GEO CONSULTANTS NORTHWEST

January 10, 2022

Taft Development, LLC
6740 SW Raleighwood Way
Portland, Oregon 97225

Attention: Reed Kirk

Subject: Geotechnical Evaluation and Geologic Hazards Study
Ebb Street Lofts - Taft Oregon
GCN Project 1564

This report presents our Geotechnical Evaluation and Geologic Hazards Study for the proposed Ebb Street Lofts development located in Lincoln City (Taft), Oregon. The report summarizes the work accomplished and provides our conclusions and recommendations for site development. Our Report has been prepared in accordance with our proposal dated April 14, 2021.

PROJECT INFORMATION

The nearly flat and level Ebb Street Lofts is planned for development with eight 3-story, residential buildings on the 11 lots. The site is mostly undeveloped, vegetated with brush and trees. One lot (Lot 1) is improved with as a mobile home site. A wetland boundary within the southwest portion of the project renders three of the 11 lots undevelopable.

We were given Architectural Site Plan Review Drawings dated April 2021, a topographic survey of the site, and a wetlands determination report to help us understand the site.

We understand the planned buildings will be three story wood frame construction. Foundations will be slab-on-grade with thickened edges.

The site relative to surrounding features is shown in Figure 1. The preliminary site layout is shown in Figure 2.

SCOPE OF WORK

The purpose of our services is to explore the site and provide recommendations for design and construction. The following describes our specific scope of services:

- Coordinate and manage the field investigation, including utility locates, authorization for site access, access preparation, exploration waste, and scheduling of contractors and GCN staff.
- Observe excavation of 5 test pits to depths up to 13 feet below the existing ground surface using a rubber-tired backhoe.
- Conduct two Dynamic Cone Penetrometer tests to depth up to 16 feet below the ground surface.
- Maintain a log of soil, rock, and groundwater, as encountered, and obtain soil samples to be classified in the field and returned to our lab for further evaluation and testing. We classify soil in general accordance with the Unified Soil Classification System.

- Determine the moisture content of selected soil samples in general accordance with ASTM D2216 and Fines Content in general accordance with ASTM C136.
- Provide a written Geotechnical Report including applicable items detailed in the Lincoln City Code Chapter 17.47. The report will summarize our geologic hazard analysis, preliminary conclusions, and preliminary recommendations that include:
  - A discussion available subsurface soil and groundwater information, geologic maps, geologic hazard maps and other information pertinent to the site, geologic conditions and the seismic setting of the site including general geologic features, tectonic faulting in the area, and seismic design criteria in accordance with the Oregon Structural Specialty Code.
  - Recommendations for site preparation, grading and drainage, materials and compaction criteria for structural fill, and wet-weather earthwork procedures.
  - Recommendations for excavation, utility trenches, backfill materials, and backfill compaction.
  - Recommendations for design and construction of shallow-spread foundations, including allowable design bearing pressures, minimum footing depth and width, lateral resistance to sliding, and estimates of settlement.
  - Geotechnical engineering recommendations for the design and construction of concrete floor slabs, including an anticipated value for subgrade modulus.
  - Geotechnical recommendations for design of the public roads and private asphalt paved spaces on the site including asphalt and base rock thicknesses.
  - A discussion of groundwater conditions on the site and recommendations for subsurface drainage of foundations, floor slabs, and pavement.

SITE CONDITIONS

The site is in an area of a sparsely developed residential area along Ebb Avenue in the Taft neighborhood of Lincoln City. The following paragraphs describe the area geology, surface, and subsurface features.

SITE GEOLOGY

The site is situated several hundred feet inland of the Pacific coastline. Ancient stream and near-beach erosion, and depositional processes contributed primarily to the current surface and subsurface conditions on the site.

Sedimentary deposits in the vicinity of the site are composed silt and fine sand with coarser sand and gravel at depth. The sedimentary deposits are underlain by mudstone and sandstone known as the Nestucca-Hamlet Formation (Eocene).[1]

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[1] *Smith, R.L and Roe, W.P., 2006, Geologic Map of Oregon: Oregon Dept. of Geology and Mineral Industries Geological Map Series, OGCD-7.*

SURFACE CONDITIONS

The approximately 1.21-acre site is located on the west and east side of Ebb Avenue between SW 48th Street and SW 50th Street. The site is mostly undeveloped except for a mobile home located within the north portion of the site. Ground surface elevation of the site is 10 to 15 feet above mean sea level.

SUBSURFACE CONDITIONS

GENERAL

We investigated subsurface conditions of the site on April 24, 2021with five test pits (TP-1 through TP-5) to depths of up to 13 feet below the existing ground surface (bgs).

The approximate locations of the borings are shown on Figure 2. The test pit logs are provided in Attachment A.

We encountered loose to medium dense fine sand in test pits TP-1 through TP-4. We encountered about 10 inches of loose sand fill at the ground surface in TP-5. In test pits TP-2 and TP-4 we encountered an approximate 6-inch-thick layer of black fine sand with trace organics at depths of about 2-1/2 to 3 feet bgs. The moisture content of samples obtained in the sand varied from 18 to 27 percent.

We supplemented our test pit explorations with two Dynamic Cone Penetrometer (DCP) tests to obtain data for liquefaction analysis and soil density correlation. Both DCPs were driven to depths of about 16 feet. Blow counts obtained from the DCP tests indicate that soft rock underlies the fine sand at about 14 feet bgs. We infer that the material is the sedimentary sandstone (bedrock) that underlies the site.

GROUNDWATER

We encountered groundwater at shallow depths in most areas, at depths generally ranging from 2-1/2 to 4 feet.

SEISMIC SETTING

The Coastal area is subject to seismic events stemming from three possible sources: the Cascadia Subduction Zone (CSZ), intraslab faults within the Juan de Fuca Plate, and crustal faults in the North American Plate.

To the south and west, the site is located near several Quaternary crustal faults that are mapped or inferred. The faults within 10 miles of the site are the multiple Siletz Bay Faults about 0.2, 4, 5.8 and 6.3 miles south, the unnamed offshore fault about 7.3 miles south, and the Cascadia fault and fold belt 8.5 miles to the northwest and, the Cape Foulweather fault about 8.6 miles southeast. The USGS considers the faults to be greater than 10,000 years old and are considered inactive.

The contribution of potential earthquake-induced ground motion from all known sources, including the faults described above, are included in probabilistic ground motion maps developed by the USGS.

Based on site explorations and geologic mapping, the site falls into Site Class B for seismic design. Seismic design parameters for the project site are provided in Table 1 on the following page.

TABLE 1 – SEISMIC DESIGN PARAMETERS

MAPPED MAXIMUM CONSIDERED EARTHQUAKE SPECTRAL RESPONSE ACCELERATION PARAMETER (SITE CLASS B)			
LAT	44.929	LON	124.019
S_S		1.35G	
S_1		0.70G	
MAPPED MAXIMUM CONSIDERED EARTHQUAKE SPECTRAL RESPONSE ACCELERATION PARAMETER			
F_A		1.2	
F_V		SEE ASCE 7-16 SECTION 11.4.8*	
S_{MS}		1.62G	
S_{M1}		SEE ASCE 7-16 SECTION 11.4.8*	
DESIGN SPECTRAL RESPONSE ACCELERATION PARAMETER			
S_{DS}		1.08G	
S_{D1}		SEE ASCE 7-16 SECTION 11.4.8*	

- *Factors depend upon structural design.

LIQUEFACTION & LATERAL SPREADING

We completed a screening level liquefaction hazard assessment for the site using the simplified method of Idriss and Boulanger (2008) based on DCP data obtained on the site. The analysis was run using LiqSVs V2.2.1.8. Based on the analysis, there is a potential for liquefaction in the loose sand layer that overlies the bedrock at the site.

The potential liquefaction-induced settlement within this layer is on the order of six to ten inches (15 - 25 cm). Because the groundwater table is near the surface, it is likely that ground disturbance and differential settlement will manifest at the ground surface during a design level CSZ event.

Lateral spreading analysis provides a potential liquefaction-induced lateral displacement value in the range of zero to 2-1/2 feet (0 - 0.8 m) in the direction of Siletz Bay. Results of the analysis are provided in Attachment B.

We provide discussion and potential remedies for these conditions in the Recommendations section of this report.

TSUNAMI INUNDATION

Recent tsunami inundation mapping of the Oregon Coast indicate that the entire site is located in a region susceptible to inundation during Cascadia Subduction Zone events from all earthquake magnitudes. Depending on the magnitude of the event, wave heights at the site extending to elevation 40 feet above means sea level (NAVD88) are expected[2].

GEOLOGIC HAZARDS

Preliminary findings report, there are no known or probable geologic conditions or hazards associated with the site other than liquefaction and tsunami inundation as discussed above. The site is not located within a mapped landslide and there are no surface indicators of ground movement. Ancient stream and near-beach erosion, and depositional processes contributed primarily to the current surface and subsurface conditions on the site. There are no areas that need to be avoided with future planned development.

The beach area is mapped by the Oregon Department of Geology and Mineral Industries (DOGAMI) as having high to moderate risk to geologic hazards related to coastal erosion and landslides. In our opinion, the site is not situated on or near a mapped landslide and the risk of beach erosion and landslide impacts is considered to be very low.

Further study of geologic hazards is beyond the purpose of our work on the site.

CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our review of available geologic information and our soil explorations and testing, it is our opinion that the site can be developed as proposed.

The site is subject to liquefaction on the order of 6 to 10 inches and lateral spreading on the order of 2 feet toward Siletz Bay during a 9.5 MW design seismic event. In general, movement of this magnitude is considered moderate and lightly-loaded buildings can be engineered to mitigate the potential for building collapse during such an event. We recommend the building foundations/floor slab be supported on 2-foot-thick granular pads to reduce differential settlement. The floor slab should be designed as a mat slab to provide additional foundation rigidity and support the building during liquefaction and lateral spreading.

The organic silt layer we encountered at depths of 2.5 to 3 feet will likely be removed for construction of the granular pad and mat foundations. This material should be completely removed in the event that it deeper that found in our explorations.

Our specific recommendations for site development are provided in the following paragraphs.

CONSTRUCTION CONSIDERATIONS

Fine-grained soil near the ground surface is easily disturbed during the wet season. If not carefully executed, site preparation can create extensive soft areas and significant extra cost can result. Earthwork should be planned and executed to minimize subgrade disturbance.

SITE PREPARATION

The existing heavily rooted zone that covers the ground surface should be removed from building and structural areas to the depth of firm compacted fill or native soil. We estimate the stripping

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[2] Tsunami Inundation Map Linc-02, Tsunami Inundation Maps for Lincoln City South, Lincoln County, Oregon, Oregon Department of Geology and Mineral Industries, Plate 1, 2013

depth will generally be 6 to 8 inches. The actual stripping depth should be based on field observations at the time of construction. Stripped material should be transported off site for disposal or used in landscaping areas.

Trees, shrubs, and brush should be removed from all building and paved areas. Root balls should be grubbed out to a depth such that roots greater than ½-inch in diameter is removed. The depth of excavation to remove root balls of trees could exceed 5 feet bgs.

Depending on the methods used, considerable disturbance and loosening of the subgrade could occur during stripping. Soil disturbed during these operations should be removed to expose firm undisturbed subgrade. The resulting excavations should be backfilled with structural fill.

After demolition, grubbing, site cutting, and other site preparation activity the site should be proof rolled with a fully loaded dump truck or similar size, rubber-tire construction equipment. The proof rolling should be observed by a member of our geotechnical staff to identify areas of excessive yielding that will require additional excavation and replacement with granular structural fill.

TRENCH EXCAVATION & BACKFILL

Trench construction and maintenance of safe working conditions, including temporary excavation stability, is the responsibility of the contractor. Local, state, and federal safety codes should be followed.

Trench backfill should consist of well-graded granular material with a maximum particle size of ¾-inch and less than 8 percent by weight passing the U.S. Standard No. 200 Sieve. The material should be free of roots, organic matter, and other unsuitable materials.

Trench backfill in the bedding zone and pipe zone should be placed and compacted in maximum lifts of 6 inches. Trench backfill above the pipe zone should be placed and compacted with a minimum of two lifts. A minimum cover of 3 feet over the top of the pipe should be placed before compacting with a hydraulic plate compactor (hoe-pack).

Trench backfill should be compacted to at least 90 percent of the maximum dry density at depths greater than 4 feet below finished grade and to 95 percent of the maximum dry density within 4 feet of finished grade. Compaction is based on ASTM D698, the standard proctor test.

STRUCTURAL FILL

Near-Surface On- Site Soil: The on-site soil is suitable for use as structural fill provided it can be moisture-conditioned and separated from demolition debris, organics or unspecified materials left over from the previous site activities. Similarly, imported soil should be clean, free from organics and debris.

Imported Granular Material: Imported granular fill material may include sand, gravel, or fragmented rock with a maximum size of 6 inches and with not more than about 8 percent passing the No. 200 sieve (washed analysis). Material satisfying these requirements can usually be placed during periods of wet weather. The first lift of granular fill placed over a fine-grained subgrade should be about 18 inches thick and subsequent lifts about 12 inches thick when using medium- to heavy-weight vibratory rollers. Granular structural fill should be limited to a maximum size of about 1-½ inches when compacted with hand-operated equipment. Lift thicknesses should be limited to less than 8 inches when using hand-operated vibratory plate compactors.

Free-Draining Fill: Free-draining material should have less than 2 percent passing the No. 200 sieve (washed analysis). Examples of materials that would satisfy this requirement include ¾ to ¼ inch, 1½ to ¾ inch, or 3- to 1-inch crushed rock.

Compaction: Fill within building, pavement, and sidewalk areas should be placed as compacted structural fill. Structural fill should be compacted to at least 95 percent of the maximum dry density as determined by ASTM D 698. Fill in non-structural areas may be compacted to 90 percent of ASTM D 698. The moisture content for compaction should be within 3 percent of optimum.

MAT FOUNDATIONS

We recommend the planned floor slabs be stiffened to act as mat slabs that are in turn supported on two-foot-thick granular pads.

The subgrade modulus for mat foundation can be complex for large and heavily loaded buildings and may involve different values for sections of the mat. For the relatively lightly loaded mat that will be constructed on granular pads we recommend using a single modulus equal to 280 pounds per cubic inch. The recommended value considers the mat will be supported on a granular pad that is in turn supported on sand.

A vapor retarder, manufactured for use beneath floor slabs, should be installed above the granular pad in inhabited spaces and spaces that will receive floor coverings. Careful attention should be made during construction to prevent perforating the retarder and to seal edges and utility penetrations. We recommend following ACI 302.1, Chapter 3 with regard to installing a vapor retarder.

GRANULAR PADS

We recommend placing floor slabs and spread footings on 24-inch-thick granular pads that extend 12 inches horizontally beyond the margins of the footings and slabs. The granular pad should be constructed from ¾-inch minus crushed quarry rock or mixed recycled concrete compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 698.

SHALLOW FOUNDATIONS

Isolated column footings that may lie outside the planned mat slab may be founded 2-foot-thick granular pads that are in-turn supported on undisturbed native sand. They should be proportioned for an allowable bearing pressure of 2,500 pounds per square foot (psf). For this allowable bearing pressure, foundations should be at least 12 inches wide. Footing embedment should be as required by the Oregon Structural Specialty Code.

The recommended allowable bearing pressure applies to the total of dead plus long-term live loads. The allowable bearing pressure may be increased by a factor of 1/3 for short-term wind or seismic loads.

The organic silt layer we encountered at depths of 2.5 to 3 feet will likely be removed for construction of the granular pad and mat foundations. This material should be completely removed in the event that it deeper that found in our explorations.

Differential and total settlement of footings under static conditions is anticipated to be less than ½ inch and 1-inch under static conditions, respectively.

LATERAL RESISTANCE

Lateral loads of buildings and retaining walls can be resisted by passive earth pressure on the sides of footings or by friction on the base of the footings but not both. We recommend using the equivalent fluid pressures and coefficients of friction provided in Table 2.

TABLE 2 – LATERAL RESISTANCE FACTORS

SOIL TYPE	EQUIVALENT FLUID PRESSURE (γ – PCF)	FRICTION COEFFICIENT (μ)
ON-SITE SAND AND GRAVEL	300	0.35
IMPORTED CRUSHED ROCK	800	0.45

To develop the tabulated capacity for passive resistance using on-site sand, concrete must be placed directly against the walls of the footing excavations. When using the value for imported crushed rock, the rock should extend a minimum horizontal distance equal to half the footing embedment and should be compacted to not less than 95 percent of the dry density as determined by ASTM D698. Adjacent floor slabs, pavements, or the upper 12-inch depth of adjacent, unpaved areas should not be considered when calculating passive resistance.

SITE DRAINAGE

We recommend that perimeter drains be installed for all fixed buildings. Foundation and crawl space drainage should be sloped to drain to a sump or low point drain outlet. Water should not be allowed to pond within crawl spaces.

Roof drains should be connected to a tightline drainpipe leading to storm drain outlet facilities. Pavement surfaces and open space areas should be sloped such that surface water runoff is collected and routed to suitable discharge points. Ground surfaces adjacent to buildings should be sloped to drain away from the buildings.

PAVEMENT

The project includes an access easement on the site. We recommend that private driveways be constructed with 3-1/2 inches of asphalt concrete over 8 inches of crushed rock base.

The pavement subgrade should be prepared in accordance with the previously described recommendations described in the "Construction Considerations", and "Structural Fill" sections of this report.

It should be expected that pavement construction in wet weather conditions may require over excavation and additional base rock.

Aggregate base should be placed in one lift and compacted to not less than 95 percent of the maximum dry density as determined by ASTM D 698. Aggregate base contaminated with soil during construction should be removed and replaced before paving.

ADDITIONAL SERVICES

Because the future performance and integrity of the structural elements will depend largely on proper site preparation, drainage, fill placement, and construction procedures, monitoring and testing (geotechnical special inspection) by experienced geotechnical personnel should be considered an integral part of the design and construction process. Consequently, we recommend that GCN be retained to provide the following post-investigation services:

- Review construction plans and specifications to verify that our design criteria presented in this report have been properly integrated into the design.
- Observe fill areas and granular pad subgrade both before fill material or crushed rock is placed to verify soil conditions and test the density of fill materials.
- Prepare a post-construction letter-of-compliance summarizing our field observations, inspections, and test results.

LIMITATIONS

This report was prepared for the exclusive use of Taft Development, LLC, and members of the design team for this specific project. It should be made available to prospective contractors for information on the factual data only, and not as a warranty of subsurface conditions, such as those interpreted from the explorations and discussed in this report.

The recommendations contained in this report are preliminary, and are based on information derived through site reconnaissance, subsurface testing, and knowledge of the site area. Variation of conditions within the area and the presence of unsuitable materials are possible and cannot be determined until exposed during construction. Accordingly, GCN's recommendations can be finalized only through GCN's observation of the project's earthwork construction. GCN accepts no responsibility or liability for any party's reliance on GCN's preliminary recommendations.

Unanticipated soil conditions are commonly encountered and cannot fully be determined by exploratory methods. Such unexpected conditions frequently require that additional expenditures be made to attain properly constructed projects. Therefore, a contingency fund is recommended to accommodate the potential for extra costs.

Within the limitations of the scope of work, schedule, and budget, the analyses, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practice in this area at the time this report was prepared. We make no warranty, either express or implied.

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We appreciate the opportunity to be of continued service to you. Please call if you have questions concerning this report or if we can provide additional services.

Sincerely,
GEO Consultants Northwest, Inc.

CERTIFIED
OREGON
DAVID K. RANKIN
E858
ENGINEERING GEOLOGIST

REGISTERED PROFESSIONAL
ENGINEER
85828PE
OREGON
NOVEMBER 8, 2011
TIMOTHY J. NORTH
EXPIRES: 12/31/22

EXPIRES 06/1/2022

David K. Rankin, C.E.G.
Principal

Timothy North, PE
Consulting Geotechnical Engineer

Figures: Figure 1 - Site Vicinity
Figure 2 - Site Layout and Explorations

Attachments: Attachment A – Field Exploration and Laboratory Testing
Attachment B - Liquefaction and Lateral Spread Analysis

IMAGE FROM ACMEMAPPER
EBB AVENUE AND SW 48TH, LINCOLN CITY
LAT 44.930 N LON 124.019 W; T7S, R11W, SEC27

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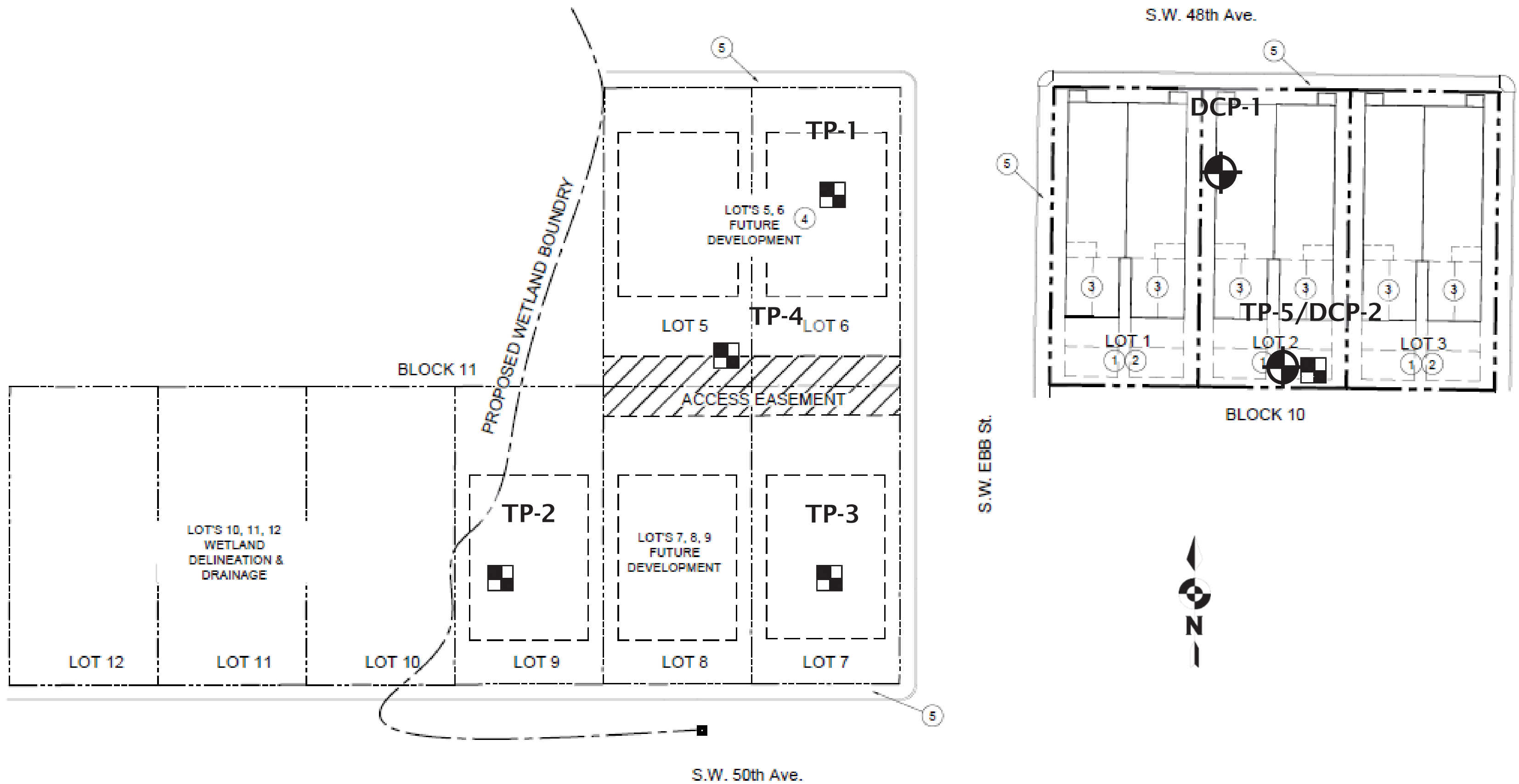
GEO CONSULTANTS NORTHWEST	PROJECT 1564	TAFT DEVELOPMENT EBB AVENUE - LINCOLN CITY
2839 SE Milwaukie Portland, OR 97202	JAN 2022 Drawn By: tac	SITE VICINITY — FIGURE 1



TP-1 TEST PITS EXCAVATED APRIL 26, 2021- LOCATIONS APPROXIMATE

DCP-1 DYNAMIC CONE PENETROMETER (DCP) TEST LOCATIONS APPROXIMATE

SOURCE DRAWING – DAVID BISSETT ARCHITECT PC - APRIL 2021.

GEO CONSULTANTS NORTHWEST	PROJECT 1564	**TAFT DEVELOPMENT** **EBB AVENUE - LINCOLN CITY**
2839 SE Milwaukie Portland, OR 97202	JAN 2022 Drawn By: tac	**SITE LAYOUT & EXPLORATIONS** — **FIGURE 2**

ATTACHMENT A

FIELD EXPLORATION PROCEDURES

LABORATORY TESTING PROCEDURES

KEY TO BORING AND TEST PIT LOGS

TEST PIT LOGS

FIELD EXPLORATION PROCEDURES

GENERAL

We explored subsurface conditions at the site by observing the excavation of five test pits to depths of up to 7 feet below the existing ground surface (bgs) and with dynamic cone penetrometer (DCP) tests to at the approximate locations shown in Figure 2.

The test pits were excavated by Road & Driveway from Newport, Oregon on April 24, 2021.

SOIL SAMPLING

A member of GCN's geotechnical staff observed subsurface explorations to record the soil, rock, and groundwater conditions encountered. Samples obtained in the exploration were sealed in airtight plastic bags to retain moisture and returned to our laboratory for additional examination and testing.

DYNAMIC CONE PENETROMETER

The Dynamic Cone Penetration Test provides a measure of a material's in-situ resistance to penetration. The test is performed by driving a metal cone into the ground by repeated striking it with a 17.6 lb (8 Kg) weight dropped from a distance of 2.26 feet (575 mm). How far the cone moves with each blow is used to determine the soil density and properties at that level.

FIELD CLASSIFICATION

Soil samples were initially classified visually in the field. Consistency, color, relative moisture, degree of plasticity, peculiar odors, and other distinguishing characteristics of the soil samples were noted. The terminology used is described in the key and glossary that follow.

SUMMARY EXPLORATION LOGS

Results from the test pits are shown in the summary exploration logs. The left-hand portion of a log provides our interpretation of the soil encountered, sample depths, and groundwater information. The right-hand, graphic portion of a log shows the results of pocket penetrometer and laboratory testing. Soil descriptions and interfaces between soil types shown in summary logs are interpretive, and actual transitions may be gradual.

LABORATORY TESTING PROCEDURES

Soil samples obtained during field explorations are examined in our laboratory, and representative samples may be selected for further testing. The testing program included visual-manual classification and natural moisture content.

VISUAL-MANUAL CLASSIFICATION

Soil samples are classified in general accordance with guidelines presented in ASTM D2488, *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)*. The physical characteristics of the samples are noted, and the field classifications are modified, where necessary, in accordance with ASTM terminology, though certain terminology that incorporates current local engineering practice may be used. The term which best described the major portion of the sample is used to describe the soil type.

BORING AND TEST PIT LOGS

DISTINCTION BETWEEN FIELD LOGS AND FINAL LOGS

A field log is prepared for exploration by our field representative. The log contains information concerning soil and groundwater encountered, sampling depths, sampler types used and identification of samples selected for laboratory analysis. The final logs presented in this report represent our interpretation of subsurface conditions based on the contents of the field logs, observations made during explorations, and the results of laboratory testing. Our recommendations are based on the contents of the final logs and the information contained therein, and not on the field logs.

SOIL CLASSIFICATION SYSTEM

Soil samples are classified in the field in general accordance with the United Soil Classification System (USCS) presented in ASTM D2488 "Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)." Final logs reflect field soil classifications and laboratory testing results. A summary of the USCS is provided on page 3. Classifications and sampling intervals are shown in the logs.

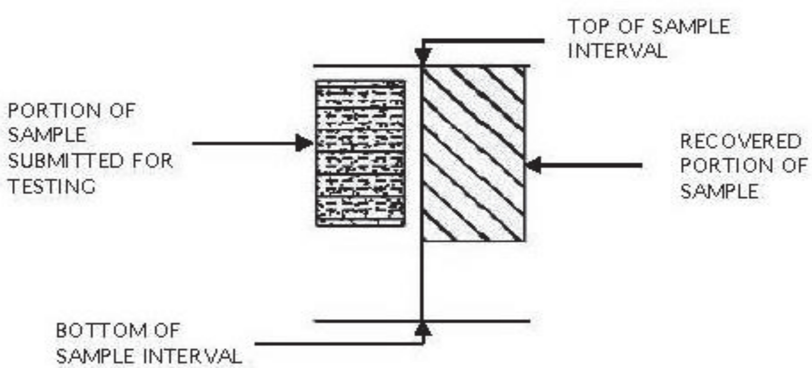
VARIATION OF SOIL BETWEEN EXPLORATIONS

The final logs and related information depict subsurface conditions only at the specific location and on the date(s) indicated. Those using the information contained herein should be aware that soil conditions at other locations or on other dates may differ.

TRANSITION BETWEEN SOIL AND ROCK CLASSIFICATIONS

The lines designating the interface between soil, fill, or rock on the final logs and on the subsurface profiles presented in the report are determined by interpolation and are, therefore, approximate. The transition between the materials may be abrupt or gradual. Only at specific exploration locations should profiles be considered as reasonably accurate and then only to the degree implied by the notes.

BORING LOG SAMPLES



GEO CONSULTANTS NORTHWEST	2019	KEY TO BORING AND TEST PIT LOGS	
2839 SE Milwaukie Avenue Portland, OR 97202	Drawn By: GCN	GENERAL INFORMATION	1/5

EXPLORATION LOG SYMBOLS

- Sample Location with No Sample Recovery
- Sample Location Using Direct Push Sampler (ASTM D 6282)
- Sample Location Using Ring-Lined Barrel Sampler (ASTM D 3550)
- Sample Location Using Split-Barrel Sampler (ASTM D 1586)
- Sample Location Using Thin-Walled Tube Sampler (ASTM D 1587)
- Rock Core Interval
- Grab Sample Location
- Soil Sample Submitted for Chemical Testing
- Soil Sample Submitted for Physical Property Testing
- Water Sample Screened Interval
- Water Sample Submitted for Chemical Testing
- Water Sample Tested in the Field
- Groundwater Level Encountered While Drilling
- Static Groundwater Level
- Perched Groundwater
- Groundwater Level at Time of Sampling

SOIL CHARACTER

Granular Soil		Cohesive Soil		
Density	Standard Penetration Test *	Consistency	Standard Penetration Test*	Unconfined Compressive Strength (tsf)
Very Loose	0 - 4	Very Soft	Less Than 2	Less Than 0.25
Loose	4 - 10	Soft	2 - 4	0.25 - 0.5
Medium Dense	10 - 30	Medium Stiff	4 - 8	0.50 - 1.0
Dense	30 - 50	Stiff	8 - 16	1.0 - 2.0
Very Dense	Greater Than 50	Very Stiff	16 - 32	2.0 - 4.0
Blows Required to Drive a Split-Barrel Sampler 12 inches		Hard	Greater Than 32	Greater Than 4.0

DEFINITIONS AND ABBREVIATIONS

AT	ATTERBERG LIMITS TEST	ND	NON DETECT	PPB	PARTS PER BILLION
BGS	BELOW GROUND SURFACE	NEG	NEGATIVE RESULT	PPM	PARTS PER MILLION
CO	CONSOLIDATION TEST	NS	NO VISIBLE SHEEN	PSF	POUNDS PER SQUARE FOOT
DS	DIRECT SHEAR TEST	OC	ORGANIC CONTENT	RS	SOIL RESISTIVITY TEST
DW	DRY UNIT WEIGHT	P	PUSHED SAMPLE	S4	SUDAN IV SOIL TEST
GS	MECHANICAL GRAIN SIZE TEST	P200	P200 FINES CONTENT TEST	SG	SPECIFIC GRAVITY TEST
HS	HEAVY SHEEN	PCF	POUNDS PER CUBIC FOOT	SPT	STD. PENETRATION TEST
HYD	HYDROMETER TEST	PH	SOIL pH	SS	SLIGHT SHEEN
MC	MOISTURE CONTENT	PID	PHOTOIONIZATION DETECTOR	TO	TOREVANE
MG/KG	MILLIGRAMS PER KILOGRAM	POS	POSITIVE RESULT	TSF	TONS PER SQUARE FOOT
MS	MODERATE SHEEN	PP	POCKET PENETROMETER	UV	ULTRAVIOLET LIGHT TEST

GRAIN SIZE DEFINITIONS			MINOR FRACTIONS IN FINE GRAINED SOIL		GROUNDWATER SEEPAGE	
SAND	FINE	No. 200 to No. 40	No Mention (CLAY, SILT)	< 15 percent	Slow	< 1 gpm
	MEDIUM	No. 40 to No. 10	With Sand, With Gravel	15 to 30 percent	Moderate	1-3 gpm
	COARSE	No. 10 to No. 4	Sandy, Gravelly	30 to 49 percent	Rapid	> 3 gpm
GRAVEL	FINE	No. 4 to 3/4-inch	FIELD MOISTURE OBSERVATION		CAVING	
	COARSE	3/4- to 3-inch	Dry	Absence of moisture, dusty, dry to touch	Minor	
COBBLE		3-inches to 12-inches	Moist	Damp but no visible water.	Moderate	
BOULDER		> 12-inches	Wet	Saturated, below groundwater	Severe	

GEO CONSULTANTS NORTHWEST	2019	KEY TO BORING AND TEST PIT LOGS
2839 SE Milwaukie Avenue Portland, OR 97202	Drawn By: GCN	SYMBOLS AND ABBREVIATIONS — 2/5

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

GEO CONSULTANTS NORTHWEST	2019	KEY TO BORING AND TEST PIT LOGS	
2839 SE Milwaukie Avenue Portland, OR 97202	Drawn By: GCN	SOIL CLASSIFICATION	3/5

ROCK CLASSIFICATION GUIDELINES

HARDNESS		DESCRIPTION
Very soft	(RH-0)	For plastic material only
Soft	(RH-1)	Carved or gouged with a knife
Moderate	(RH-2)	Scratched with a knife
Hard	(RH-3)	Difficult to scratch with a knife
Very hard	(RH-4)	Rock scratches metal; rock cannot be scratched with a knife

STRENGTH	DESCRIPTION
Plastic	Easily deformable with finger pressure
Friable	Crumbles by rubbing with fingers
Weak	Crumbles only under light hammer blows
Moderately Strong	Few heavy hammer blows before breaking
Strong	Withstands few heavy hammer blows and yields large fragments
Very Strong	Withstands many heavy hammer blows, yields dust and small fragments

WEATHERING	DESCRIPTION
Severe	Rock decomposed; thorough discoloration; all fractures extensively coated with clay, oxides, or carbonates.
Moderate	Intense localized discoloration of rock; fracture surfaces coated with weathering minerals.
Little	Slight and intermittent discoloration of rock; few stains on fracture surfaces.
Fresh	Rock unaffected by weathering

FRACTURING	FRACTURE SPACING
Crushed	Less than 5/8 inch to contains clay
Highly Fractured	5/8 inch to 2 inches
Closely Fractured	2 inches to 6 inches
Moderately fractured	6 inches to 1 foot
Little Fractured	1 foot to 4 feet
Massive	Greater than 4 feet

JOINT SPACING	DESCRIPTION
Papery	Less than 1/8 inch
Shaley or Platey	1/8 inch to 5/8 inch
Very Close	5/8 inch to 3 inches
Close	3 inches to 2 feet
Blocky	2 to 4 feet
Massive	Greater than 4 feet

GEO CONSULTANTS NORTHWEST	2019	KEY TO BORING AND TEST PIT LOGS
2839 SE Milwaukie Avenue Portland, OR 97202	Drawn By: GCN	ROCK CLASSIFICATION

GLOSSARY

Alluvial – Made up of or found in the materials that are left by the water of rivers, streams, floods, etc.

Bearing pressure – The total stress transferred from the structure to the foundation, then to the soil below the foundation.

Bulk density (Soil density) – The total mass of water and soil particles contained in a unit volume of soil: lb/ft^3.

Coefficient of active earth pressure – The ratio of the minimum horizontal effective stress of a soil to the vertical effective stress at a single point in a soil mass retained by a retaining wall as the wall moves away from the soil.

Cohesive soil – Clay type soil with angles of internal friction close to zero. Cohesion is the force that holds together molecules or like-particles within a substance.

Colluvium – A loose accumulation of soil and rock fragments deposited through the action of gravity, such as erosion and soil creep.

Differential settlement – The vertical displacement due to settlement of one point in a foundation with respect to another point of the foundation.

Engineered fill – Soil used as fill, such as retaining wall backfill, foundation support, dams, slopes, etc., that are to be placed in accordance with engineered specifications. These specifications may delineate soil grain-size, plasticity, moisture, compaction, angularity, and many other index properties depending on the application.

Excess pore pressure – That increment of pore water pressures greater than hydro-static values, produced by consolidation stresses in compressible materials or by shear strain; excess pore pressure is dissipated during consolidation.

Factor of safety – The ratio of a limiting value of a quantity to the design value of that quantity.

Fines – Material by weight passing the U.S. Standard No. 200 Sieve by washed analysis.

Fluvial – Produced by the action of rivers or streams.

Homogenous soil – A mass of soil where the soil is of one characteristic having the same engineering and index properties.

In situ – Undisturbed, existing field conditions.

Lacustrine – Of a lake, e.g., the depositional environment of a lake.

Liquefaction – The sudden, large decrease of shear strength of cohesionless soil caused by collapse of the soil structure, produced by small shear strains associated with sudden but temporary increase of pore water pressure. Usually a problem in submerged, poorly graded sands within the upper 50 feet of subgrade in earthquake-prone environments.

Maximum dry density – A soil property obtained in the laboratory from a Proctor test. Density of soil at 100% compaction.

Overbank deposit – Sediment that has been deposited on the floodplain of a river or stream by flood waters that have broken through or overtopped the banks.

Permeability – A measure of continuous voids in a soil. The property which allows the flow of water through a soil. See also coefficient of permeability.

Porosity (Pore space) – The ratio of the volume of voids to the total volume: unitless or expressed as a percentage.

Residual soil – Soil that has been formed in place by rock decay.

Shear strength – The maximum shear stress which a soil can sustain under a given set of conditions. For clay, shear strength = cohesion. For sand, shear strength = the product of effective stress and the tangent of the angle of internal friction.

Surcharge – An additional force applied at the exposed upper surface of a restrained soil.

Tuff – An igneous rock (from molten material) that forms from the debris ejected by an explosive volcanic eruption.

Unit weight – The ratio of the total weight of soil to the total volume of a unit of soil: lb/ft^3.

NATURAL MOISTURE CONTENT

Natural moisture content is determined in general accordance with guidelines presented in ASTM D2216, *Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass*. The natural moisture content is the ratio, expressed as a percentage, of the weight of water to the weight of soil particles.

FINES CONTENT

Fines content testing is performed in general accordance with guidelines presented in ASTM D1140, *Standard Test Methods for Determining the Amount of Material Finer than 75-μm (No. 200) Sieve in Soils by Washing.* The fines content is the fraction of soil that passes the U.S. Standard Number 200 Sieve. This sieve differentiates fines (silt and clay) from fine sand. Soil material that remains on the 200 sieve is sand. Material that passes the sieve is fines. The test is used to refine soil type.

DEPTH (feet bg)	G PHIC LOG	USCS SYMBOL	OIL DESCRIPTION	SAMPLE	WATER CONTENT (%)	GROUND WATER	ELD TES NG	TESTING ND LABORATORY DATA
0		SM	Loose, silty, fine SAND; organics (roots); moist. (TOPSOIL)					
		SP	Loose to medium dense, brown, fine SAND; moist to wet.	1	27	▽		
5		SM	Loose to medium dense, dark brown, silty, fine SAND; moist.	2	22			
10				3	23			
15			nd at 13 feet in loose sand. Moderate caving at 2 feet. Severe caving below 5 feet. Groundwater seepage at 2.7 feet.					
20								

Station:	LOGGED Y: avid Rankin
Approximate levation:	xcavator: JD135G
xcavation Started: 4/26/21	xcavation ompleted: 4/26/21

1564 Taft ev - bb Lofts	GEO Consultants Northwest 2839 SE Milwaukie venue Portland OR 202 Tel 503-616-9425 Fax 1-866-293-9037	LOG OF EST IT -1 Page 1 of 1

DEPTH (feet bgs)	G PHIC LOG	USCS SYMBOL	SOIL DESCRIPTION	SAMPLE	WATER CONTENT (%)	GROUND WATER	ELD TES NG	TESTING ND LABORATORY DATA
0		SP	Brown, silty, fine SAND; organics (roots); moist. (TOPSOIL)	1	5			
		SP	Brown-gray, fine SAND; moist to wet.		7			
		SP	Dark black, fine SAND with trace organics; moist.		104			
5		SP	Gray, fine sand; moist.		6			
10				5	6			
				6	0			
						▽		
15			End at 13 feet in sand. Severe caving below 3 feet. Groundwater seepage at 13 feet (approx. 0 gallons/minute).					
20								

Station:	LOGGED Y: David Rankin
Approximate levation:	xcavator: DBEC JD135G
Excavation Started: 4/26/21	xcavation ompleted: 4/26/21

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Portland OR 202
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Fax 1-866-293-9037

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LOG OF EST IT -2

DEPT (feet bgs)	G PHIC LOG	USCS SYMBOL	OIL DESCRIPTION	SAMPLE	WATE CONTENT (%)	G OUND WATE	ELD TES NG	TESTING ND LABORATORY DATA
0		SP	Loose, silty, fine SAND; organics (roots); moist. (TOPSOIL)					
		SP	Loose, brown, fine SAND; moist to wet.	1	22			
		SM	Loose to medium dense, silty, fine SAND; moist.	2	24	∇		
5								
				3	22			
10								
			End at 11 feet in loose sand. Moderate caving at 2 feet. Severe caving below feet.					
15			Perched groundwater seepage at feet. Flow rate estimate 2 to 3 gallons/minute.					
20								

Station:

pproximate Elevation:

Excavation Started: 4/26/21

LOGGED Y: avid Rankin

Excavator: EC JD135G

Excavation ompleted: 4/26/21

1564 Taft ev - Ebb Lofts

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Portland OR 202
Tel 503-616-9425
Fax 1-866-293-9037

LOG OF EST IT
-3

DEPTH (feet bgs)	GRAPHIC LOG	USCS SYMBOL	SOIL DESCRIPTION	SAMPLE	WATER CONTENT (%)	GROUND WATER	ELD TES NG	TESTING ND LABORATORY DATA
0		SP	Brown, silty, fine SAND; organics (roots); moist. (TOPSOIL)					
		SP	rown-gray, fine SAND; moist to wet.	1	23			FC = 0.7%
		SP	Dark black, fine SAND with organics; moist.					
5		SP	Gray, fine SAND; moist.			▽		
10								
			End at 11 feet in sand. Severe caving below 3 feet. Groundwater seepage at 5 feet.	2	20			
15								
20								

Station:

Approximate Elevation:

Excavation Started: 4/26/21

LOGGED Y: David Rankin

Excavator: DBEC JD135G

Excavation ompleted: 4/26/21

1564 Taft Dev - Ebb Lofts

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GEO CONSULTANTS NORTHWEST

LOG OF EST IT
P-4

DEPTH (feet bgs)	USCS SYMBOL	SOIL DESCRIPTION	SAMPLE	WATER CONTENT (%)	GROUND WATER	TESTING AND LABORATORY DATA
0	SP	Loose, brown, silty, fine SAND; organics (roots); moist. (FILL)	1	47		
	SM	Loose, brown, silty, fine SAND; moist.				
	SP	Loose to medium dense, gray, fine SAND; oist. Thin organic seam at 2.2 feet.	2	18		
			3		∇	
			4	25		FC = 3.5%
		End at 11 feet in medium dense sand. Severe caving below 3 feet. Groundwater seepage below 3 feet.				

FIELD TES NG: 5, 6, 6, 6, 6, 5, 4, 3, 4, 3, 4, 7, 8, 9, 11, 10, 10, 12, 12, 15, 18, 19, 15, 16, 13, 9, 5, 2, 1, 1, 2, 4, 3, 3, 3, 3, 3, 4, 10, 19, 9, 9, 14, 9, 9, 11, 19, 2

Station:

Approximate Elevation:

Excavation Started: 4/26/21

LOGGED Y: David Rankin

Excavator: DBEC JD135G

Excavation ompleted: 4/26/21

1564 Taft Dev - Ebb Lofts

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Portland OR 202
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GEO CONSULTANTS NORTHWEST

LOG OF EST IT
P-5

WILDCAT DYNAMIC CONE LOG

GCN
2839 SE Milwaukie Ave.
Portland, Oregon 97202

PROJECT NUMBER: 1564
DATE STARTED: 04-26-2021
DATE COMPLETED: 04-26-2021

HOLE #: DCP-1
CREW: MP/DKR
PROJECT: 2nd Story - Taft
ADDRESS: 0
LOCATION: Lincoln City

SURFACE ELEVATION: Ground surface
WATER ON COMPLETION:
HAMMER WEIGHT: 35 lbs.
CONE AREA: 10 sq. cm

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY SAND & SILT	TESTED CONSISTENCY CLAY
-	1	4.4	•	1	VERY LOOSE	
-	3	13.3	•••	3	VERY LOOSE	
- 1 ft	7	31.1	•••••••••	8	LOOSE	
-	8	35.5	••••••••••	10	LOOSE	
-	7	31.1	•••••••••	8	LOOSE	
- 2 ft	5	22.2	••••••	6	LOOSE	
-	3	13.3	•••	3	VERY LOOSE	
-	2	8.9	••	2	VERY LOOSE	
- 3 ft	2	8.9	••	2	VERY LOOSE	
- 1 m	10	44.4	••••••••••••	12	MEDIUM DENSE	
-	11	42.5	••••••••••••	12	MEDIUM DENSE	
- 4 ft	10	38.6	•••••••••••	11	MEDIUM DENSE	
-	7	27.0	•••••••	7	LOOSE	
-	7	27.0	•••••••	7	LOOSE	
- 5 ft	7	27.0	•••••••	7	LOOSE	
-	7	27.0	•••••••	7	LOOSE	
-	8	30.9	••••••••	8	LOOSE	
- 6 ft	8	30.9	••••••••	8	LOOSE	
-	12	46.3	•••••••••••••	13	MEDIUM DENSE	
- 2 m	13	50.2	••••••••••••••	14	MEDIUM DENSE	
- 7 ft	11	37.6	••••••••••	10	LOOSE	
-	15	51.3	••••••••••••••	14	MEDIUM DENSE	
-	15	51.3	••••••••••••••	14	MEDIUM DENSE	
- 8 ft	16	54.7	•••••••••••••••	15	MEDIUM DENSE	
-	14	47.9	•••••••••••••	13	MEDIUM DENSE	
-	#REF!	#REF!	#REF!	#REF!	#REF!	
- 9 ft	16	54.7	•••••••••••••••	15	MEDIUM DENSE	
-	15	51.3	••••••••••••••	14	MEDIUM DENSE	
-	8	27.4	•••••••	7	LOOSE	
- 3 m 10 ft	2	6.8	•	1	VERY LOOSE	
-	2	6.1	•	1	VERY LOOSE	
-	3	9.2	••	2	VERY LOOSE	
-	3	9.2	••	2	VERY LOOSE	
- 11 ft	10	30.6	••••••••	8	LOOSE	
-	13	39.8	•••••••••••	11	MEDIUM DENSE	
-	6	18.4	•••••	5	LOOSE	
- 12 ft	5	15.3	••••	4	VERY LOOSE	
-	6	18.4	•••••	5	LOOSE	
-	3	9.2	••	2	VERY LOOSE	
- 4 m 13 ft	6	18.4	•••••	5	LOOSE	

HOLE #: DCP-1

WILDCAT DYNAMIC CONE LOG

PROJECT: 2nd Story - Taft

PROJECT NUMBER: 1564

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY SAND & SILT	TESTED CONSISTENCY CLAY
-	12	33.2	•••••••••	9	LOOSE	
-	16	44.3	••••••••••••	12	MEDIUM DENSE	
- 14 ft	16	44.3	••••••••••••	12	MEDIUM DENSE	
-						
-						
- 15 ft						
-						
-						
- 16 ft						
- 5 m						
-						
- 17 ft						
-						
-						
- 18 ft						
-						
-						
- 19 ft						
-						
- 6 m						
- 20 ft						
-						
-						
- 21 ft						
-						
-						
- 22 ft						
-						
-						
- 7 m 23 ft						
-						
-						
- 24 ft						
-						
-						
- 25 ft						
-						
-						
- 26 ft						
- 8 m						
-						
- 27 ft						
-						
-						
- 28 ft						
-						
-						
- 29 ft						
-						
- 9 m						

ATTACHMENT B

LIQUEFACTION AND LATERAL SPREADING ANALYSIS

GEO Consultants Northwest, Inc.
2839 SE Milwaukie Avenue
Portland< Oregon 97202

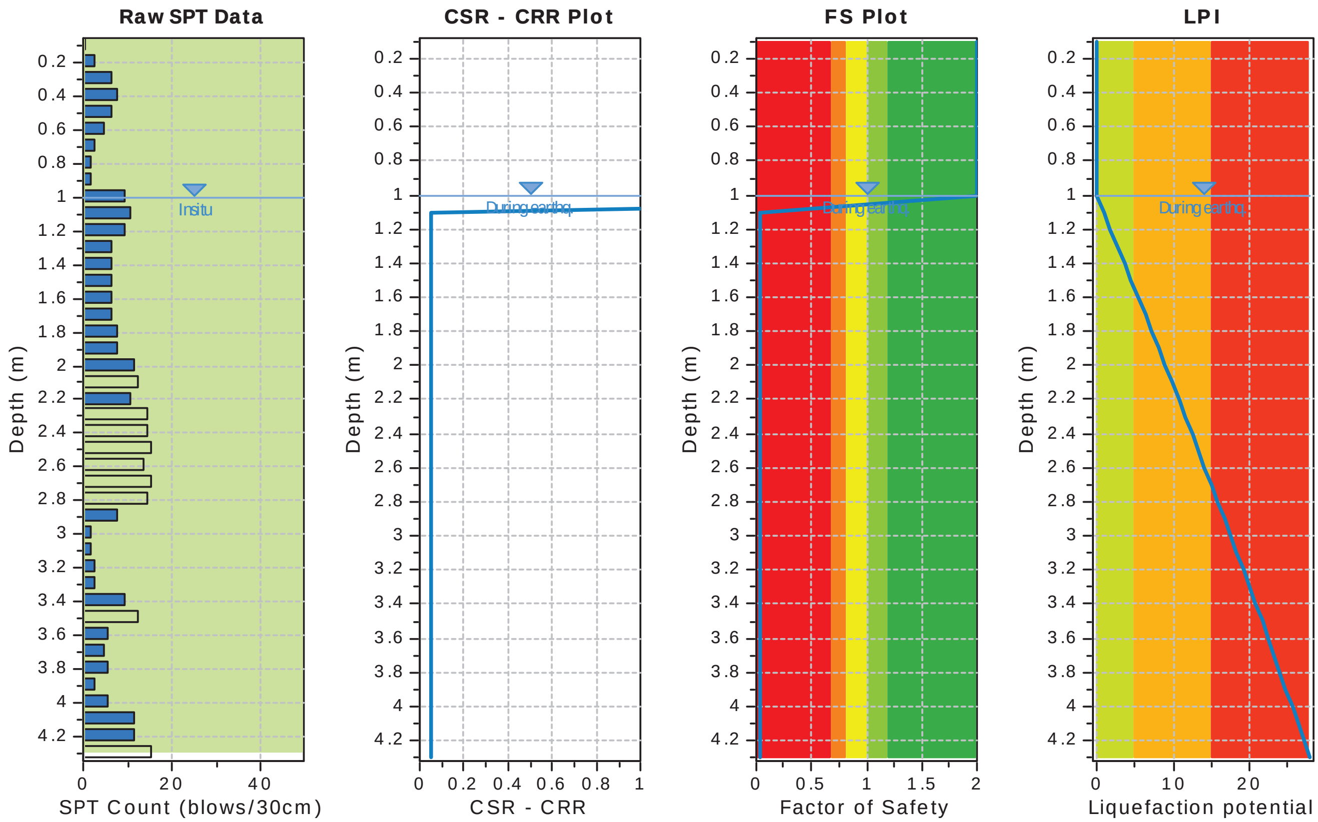
SPT BASED LIQUEFACTION ANALYSIS REPORT

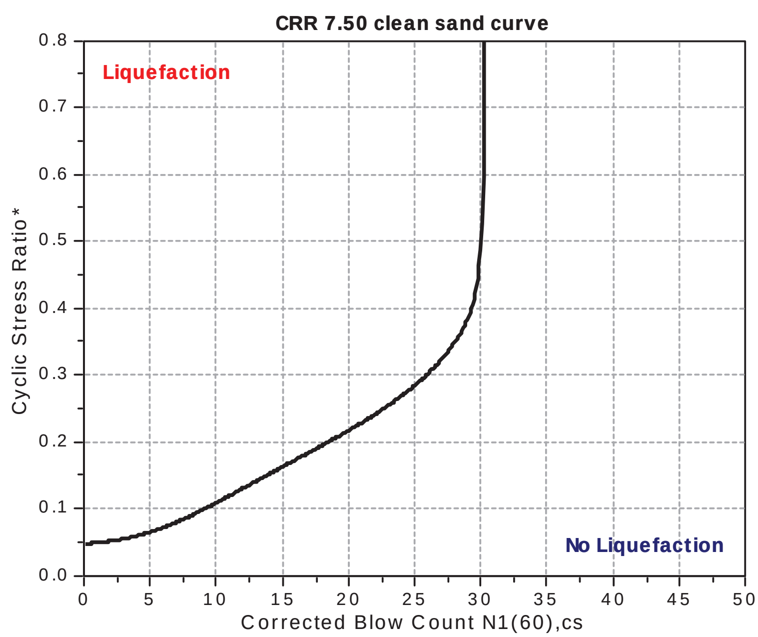
Project title : **SPT Name: DCP 1**

Location :

:: Input parameters and analysis properties ::

Analysis method:	NCEER 1998	G.W.T. (in-situ):	1.00 m
Fines correction method:	NCEER 1998	G.W.T. (earthq.):	1.00 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	9.50
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.80 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		

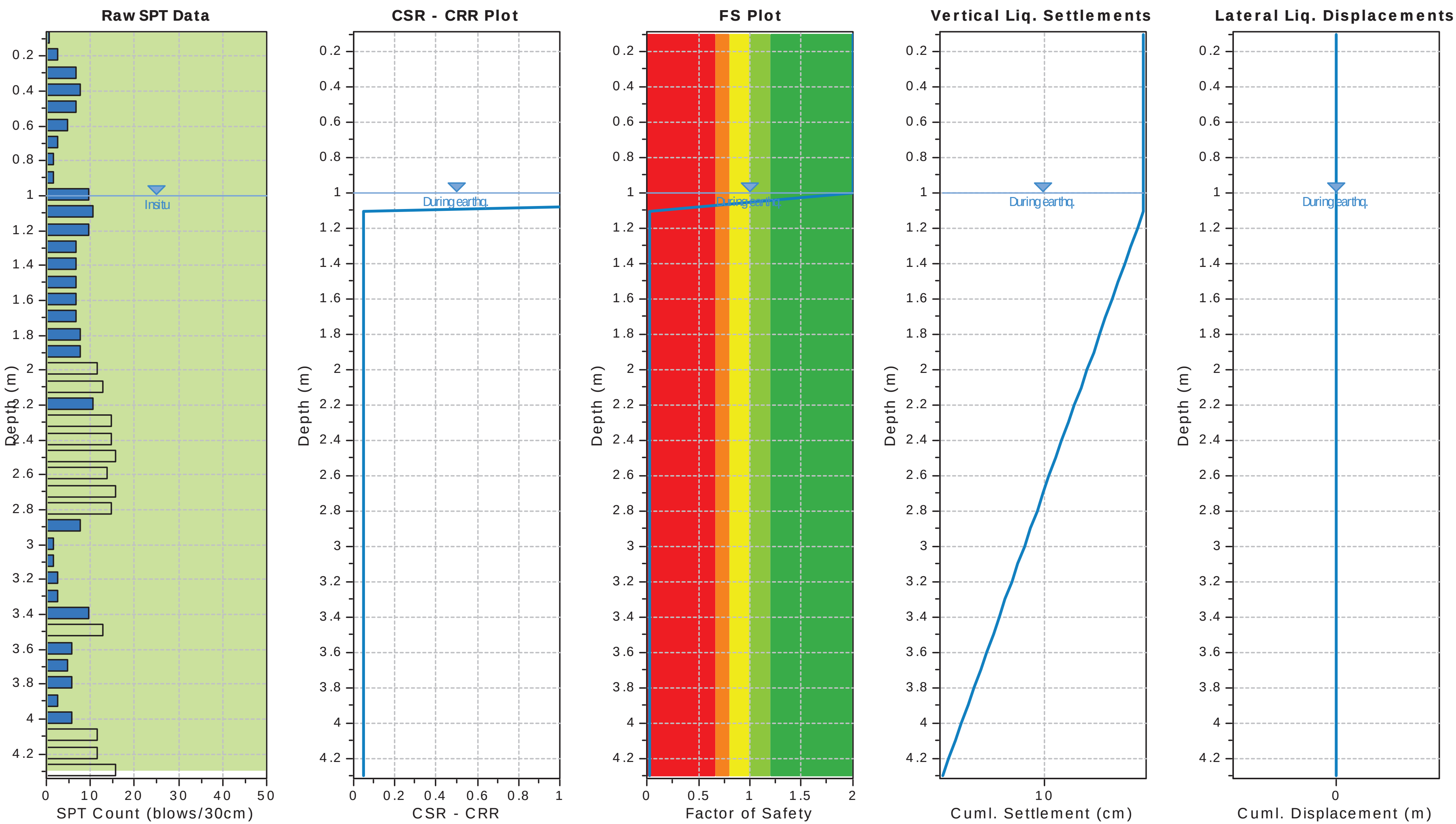




F.S. color scheme
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

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

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (m)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (kN/m³)	Infl. Thickness (m)	Can Liquefy
0.10	1	1.00	1520.00	0.10	No
0.20	3	1.00	1520.00	0.10	No
0.30	7	1.00	1520.00	0.10	No
0.40	8	1.00	1520.00	0.10	No
0.50	7	1.00	1520.00	0.10	No
0.60	5	1.00	1520.00	0.10	No
0.70	3	1.00	1520.00	0.10	No
0.80	2	1.00	1520.00	0.10	No
0.90	2	1.00	1520.00	0.10	No
1.00	10	1.00	1520.00	0.10	No
1.10	11	1.00	1520.00	0.10	Yes
1.20	10	1.00	1520.00	0.10	Yes
1.30	7	1.00	1520.00	0.10	Yes
1.40	7	1.00	1520.00	0.10	Yes
1.50	7	1.00	1520.00	0.10	Yes
1.60	7	1.00	1520.00	0.10	Yes
1.70	7	1.00	1520.00	0.10	Yes
1.80	8	1.00	1520.00	0.10	Yes
1.90	8	1.00	1520.00	0.10	Yes
2.00	12	1.00	1520.00	0.10	Yes
2.10	13	1.00	1520.00	0.10	Yes
2.20	11	1.00	1520.00	0.10	Yes
2.30	15	1.00	1520.00	0.10	Yes
2.40	15	1.00	1520.00	0.10	Yes
2.50	16	1.00	1520.00	0.10	Yes
2.60	14	1.00	1520.00	0.10	Yes
2.70	16	1.00	1520.00	0.10	Yes
2.80	15	1.00	1520.00	0.10	Yes
2.90	8	1.00	1520.00	0.10	Yes
3.00	2	1.00	1520.00	0.10	Yes
3.10	2	1.00	1520.00	0.10	Yes
3.20	3	1.00	1520.00	0.10	Yes
3.30	3	1.00	1520.00	0.10	Yes
3.40	10	1.00	1520.00	0.10	Yes
3.50	13	1.00	1520.00	0.10	Yes
3.60	6	1.00	1520.00	0.10	Yes
3.70	5	1.00	1520.00	0.10	Yes
3.80	6	1.00	1520.00	0.10	Yes
3.90	3	1.00	1520.00	0.10	Yes
4.00	6	1.00	1520.00	0.10	Yes
4.10	12	1.00	1520.00	0.10	Yes
4.20	12	1.00	1520.00	0.10	Yes
4.30	16	1.00	1520.00	0.10	Yes

:: Field input data ::

Test Depth (m)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (kN/m³)	Infl. Thickness (m)	Can Liquefy

Abbreviations

Depth: Depth at which test was performed (m)
SPT Field Value: Number of blows per 30 cm
Fines Content: Fines content at test depth (%)
Unit Weight: Unit weight at test depth (kN/m³)
Infl. Thickness: Thickness of the soil layer to be considered in settlements analysis (m)
Can Liquefy: User defined switch for excluding/including test depth from the analysis procedure

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

Depth (m)	SPT Field Value	Unit Weight (kN/m³)	σ_v (kPa)	u_o (kPa)	σ'_{vo} (kPa)	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	Fines Content (%)	α	β	$(N_1)_{60cs}$	$CRR_{7.5}$
0.10	1	1520.00	152.00	0.00	152.00	0.81	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	4.000
0.20	3	1520.00	304.00	0.00	304.00	0.52	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	4.000
0.30	7	1520.00	456.00	0.00	456.00	0.39	1.00	1.00	0.75	1.00	2	1.00	0.00	1.00	2	4.000
0.40	8	1520.00	608.00	0.00	608.00	0.31	1.00	1.00	0.75	1.00	2	1.00	0.00	1.00	2	4.000
0.50	7	1520.00	760.00	0.00	760.00	0.25	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	4.000
0.60	5	1520.00	912.00	0.00	912.00	0.22	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	4.000
0.70	3	1520.00	1064.00	0.00	1064.00	0.19	1.00	1.00	0.75	1.00	0	1.00	0.00	1.00	0	4.000
0.80	2	1520.00	1216.00	0.00	1216.00	0.17	1.00	1.00	0.75	1.00	0	1.00	0.00	1.00	0	4.000
0.90	2	1520.00	1368.00	0.00	1368.00	0.15	1.00	1.00	0.75	1.00	0	1.00	0.00	1.00	0	4.000
1.00	10	1520.00	1520.00	0.00	1520.00	0.14	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	4.000
1.10	11	1520.00	1672.00	0.98	1671.02	0.12	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	0.050
1.20	10	1520.00	1824.00	1.96	1822.04	0.11	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	0.050
1.30	7	1520.00	1976.00	2.94	1973.06	0.11	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	0.050
1.40	7	1520.00	2128.00	3.92	2124.08	0.10	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	0.050
1.50	7	1520.00	2280.00	4.91	2275.09	0.09	1.00	1.00	0.75	1.00	0	1.00	0.00	1.00	0	0.048
1.60	7	1520.00	2432.00	5.89	2426.11	0.09	1.00	1.00	0.75	1.00	0	1.00	0.00	1.00	0	0.048
1.70	7	1520.00	2584.00	6.87	2577.13	0.08	1.00	1.00	0.75	1.00	0	1.00	0.00	1.00	0	0.048
1.80	8	1520.00	2736.00	7.85	2728.15	0.08	1.00	1.00	0.75	1.00	0	1.00	0.00	1.00	0	0.048
1.90	8	1520.00	2888.00	8.83	2879.17	0.07	1.00	1.00	0.75	1.00	0	1.00	0.00	1.00	0	0.048
2.00	12	1520.00	3040.00	9.81	3030.19	0.07	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	0.050
2.10	13	1520.00	3192.00	10.79	3181.21	0.07	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	0.050
2.20	11	1520.00	3344.00	11.77	3332.23	0.06	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	0.050
2.30	15	1520.00	3496.00	12.75	3483.25	0.06	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	0.050
2.40	15	1520.00	3648.00	13.73	3634.27	0.06	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	0.050
2.50	16	1520.00	3800.00	14.72	3785.29	0.06	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	0.050
2.60	14	1520.00	3952.00	15.70	3936.30	0.05	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	0.050
2.70	16	1520.00	4104.00	16.68	4087.32	0.05	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	0.050
2.80	15	1520.00	4256.00	17.66	4238.34	0.05	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	0.050
2.90	8	1520.00	4408.00	18.64	4389.36	0.05	1.00	1.00	0.75	1.00	0	1.00	0.00	1.00	0	0.048
3.00	2	1520.00	4560.00	19.62	4540.38	0.05	1.00	1.00	0.75	1.00	0	1.00	0.00	1.00	0	0.048
3.10	2	1520.00	4712.00	20.60	4691.40	0.05	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048
3.20	3	1520.00	4864.00	21.58	4842.42	0.04	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048
3.30	3	1520.00	5016.00	22.56	4993.44	0.04	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048
3.40	10	1520.00	5168.00	23.54	5144.46	0.04	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048
3.50	13	1520.00	5320.00	24.53	5295.48	0.04	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048
3.60	6	1520.00	5472.00	25.51	5446.49	0.04	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048
3.70	5	1520.00	5624.00	26.49	5597.51	0.04	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048

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

Depth (m)	SPT Field Value	Unit Weight (kN/m³)	σ_v (kPa)	u_o (kPa)	σ'_{vo} (kPa)	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	Fines Content (%)	α	β	$(N_1)_{60cs}$	$CRR_{7.5}$
3.80	6	1520.00	5776.00	27.47	5748.53	0.04	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048
3.90	3	1520.00	5928.00	28.45	5899.55	0.04	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048
4.00	6	1520.00	6080.00	29.43	6050.57	0.04	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048
4.10	12	1520.00	6232.00	30.41	6201.59	0.04	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048
4.20	12	1520.00	6384.00	31.39	6352.61	0.03	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048
4.30	16	1520.00	6536.00	32.37	6503.63	0.03	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048

Abbreviations

σ_v:	Total stress during SPT test (kPa)
u_o:	Water pore pressure during SPT test (kPa)
σ'_{vo}:	Effective overburden pressure during SPT test (kPa)
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
α, β:	Clean sand equivalent clean sand formula coefficients
$N_{1(60)cs}$:	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 (m)	Unit Weight (kN/m³)	$\sigma_{v,eq}$ (kPa)	$u_{o,eq}$ (kPa)	$\sigma'_{vo,eq}$ (kPa)	r_d	α	CSR	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
0.10	1520.00	152.00	0.00	152.00	1.00	1.00	0.518	0.55	0.950	0.92	1.030	2.000
0.20	1520.00	304.00	0.00	304.00	1.00	1.00	0.516	0.55	0.945	0.80	1.178	2.000
0.30	1520.00	456.00	0.00	456.00	1.00	1.00	0.513	0.55	0.941	0.74	1.272	2.000
0.40	1520.00	608.00	0.00	608.00	1.00	1.00	0.511	0.55	0.936	0.70	1.341	2.000
0.50	1520.00	760.00	0.00	760.00	1.00	1.00	0.508	0.55	0.932	0.67	1.395	2.000
0.60	1520.00	912.00	0.00	912.00	1.00	1.00	0.506	0.55	0.927	0.64	1.440	2.000
0.70	1520.00	1064.00	0.00	1064.00	1.00	1.00	0.504	0.55	0.923	0.62	1.477	2.000
0.80	1520.00	1216.00	0.00	1216.00	1.00	1.00	0.501	0.55	0.918	0.61	1.510	2.000
0.90	1520.00	1368.00	0.00	1368.00	1.00	1.00	0.499	0.55	0.913	0.59	1.538	2.000
1.00	1520.00	1520.00	0.00	1520.00	0.99	1.00	0.496	0.55	0.909	0.58	1.563	2.000
1.10	1520.00	1672.00	0.98	1671.02	0.99	1.00	0.494	0.55	0.905	0.57	1.586	0.032
1.20	1520.00	1824.00	1.96	1822.04	0.99	1.00	0.492	0.55	0.901	0.56	1.606	0.031
1.30	1520.00	1976.00	2.94	1973.06	0.99	1.00	0.489	0.55	0.897	0.55	1.624	0.031
1.40	1520.00	2128.00	3.92	2124.08	0.99	1.00	0.487	0.55	0.892	0.54	1.641	0.031
1.50	1520.00	2280.00	4.91	2275.09	0.99	1.00	0.485	0.55	0.888	0.54	1.656	0.029
1.60	1520.00	2432.00	5.89	2426.11	0.99	1.00	0.482	0.55	0.884	0.53	1.669	0.029
1.70	1520.00	2584.00	6.87	2577.13	0.99	1.00	0.480	0.55	0.879	0.52	1.681	0.029
1.80	1520.00	2736.00	7.85	2728.15	0.99	1.00	0.478	0.55	0.875	0.52	1.692	0.029
1.90	1520.00	2888.00	8.83	2879.17	0.99	1.00	0.475	0.55	0.871	0.51	1.702	0.028
2.00	1520.00	3040.00	9.81	3030.19	0.99	1.00	0.473	0.55	0.866	0.51	1.711	0.029
2.10	1520.00	3192.00	10.79	3181.21	0.99	1.00	0.471	0.55	0.862	0.50	1.719	0.029
2.20	1520.00	3344.00	11.77	3332.23	0.99	1.00	0.468	0.55	0.858	0.50	1.726	0.029
2.30	1520.00	3496.00	12.75	3483.25	0.98	1.00	0.466	0.55	0.853	0.49	1.733	0.029
2.40	1520.00	3648.00	13.73	3634.27	0.98	1.00	0.463	0.55	0.849	0.49	1.738	0.029
2.50	1520.00	3800.00	14.72	3785.29	0.98	1.00	0.461	0.55	0.845	0.48	1.744	0.029
2.60	1520.00	3952.00	15.70	3936.30	0.98	1.00	0.459	0.55	0.840	0.48	1.748	0.029
2.70	1520.00	4104.00	16.68	4087.32	0.98	1.00	0.456	0.55	0.836	0.48	1.752	0.029

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

Depth (m)	Unit Weight (kN/m³)	$\sigma_{v,eq}$ (kPa)	$u_{o,eq}$ (kPa)	$\sigma'_{vo,eq}$ (kPa)	r_d	α	CSR	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS	
2.80	1520.00	4256.00	17.66	4238.34	0.98	1.00	0.454	0.55	0.832	0.47	1.756	0.029	●
2.90	1520.00	4408.00	18.64	4389.36	0.98	1.00	0.452	0.55	0.827	0.47	1.759	0.028	●
3.00	1520.00	4560.00	19.62	4540.38	0.98	1.00	0.449	0.55	0.823	0.47	1.762	0.027	●
3.10	1520.00	4712.00	20.60	4691.40	0.98	1.00	0.447	0.55	0.819	0.46	1.764	0.027	●
3.20	1520.00	4864.00	21.58	4842.42	0.98	1.00	0.444	0.55	0.814	0.46	1.766	0.027	●
3.30	1520.00	5016.00	22.56	4993.44	0.98	1.00	0.442	0.55	0.810	0.46	1.767	0.027	●
3.40	1520.00	5168.00	23.54	5144.46	0.98	1.00	0.440	0.55	0.806	0.46	1.768	0.027	●
3.50	1520.00	5320.00	24.53	5295.48	0.98	1.00	0.437	0.55	0.801	0.45	1.769	0.027	●
3.60	1520.00	5472.00	25.51	5446.49	0.98	1.00	0.435	0.55	0.797	0.45	1.769	0.027	●
3.70	1520.00	5624.00	26.49	5597.51	0.97	1.00	0.433	0.55	0.793	0.45	1.770	0.027	●
3.80	1520.00	5776.00	27.47	5748.53	0.97	1.00	0.430	0.55	0.788	0.45	1.769	0.027	●
3.90	1520.00	5928.00	28.45	5899.55	0.97	1.00	0.428	0.55	0.784	0.44	1.769	0.027	●
4.00	1520.00	6080.00	29.43	6050.57	0.97	1.00	0.426	0.55	0.780	0.44	1.768	0.027	●
4.10	1520.00	6232.00	30.41	6201.59	0.97	1.00	0.423	0.55	0.775	0.44	1.767	0.027	●
4.20	1520.00	6384.00	31.39	6352.61	0.97	1.00	0.421	0.55	0.771	0.44	1.766	0.027	●
4.30	1520.00	6536.00	32.37	6503.63	0.97	1.00	0.419	0.55	0.767	0.43	1.764	0.027	●

Abbreviations

$\sigma_{v,eq}$:	Total overburden pressure at test point, during earthquake (kPa)
$u_{o,eq}$:	Water pressure at test point, during earthquake (kPa)
$\sigma'_{vo,eq}$:	Effective overburden pressure, during earthquake (kPa)
r_d :	Nonlinear shear mass factor
α:	Improvement factor due to stone columns
CSR :	Cyclic Stress Ratio (adjusted for improvement)
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 (m)	FS	F	wz	Thickness (m)	I_L
0.10	2.000	0.00	9.95	0.10	0.00
0.20	2.000	0.00	9.90	0.10	0.00
0.30	2.000	0.00	9.85	0.10	0.00
0.40	2.000	0.00	9.80	0.10	0.00
0.50	2.000	0.00	9.75	0.10	0.00
0.60	2.000	0.00	9.70	0.10	0.00
0.70	2.000	0.00	9.65	0.10	0.00
0.80	2.000	0.00	9.60	0.10	0.00
0.90	2.000	0.00	9.55	0.10	0.00
1.00	2.000	0.00	9.50	0.10	0.00
1.10	0.032	0.97	9.45	0.10	0.92
1.20	0.031	0.97	9.40	0.10	0.91
1.30	0.031	0.97	9.35	0.10	0.91
1.40	0.031	0.97	9.30	0.10	0.90
1.50	0.029	0.97	9.25	0.10	0.90
1.60	0.029	0.97	9.20	0.10	0.89
1.70	0.029	0.97	9.15	0.10	0.89

:: Liquefaction potential according to Iwasaki ::

Depth (m)	FS	F	wz	Thickness (m)	I_L
1.80	0.029	0.97	9.10	0.10	0.88
1.90	0.028	0.97	9.05	0.10	0.88
2.00	0.029	0.97	9.00	0.10	0.87
2.10	0.029	0.97	8.95	0.10	0.87
2.20	0.029	0.97	8.90	0.10	0.86
2.30	0.029	0.97	8.85	0.10	0.86
2.40	0.029	0.97	8.80	0.10	0.85
2.50	0.029	0.97	8.75	0.10	0.85
2.60	0.029	0.97	8.70	0.10	0.85
2.70	0.029	0.97	8.65	0.10	0.84
2.80	0.029	0.97	8.60	0.10	0.84
2.90	0.028	0.97	8.55	0.10	0.83
3.00	0.027	0.97	8.50	0.10	0.83
3.10	0.027	0.97	8.45	0.10	0.82
3.20	0.027	0.97	8.40	0.10	0.82
3.30	0.027	0.97	8.35	0.10	0.81
3.40	0.027	0.97	8.30	0.10	0.81
3.50	0.027	0.97	8.25	0.10	0.80
3.60	0.027	0.97	8.20	0.10	0.80
3.70	0.027	0.97	8.15	0.10	0.79
3.80	0.027	0.97	8.10	0.10	0.79
3.90	0.027	0.97	8.05	0.10	0.78
4.00	0.027	0.97	8.00	0.10	0.78
4.10	0.027	0.97	7.95	0.10	0.77
4.20	0.027	0.97	7.90	0.10	0.77
4.30	0.027	0.97	7.85	0.10	0.76

Overall potential I_L : 27.73

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 (m)	$(N_1)_{60}$	τ_{av}	p	G_{max} (MPa)	α	b	γ (%)	ε_{15}	N_c	ε_{Nc} (%)	Δh (m)	ΔS (cm)
0.10	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.000
0.20	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.000
0.30	2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.000
0.40	2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.000
0.50	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.000
0.60	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.000
0.70	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.000
0.80	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.000
0.90	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.000

:: Vertical settlements estimation for dry sands ::

Depth (m)	$(N_1)_{60}$	τ_{av}	p	G_{max} (MPa)	α	b	γ (%)	ε_{15}	N_c	ε_{Nc} (%)	Δh (m)	ΔS (cm)

Cumulative settlemetns: 0.000

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus (MPa)
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 (cm)
ΔS: Settlement of soil layer (cm)

:: Vertical settlements estimation for saturated sands ::

Depth (m)	D_{50} (mm)	q_c / N	e_v weight factor	e_v (%)	Δh (ft)	s (in)
1.00	2.50	5.00	1.00	0.00	0.10	0.000
1.10	2.50	5.00	1.00	5.80	0.10	0.580
1.20	2.50	5.00	1.00	5.80	0.10	0.580
1.30	2.50	5.00	1.00	5.80	0.10	0.580
1.40	2.50	5.00	1.00	5.80	0.10	0.580
1.50	2.50	5.00	1.00	5.80	0.10	0.580
1.60	2.50	5.00	1.00	5.80	0.10	0.580
1.70	2.50	5.00	1.00	5.80	0.10	0.580
1.80	2.50	5.00	1.00	5.80	0.10	0.580
1.90	2.50	5.00	1.00	5.80	0.10	0.580
2.00	2.50	5.00	1.00	5.80	0.10	0.580
2.10	2.50	5.00	1.00	5.80	0.10	0.580
2.20	2.50	5.00	1.00	5.80	0.10	0.580
2.30	2.50	5.00	1.00	5.80	0.10	0.580
2.40	2.50	5.00	1.00	5.80	0.10	0.580
2.50	2.50	5.00	1.00	5.80	0.10	0.580
2.60	2.50	5.00	1.00	5.80	0.10	0.580
2.70	2.50	5.00	1.00	5.80	0.10	0.580
2.80	2.50	5.00	1.00	5.80	0.10	0.580
2.90	2.50	5.00	1.00	5.80	0.10	0.580
3.00	2.50	5.00	1.00	5.80	0.10	0.580
3.10	2.50	5.00	1.00	5.80	0.10	0.580
3.20	2.50	5.00	1.00	5.80	0.10	0.580
3.30	2.50	5.00	1.00	5.80	0.10	0.580
3.40	2.50	5.00	1.00	5.80	0.10	0.580
3.50	2.50	5.00	1.00	5.80	0.10	0.580
3.60	2.50	5.00	1.00	5.80	0.10	0.580
3.70	2.50	5.00	1.00	5.80	0.10	0.580
3.80	2.50	5.00	1.00	5.80	0.10	0.580
3.90	2.50	5.00	1.00	5.80	0.10	0.580
4.00	2.50	5.00	1.00	5.80	0.10	0.580
4.10	2.50	5.00	1.00	5.80	0.10	0.580
4.20	2.50	5.00	1.00	5.80	0.10	0.580

:: Vertical settlements estimation for saturated sands ::

Depth (m)	D_{50} (mm)	q_c / N	e_v weight factor	e_v (%)	Δh (ft)	s (in)
4.30	2.50	5.00	1.00	5.80	0.10	0.580

Cumulative settlements: 19.140

Abbreviations

D_{50}: Median grain size (mm)
q_c/N: Ratio of cone resistance to SPT
e_v: Post liquefaction volumetric strain (%)
Δh: Thickness of soil layer to be considered (m)
s: Estimated settlement (cm)

:: Lateral displacements estimation for saturated sands ::

Depth (m)	$(N_1)_{60}$	D_r (%)	γ_{max} (%)	d_z (m)	LDI	LD (m)
0.10	1	14.00	0.00	0.10	0.000	0.00
0.20	1	14.00	0.00	0.10	0.000	0.00
0.30	2	19.80	0.00	0.10	0.000	0.00
0.40	2	19.80	0.00	0.10	0.000	0.00
0.50	1	14.00	0.00	0.10	0.000	0.00
0.60	1	14.00	0.00	0.10	0.000	0.00
0.70	0	0.00	0.00	0.10	0.000	0.00
0.80	0	0.00	0.00	0.10	0.000	0.00
0.90	0	0.00	0.00	0.10	0.000	0.00
1.00	1	14.00	0.00	0.10	0.000	0.00
1.10	1	14.00	51.20	0.10	0.000	0.00
1.20	1	14.00	51.20	0.10	0.000	0.00
1.30	1	14.00	51.20	0.10	0.000	0.00
1.40	1	14.00	51.20	0.10	0.000	0.00
1.50	0	0.00	51.20	0.10	0.000	0.00
1.60	0	0.00	51.20	0.10	0.000	0.00
1.70	0	0.00	51.20	0.10	0.000	0.00
1.80	0	0.00	51.20	0.10	0.000	0.00
1.90	0	0.00	51.20	0.10	0.000	0.00
2.00	1	14.00	51.20	0.10	0.000	0.00
2.10	1	14.00	51.20	0.10	0.000	0.00
2.20	1	14.00	51.20	0.10	0.000	0.00
2.30	1	14.00	51.20	0.10	0.000	0.00
2.40	1	14.00	51.20	0.10	0.000	0.00
2.50	1	14.00	51.20	0.10	0.000	0.00
2.60	1	14.00	51.20	0.10	0.000	0.00
2.70	1	14.00	51.20	0.10	0.000	0.00
2.80	1	14.00	51.20	0.10	0.000	0.00
2.90	0	0.00	51.20	0.10	0.000	0.00
3.00	0	0.00	51.20	0.10	0.000	0.00
3.10	0	0.00	51.20	0.10	0.000	0.00
3.20	0	0.00	51.20	0.10	0.000	0.00
3.30	0	0.00	51.20	0.10	0.000	0.00
3.40	0	0.00	51.20	0.10	0.000	0.00
3.50	0	0.00	51.20	0.10	0.000	0.00

:: Lateral displacements estimation for saturated sands ::

Depth (m)	$(N_1)_{60}$	D_r (%)	γ_{max} (%)	d_z (m)	LDI	LD (m)
3.60	0	0.00	51.20	0.10	0.000	0.00
3.70	0	0.00	51.20	0.10	0.000	0.00
3.80	0	0.00	51.20	0.10	0.000	0.00
3.90	0	0.00	51.20	0.10	0.000	0.00
4.00	0	0.00	51.20	0.10	0.000	0.00
4.10	0	0.00	51.20	0.10	0.000	0.00
4.20	0	0.00	51.20	0.10	0.000	0.00
4.30	0	0.00	51.20	0.10	0.000	0.00

Cumulative lateral displacements: 0.00

Abbreviations

D_r: Relative density (%)
γ_{max}: Maximum amplitude of cyclic shear strain (%)
d_z: Soil layer thickness (m)
LDI: Lateral displacement index (m)
LD: Actual estimated displacement (m)

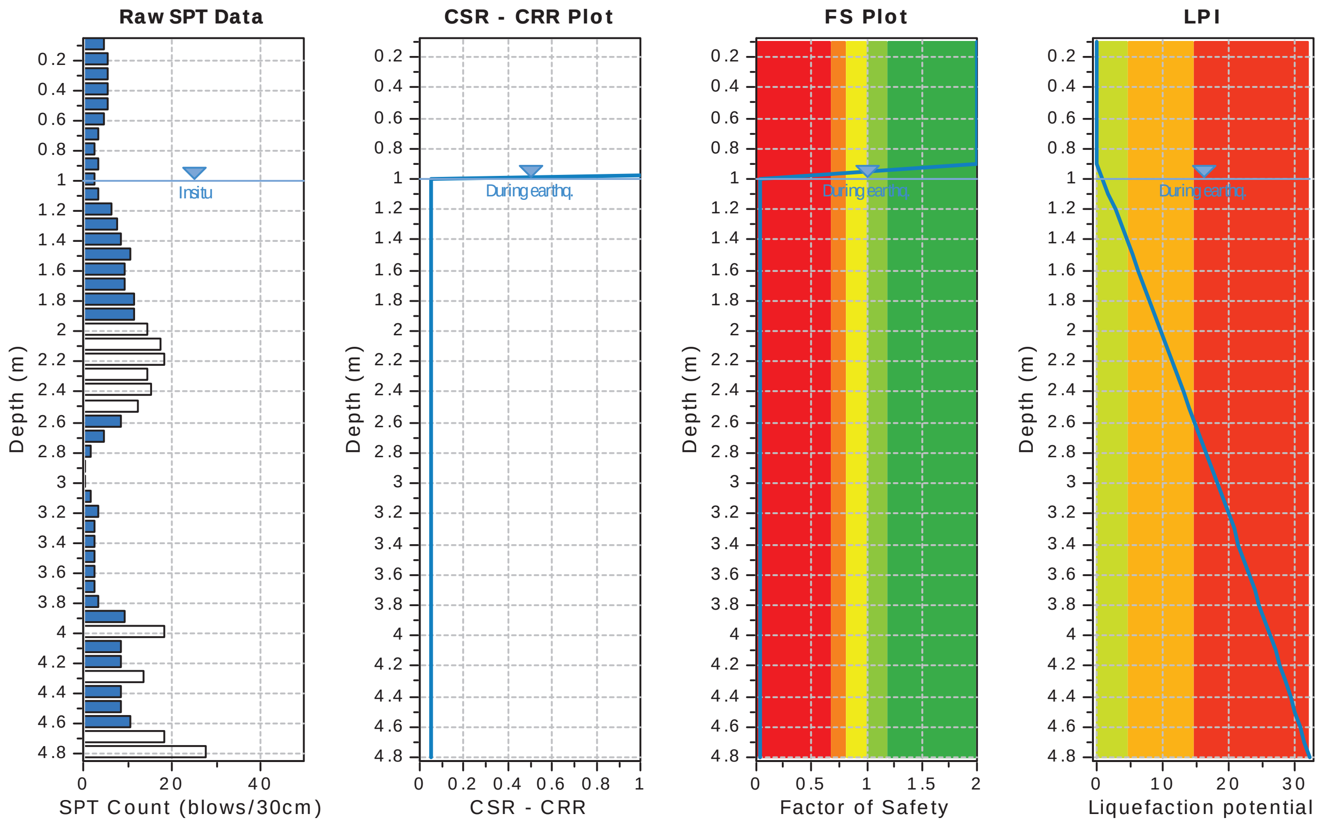
SPT BASED LIQUEFACTION ANALYSIS REPORT

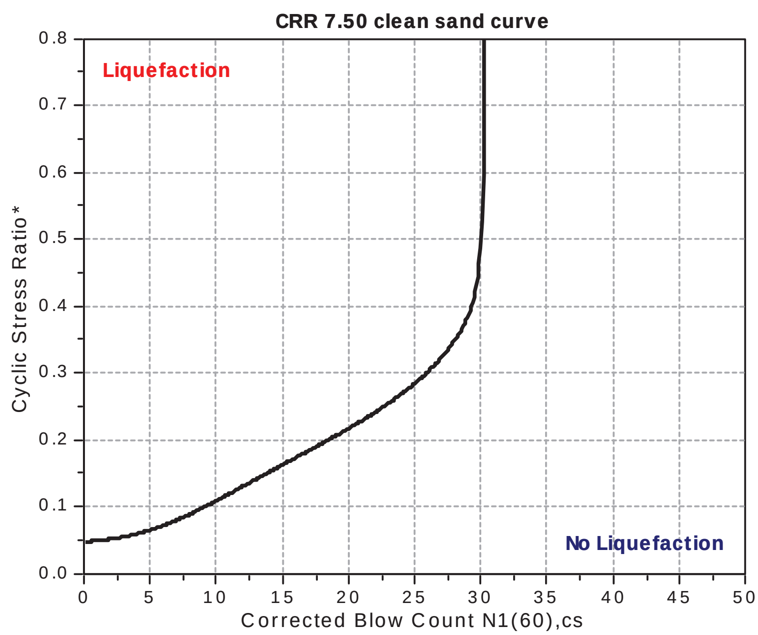
Project title : **SPT Name: DCP 2**

Location :

:: Input parameters and analysis properties ::

Analysis method:	NCEER 1998	G.W.T. (in-situ):	1.00 m
Fines correction method:	NCEER 1998	G.W.T. (earthq.):	1.00 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	9.50
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.80 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		





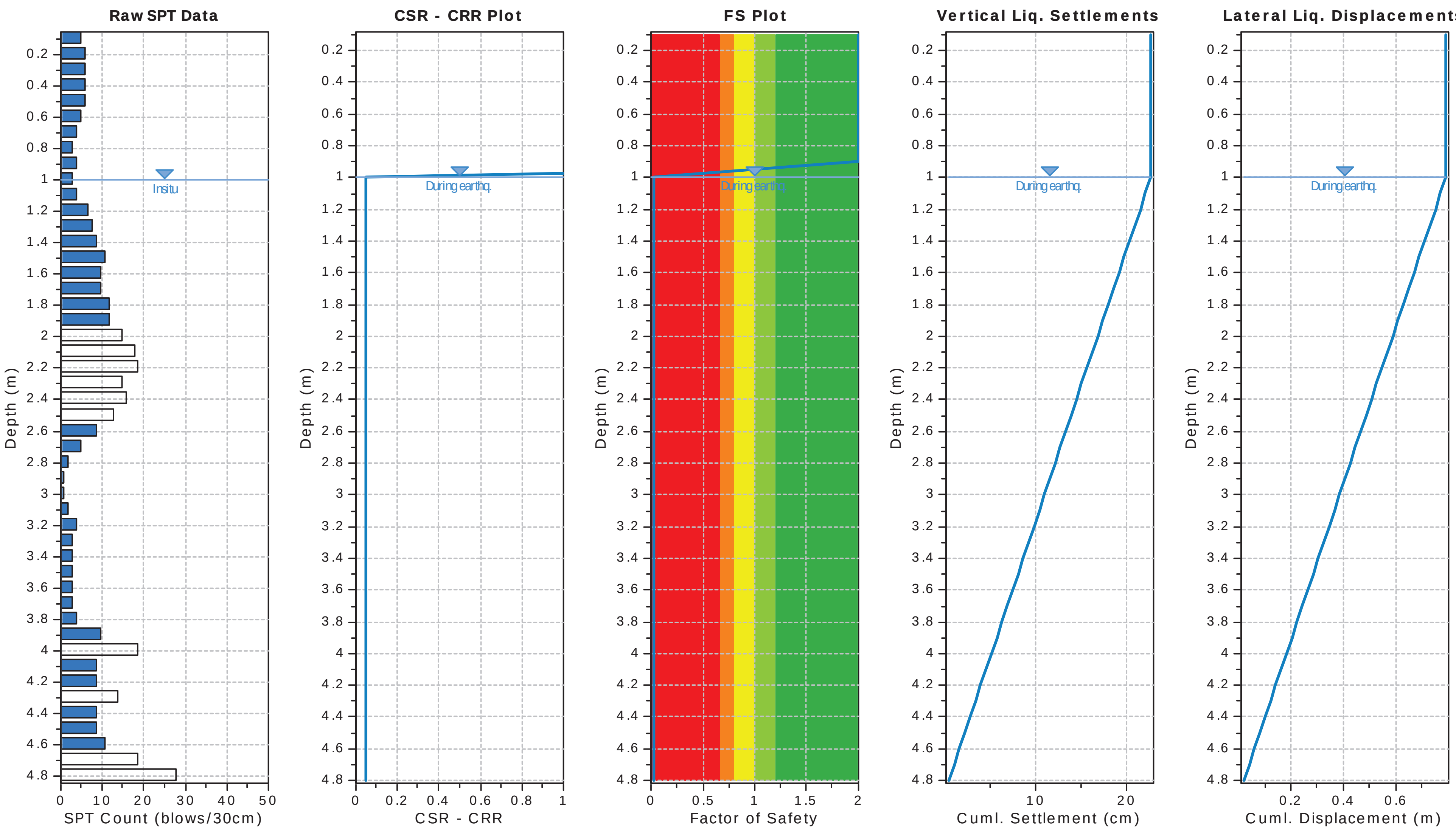
F.S. color scheme

- 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 (m)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (kN/m³)	Infl. Thickness (m)	Can Liquefy
0.10	5	1.00	1520.00	0.10	Yes
0.20	6	1.00	1520.00	0.10	Yes
0.30	6	1.00	1520.00	0.10	Yes
0.40	6	1.00	1520.00	0.10	Yes
0.50	6	1.00	1520.00	0.10	Yes
0.60	5	1.00	1520.00	0.10	Yes
0.70	4	1.00	1520.00	0.10	Yes
0.80	3	1.00	1520.00	0.10	Yes
0.90	4	1.00	1520.00	0.10	Yes
1.00	3	1.00	1520.00	0.10	Yes
1.10	4	1.00	1520.00	0.10	Yes
1.20	7	1.00	1520.00	0.10	Yes
1.30	8	1.00	1520.00	0.10	Yes
1.40	9	1.00	1520.00	0.10	Yes
1.50	11	1.00	1520.00	0.10	Yes
1.60	10	1.00	1520.00	0.10	Yes
1.70	10	1.00	1520.00	0.10	Yes
1.80	12	1.00	1520.00	0.10	Yes
1.90	12	1.00	1520.00	0.10	Yes
2.00	15	1.00	1520.00	0.10	Yes
2.10	18	1.00	1520.00	0.10	Yes
2.20	19	1.00	1520.00	0.10	Yes
2.30	15	1.00	1520.00	0.10	Yes
2.40	16	1.00	1520.00	0.10	Yes
2.50	13	1.00	1520.00	0.10	Yes
2.60	9	1.00	1520.00	0.10	Yes
2.70	5	1.00	1520.00	0.10	Yes
2.80	2	1.00	1520.00	0.10	Yes
2.90	1	1.00	1520.00	0.10	Yes
3.00	1	1.00	1520.00	0.10	Yes
3.10	2	1.00	1520.00	0.10	Yes
3.20	4	1.00	1520.00	0.10	Yes
3.30	3	1.00	1520.00	0.10	Yes
3.40	3	1.00	1520.00	0.10	Yes
3.50	3	1.00	1520.00	0.10	Yes
3.60	3	1.00	1520.00	0.10	Yes
3.70	3	1.00	1520.00	0.10	Yes
3.80	4	1.00	1520.00	0.10	Yes
3.90	10	1.00	1520.00	0.10	Yes
4.00	19	1.00	1520.00	0.10	Yes
4.10	9	1.00	1520.00	0.10	Yes
4.20	9	1.00	1520.00	0.10	Yes
4.30	14	1.00	1520.00	0.10	Yes
4.40	9	1.00	1520.00	0.10	Yes
4.50	9	1.00	1520.00	0.10	Yes
4.60	11	1.00	1520.00	0.10	Yes
4.70	19	1.00	1520.00	0.10	Yes
4.80	28	1.00	1520.00	0.10	Yes

:: Field input data ::

Test Depth (m)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (kN/m³)	Infl. Thickness (m)	Can Liquefy

Abbreviations

Depth: Depth at which test was performed (m)
SPT Field Value: Number of blows per 30 cm
Fines Content: Fines content at test depth (%)
Unit Weight: Unit weight at test depth (kN/m³)
Infl. Thickness: Thickness of the soil layer to be considered in settlements analysis (m)
Can Liquefy: User defined switch for excluding/including test depth from the analysis procedure

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

Depth (m)	SPT Field Value	Unit Weight (kN/m³)	σ_v (kPa)	u_o (kPa)	σ'_{vo} (kPa)	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	Fines Content (%)	α	β	$(N_1)_{60cs}$	$CRR_{7.5}$
0.10	5	1520.00	152.00	0.00	152.00	0.81	1.00	1.00	0.75	1.00	3	1.00	0.00	1.00	3	4.000
0.20	6	1520.00	304.00	0.00	304.00	0.52	1.00	1.00	0.75	1.00	2	1.00	0.00	1.00	2	4.000
0.30	6	1520.00	456.00	0.00	456.00	0.39	1.00	1.00	0.75	1.00	2	1.00	0.00	1.00	2	4.000
0.40	6	1520.00	608.00	0.00	608.00	0.31	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	4.000
0.50	6	1520.00	760.00	0.00	760.00	0.25	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	4.000
0.60	5	1520.00	912.00	0.00	912.00	0.22	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	4.000
0.70	4	1520.00	1064.00	0.00	1064.00	0.19	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	4.000
0.80	3	1520.00	1216.00	0.00	1216.00	0.17	1.00	1.00	0.75	1.00	0	1.00	0.00	1.00	0	4.000
0.90	4	1520.00	1368.00	0.00	1368.00	0.15	1.00	1.00	0.75	1.00	0	1.00	0.00	1.00	0	4.000
1.00	3	1520.00	1520.00	0.00	1520.00	0.14	1.00	1.00	0.75	1.00	0	1.00	0.00	1.00	0	0.048
1.10	4	1520.00	1672.00	0.98	1671.02	0.12	1.00	1.00	0.75	1.00	0	1.00	0.00	1.00	0	0.048
1.20	7	1520.00	1824.00	1.96	1822.04	0.11	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	0.050
1.30	8	1520.00	1976.00	2.94	1973.06	0.11	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	0.050
1.40	9	1520.00	2128.00	3.92	2124.08	0.10	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	0.050
1.50	11	1520.00	2280.00	4.91	2275.09	0.09	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	0.050
1.60	10	1520.00	2432.00	5.89	2426.11	0.09	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	0.050
1.70	10	1520.00	2584.00	6.87	2577.13	0.08	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	0.050
1.80	12	1520.00	2736.00	7.85	2728.15	0.08	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	0.050
1.90	12	1520.00	2888.00	8.83	2879.17	0.07	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	0.050
2.00	15	1520.00	3040.00	9.81	3030.19	0.07	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	0.050
2.10	18	1520.00	3192.00	10.79	3181.21	0.07	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	0.050
2.20	19	1520.00	3344.00	11.77	3332.23	0.06	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	0.050
2.30	15	1520.00	3496.00	12.75	3483.25	0.06	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	0.050
2.40	16	1520.00	3648.00	13.73	3634.27	0.06	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	0.050
2.50	13	1520.00	3800.00	14.72	3785.29	0.06	1.00	1.00	0.75	1.00	1	1.00	0.00	1.00	1	0.050
2.60	9	1520.00	3952.00	15.70	3936.30	0.05	1.00	1.00	0.75	1.00	0	1.00	0.00	1.00	0	0.048
2.70	5	1520.00	4104.00	16.68	4087.32	0.05	1.00	1.00	0.75	1.00	0	1.00	0.00	1.00	0	0.048
2.80	2	1520.00	4256.00	17.66	4238.34	0.05	1.00	1.00	0.75	1.00	0	1.00	0.00	1.00	0	0.048
2.90	1	1520.00	4408.00	18.64	4389.36	0.05	1.00	1.00	0.75	1.00	0	1.00	0.00	1.00	0	0.048
3.00	1	1520.00	4560.00	19.62	4540.38	0.05	1.00	1.00	0.75	1.00	0	1.00	0.00	1.00	0	0.048
3.10	2	1520.00	4712.00	20.60	4691.40	0.05	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048
3.20	4	1520.00	4864.00	21.58	4842.42	0.04	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048
3.30	3	1520.00	5016.00	22.56	4993.44	0.04	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048
3.40	3	1520.00	5168.00	23.54	5144.46	0.04	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048
3.50	3	1520.00	5320.00	24.53	5295.48	0.04	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048
3.60	3	1520.00	5472.00	25.51	5446.49	0.04	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048
3.70	3	1520.00	5624.00	26.49	5597.51	0.04	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048

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

Depth (m)	SPT Field Value	Unit Weight (kN/m³)	σ_v (kPa)	u_o (kPa)	σ'_{vo} (kPa)	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	Fines Content (%)	α	β	$(N_1)_{60cs}$	$CRR_{7.5}$
3.80	4	1520.00	5776.00	27.47	5748.53	0.04	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048
3.90	10	1520.00	5928.00	28.45	5899.55	0.04	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048
4.00	19	1520.00	6080.00	29.43	6050.57	0.04	1.00	1.00	0.85	1.00	1	1.00	0.00	1.00	1	0.050
4.10	9	1520.00	6232.00	30.41	6201.59	0.04	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048
4.20	9	1520.00	6384.00	31.39	6352.61	0.03	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048
4.30	14	1520.00	6536.00	32.37	6503.63	0.03	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048
4.40	9	1520.00	6688.00	33.35	6654.65	0.03	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048
4.50	9	1520.00	6840.00	34.34	6805.67	0.03	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048
4.60	11	1520.00	6992.00	35.32	6956.68	0.03	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048
4.70	19	1520.00	7144.00	36.30	7107.70	0.03	1.00	1.00	0.85	1.00	0	1.00	0.00	1.00	0	0.048
4.80	28	1520.00	7296.00	37.28	7258.72	0.03	1.00	1.00	0.85	1.00	1	1.00	0.00	1.00	1	0.050

Abbreviations

σ_v: Total stress during SPT test (kPa)
u_o: Water pore pressure during SPT test (kPa)
σ'_{vo}: Effective overburden pressure during SPT test (kPa)
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
α, β: Clean sand equivalent clean sand formula coefficients
$N_{1(60)cs}$: 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 (m)	Unit Weight (kN/m³)	$\sigma_{v,eq}$ (kPa)	$u_{o,eq}$ (kPa)	$\sigma'_{vo,eq}$ (kPa)	r_d	α	CSR	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
0.10	1520.00	152.00	0.00	152.00	1.00	1.00	0.518	0.55	0.950	0.92	1.030	2.000
0.20	1520.00	304.00	0.00	304.00	1.00	1.00	0.516	0.55	0.945	0.80	1.178	2.000
0.30	1520.00	456.00	0.00	456.00	1.00	1.00	0.513	0.55	0.941	0.74	1.272	2.000
0.40	1520.00	608.00	0.00	608.00	1.00	1.00	0.511	0.55	0.936	0.70	1.341	2.000
0.50	1520.00	760.00	0.00	760.00	1.00	1.00	0.508	0.55	0.932	0.67	1.395	2.000
0.60	1520.00	912.00	0.00	912.00	1.00	1.00	0.506	0.55	0.927	0.64	1.440	2.000
0.70	1520.00	1064.00	0.00	1064.00	1.00	1.00	0.504	0.55	0.923	0.62	1.477	2.000
0.80	1520.00	1216.00	0.00	1216.00	1.00	1.00	0.501	0.55	0.918	0.61	1.510	2.000
0.90	1520.00	1368.00	0.00	1368.00	1.00	1.00	0.499	0.55	0.913	0.59	1.538	2.000
1.00	1520.00	1520.00	0.00	1520.00	0.99	1.00	0.496	0.55	0.909	0.58	1.563	0.031
1.10	1520.00	1672.00	0.98	1671.02	0.99	1.00	0.494	0.55	0.905	0.57	1.586	0.031
1.20	1520.00	1824.00	1.96	1822.04	0.99	1.00	0.492	0.55	0.901	0.56	1.606	0.031
1.30	1520.00	1976.00	2.94	1973.06	0.99	1.00	0.489	0.55	0.897	0.55	1.624	0.031
1.40	1520.00	2128.00	3.92	2124.08	0.99	1.00	0.487	0.55	0.892	0.54	1.641	0.031
1.50	1520.00	2280.00	4.91	2275.09	0.99	1.00	0.485	0.55	0.888	0.54	1.656	0.030
1.60	1520.00	2432.00	5.89	2426.11	0.99	1.00	0.482	0.55	0.884	0.53	1.669	0.030
1.70	1520.00	2584.00	6.87	2577.13	0.99	1.00	0.480	0.55	0.879	0.52	1.681	0.030
1.80	1520.00	2736.00	7.85	2728.15	0.99	1.00	0.478	0.55	0.875	0.52	1.692	0.030
1.90	1520.00	2888.00	8.83	2879.17	0.99	1.00	0.475	0.55	0.871	0.51	1.702	0.029
2.00	1520.00	3040.00	9.81	3030.19	0.99	1.00	0.473	0.55	0.866	0.51	1.711	0.029
2.10	1520.00	3192.00	10.79	3181.21	0.99	1.00	0.471	0.55	0.862	0.50	1.719	0.029
2.20	1520.00	3344.00	11.77	3332.23	0.99	1.00	0.468	0.55	0.858	0.50	1.726	0.029

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

Depth (m)	Unit Weight (kN/m³)	$\sigma_{v,eq}$ (kPa)	$u_{o,eq}$ (kPa)	$\sigma'_{vo,eq}$ (kPa)	r_d	α	CSR	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
2.30	1520.00	3496.00	12.75	3483.25	0.98	1.00	0.466	0.55	0.853	0.49	1.733	0.029
2.40	1520.00	3648.00	13.73	3634.27	0.98	1.00	0.463	0.55	0.849	0.49	1.738	0.029
2.50	1520.00	3800.00	14.72	3785.29	0.98	1.00	0.461	0.55	0.845	0.48	1.744	0.029
2.60	1520.00	3952.00	15.70	3936.30	0.98	1.00	0.459	0.55	0.840	0.48	1.748	0.028
2.70	1520.00	4104.00	16.68	4087.32	0.98	1.00	0.456	0.55	0.836	0.48	1.752	0.028
2.80	1520.00	4256.00	17.66	4238.34	0.98	1.00	0.454	0.55	0.832	0.47	1.756	0.028
2.90	1520.00	4408.00	18.64	4389.36	0.98	1.00	0.452	0.55	0.827	0.47	1.759	0.028
3.00	1520.00	4560.00	19.62	4540.38	0.98	1.00	0.449	0.55	0.823	0.47	1.762	0.027
3.10	1520.00	4712.00	20.60	4691.40	0.98	1.00	0.447	0.55	0.819	0.46	1.764	0.027
3.20	1520.00	4864.00	21.58	4842.42	0.98	1.00	0.444	0.55	0.814	0.46	1.766	0.027
3.30	1520.00	5016.00	22.56	4993.44	0.98	1.00	0.442	0.55	0.810	0.46	1.767	0.027
3.40	1520.00	5168.00	23.54	5144.46	0.98	1.00	0.440	0.55	0.806	0.46	1.768	0.027
3.50	1520.00	5320.00	24.53	5295.48	0.98	1.00	0.437	0.55	0.801	0.45	1.769	0.027
3.60	1520.00	5472.00	25.51	5446.49	0.98	1.00	0.435	0.55	0.797	0.45	1.769	0.027
3.70	1520.00	5624.00	26.49	5597.51	0.97	1.00	0.433	0.55	0.793	0.45	1.770	0.027
3.80	1520.00	5776.00	27.47	5748.53	0.97	1.00	0.430	0.55	0.788	0.45	1.769	0.027
3.90	1520.00	5928.00	28.45	5899.55	0.97	1.00	0.428	0.55	0.784	0.44	1.769	0.027
4.00	1520.00	6080.00	29.43	6050.57	0.97	1.00	0.426	0.55	0.780	0.44	1.768	0.028
4.10	1520.00	6232.00	30.41	6201.59	0.97	1.00	0.423	0.55	0.775	0.44	1.767	0.027
4.20	1520.00	6384.00	31.39	6352.61	0.97	1.00	0.421	0.55	0.771	0.44	1.766	0.027
4.30	1520.00	6536.00	32.37	6503.63	0.97	1.00	0.419	0.55	0.767	0.43	1.764	0.027
4.40	1520.00	6688.00	33.35	6654.65	0.97	1.00	0.416	0.55	0.763	0.43	1.762	0.027
4.50	1520.00	6840.00	34.34	6805.67	0.97	1.00	0.414	0.55	0.758	0.43	1.760	0.028
4.60	1520.00	6992.00	35.32	6956.68	0.97	1.00	0.412	0.55	0.754	0.43	1.758	0.028
4.70	1520.00	7144.00	36.30	7107.70	0.97	1.00	0.409	0.55	0.750	0.43	1.755	0.028
4.80	1520.00	7296.00	37.28	7258.72	0.97	1.00	0.407	0.55	0.745	0.43	1.753	0.029

Abbreviations

$\sigma_{v,eq}$:	Total overburden pressure at test point, during earthquake (kPa)
$u_{o,eq}$:	Water pressure at test point, during earthquake (kPa)
$\sigma'_{vo,eq}$:	Effective overburden pressure, during earthquake (kPa)
r_d :	Nonlinear shear mass factor
α:	Improvement factor due to stone columns
CSR :	Cyclic Stress Ratio (adjusted for improvement)
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 (m)	FS	F	wz	Thickness (m)	I_L
0.10	2.000	0.00	9.95	0.10	0.00
0.20	2.000	0.00	9.90	0.10	0.00
0.30	2.000	0.00	9.85	0.10	0.00
0.40	2.000	0.00	9.80	0.10	0.00
0.50	2.000	0.00	9.75	0.10	0.00
0.60	2.000	0.00	9.70	0.10	0.00
0.70	2.000	0.00	9.65	0.10	0.00

:: Liquefaction potential according to Iwasaki ::

Depth (m)	FS	F	wz	Thickness (m)	I_L
0.80	2.000	0.00	9.60	0.10	0.00
0.90	2.000	0.00	9.55	0.10	0.00
1.00	0.031	0.97	9.50	0.10	0.92
1.10	0.031	0.97	9.45	0.10	0.92
1.20	0.031	0.97	9.40	0.10	0.91
1.30	0.031	0.97	9.35	0.10	0.91
1.40	0.031	0.97	9.30	0.10	0.90
1.50	0.030	0.97	9.25	0.10	0.90
1.60	0.030	0.97	9.20	0.10	0.89
1.70	0.030	0.97	9.15	0.10	0.89
1.80	0.030	0.97	9.10	0.10	0.88
1.90	0.029	0.97	9.05	0.10	0.88
2.00	0.029	0.97	9.00	0.10	0.87
2.10	0.029	0.97	8.95	0.10	0.87
2.20	0.029	0.97	8.90	0.10	0.86
2.30	0.029	0.97	8.85	0.10	0.86
2.40	0.029	0.97	8.80	0.10	0.85
2.50	0.029	0.97	8.75	0.10	0.85
2.60	0.028	0.97	8.70	0.10	0.85
2.70	0.028	0.97	8.65	0.10	0.84
2.80	0.028	0.97	8.60	0.10	0.84
2.90	0.028	0.97	8.55	0.10	0.83
3.00	0.027	0.97	8.50	0.10	0.83
3.10	0.027	0.97	8.45	0.10	0.82
3.20	0.027	0.97	8.40	0.10	0.82
3.30	0.027	0.97	8.35	0.10	0.81
3.40	0.027	0.97	8.30	0.10	0.81
3.50	0.027	0.97	8.25	0.10	0.80
3.60	0.027	0.97	8.20	0.10	0.80
3.70	0.027	0.97	8.15	0.10	0.79
3.80	0.027	0.97	8.10	0.10	0.79
3.90	0.027	0.97	8.05	0.10	0.78
4.00	0.028	0.97	8.00	0.10	0.78
4.10	0.027	0.97	7.95	0.10	0.77
4.20	0.027	0.97	7.90	0.10	0.77
4.30	0.027	0.97	7.85	0.10	0.76
4.40	0.027	0.97	7.80	0.10	0.76
4.50	0.028	0.97	7.75	0.10	0.75
4.60	0.028	0.97	7.70	0.10	0.75
4.70	0.028	0.97	7.65	0.10	0.74
4.80	0.029	0.97	7.60	0.10	0.74

Overall potential I_L : 32.39

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 (m)	$(N_1)_{60}$	τ_{av}	p	G_{max} (MPa)	α	b	γ (%)	ε_{15}	N_c	ε_{Nc} (%)	Δh (m)	ΔS (cm)
0.10	3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.000
0.20	2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.000
0.30	2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.000
0.40	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.000
0.50	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.000
0.60	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.000
0.70	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.000
0.80	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.000
0.90	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.000

Cumulative settlemetns: 0.000

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus (MPa)
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 (cm)
ΔS: Settlement of soil layer (cm)

:: Vertical settlements estimation for saturated sands ::

Depth (m)	D_{50} (mm)	q_c / N	e_v weight factor	e_v (%)	Δh (ft)	s (in)
1.00	2.50	5.00	1.00	5.80	0.10	0.580
1.10	2.50	5.00	1.00	5.80	0.10	0.580
1.20	2.50	5.00	1.00	5.80	0.10	0.580
1.30	2.50	5.00	1.00	5.80	0.10	0.580
1.40	2.50	5.00	1.00	5.80	0.10	0.580
1.50	2.50	5.00	1.00	5.80	0.10	0.580
1.60	2.50	5.00	1.00	5.80	0.10	0.580
1.70	2.50	5.00	1.00	5.80	0.10	0.580
1.80	2.50	5.00	1.00	5.80	0.10	0.580
1.90	2.50	5.00	1.00	5.80	0.10	0.580
2.00	2.50	5.00	1.00	5.80	0.10	0.580
2.10	2.50	5.00	1.00	5.80	0.10	0.580
2.20	2.50	5.00	1.00	5.80	0.10	0.580
2.30	2.50	5.00	1.00	5.80	0.10	0.580
2.40	2.50	5.00	1.00	5.80	0.10	0.580
2.50	2.50	5.00	1.00	5.80	0.10	0.580
2.60	2.50	5.00	1.00	5.80	0.10	0.580
2.70	2.50	5.00	1.00	5.80	0.10	0.580
2.80	2.50	5.00	1.00	5.80	0.10	0.580
2.90	2.50	5.00	1.00	5.80	0.10	0.580
3.00	2.50	5.00	1.00	5.80	0.10	0.580
3.10	2.50	5.00	1.00	5.80	0.10	0.580
3.20	2.50	5.00	1.00	5.80	0.10	0.580
3.30	2.50	5.00	1.00	5.80	0.10	0.580

:: Vertical settlements estimation for saturated sands ::

Depth (m)	D_{50} (mm)	q_c / N	e_v weight factor	e_v (%)	Δh (ft)	s (in)
3.40	2.50	5.00	1.00	5.80	0.10	0.580
3.50	2.50	5.00	1.00	5.80	0.10	0.580
3.60	2.50	5.00	1.00	5.80	0.10	0.580
3.70	2.50	5.00	1.00	5.80	0.10	0.580
3.80	2.50	5.00	1.00	5.80	0.10	0.580
3.90	2.50	5.00	1.00	5.80	0.10	0.580
4.00	2.50	5.00	1.00	5.80	0.10	0.580
4.10	2.50	5.00	1.00	5.80	0.10	0.580
4.20	2.50	5.00	1.00	5.80	0.10	0.580
4.30	2.50	5.00	1.00	5.80	0.10	0.580
4.40	2.50	5.00	1.00	5.80	0.10	0.580
4.50	2.50	5.00	1.00	5.80	0.10	0.580
4.60	2.50	5.00	1.00	5.80	0.10	0.580
4.70	2.50	5.00	1.00	5.80	0.10	0.580
4.80	2.50	5.00	1.00	5.80	0.10	0.580

Cumulative settlements: 22.620

Abbreviations

D_{50}: Median grain size (mm)
q_c/N: Ratio of cone resistance to SPT
e_v: Post liquefaction volumetric strain (%)
Δh: Thickness of soil layer to be considered (m)
s: Estimated settlement (cm)

:: Lateral displacements estimation for saturated sands ::

Depth (m)	$(N_1)_{60}$	D_r (%)	γ_{max} (%)	d_z (m)	LDI	LD (m)
0.10	3	24.25	0.00	0.10	0.000	0.00
0.20	2	19.80	0.00	0.10	0.000	0.00
0.30	2	19.80	0.00	0.10	0.000	0.00
0.40	1	14.00	0.00	0.10	0.000	0.00
0.50	1	14.00	0.00	0.10	0.000	0.00
0.60	1	14.00	0.00	0.10	0.000	0.00
0.70	1	14.00	0.00	0.10	0.000	0.00
0.80	0	0.00	0.00	0.10	0.000	0.00
0.90	0	0.00	0.00	0.10	0.000	0.00
1.00	0	0.00	51.20	0.10	0.051	0.02
1.10	0	0.00	51.20	0.10	0.051	0.02
1.20	1	14.00	51.20	0.10	0.051	0.02
1.30	1	14.00	51.20	0.10	0.051	0.02
1.40	1	14.00	51.20	0.10	0.051	0.02
1.50	1	14.00	51.20	0.10	0.051	0.02
1.60	1	14.00	51.20	0.10	0.051	0.02
1.70	1	14.00	51.20	0.10	0.051	0.02
1.80	1	14.00	51.20	0.10	0.051	0.02
1.90	1	14.00	51.20	0.10	0.051	0.02
2.00	1	14.00	51.20	0.10	0.051	0.02
2.10	1	14.00	51.20	0.10	0.051	0.02

:: Lateral displacements estimation for saturated sands ::

Depth (m)	$(N_1)_{60}$	D_r (%)	γ_{max} (%)	d_z (m)	LDI	LD (m)
2.20	1	14.00	51.20	0.10	0.051	0.02
2.30	1	14.00	51.20	0.10	0.051	0.02
2.40	1	14.00	51.20	0.10	0.051	0.02
2.50	1	14.00	51.20	0.10	0.051	0.02
2.60	0	0.00	51.20	0.10	0.051	0.02
2.70	0	0.00	51.20	0.10	0.051	0.02
2.80	0	0.00	51.20	0.10	0.051	0.02
2.90	0	0.00	51.20	0.10	0.051	0.02
3.00	0	0.00	51.20	0.10	0.051	0.02
3.10	0	0.00	51.20	0.10	0.051	0.02
3.20	0	0.00	51.20	0.10	0.051	0.02
3.30	0	0.00	51.20	0.10	0.051	0.02
3.40	0	0.00	51.20	0.10	0.051	0.02
3.50	0	0.00	51.20	0.10	0.051	0.02
3.60	0	0.00	51.20	0.10	0.051	0.02
3.70	0	0.00	51.20	0.10	0.051	0.02
3.80	0	0.00	51.20	0.10	0.051	0.02
3.90	0	0.00	51.20	0.10	0.051	0.02
4.00	1	14.00	51.20	0.10	0.051	0.02
4.10	0	0.00	51.20	0.10	0.051	0.02
4.20	0	0.00	51.20	0.10	0.051	0.02
4.30	0	0.00	51.20	0.10	0.051	0.02
4.40	0	0.00	51.20	0.10	0.051	0.02
4.50	0	0.00	51.20	0.10	0.051	0.02
4.60	0	0.00	51.20	0.10	0.051	0.02
4.70	0	0.00	51.20	0.10	0.051	0.02
4.80	1	14.00	51.20	0.10	0.051	0.02

Cumulative lateral displacements: 0.79

Abbreviations

D_r: Relative density (%)
γ_{max}: Maximum amplitude of cyclic shear strain (%)
d_z: Soil layer thickness (m)
LDI: Lateral displacement index (m)
LD: Actual estimated displacement (m)

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