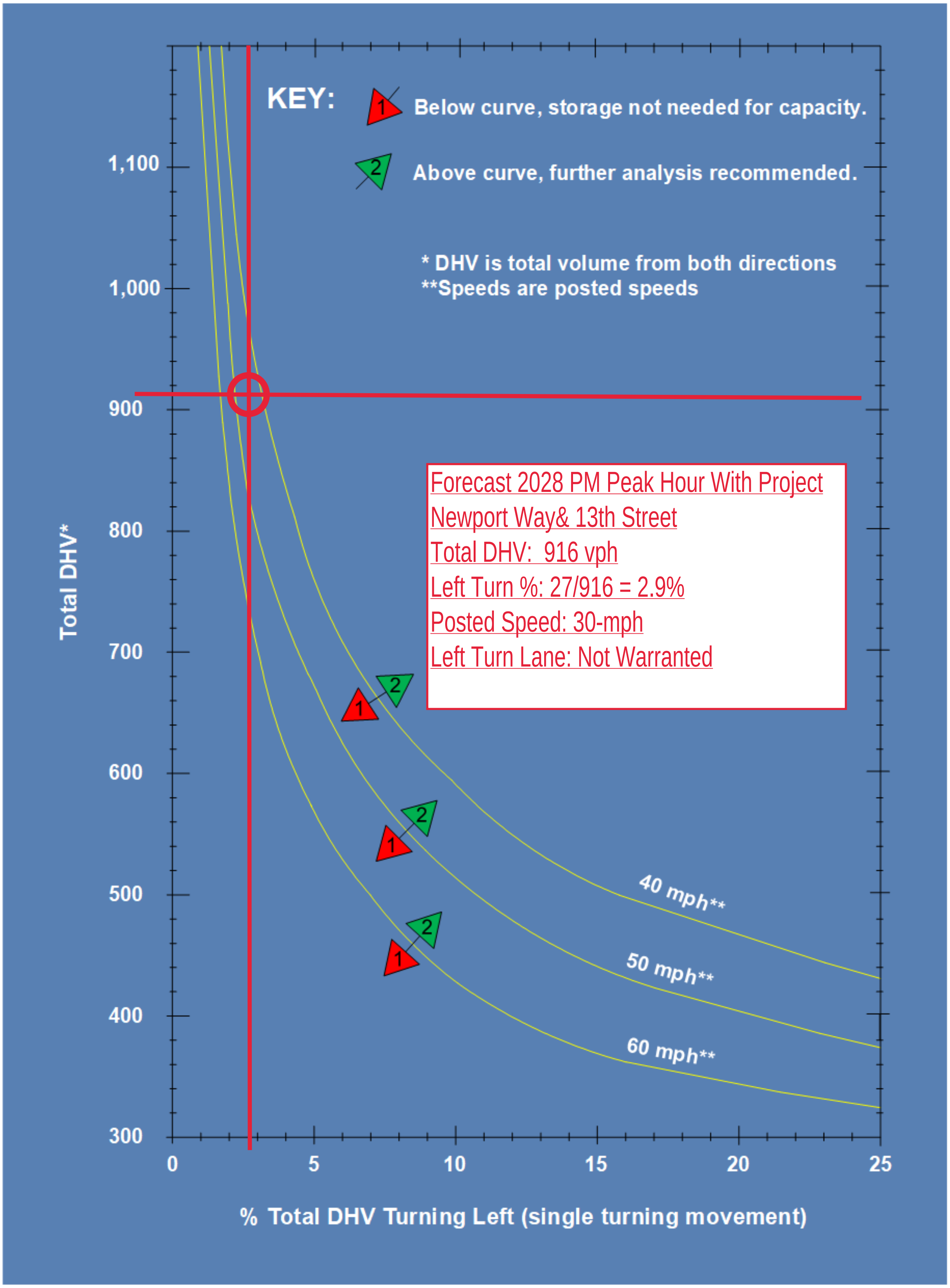
Approach	EB		NB		SB	
HCM Ctrl Dly, s/v	8.76		3.65		0	
HCM LOS	A					

Minor Lane/Major Mvmt	NBL	NBT	EBLn1	SBT	SBR
Capacity (veh/h)	900	-	973	-	-
HCM Lane V/C Ratio	0.015	-	0.016	-	-
HCM Ctrl Dly (s/v)	7.3	0	8.8	-	-
HCM Lane LOS	A	A	A	-	-
HCM 95th %tile Q(veh)	0	-	0	-	-

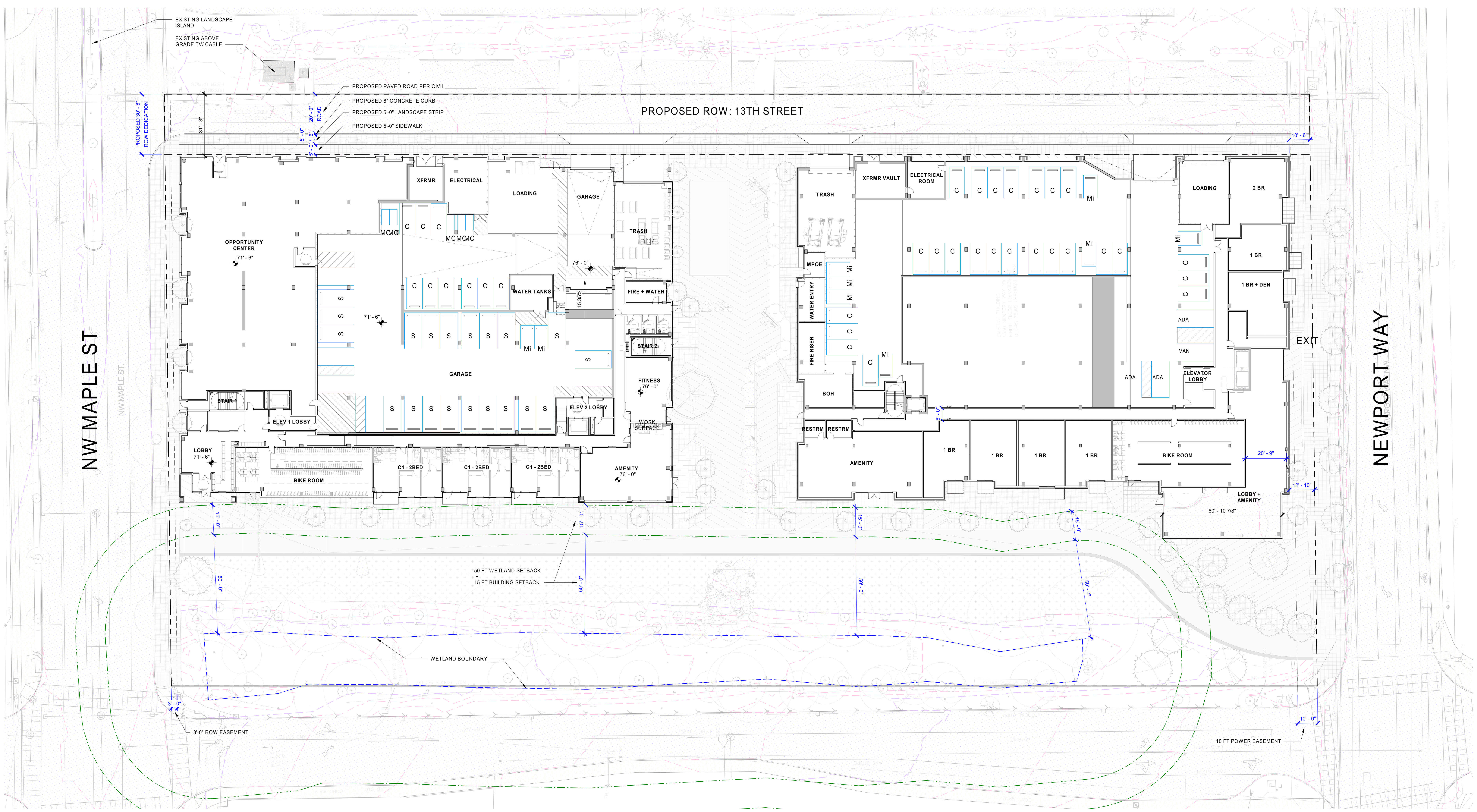
Intersection: 17: Newport Way NW & 13th Street

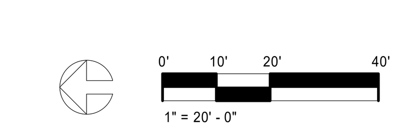
Movement	EB	WB	SB
Directions Served	LT	TR	LR
Maximum Queue (ft)	221	456	70
Average Queue (ft)	38	317	28
95th Queue (ft)	148	701	74
Link Distance (ft)	252	521	175
Upstream Blk Time (%)	0	17	
Queuing Penalty (veh)	1	77	
Storage Bay Dist (ft)			
Storage Blk Time (%)			
Queuing Penalty (veh)			

Exhibit 1310-9 Left-Turn Storage Guidelines: Two-Lane, Unsignalized



PROPOSED NEW ROW IMPACT	WETLAND IMPACT
PROPOSED ROW 30'-6" WIDE = 17,700 SF DEVELOPMENT AREA LOST (10.16%)	50 FT WETLAND SETBACK 38,634 SF DEVELOPMENT AREA LOST (22%) 15 FT BUILDING SETBACK 9,170 SF DEVELOPMENT AREA LOST (5%) **TOTAL WETLAND IMPACT: (27%)**





AREA RESERVED FOR CITY PERMIT STAMP

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Project:
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Issue:

Construction Revision:

Phase:	Client Approval	Quality Assurance
Schematics		
Design Dev.		
Permit Doc.		
Bid Doc.		
Const Doc.		

Drawn By: Author
Project Manager: NW
Principal In Charge: JR

SITE PLAN + L1 PLANS

A1.01

Project Number: 24-027

NOT FOR CONSTRUCTION
100% SCHEMATIC DESIGN 12.20.2024