THE GALLI GROUP
Engineering Consulting

GEOTECHNICAL DESIGN REPORT
NEW CHILD LEARNING CENTER
KLAMATH COMMUNITY COLLEGE
7390 SOUTH 6TH STREET
KLAMATH FALLS, OREGON

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1.0 INTRODUCTION

This report presents the results of our geotechnical evaluation for the on-campus site of the proposed Child Learning Center for Klamath Community College. The project site is located in the northwestern portion of the campus, near the entrance on the south side of South 6th Street, in Klamath Falls, Oregon. See *Figure 1, Vicinity Map* for the site location.

The purpose of this investigation and report was to evaluate the site surface and subsurface conditions with exploratory borings and a seismic survey, to provide geotechnical recommendations for design and construction of the proposed development, including geologic hazards evaluation, seismic design parameters, structure foundations, and support for roadway/access lanes and parking areas.

2.0 SITE AND PROJECT DESCRIPTION

The project site consists of the area immediately south and west of the entrance road to the KCC campus from South 6th Street. The planned area of development is under 5 acres of land that is currently utilized for agriculture. The single-story structure will have a footprint of over 25,000 square feet and be surrounded by lawn/landscaping/playground areas, paved access roads/parking lots, exterior walkways, and associated utilities.

We understand the structure is anticipated to have loads of 1-5 kips per lineal foot for continuous footings and 20-60 kips for isolated footing loads. The structure will have a slab on grade floor, and no underground levels or retaining walls are currently planned.

3.0 FIELD EXPLORATION

3.1 EXPLORATORY BORINGS

On May 2, 2025, Engineering Associate, Kristen S. Pierce, E.I.T., and our drilling crew visited the site to accomplish the subsurface investigation. Two exploratory borings were accomplished for the site at the location shown on *Figure 2 - Site Plan with Exploration*

Locations. The drilling was accomplished with our ATV-mounted, solid stem auger drill rig. Standard Penetration Testing (SPT) was accomplished in the borings. This entails driving a 1½-inch diameter, steel split spoon sampler, by dropping a 140-pound weight for a 30-inch drop. The total number of blows it takes to drive the sampler the last 12 inches of an 18-inch drive is called the SPT N-value. The results are an indication of the relative density or consistency of the soil and can be correlated with soil strength and density parameters from testing on thousands of other projects.

Our representative identified the exploration locations away from marked utilities, logged subsurface soils and water conditions and obtained soil samples for transport to our laboratory. The borings were located near the southwestern and northeastern perimeters of the planned structure. The borings were advanced to depths of 16.5 and 6.5 feet, terminating in very dense soil conditions. Groundwater was encountered in the deeper boring, which was filled to above the groundwater level with bentonite chips. Both borings were backfilled to the ground surface with soil spoils, after completion of the drilling operations, leaving the boring locations clear of most soil debris.

Visual classifications of the soils were made in the field and are presented in the *Boring Logs* in *Appendix A*, at the end of this report. The N-values shown in the logs are raw data from the field and have not been adjusted for sampling equipment type, adjusted for sampler size, or overburden pressure. Please note that in the logs, soil changes are depicted as distinct layers, while in nature they may be more gradual.

3.2 MULTI-CHANNEL ANALYSIS OF SURFACE WAVE (MASW)

During the field exploration on May 2, 2025, an MASW survey was conducted to develop a 1-dimensional shear wave velocity model of the subsurface. The survey array was oriented north-south and located along the central portion of the planned structure, as shown on *Figure 2.* A 20lb sledgehammer was used as an active source to generate seismic energy. An ES-3000 exploration seismograph with 24 channels utilizing 4.5Hz vertical geophones was used for the acquisition of the seismic data from the active source.

The field seismic data acquired was processed in the office and a 1-dimensional shear wave velocity profile of the site's subsurface was generated, modeling the S-wave velocities to depths of up to 140 feet. This information was used for making the seismic site class determination. This information was also used in combination with published correlations to estimate other geotechnical parameters and provide geotechnical design recommendations for this project. Graphical representation of the MASW survey result is shown on *Figure 3, MASW 1-Dimensional Shear Wave Velocity Profile*.

The shear wave velocity profile was developed using a ten-layer model of the subsurface. The upper 12.5 feet of the subsurface at the project site is modeled with material shear wave velocities of 700-800 feet/second, with velocities ranging from 1,140-1,340 feet/second extending from a depth of approximately 20 feet to over 100 feet in depth.

4.0 LABORATORY TESTING

Laboratory tests were performed on selected samples to measure soil index properties to provide a basis for estimating engineering properties. All soil samples collected during our investigation were tested for natural moisture content (ASTM), and are presented in the *Boring Logs* in *Appendix A.* In addition, three Washed Sieve Analyses with Hydrometer test (ASTM D1140 and ASTM D7928), and an Atterberg Limits test (ASTM D4318), were conducted on selected collected samples. Results of the laboratory soils tests are summarized below, in Table 1; individual lab test results are attached in *Appendix B.*

Table 1: LABORATRORY TESTING

Boring/Sample Depth (ft bgs)	Test Performed	Test Results			
B-1/S-4 7.5 – 9.0	ASTM D4318	Liquid Limit	Plastic Limit	Plasticity Index	
		71	58	13	
B-1/S-2 2.5 – 4.0	ASTM D1440/ ASTM D7928	Gravel	Sand	Silt	Clay
		0%	76%	13%	11%
B-1/S-5 10 – 11.5	ASTM D1440/ ASTM D7928	Gravel	Sand	Silt	Clay
		0%	30%	58%	12%
B-2/S-2 2.5 – 4.0	ASTM D1440/ ASTM D7928	Gravel	Sand	Silt	Clay
		0%	64%	31%	5%

5.0 SUBSURFACE CONDITIONS

5.1 SOIL CONDITIONS

According to the *Custom Soil Resource Report* for this area, provided by the USDA Natural Resources Conservation Service Web Soil Survey website, the soil in the upper 5 feet of the project area is described as fine sandy loam (NRCS, 2024). These soils are typically alluvium and/or lacustrine sediments derived from tuff, basalt, ash and diatomite.

The exploratory borings predominantly encountered silty Sand, with varying amounts of clay, though conditions did vary between the two exploratory borings. Boring B-1, located near the southwestern perimeter of the structure, extended to a depth of 16.5 feet. In this boring, dry, medium stiff silt was present at the near surface. The soil quickly increased in moisture and sand content with depth, with a wet, loose, layer of sand present from approximately 4 to 6 feet in depth. Beneath this wet, loose Sand is a layer of stiff to very stiff, sandy Silt with a little clay. At a depth of approximately 13 feet the soil transitions to very dense, silty Sand with trace clay.

Boring B-2, located near the northeastern perimeter of the planned structure, only extended to a depth of 6.5 feet, with a similar dry, medium stiff S\silt present at the near surface. B-2 similarly increased in sand content, but did not increase as quickly in water content with depth. No groundwater was encountered in boring B-2, and relative density of the silty Sand soil reached very dense conditions as shallow as 2.5 feet beneath the ground surface.

The loose sand layer present from approximately 4 to 6 feet in boring B-1, at the southern portion of the development, was not encountered in B-2, at the northern end of the planned structure. Additionally, the MASW seismic survey models the upper 12.5 feet of soil with a shear wave velocity of over 600 feet/second, indicative of soil in stiff/dense conditions. In our experience, loose saturated sand soils tend to have a shear wave velocity of less than 500 ft/s.

5.2 GROUNDWATER

Free groundwater was initially encountered at a depth of 4.5 feet in boring B-1. The water level dropped by approximately 2.0 feet, to 6.5 feet in depth, during drilling operations over the course of approximately 1 hour. Groundwater was not encountered in boring B-2, which is located at a slightly higher elevation than B-1. The elevation difference between boring B-1 and B-2 is estimated to be 3 feet.

Based on the reviewed water well logs and geotechnical boring log data in the vicinity of the project area, the groundwater levels are typically around 5 feet below the ground surface (ORWD, 2023). Groundwater levels change with seasonal rainfall and other climatic occurrences. As such, the groundwater level at the project site may be higher or lower than estimated at the time of this report.

6.0 SITE GEOLOGY AND SEISMIC DESIGN

6.1 REVIEW OF SITE GEOLOGY AND SEISMICITY

6.1.1 Regional Geologic Setting

The project site is within Oregon’s Basin and Range Physiographic Province, which consists of a series of mountain ranges separated by rift valleys. The mountain ranges are generally oriented north-south and the basins typically contain lakes or dried lake beds. The project site is located on the eastern edge of a basin, less than 10 miles wide, between the mountain ranges to the east and west. These mountain ranges are primarily composed of Pliocene and Pleistocene-aged basalt and basaltic andesite (Priest, et al, 2008). The project site is located just over 0.6 miles to the west of the southwestern slopes of Hogback Mountain. The summit of Hogback Mountain, composed of Pliocene-aged basalt, rises over 2000 feet above the elevation at the project site. The basin is primarily infilled with older and younger, consolidated and unconsolidated, sediments.

6.1.2 Site Geology

The project area is located in the Altamont 7.5-minute USGS topographic quadrangle. The mapped geologic unit at the surface of the project site is older Quaternary (Pleistocene) valley sedimentary deposits. These surficial deposits are expected to overlie late Tertiary aged (Miocene-Pliocene) terrestrial sedimentary rocks, classified as lacustrine mudstone, which are present throughout much of the basin. Review of nearby well log information indicates the near surface sediment layers alternate between clay, silt, and sand; drillers occasionally describe subsurface rock (generally sandstone, but occasionally claystone) occurring at depths around 25-30 feet, however some wells extending hundreds of feet deep do not describe subsurface rock at this depth (OWRD, 2024). The exploratory borings encountered very dense sand at depths of 5 and 15 feet (see Appendix A). In deep, basin-fill sediment deposits, compaction, as well as partial cementation by mineral precipitation, can lead to very dense/hard soil conditions. The MASW survey conducted indicates that sediment conditions become very dense/hard at approximately 30 feet in depth, where material shear wave velocities exceed 1,200 ft/s (see Figure 3). Shear wave velocities in this range could also indicate the presence of very soft sedimentary rocks, however the MASW model does not indicate the presence of competent rock strata in the upper 150 feet of the subsurface at the project site.

6.1.3 Tectonic Setting

Basin and range topography in this area is a result of an extensional environment, characterized by a series of semi-parallel, opposing, normal faults. These faults result in down-dropped blocks (basins/Grabens) and up-thrown blocks (ranges). The vicinity of the project area is a seismically active area known as the Klamath Graben Fault System, which has been further divided into three fault sections. The project site is located within the South Klamath Lake Section (USGS, Quaternary fault and fold data base for the United States, 2022). This fault system is capable of earthquakes of magnitude over Mw7.0. This system has produced numerous earthquakes in recent history, with events occurring generally once every one to two decades. Faults in the Klamath Graben Fault System are normal faults that typically have a north to northwest trending strike. The project site is on the east side of the basin, where faults tend to dip to the west and southwest. Predictive models of earthquakes originating in this fault system indicate that a M7.0 event will produce moderately severe to violent perceived shaking, and potentially result in moderate to very heavy damage in the project area (USGS, Earthquake Scenarios, 2017).

The two closest mapped active faults to the project site are located slightly over 5 and 7 kilometers, from the project site (HazVu, 2018). Both are normal faults, which bound Hogback Mountain. The closer fault runs along the northeast side of the mountain with an eastern dip. This fault is classified as having most recent activity within the last 1.6 million years. The fault that is located approximately 7 kilometers away runs along the southwestern side of the mountain, with a western dip. This fault has evidence of activity within the last 15,000 years (USGS, Quaternary fault and fold data base for the United States, 2022). A detailed geology map of the project area was completed in 2008, indicating evidence of the younger, western dipping fault extending much further south, placing the fault just over 1 kilometer from the project site (Priest, Hladky, & Murray, 2008).

In addition to the project site being located within the Klamath Graben fault system, the site may be impacted by seismic events occurring along the Cascadia Subduction Zone (CSZ) off the Oregon coast. This system is considered capable of M8.5 or greater earthquakes. Various tectonic models estimate the eastern, down-dip seismogenic zone of the CSZ to be located approximately 160km of the project area (USGS, 2014). The predicted ground shaking in the project area for a M9.0 CSZ earthquake is classified as Strong, and the probability of damage caused by shaking is between 10%-30% (USGS, 2020).

The design earthquake for the proposed structures is one originating from the Klamath Graben Fault System. The predicted ground shaking in the project area for a M6.9 earthquake originating from the South Klamath Lake Section of the Klamath Graben Fault System is classified as very strong to severe, with a medium-high risk of damage caused by the shaking (USGS, 2017). The design earthquake is expected to produce negligible to slight damage in specially designed structures, with moderate to considerable damage in well-built, ordinary buildings.

6.2 GEOLOGIC HAZARDS EVALUATION

Flooding. The project site is located over two miles from any natural waterways; therefore, the risk of flooding is very low.

Expansive Soil. The soils encountered in the borings were not specifically tested for expansion. In the absence of an expansion index test, soils can be screened for expansion potential based on their clay content and plasticity index (PI). Soils with more than 10% clay ***and*** a PI of greater than 15 have the potential for expansion. The Atterberg Limit test and Washed Sieve and Hydrometer Analysis performed on the collected clayey soil samples indicate these soils are not expansive.

Landslides / Slope Instability. The project site is relatively flat. The nearest slope exceeding 20% grade occurs over 0.5 miles northeast of the project site. The State Landslide Inventory does not map the site, or the immediate areas surrounding the site, as being susceptible to landslides (SLIDO, 2021). Given the topography at and surrounding the project site, in our professional opinion the risk of landslide/slope instability to the project is very low.

Liquefaction/Lateral Spread. Liquefaction is known to occur in *cohesionless* soils (non-plastic silts and sands) that are saturated and loose. Lateral spread is a liquefaction induced ground failure that can occur at or near abrupt downslope areas or free-faces (cut slopes, river banks, etc.). The state of Oregon has mapped the project site as having very low susceptibility to liquefaction (HazVu, 2018). A general screening of liquefaction and lateral spread hazards includes evaluation of the following: seismic source potential to cause liquefaction, historic occurrence of liquefaction, depth to the water table, geologic age, and composition of subsurface material, including density of material. The soils underlying portions of the project site (loose, saturated, sandy soil) are susceptible to liquefaction. Analysis of the soils encountered at the project site indicate, if left unmitigated, liquefaction induced settlement of less than 0.5 inches *may occur* during the design seismic event, in the portions of the project area underlain by this loose, saturated sand. The risk of liquefaction and/or lateral spread at the project site during a seismic event is low.

Seismic Ground Amplification or Resonance. Seismic waves can be modified by local geologic conditions, as the waves pass from deeper, harder rocks to shallower, softer rocks, they slow down and increase in amplitude. The sharp density contrast of the soft basin rocks and sediments with surrounding geology can cause seismic waves to reflect, trapping energy in the basin for a period and extending the duration of shaking. Sediment filled basins are susceptible to seismic wave amplification and seismic wave resonance. However, no unusually hazardous amplification or resonance effects on seismic waves have been associated with the soil/bedrock subsurface conditions in the project area. The probabilistic peak horizontal ground acceleration at the site, PGA_M, for an earthquake with a 2% probability of occurring in a 50-year period is 0.48g for the project site (see *Section 6.3,* below, for more information). This PGA_M value can be used with an appropriate seismic coefficient in pseudo static analysis for design of the pertinent structural components of the proposed development.

Tsunami/Seiche Hazard. The project is located over 130 miles inland and is not subject to tsunami hazard**.** Inland bodies of water in the vicinity are located at distances too great to produce seiche that would affect the project site, therefore, the risk of seiche is very low.

Surface Rupture. The 1993 earthquakes and aftershocks resulted in numerous mapped ground ruptures, with most of them at distances ranging from 3km to 8km away from the epicenters (Wiley, et al., 1993). Most of the mapped ground ruptures were within 1km of mapped faults. The ground ruptures tended to occur in artificial fill with a maximum vertical offset of 1.5ft. Most of the ruptures were 3-6m in length, with some as long as 30m. In one instance of the longer ground ruptures, the crack extended beyond the artificial fill and into regolith (Wiley, et al., 1993). The ground ruptures tended to coincide with not only mapped faults, but also mapped geologic boundaries. Most ruptures were located near the boundaries of unconsolidated Quaternary sediment infill and volcanics of late Tertiary/early Quaternary ages (Priest, et al; 2008; Jenks, M. D.; 2007).

The ground rupture locations from the 1993 earthquakes are near fault lines and primarily occurred in deposits of sediment infill that likely directly overlie older volcanic deposits. The project site is located on deposits of unconsolidated to semi-consolidated sediments that are expected to overlie a very thick, estimated at nearly 1000ft, layer of older lacustrine sedimentary rocks (Priest, Hladky, & Murray, 2008). The active faults near the project site are located over 1km from the project site. It is expected that a seismic event causing movement on any faults mapped near the project site could cause ground rupture through the overlying unconsolidated sediments. Slip rates of the active portions of the Klamath Graben fault system are estimated to be between 0.2 to 1.0 mm/year, with actual measurement on the floor of the Upper Klamath Lake indicating a 0.43mm/year slip rate (Coleman, Rosenbaum, Reynolds, & Sarna-Wojcicki, 2000). The active faults in the area are located a sufficient distance from the project site such that significant ground rupture at the proposed development is unlikely. In our professional opinion, surface rupture impact to proposed development cannot be ruled out, however, it is a relatively low hazard.

6.3 ASCE DESIGN EARTHQUAKE

The design earthquake for the project area is based upon the established values and methodology in ASCE 7-16, as recommended by the Oregon Structural Specialty Code (OSSC, 2022). The Maximum Considered Earthquake (MCE_R), and spectral response accelerations were established as set forth in Chapter 11 of the ASCE 7-16, and were partly obtained from the online American Society of Civil Engineers Hazard Tool (ASCE, 2024). The subject structure is intended to serve education and assembly activities and in accordance with Chapter 1 of the ASCE 7-16 is assigned an occupancy risk category of III. The soils at the project site are mapped with a National Earthquake Hazards Reduction Program (NEHRP) site class D designation (HazVu, 2018). The MASW survey conducted at the project site indicates that the average shear wave velocity (Vs) in the upper 100 feet of the subsurface at the project site is approximately 1,130 feet per second. Site class determination using the parameters set forth in Chapter 20 of ASCE 7-16, (600 ft/s < Vs < 1,200 ft/s) also yield a site class D for the project site. Table 2 provides the design acceleration parameters recommended to be used during the project design.

TABLE 2 – DESIGN EARTHQUAKE (ASCE 7-16)

Klamath Community College, Child Learning Center, 7390 6th Street		
Project Area: Klamath Falls, Oregon	Latitude: 42.195952	
	Longitude: -121.701343	
Risk Category (Table 1.5-1, ASCE 7-16)	III	
Mapped Spectral Response Acceleration, MCE_R Short Period **Ss**, 0.2s (from Figure 22-1) ASCE 7-16	89.8% of g =	0.898 g
MCE_R 1 sec Period **S1**, (from Figure 22-2) ASCE 7-16	34.9% of g =	0.349 g
Site Class	D	
Site Coefficients **Fa**, Short Period (Table 11.4-1 ASCE 7-16)	1.141	
Site Coefficients **Fv**, 1 sec Period (Table 11.4-2 ASCE 7-16)	1.95	
Spectral Response Acceleration, **SMS**, Short Period (F_a*Ss equation 11.4-1 ASCE 7-16)	1.024 g	

Spectral Response Acceleration, **S_{M1}**, 1 sec Period ($F_V * S_1$ equation 11.4-2 ASCE 7-16)	0.681 g	
Design Spectral Acceleration **S_{DS}**, Short Period ($(2/3) * S_{MS}$ equation 11.4-3 ASCE 7-16)	0.683 g	
Design Spectral Acceleration **S_{D1}**, 1 sec Period ($(2/3) * S_{M1}$ equation 11.4-3 ASCE 7-16)	0.454 g	
MCE_G, PGA (Figure 22-9 ASCE 7-16)	40% of g = 0.4 g	
Site coefficient, **F_{PGA}** (Table 11.8-1 ASCE 7-16)	1.2	
MCE_G adjusted for site class effects, PGA_M (F_{PGA}*PGA equation 11.8-1 ASCE 7-16)	0.48 g	
Seismic Design Category **SDC** (Tables 1613.2.5(1) and 1613.2.5(2), (OSSC, 2022))	**D**	0.50 g < S_{DS} 0.20 g < S_{D1}

7.0 CONCLUSIONS

The project site is underlain by predominantly sandy/silty soils. This soil tends to be loose/medium stiff in the southern portion of the project site and very dense/very stiff in the southern portion of the project area. Groundwater was encountered at approximately 5 feet below the southern (lower elevation) portion of the project area. In our professional opinion, based on observations at the project site and evaluation of the results of our field exploration program, contained in this report, the site is suitable for the proposed development, provided adherence to the recommendations of this report. A conventional foundation design is adequate to support the planned development, however the structures must de designed to withstand the predicted ground motions of the anticipated seismic event.

The following sections of this report provide our geotechnical recommendations for site preparation and grading, slab, footing and asphaltic concrete pavement subgrade preparation recommendations and foundation support for the proposed development.

8.0 GEOTECHNICAL RECOMMENDATIONS

8.1 SITE PREPARATION

The site slopes very gently down to the south and is currently used for light agricultural purposes. There is evidence of abandoned utilities or other manmade debris. Therefore, normal methods of debris removal, clearing, grubbing, stripping for organic soil removal and subgrade soil preparation will apply.

8.1.1 Manmade Fill & Debris Considerations

The site has no evidence of previous development, however any old fill or debris encountered during construction must be removed. Abandoned utility lines underground tanks, inert construction debris, or other items which provide void space beneath the surface can concentrate movement of surface and/or groundwater and create the potential for piping of soils (the removal of soil fines by water seeping into the void spaces or through conduits), resulting in subsidence of the surface or settlement of structures and paved areas. This is not anticipated to be an issue for construction of the project.

8.1.2 Clearing, Grubbing and Stripping

All areas proposed for the new structures, access roads, parking areas, sidewalks, or structural fill beneath these items shall be cleared and grubbed of all trees, stumps, brush, and other debris and/or deleterious materials encountered. The areas of the site designated for development shall then be stripped and cleared of all remaining vegetation, sod, and organic topsoil. Any stripped materials should either be hauled from the site or stockpiled for use in landscape areas only. This material must <u>not</u> be used in structural fill or trench backfill on this project.

Holes or depressions resulting from excavations or the removal of deleterious materials that extend below the finish subgrade, and will be beneath structures and/or AC sections, shall be cleared of all loose native subgrade material and dished to provide access for compaction equipment. These areas shall then be backfilled and compacted to grade with structural fill, as described later in this report.

Note: Due to the presence of a loose and wet sand soil horizon, that exists at finish subgrade for the planned structure, areas of over excavation are anticipated.

It is recommended that compaction of depressions *below finish subgrade* resulting from grubbing and stripping of the site be observed/documented by the geotechnical engineer or his representative from The Galli Group.

8.1.3 Subgrade Preparation and Densification

Due to the presence of a loose sand soil horizon, subgrade preparation and densification will be critical to minimize the possibility of additional future settlement and subsidence. After removal of all vegetation, organic soil, and deleterious materials within the areas planned for development, and an area has been cut to grade, the exposed subgrade must be redensified by numerous passes with a heavy vibratory roller. We strongly recommend utilizing a segmented pad or sheepsfoot roller for compacting the onsite native soils. This densification shall be accomplished under all areas of the site planned for development. This will generally provide a reasonably “stable” subgrade for structural fill beneath structures, asphalt pavement sections and walkways This also includes the area around the outside of planned structures parking, access roads, and all concrete sidewalks.

The contractor should be aware that the surficial soils at the site will be highly susceptible to disturbance during the wet winter months. Care must be taken to not over-vibrate and disturb the subgrade soils during wet weather. *Redensification shall be discontinued if it starts to "pump up" the subgrade.*

8.1.4 Subgrade Proofrolling

The exposed subgrade throughout the site, which will support structural fill, building foundations, roadways, etc. should exhibit a smooth, firm and unyielding surface. After stripping, over-excavation, and redensification, all subgrade soils shall be proofrolled under the observation of a representative from The Galli Group. Proofrolling can be performed with a loaded to partially loaded dump truck, water truck or large heavy roller (no vibration) to verify all areas are dense and stable. When proofrolling, a successful test is when the tires of a loaded or partially loaded truck do not deflect the soils more than 3/8-inch. Proofrolling shall be discontinued if it appears the operation is pumping moisture up to the surface or otherwise disturbing the in-place soils.

Where subgrade soils are disturbed or do not demonstrate a firm, unyielding condition when proofrolled, the soil shall be redensified or aerated and redensified, or replaced with imported granular fill. The imported fill material shall be compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM Test Method D-698 (Standard Proctor). All soft and/or unstable areas must be over-excavated and backfilled with granular structural fill.

We recommend our firm observe the excavated subgrade and conduct proofrolling after excavations are complete and prior to placement of structural fill. This subgrade verification and proofrolling shall be accomplished on the exposed subgrade of over-excavated areas as well as the lifts of structural fill and the finish subgrade surfaces. After completion of site stripping, excavation to subgrade, redensification and proofrolling, the contractor must take care to protect the subgrade from disturbance due to construction equipment, especially during wet weather.

8.2 UTILITY AND SITE EXCAVATIONS

During the construction of the project, we anticipate trench excavations will be performed for utility lines and structure footings. Excavations will encounter silty Sand and sandy Silt soils, with minor amounts of clay. The soil conditions range from loose/medium stiff to very dense/hard throughout the site.

Excavations. All sizes of excavators should have no difficulty in excavating the native soils across the project site to the depths required. Trench excavations during dry weather should stand for short periods of time in shallow trenches in soils (less than 4 feet) which are not subjected to emerging groundwater seepages or surface water. Seepage or wet weather and long-term dry weather, can cause the soils to cave and slough into the trench. Deeper (greater than 4 feet) trenches or excavations into the native are susceptible to instability and wall collapse, the possibility of this increases in areas where groundwater seepages are encountered. These deeper excavations will likely require some form of shoring measures or temporary cut slopes (see below) to effectively and safely install and backfill the utilities at these deeper locations.

Temporary Cut Slopes

In dry weather conditions, temporary cut slopes in the native soil, which are not subject to emerging groundwater seepage, may be cut at 1H:1V or flatter. During wet weather conditions, temporary cuts slopes must be 1.5H:1V or flatter. For construction during periods of extended precipitation, the contractor should be prepared to further flatten temporary cut slopes to maintain slope stability.

Please note, that while we have commented on the anticipated stability of the soil in trenches and cuts, we are not responsible for job site safety. The contractor is at all times responsible for job site safety, including excavation safety. We recommend all local, state, and federal safety regulations be adhered to.

8.3 STRUCTURAL FILL PLACEMENT AND COMPACTION

8.3.1 Beneath Structures and Roadways

Structural fill is defined as any fill, placed and compacted to specified densities, used in areas that will be under structures, pavements, sidewalks, and other load-bearing improvements. The footings, floor slabs and pavements will require crushed rock structural fill below them when the loose and easily disturbed surficial soils are removed and replaced with structural fill as described in the Structure Support Recommendations (section 8.5) later in this report. The subgrade needs to be prepared properly and the fill must be placed and compacted correctly for proper long-term performance.

Structural Fill Materials. Ideally, and particularly for wet weather construction, structural fill must consist of a free-draining, angular, granular material (non-expansive) with a maximum particle size of six inches. The material should be reasonably well-graded with less than 5 percent fines (silt and clay size passing the No. 200 mesh sieve). During dry weather, any organic-free, non-expansive, compactable granular material, meeting the maximum size criteria, is acceptable for this purpose. Locally available crushed rock and jaw-run crushed "shale" have performed adequately for most applications of structural fill *(* See *Section 9.0* for structural fill specifications). The on-site surficial silty Sand soils should not be used for structural fill on this project. Imported Structural Fill shall meet the requirements of Aggregate Base Rock (AB) or Aggregate Subbase Rock (ASB) as specified in *Section 9.0*. During dry weather only, it may consist of Embankment Fill which is any angular rock, sand and silt combination with specifications as listed in *Section 9.0*. **Note:** Structural Fill for the building pad must be crushed rock as specified in *Section 8.5,* Structure Support Recommendations.

***Note:** It is the contractor's responsibility to understand the impending weather and plan for use of structural fill that will be capable of being compacted properly and remain stable under the expected construction traffic in all weather that could arise during the project construction.*

Structural Fill Placement. Structural fill shall be placed in horizontal lifts not exceeding 8 inches loose thickness (less, if necessary, to obtain proper compaction) for heavy compaction equipment. We recommend lifts of four inches or less for light and hand-operated equipment. Each lift must be compacted to a minimum of 98 percent of the maximum dry density, unless otherwise specified, as determined by the Standard Proctor test, (ASTM D698/AASHTO T99). Mechanical means must be used for compaction of structural fill; compaction by "jetting" or water settling will not be allowed. A large smooth drum roller may be utilized when compacting granular rock materials such as imported crushed rock or jaw-run “shale”.

When structural fill is used beneath footings or other structural elements it must extend beyond all sides of such elements a distance equal to at least ½ the total depth of the structural fill beneath the structural element in question for vertical support (i.e., for 2 feet of structural fill beneath footings, extend the fill at least 1 foot past all edges of the footing).

To facilitate the earthwork and compaction process, the earthwork contractor must place and compact fill materials at or slightly above their optimum moisture content. If fill soils are wet of optimum, they can be dried by continuous windrowing and aeration or by intermixing lime or Portland Cement to absorb excess moisture and improve soil properties. If soils become dry during the summer months, a water truck must be available to help keep the moisture content at or near optimum during compaction operations.

Fill Placement Observation and Testing Methods. The required construction monitoring of the structural fill utilizing standard nuclear density gauge testing and standard laboratory compaction curves (ASTM D-698 specified) is not applicable to larger jaw run shale (2” or above) or larger crushed rock. The high percentage of rock particles greater than ¾’s of an inch in these materials causes laboratory and field density test results to be erratic and does not provide an adequate representation of the density achieved. Therefore, construction specifications for this type of material typically specify method of placement and compaction coupled with visual observation during the placement and compaction operations.

For these larger rock materials, we recommend the 8-inch lift be compacted by a minimum of 3 passes with a heavy vibratory roller. One “pass” is defined as the roller moving across an area once in both directions. The placement and compaction shall be observed by our representative. After compaction as specified above is completed, the entire area shall be proofrolled with a loaded dump truck to verify density has been achieved. All areas which exhibit movement or compression of the rock material under proofrolling shall be reworked or removed and replaced as specified above.

Nuclear Density Testing of Structural Fill. Field density testing by “nuclear” methods would be adequate for verifying compaction of 2-inch to ¾-inch minus crushed base rock, Decomposed Granite, and other materials 2 inches or smaller in size. Therefore, typical specifications would suffice. Testing shall be accomplished in a systematic manner on all lifts as they are placed. Testing only the upper lifts is not adequate.

8.3.2 Non-Structural Fill

Any waste soil, organic strippings or other deleterious soil, reasonably free of debris would be considered non-structural fill. These materials may be utilized in landscape area. This material may be placed in landscape areas as berms with slopes at 3.0H:1.0V or flatter. This material cannot be placed under structures, sidewalks, roadways, parking areas or as part of a structural fill slope. It is recommended that when these soils are used, they be given a moderate level of compaction (90 to 92 percent) to help seal them from excessive surface water infiltration that would contribute to accelerated erosion.

8.4 UTILITY LINE RECOMMENDATIONS

Below we have provided general recommendations for utility construction for the project. Recommendations are based upon observations from our field investigation and experience on other projects in the area.

Trench Excavation. Trenches will be required across the site for utility installation of various kinds. Shallow (4 feet or less) trench excavations should be easy to excavate in most areas of the site. Excavation difficulty may increase on the northern portion of the site, but all areas of the site are excavatable (ie. no hard rock will be encountered). Trench sidewalls can be expected to ravel and slough at times, with emerging groundwater exacerbating instability. To reduce the chance of sloughing and caving, and to protect workmen during construction, temporary cut slopes recommended in this report, or use of trench boxes, must be utilized for trenches deeper than 4 feet, or encountering emerging seepage.

Trench Backfill and Compaction. New utility lines will require trench backfill and compaction along the entire alignment. Pipes need to be adequately supported and the trenches need to be backfilled and compacted properly to prevent subsidence of the surface or damage to utility lines or the potential overlying footings, slabs, and pavement sections.

In our experience, utility trench backfill has been a source of post-construction fill settlement problems. They are also areas which cause early pavement failure due to inadequate subgrade support.

Pipe Bedding. The bottom of the trench must be shaped out of acceptable bedding materials (refer to manufacturer's recommendations) to fit the pipe base prior to placement of the pipe. It is critical to the long-term performance of the pipe that the bottom and haunches be fully supported by a dense bedding which decreases pipe distortion from load. Finer crushed rock materials (such as ¾-inch minus crushed rock) usually provide the best bedding material.

Pipe bedding shall be compacted to 95% of ASTM D-698 (Standard Proctor) or to that which is specified by the pipeline designer. Cement-treated pea-gravel or sand/cement slurry (with at least 200 pounds of cement per cubic yard) will solidify and would typically not require compaction after placement and also makes good bedding material. Care must be taken to make sure the pipe does not "float" up in the fluid mix prior to its "setting."

Pipe Zone Material. All of the lines shall be backfilled around and to approximately 12-inches above the pipe (or more, if required by manufacturer) with an acceptable "pipe zone" material. This may consist of finer crushed rock, cement-treated pea gravel, sand/cement slurry, coarse sand with fine gravel (Decomposed Granite), or other material acceptable to the client and pipeline designers. The pipe zone material shall be well compacted on each side of the pipe, and to at least 12 inches above the pipe. Mechanical means will be required to densify these materials to the required densities (unless a cement-treated material is used).

Density requirements for "pipe zone" backfill shall be per the manufacturer's specifications for the type of pipe being used (we recommend using 95% to 98% of ASTM D-698). Care must be taken when compacting close to and immediately above the pipe so as to not damage the pipe.

General Trench Backfill. Above the "pipe zone" the backfill materials would typically consist of any compactable granular material that does not have excessive voids (such as gap-graded large gravels and cobbles), organics, expansive clay, debris, or other deleterious material. Crushed rock, jaw-run shale and sandy Gravels work well for general trench backfill.

Where laterals of any kind, or valving, extend upward from the lines, we recommend the trench areas adjacent to these items be backfilled with the "pipe zone" backfill materials. This will prevent the larger pieces of other backfill materials from damaging the valves and/or other equipment.

We strongly recommend that all general trench backfill be placed and compacted in the same manner as for general structural fill. Trench backfills beneath structures shall be compacted to at least 95 percent of the maximum dry density, as determined by ASTM Test Method D-698 (Standard Proctor). Trench backfills beneath asphalt pavements should be compacted to at least 98 percent of the maximum dry density, for the upper 48 inches, below 48 inches the trench backfill shall be compacted to at least 95 percent of the maximum dry density. Trench backfills in landscape areas may be compacted to at least 92 percent of the maximum dry density.

8.5 STRUCTURE SUPPORT RECOMMENDATIONS

During the field investigation we encountered loose, sandy native soils in the upper 2.5 feet (B-2) to 5 feet (B-1). The primary concerns are to limit total and differential settlement between footing types, and between footings and the slab-on-grade, in and around the structure, and to ensure adequate drainage and diversion of groundwater and surface water away from structures. When redensified, these upper layer soils will provide marginal subgrade support.

Based on our site investigation, settlement analyses, and experience with these types of soils conditions, we recommend that all loose soils, or at least the upper 18 inches of native sandy soils below all footings be removed. We recommend all exposed subgrades should be redensified, regardless of bottom-of-footing subgrade elevations. Allow for at least 18 inches of crushed rock structural fill above the redensified subgrade and beneath all footings. All structure footings must be supported on at least 18 inches (see note below) of crushed rock structural fill over the redensified subgrade.

Foundation Support:

To provide uniform support and improve bearing capacity, the structure footings should be supported on a minimum of 18 inches (see note below) of crushed rock over the redensified native subgrade. This will prevent excessive total and differential settlement. It will also prevent the unlikely possibility of punching failures due to loss of footing support in isolated areas of loose soils. Footings shall be designed based on the following recommendations

1. Footing areas designated to receive structural fill should first be excavated to remove the existing native soils to at least 18 inches below the bottom of the deepest footing. Base of excavation must be level and must allow for at least 18 inches below the bottom of the footings. Removal shall extend laterally at least 12 inches beyond the outside edges of <u>all</u> footings. Note: Excavations greater than 4 feet in height must be back sloped, as described in Section 8.2 for safety.
2. Redensify the subgrade soils, as described in previous sections of this report (8.1.2 - 8.1.4), to achieve a dense native subgrade. <u>DO NOT</u> over-vibrate if excessive moisture is present.
3. Place a woven geotextile support fabric (ACF S200 or equivalent) on the redensified native subgrade, pull tight.
4. Place and compact at least 18 inches of crushed rock structural fill (3/4" minus to 4" minus crushed rock structural fill) in recommended lifts over the support fabric. Compact to 98% of the maximum dry density (ASTM D-698). The upper 6 inches of structural fill shall consist of ¾” minus crushed rock)
5. Footings placed on at least 18" of compacted crushed rock over the redensified subgrade as listed above, may be designed for an allowable bearing pressure of 2000 pounds per square foot. A 1/3 increase in this allowable bearing pressure may be used when considering short-term transitory wind and seismic loads.

6. All exterior footings shall be buried a minimum of 30 inches below finish exterior grades in order to provide lateral support and frost protection.
7. We recommend minimum lateral dimensions of 18 inches for continuous load bearing footings and 36 inches for isolated piers constructed in this manner.

Anticipated Settlements. For properly constructed foundations, as described above, we have computed anticipated maximum total and differential settlement (long term loading) to be less than 3/4-inch and 1/2-inch, respectively. The settlement values assume that these footings are constructed on at least 18 inches of compacted structural fill and that existing native soil removal and replacement has been accomplished, as described above.

Note: Some footings may be deep such that all loose soils are removed from beneath them. Where all the loose soils are removed (possible excavation is between 2.5 and 5 feet) below existing grade, ensuring that footings are bearing on at least the medium dense sand, then only 6 inches of crushed rock structural fill is needed below the bottom of footings for foundation support. The extent of excavation must be verified by the geotechnical engineer or their representative from The Galli Group.

Foundation Drains. We recommend all exterior footings be installed with a footing drain to intercept groundwater seepage. Footing drains consisting of a rigid, smooth-wall perforated pipe surrounded by drain rock (one side and above), all wrapped in a non-woven geotextile fabric, shall be placed adjacent to the footings. See *Figure 4, Typical Foundation Drain Slab on Grade Floor*, this is addressed more fully later in this report (*see Section 8.9*).

8.6 INTERIOR FLOOR SLABS

Properly prepared crushed rock structural fill over redensified native subgrade soils will provide adequate support for the interior concrete slabs-on-grade. Proper site preparation, as specified in *Section 8.1*, under a minimum 12-inch layer of properly prepared crushed rock structural fill, as specified in *Section 8.3*, would provide adequate support for concrete slab-on-grade floors.

Standard Slab Section. The following recommendations are provided for the building floor slab constructed on the properly prepared native soil or crushed rock pad subgrade.

1. Excavate the slab area to a minimum of 12 inches below the bottom of slab.
2. Redensify exposed native soil subgrade (or crushed rock structural fill where additional over excavation was required) to at least 98% of maximum density from ASTM D-698.
3. Backfill up to the bottom-of-slab elevation with a minimum of 12 inches of compacted crushed rock structural fill, compacted to at least 95% of ASTM D-698. The upper 6 inches of structural fill shall consist of ¾” minus crushed rock. **Note:** If site grades indicate surface water may infiltrate into the rock below the slab; the top 6 inches of the 3/4" minus shall be replaced with a layer of <u>clean</u> (less than 2% passing the no. 200 sieve and less than 5% passing the No. 10 sieve)

crushed rock (1/2" to 3/4" clean crushed rock works well) over the compacted subgrade to provide a positive capillary moisture break and uniform slab support. The capillary break is especially helpful in areas with floors that will not "breathe".

4. A tough impermeable membrane, such as Stego Industries 15-mil vapor barrier (or an equivalent product) shall be placed over the crushed rock layer to further retard upward migration of moisture vapor into and through the concrete slab. Seal all seams, punctures, penetrations and tears per the manufacturer's recommended method.

Note: If it appears water may pond in the rock below the slab, a series of slab subdrains should be installed. These shall be constructed as shown on *Figure 5, Interior Floor Slab Subdrain Detail* and as described later in report *Section 8.9*.

8.7 MODULUS OF SUBGRADE REACTION

We have estimated the modulus of subgrade reaction (k) of the subgrade soils encountered during our site investigation. We estimated the k_1 for the silty Sand to be 60 pci (pounds per cubic inch). The estimated k_1 refers to the reaction for a 1ft x 1ft foundation bearing on the prepared subgrade. For design purposes, project designers must adjust the k for the size of footing or slab using the following correlation from Terzaghi, 1955, or other acceptable correlation.

$$k_s = k_1 \left(\frac{B+1}{2B}\right)^2$$

Where k_s= Static modulus of subgrade reaction for given footing length L and width B
B = Footing width
k_1 = Modulus of subgrade reaction for 1ft plate (estimated)

The static modulus will be significantly increased following the construction of the crushed rock support section beneath footings and slab areas, to approximately 100 pci (pounds per square inch per 1 inch deflection).

8.8 LATERAL LOAD RESISTANCE

Lateral loads exerted upon structural members can be resisted by passive pressure acting on buried portions of the foundations and other buried structures and by friction between the bottom of structural elements and the underlying soil.

We recommend the use of passive equivalent fluid pressures of the following values for portions of the structure and foundations embedded into the native soils.

• Native, silty Sand	250 pcf
• Dense Compacted Crushed Rock	450 pcf

We recommend that the first 12 inches below the native ground surface be ignored when computing the passive resistance. Where granular fill or crushed rock/shale is used, the entire embedment depth may be used. A coefficient of friction of 0.45 or 0.35 can be used for elements poured neat against crushed rock structural fill or native soils, respectively. This should be reduced to 0.2 for areas over a plastic vapor barrier.

8.9 FOUNDATION AND FLOOR DRAINS

All exterior foundations and embedded structures shall have proper drainage.

Footing Drains. Foundation drainage should consist of a rigid, smooth wall perforated pipe with at least 6 inches of drain rock on top and one side, all wrapped in a non-woven geotextile designed as a filter fabric (such as Mirafi 140N or equivalent). The perforated pipe should be located on the footing next to the stem wall (or beside the footing), provided this is at least 12 inches below underslab drain rock. The pipes are sloped to drain and be collected by a tightline which leads to the stormwater disposal system. Please see *Figure 4* for more information.

Floor Slab Subdrains. Where the drain rock layer below slabs will be lower than the adjacent exterior grades and there are water bearing zones that can saturate the underslab rock (or where the site slopes towards the structure), water will usually tend to accumulate in this low area. One method to drain this water is to include a series of subdrains at the bottom of a capillary break drain rock layer beneath the slab. The drain rock section should be thickened to at least 8-inches for such lower areas. The subdrain lines typically consist of 3-inch diameter, smooth interior, solid wall, perforated pipe at spacing of 10 feet (or less) across the structure (and around the interior perimeter). The perforated pipe is placed in a deepened zone of the drain layer as shown on *Figure 5.* The pipes are sloped to drain and collected by a tightline which leads to the stormwater disposal system. We recommend we be allowed to review the subdrain system design prior to final plan submittal or construction bidding.

All drains shall be tightlined and positively sloped to an approved stormwater disposal location in the public storm drain system or detention system. **Note:** In no case shall water be collected and/or directed or discharged close to the foundations. Such improper water discharge can cause added water related problems.

We strongly recommend against connecting roof drains or surface area drains to foundation or floor subdrains. Foundation drains (at the base of the wall footings) should consist of rigid smooth-wall perforated pipe. The rigid smooth-wall pipe can be cleaned out by means of a “roto-rooter” type system should it become plugged with sediment or fine roots. We recommend cleanouts be placed periodically by the designer to facilitate cleaning and maintenance of the drains.

8.10 EXTERIOR CONCRETE FLATWORK DESIGN

Reinforced concrete should be utilized for sidewalks, walkways, entrance ways, patios and other exterior flatwork. Concrete exterior flatwork for this project will be likely be founded on silty Sand/sandy Silt soils that are in loose/medium stiff conditions. These soils are easily disturbed. Uniform support can be achieved by placing a thickened section of structural fill beneath these areas.

We assume that light duty concrete sections will be used for walkways and other areas which will not be regularly subjected to heavy loading. Light duty concrete is NOT intended for heavy loads or vehicular traffic.

Light Duty Concrete.
4" Portland Cement Concrete (3,500 psi mix, minimum)
4" Aggregate Base (3/4" or 1" minus Crushed Rock)
8" Aggregate Subbase
Woven Geotextile Fabric
Redensified Subgrade

We assume that standard duty concrete slab areas may be utilized for entrance ways and other areas of the development. Standard duty concrete may be used in areas which would experience occasional traffic from standard sized (personal/family) vehicles.

Standard Duty Concrete.
4" Portland Cement Concrete (3,500 psi mix, minimum)
4" Aggregate Base (3/4" or 1" minus Crushed Rock)
12" Aggregate Subbase
Woven Geotextile Fabric
Redensified Subgrade

We assume heavy duty concrete slab areas may also be utilized for the development in areas such as trash enclosures, or storm water conveyance swales through parking areas and driveways, or any area which will experience frequent vehicle traffic.

Heavy Duty Concrete.
6" Portland Cement Concrete (3,500 psi mix, minimum)
6" Aggregate Base (3/4" or 1" minus Crushed Rock)
12" Aggregate Subbase
Woven Geotextile Fabric
Redensified Subgrade

Note: These concrete section designs assume the subgrade is properly prepared per *Section 8.1*. Extend prepared subgrade and base/subbase sections at least 6 inches beyond the edges of planned light-duty concrete coverage and at least 12 inches for standard and heavy-duty concrete. All details for concrete work must be reviewed by the project structural engineer and/or architect.

8.11 ASPHALTIC CONCRETE (AC) PAVEMENTS

It is our understanding that access entrance/exit, drive lanes and parking areas will be constructed as part of this project and will consist of Hot Mix Asphaltic Concrete (AC) paved surfaces. The following sections provide recommendations for asphaltic concrete section design and construction. The following asphalt sections were designed utilizing a Crushed Rock Equivalent (CRE) method based on gravel equivalency; CRE factors of 0.8 (4" minus crushed rock or jaw-run "shale" subbase, or ASB), 1.0 (3/4" or 1" minus crushed rock aggregate base, or AB), and 2.0 (asphaltic concrete, or AC), were used.

The subject site is underlain by medium stiff, loose to dense, silty Sand soils. The subgrade soils for the anticipated access roads and parking sites consist of medium stiff silt. Our firm utilized the results of a California Bearing Ratio (CBR) laboratory test (ASTM D1883) value of 5.5 (equivalent R-value of 15) for the nearby onsite, native soils.

We have assumed the traffic loading for the access roadways and parking areas based on primary traffic consisting of light automobiles and school busses, with occasional delivery truck, garbage truck or full-sized fire service truck loading. The design Traffic Indices (TI) were determined to be 6.5 for the access roadways and drive lanes and 5.0 for parking stalls. The TI values are based on the anticipated traffic numbers, axle loads from trucks and for a 20-year design life. The successful performance of pavement structures is a function of subgrade material properties, traffic conditions, drainage conditions, the pavement material properties and design, careful construction, and ongoing maintenance.

Access Roadways/Drive Lanes (TI=6.5)	**Parking Stalls (TI = 5.0)**
4” Asphaltic Concrete	3” Asphaltic Concrete
8” Aggregate Base Rock	6” Aggregate Base Rock
12” Aggregate Subbase*	10” Aggregate Subbase*
Woven Geotextile Support Fabric	Woven Geotextile Support Fabric
Redensified Subgrade	Redensified Subgrade
OR	**OR**
4” Asphaltic Concrete	3” Asphaltic Concrete
14” Aggregate Base Rock	12” Aggregate Base Rock*
Woven Geotextile Support Fabric	Woven Geotextile Support Fabric
Redensified Subgrade	Redensified Subgrade

Subgrade Preparation. Subgrade preparation should begin with removal of debris and loose and disturbed soils. All debris and organic material should be disposed of properly and is not permitted as subgrade or structural fill material.

All finish subgrades should be shaped to a uniform surface running reasonably true to established line and grade described in the contract documents. Areas so specified must be redensified and/or backfilled with structural fill. It is important that dense, stable conditions of the subgrade be maintained until the subgrade is covered with the subbase aggregate. Subgrade preparation should include clearing, redensification to at least 95% of ASTM D-698, and proofrolling (as described earlier in this report) to identify soft and disturbed subgrade areas.

After subgrade preparation is completed, the exposed subgrade prepared for the pavement structure should demonstrate a firm and unyielding condition as shown by proofrolling. Soft or loose materials disturbed during the site preparation process, incapable of achieving the compaction criteria, should be removed to appropriate bearing materials prior to replacing with structural fill. Where loose or softened subgrade areas are identified, the area should be over-excavated and replaced with imported granular fill with less than 10 percent passing the number 200 sieve.

It shall be noted that in no case shall construction trucks be allowed to "run" directly on top of the subgrade soil until it is covered with rock. This might result in the disturbance of the subgrade soils due to the heavily loaded vehicles (which will result in additional over-excavation to remove softened soils). We recommend covering the subgrade soils with at least 12 inches (more if directed by the Geotechnical Engineer or his personnel) of compacted crushed rock over the woven fabric prior to construction truck traffic traversing the area. Therefore, construction traffic must be carefully coordinated in order to minimize disturbance to the underlying fine-grained soils.

Geotextile Fabric Placement. When the subgrade soils have been properly prepared, the described subgrade areas shall be covered with the woven geotextile support fabric prior to placement and compaction of structural fill. We recommend a fabric such as ACF S200 or equivalent. The fabric shall be laid longitudinally with the direction of traffic. All ends and edges should be overlapped a minimum of 5 and 2 feet, respectively. Care must be taken to not damage the fabric. It should be noted that construction trucks should not be allowed to "run" directly on top of the fabric until it is covered with rock. We recommend covering the subgrade soils with at least 6 inches of crushed rock or "shale" over the woven fabric, during construction, prior to light construction truck traffic traversing the area.

Wet Weather Construction. During wet weather the unprotected subgrade materials will become disturbed rather easily. We recommend that for construction during very wet weather or on wet subgrades, all construction roads and drive lane where construction traffic will concentrate, the subgrades should be covered with a woven geotextile support fabric (ACF 200 or equivalent) and a minimum of 12 inches of imported granular 4-inch minus crushed rock. Compaction of the fill should not begin until a minimum of 8 inches of rock is placed above the fabric. Compact carefully so as not to disturb the subgrade. This should provide an adequate working surface and help protect the subgrade from damage from construction traffic. Construction traffic should not be allowed to traverse the area until the minimum of 12 or more inches of compacted material has been placed and compacted over the support fabric.

Note: If construction traffic begins to "pump" the subgrade soils, "haul roads" with 18" or more of crushed rock over fabric shall be established. These are particularly helpful near the structure where concrete trucks and lift trucks will be situated during building construction. The excess rock on these "roads" may be pulled off and used in the AC areas when final rock placement takes place.

Materials. All materials used and construction techniques applied at the site must result in conditions as assumed for design of the pavement sections. We recommend materials used in the pavement support sections be as listed in *Section 9.0*.

Drainage. Adequate provision should be made to direct surface water away from the pavement section and subgrade. Ponded water adjacent to the asphalt areas can saturate the subgrade resulting in loss of support. Therefore, we recommend the areas along the edge of the asphalt be well drained. All paved areas should be sloped and drainage gradients maintained to carry surface water to catch basins or ditches for transmission off the roadway and parking areas. Excessive landscape watering can also saturate the subgrade and decrease pavement life. Deep curbs, drip irrigation and/or use of dry-land plants will mitigate these effects.

Maintenance. Pavement life can be extended by providing proper maintenance and overlays as needed. Cracks in the pavement should be filled to prevent intrusion of surface water into the subbase. Asphalt pavements typically require seal coats or overlays after 10 to 12 years to maintain structural performance and aesthetic appearance.

9.0 MATERIALS SPECIFICATIONS

The following materials specifications shall apply to the materials as used on this project.

Aggregate Base Rock (Acceptable for Structural Fill)

- Angular Crushed Rock (3/4" or 1" Minus); R=80 or greater; Well Graded (No Gaps and at least 60% retained on the No. 4 sieve).
- Maximum passing the No. 200 sieve ≤ 5%.
- Compacted to 98% of the maximum dry density as determined by ASTM D698 or AASHTO T-99.

Aggregate Subbase Rock (Acceptable for Structural Fill)

- Angular Clean Crushed (jaw run) hard "Shale" (4" Minus Jaw-Run) or Crushed Rock (2" to 4" Minus); R=40 or greater; Angular and Reasonably Well Graded.
- At Least 60% retained on the No. 4 Sieve.
- Maximum passing the No. 200 sieve ≤ 7% Total; ≤ 3% Clay Size
- During wet weather; passing No. 200 sieve ≤ 5%.
- Compacted to 95% of the maximum dry density as determined by ASTM D698 or AASHTO T-99; initial lift may not attain 95% due to soft subgrade; Engineer to decide in the field.

Embankment Fill (Acceptable for Structural Fill During Dry Weather)

- Reasonably well graded (not open work).
- Has at least 60% retained on the No. 4 sieve.
- Has no more than 30% passing No. 200 sieve.
- Passing No. 200 sieve must have less than 20% clay size.

On-Site Silty Sand

- Used in landscaping areas only including general backfill portions of utility trenches in landscape areas.

Clean Sand

- Clean washed sand or sand and gravel, less than 2% passing No. 200.
- Gravel to be rounded or subrounded (no fracture faces), 1" or less.
- Must have less than 30% gravel by weight.

Note: Some fill materials will be difficult to nearly impossible to compact during wet weather. *The contractor must select the type of structural fill that will be able to be placed and compacted to specified conditions during the weather conditions that can take place during the construction schedule.*

Drain Rock (For Drainage Sections)

- Clean washed rounded or angular openwork drain rock.
- Gradation to be 1/4" and greater, sized to not move into and through perforations in the pipe.
- 1/4" to 3/4" clean crushed, 3/4" to 1" clean rounded rock, and 1" to 2" clean angular rock are all acceptable.
- Clean means washed rock with NO coating of silt, clay or sand; less than 2% passing No. 200 sieve.

Note: All types may be used in all applications of drain rock that are not beneath Asphaltic Concrete paved areas. In all AC areas angular clean drain rock must be used for AC support. Drainage layer drain rock that is beneath the floor slab must be angular clean drain rock.

Non-Woven Geotextile Filter Fabric

- Non-woven geotextile filter fabric for wrapping drainage sections and separation of openwork rock from sands or soils fines.
- Meet specifications as per Mirafi 140N or equivalent (unless otherwise specified).
- Overlap all edges at least 24 inches (12" for drainage section envelope).
- Secure in place such that overlaps will not move during covering operation.

Woven Geotextile Support Fabric

- Woven geotextile support fabric designed for separation of crushed rock and subgrade soil and for rock section support.
- Meet specifications as per ACF S200 woven support fabric (unless otherwise specified).
- Overlap edges at least 2 feet and ends at least 5 feet.
- Align roll lengthwise with direction of traffic in all drive lanes.
- Pull tight full length and keep tight during placement of crushed rock above fabric.
- Do not drive on the fabric until it is covered with rock.

Perforated Pipe

- 3", 4" or 6" rigid wall, smooth interior perforated pipe.
- Secure all joints with solvent weld glue. DO NOT use only compression push together fittings.
- Slope to drain per specifications in report or on plan sheets (minimum 1%).
- Align perforations in the downward direction.
- Must always be placed within filter fabric wrap unless specified otherwise.
- Protect from construction traffic until buried at least 2 times pipe diameter (minimum 8 inches) of angular rock fill.

Asphaltic Concrete

- Type 2 Dense Graded HMAC
- PG 64-22
- The 3-4” AC may be placed in 1 lift if vibratory rollers are used.
- Compacted to between 91% and 95% of “Maximum Specific Gravity” for first courses; between 92% and 95% for wearing course.
- Must have densification completed while temperature is above 185 degrees F.
- Do not over densify as this will significantly decrease frost heave protection of internal air voids.
- The contractor must provide a HMAC design mix for review and approval.
- All aspects of the asphaltic paving shall be accomplished in accordance with applicable ODOT standards and recommendations.

These materials shall be used on this project as specified in this report and on project plans or specifications.

NOTE: DEVIATIONS FROM SPECIFIED MATERIALS MUST BE APPROVED IN WRITING BY THE GEOTECHNICAL ENGINEER, OWNER AND OWNER'S OTHER CONSULTANTS/DESIGN ENGINEERS PRIOR TO USE AT THE SITE.

10.0 SITE DRAINAGE

The site shall be graded during construction such that surface water does not pond within building footprints or beneath pavement areas. **Note:** This is critical to limiting subgrade damage during wet weather. Surface runoff shall be controlled during construction and with final site grading. All areas adjacent to the structures shall have a permanent slope away from the foundations at an inclination of at least 6 inches in eight (8) feet. This surface water shall be channeled into landscape area drains or catch basins, or shall be conveyed around the structures and to the detention pond. Where items such as landscape areas and walkways block the flow of surface water, small area drains should be installed to collect the surface runoff. Good site design accommodates all site runoff and conveys it away from the structures and off the site to an acceptable disposal location or to a detention pond.

All roof downspouts shall be connected to a sealed tightline system, which discharges to an acceptable disposal location. In no case should these be connected to footing drains or subdrains beneath floors.

11.0 EROSION CONTROL

The site soils are moderately susceptible to erosion. The site grades are relatively flat; therefore, erosion should be low.

Construction Erosion Control. All disturbed areas shall have the low side surrounded by a silt fence with the bottom edge embedded in the soil at least three (3) inches. Also, at select locations settling ponds of hay-bale backed silt fence are typically established to decrease silt content of water flowing off site. Hay bales or wattles should be used to protect roadside ditches and cross culverts within 300 feet of the site (if water flows from the site can reach them).

The site will also require crushed rock (or shale) entrances to prevent "tracking" of soil by construction vehicles onto easements or public roadways. These are typically required to be 50 feet long and should be constructed of a 12” section of ASB rock over a woven fabric (more rock may be needed to protect the subgrade soils, especially in wet weather conditions).

Permanent Erosion Control. Permanent project landscaping and paving as required by the City/County will meet most needs of long-term erosion control. All disturbed areas on the site, outside of the structural developments of the project, must be reseeded with local native grasses for erosion prevention. Ideally, these areas would be graded reasonably smooth and the surface scarified to 1/2 inch deep, then hydroseeded with a combination of erosion control grass seed, fertilizer and mulch. Alternatively, and at a minimum, these areas should be covered with a thin layer of crushed rock.

12.0 ADDITIONAL SERVICES AND LIMITATIONS

12.1 ADDITIONAL SERVICES

We should review construction plans and specifications for this project as they are being developed. In addition, The Galli Group should be retained to review all geotechnical-related portions of the plans and specifications to evaluate whether they are in conformance with the recommendations provided in our report. Additionally, to observe compliance with the intent of our recommendations, design concepts, and the plans and specifications, all construction operations dealing with earthwork, foundations and structural fill placement and compaction should be observed by a representative from The Galli Group.

For this project, we anticipate additional services could include the following:

- Review of construction plans and specifications for compliance with geotechnical recommendations and to verify adverse conditions are not created. Such review must be accomplished prior to start of construction bidding.
- Possible project team meetings to clarify issues and proceed smoothly into and through the construction process.
- Observation and/or testing of over-excavated areas, subgrade preparation, subgrade proofrolling, structural fill placement and compaction, pavement subgrade preparation, footing subgrade verification, aggregate base placement and compaction, site grading, surface drainage, footing drainage, and floor drainage.
- Periodic construction field reports, as requested by the client and required by the building department.

We would provide these additional services on a time-and-expense basis in accordance with our current Standard Fee Schedule and General Conditions at the time of construction. If we are not retained to provide these services, we cannot be held responsible for the decisions by others or geotechnical related issues in the constructed product which we do not verify. The firm providing these services must then become the Geotechnical Engineer of Record for the project.

12.2 LIMITATIONS

The analyses, conclusions and recommendations contained in this report are based on site conditions and assumed development plans as they existed at the time of the study, and assume soils, rock and groundwater conditions exposed at the site and observed in the borings during our investigation are representative of soils and groundwater conditions throughout the site. If during construction, subsurface conditions or assumed design information is found to be different, we should be advised at once so that we can review this report and reconsider our recommendations in light of the changed conditions. If there is a significant lapse of time (5 years) between submission of this report and the start of work at the site, if the project is changed, or if conditions have changed due to acts of God or construction at or adjacent to the site, it is recommended that this report be reviewed considering the changed conditions and/or time lapse.

This report was prepared for the use of the Klamath Community College and their design and construction team for the design and construction of the project. It should be made available to contractors for information and factual data only. This report should not be used for contractual purposes as a warranty of site subsurface conditions. It should also not be used at other sites or for projects other than the one intended.

We have performed these services in accordance with generally accepted geotechnical engineering practices in the state of Oregon, at the time the study was accomplished. No other warranties, either expressed or implied, are provided.

THE GALLI GROUP
GEOTECHNICAL CONSULTING

Kristen S. Pierce, EIT
Engineering Associate

Dennis Duru

Dennis Duru, PE, CEG, RG.
Principal

REGISTERED PROFESSIONAL ENGINEER
100601PE
Dennis Duru
OREGON
JULY 12, 2022
DENNIS C. DURU
EXPIRES: 12/31/2025

CERTIFIED ENGINEERING GEOLOGIST
OREGON
DENNIS C. DURU
Dennis Duru
E2797
Expires:
01/01/2025

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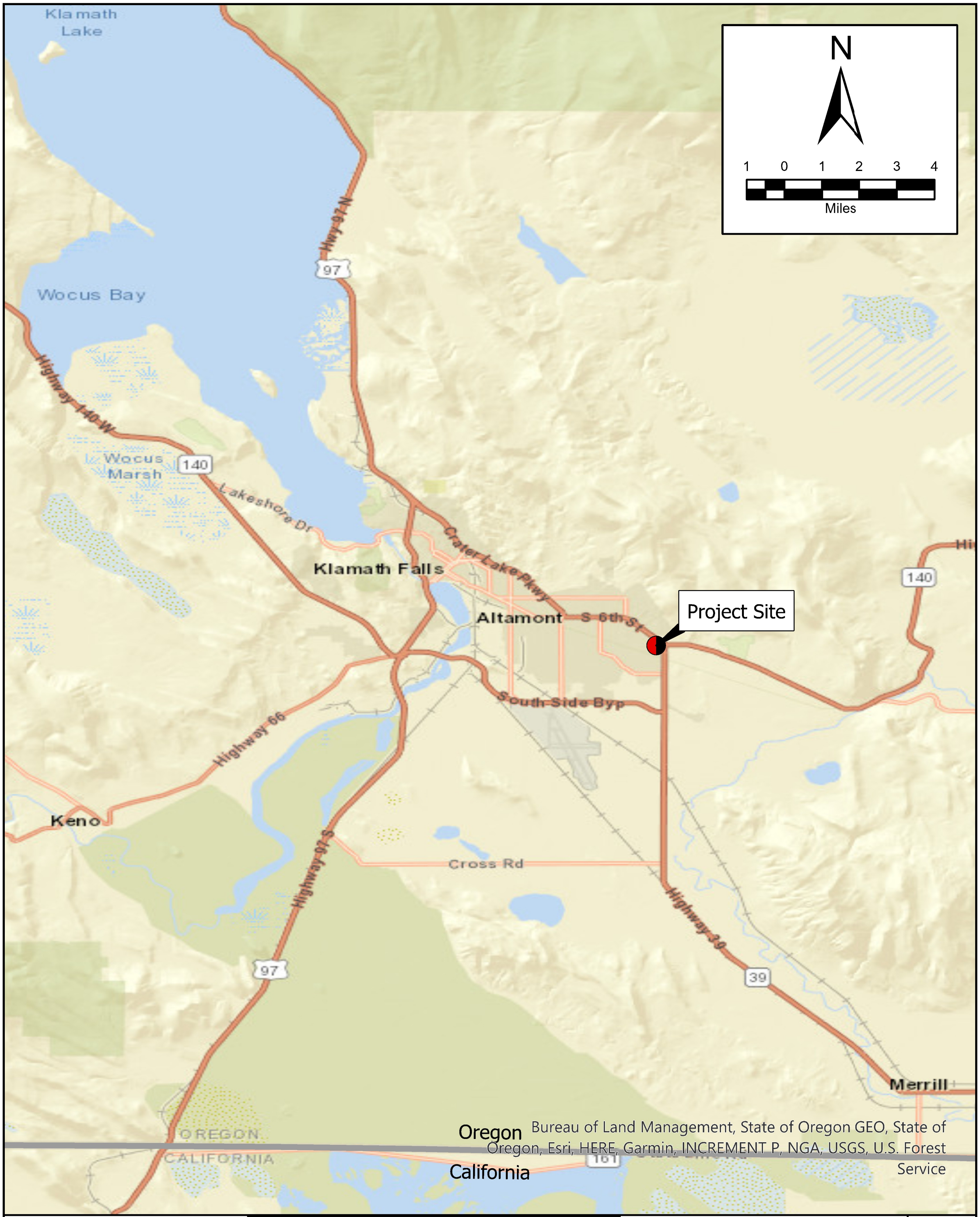

Bureau of Land Management, State of Oregon GEO, State of Oregon, Esri, HERE, Garmin, INCREMENT P, NGA, USGS, U.S. Forest Service

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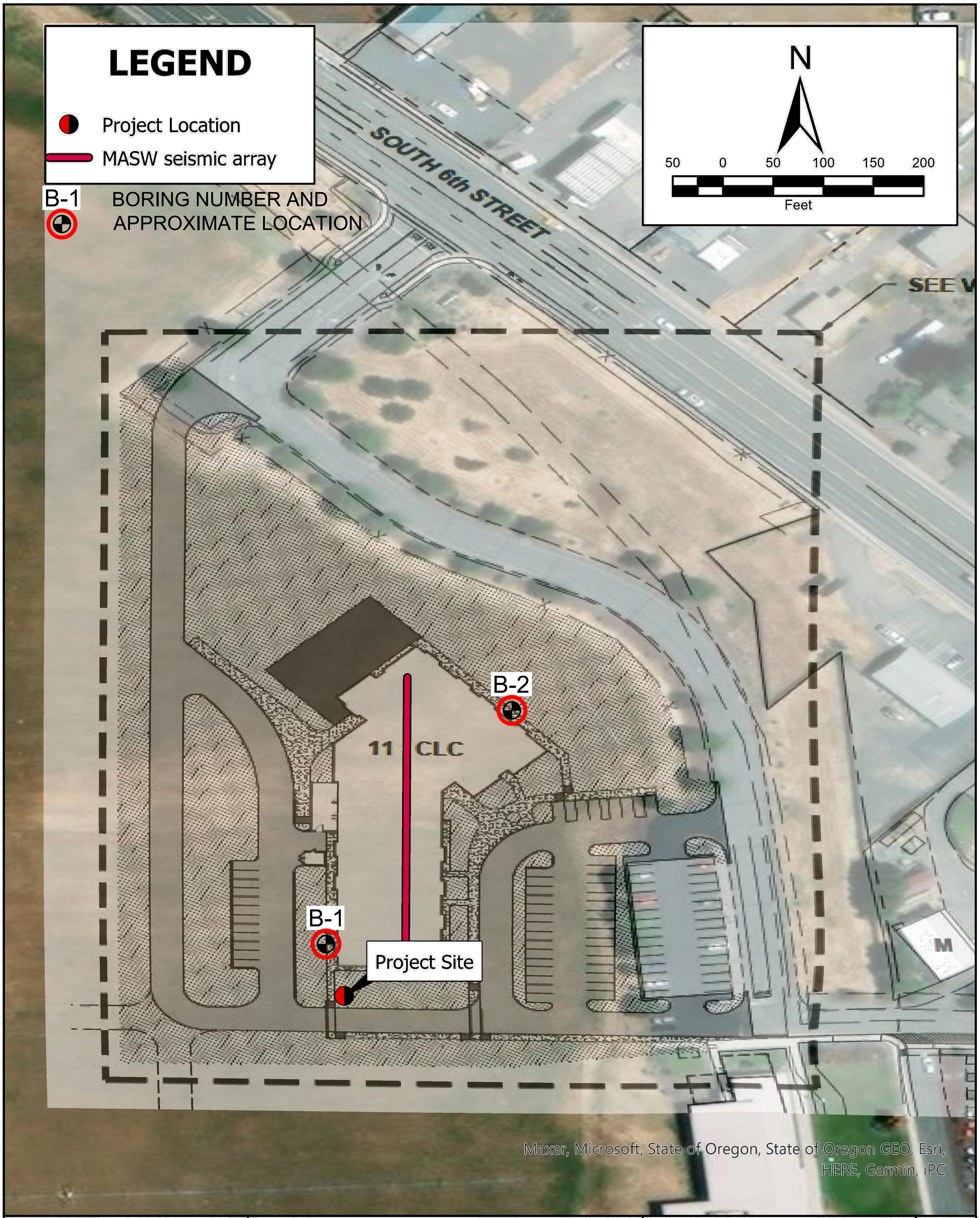
Grants Pass, Oregon 97526

VICINITY MAP

NEW CHILD LEARNING CENTER
KLAMATH COMMUNITY COLLEGE
7390 SOUTH 6TH STREET, KLAMATH FALLS, OR

DATE: MAY, 2025
JOB NO: 02-4434-04
REV: 05/21/2025 5:15 PM
PREPARED BY: KSP
4434-04: KCC Child Learning Center-01-Vicinity Map

FIGURE:
1



Maxar, Microsoft, State of Oregon, State of Oregon GEO, Esri, HERE, Garmin, iPC

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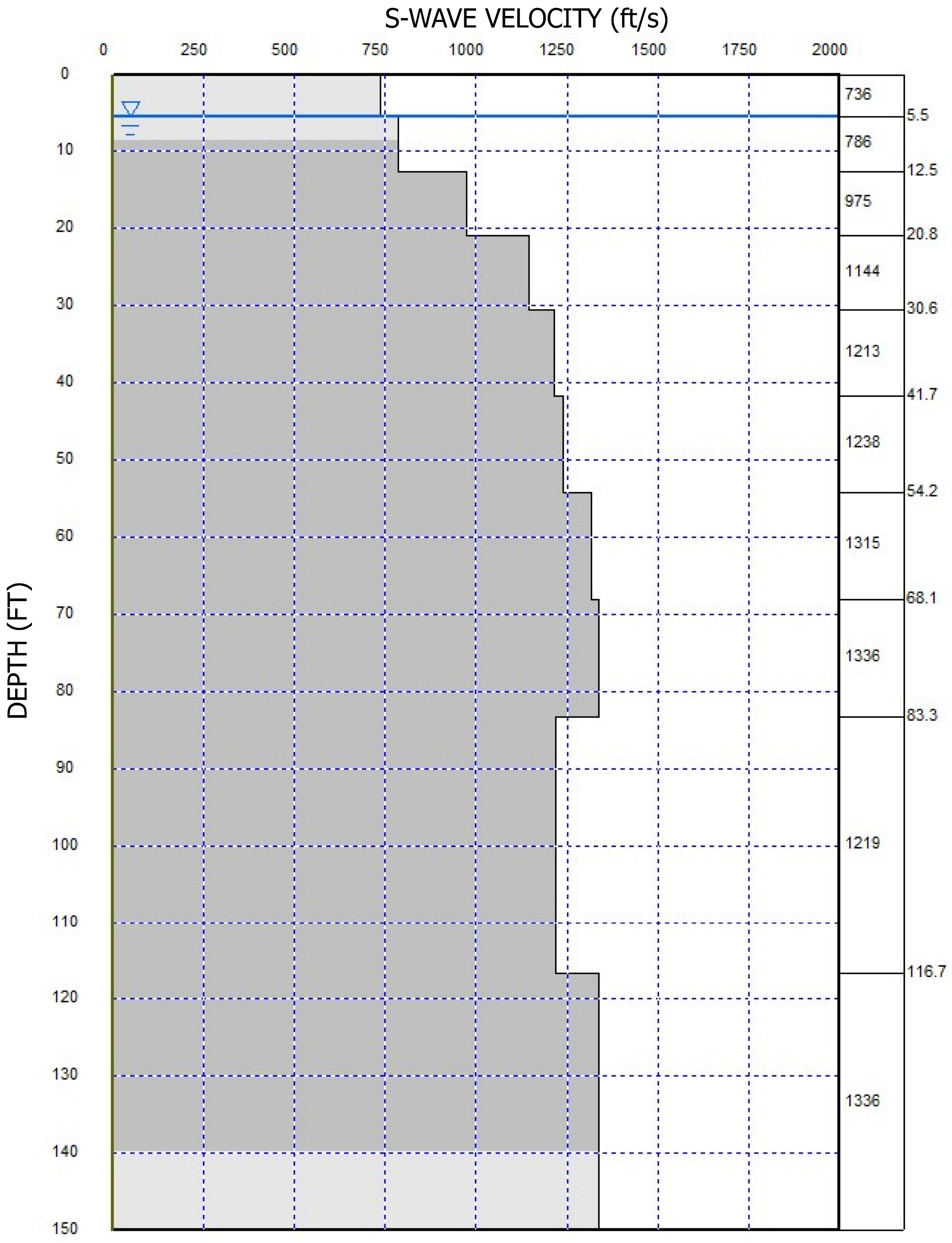
Grants Pass, Oregon 97526

SITE PLAN
WITH EXPLORATION LOCATIONS

NEW CHILD LEARNING CENTER
KLAMATH COMMUNITY COLLEGE
7390 SOUTH 6TH STREET, KLAMATH FALLS, OR

DATE: MAY, 2025
JOB NO: 02-4434-04
REV: 05/21/2025 5:11 PM
PREPARED BY: KSP
4434-04: KCC Child Learning Center-02-Site Plan

FIGURE: 2



Average Vs to 100 ft = 1132.4 ft/s
Site Class = D (Per Section 20 of ASCE 7-16)

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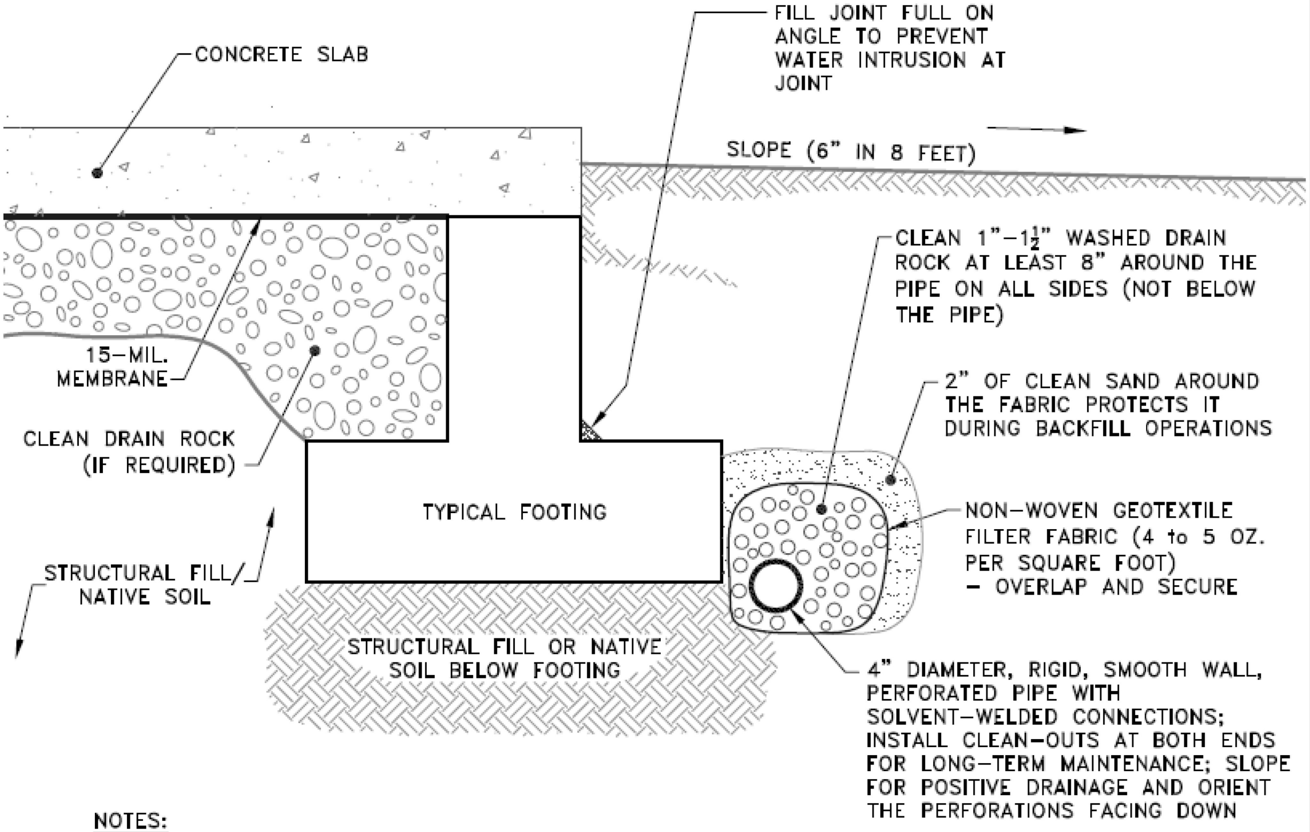
MASW 1-DIMENSIONAL SHEAR WAVE VELOCITY MODEL

NEW CHILD LEARNING CENTER
KLAMATH COMMUNITY COLLEGE
7390 SOUTH 6TH STREET, KLAMATH FALLS, OR

DATE: MAY 2025
JOB NO: 02-4434-04
REV: 05/27/2025 9:40 AM
PREPARED BY: KSP
4434-04: KCC Child Learning Center-03-MASW

FIGURE:
3



NOTES:

(1) VAPOR BARRIER TO BE STEGO INDUSTRIES 15mil STEGO WRAP OR EQUIVALENT.

(2) CAPILLARY BREAK ROCK BELOW VAPOR BARRIER TO BE 1/4" TO 3/4" CLEAN CRUSHED ROCK OR EQUIVALENT.

FOR ILLUSTRATION PURPOSES ONLY
NOT TO SCALE

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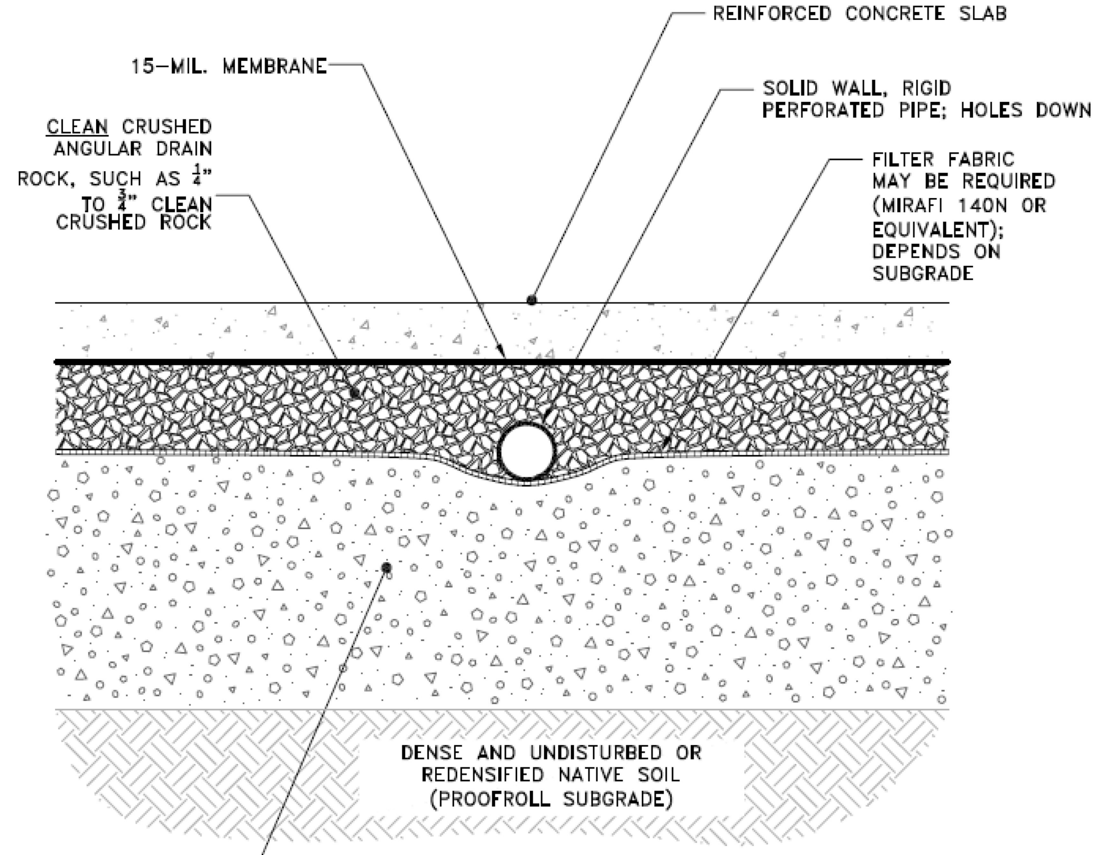
Grants Pass, Oregon 97526

TYPICAL FOUNDATION DRAIN SLAB ON GRADE FLOOR

NEW CHILD LEARNING CENTER
KLAMATH COMMUNITY COLLEGE
7390 SOUTH 6TH STREET, KLAMATH FALLS, OR

DATE: MAY 2025
JOB NO: 02-4434-04
REV: 05/27/2025 10:02 AM
PREPARED BY: KSP
4434-04: KCC Child Learning Center-04-fdn drain SOG

FIGURE: 4



NOTES:

(1) MAXIMUM SPACING IS 10 TO 15 FEET.

(2) ORIENT PIPE PERFORATIONS TO BOTTOM.

(3) ASSEMBLE PIPE USING SOLVENT–WELDED CONNECTIONS.

(4) DO NOT DRIVE OVER DRAIN LINES.

(5) DRAIN ROCK AND STRUCTURAL FILL TO MEET SPECS. IN REPORT BODY – SLOPE PIPE TO DRAIN.

(6) MAY REQUIRE FILTER FABRIC ON NATIVE SUBGRADE OR IF STRUCTURAL FILL IS VERY SILTY OR SANDY.

(7) BEST VAPOR BARRIER IS STEGO INDUSTRIES 15mil STEGO WRAP (OR EQUIVALENT).

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INTERIOR FLOOR SLAB SUBDRAIN DETAIL

NEW CHILD LEARNING CENTER
KLAMATH COMMUNITY COLLEGE
7390 SOUTH 6TH STREET, KLAMATH FALLS, OR

DATE: MAY 2025
JOB NO: 02-4434-04
REV: 05/27/2025 8:59 AM
PREPARED BY: KSP
4434-04: KCC Child Learning Center-05-slab subdrain

FIGURE:
5

APPENDIX A

BORING LOGS

BOREHOLE LOG:B-1

LOGGED BY: K.S. Pierce
START DATE: 2025-05-09
END DATE: 2025-05-09
CONTRACTOR: TGG
DRILL EQUIPMENT: ATV mounted 4" diameter solid stem auger

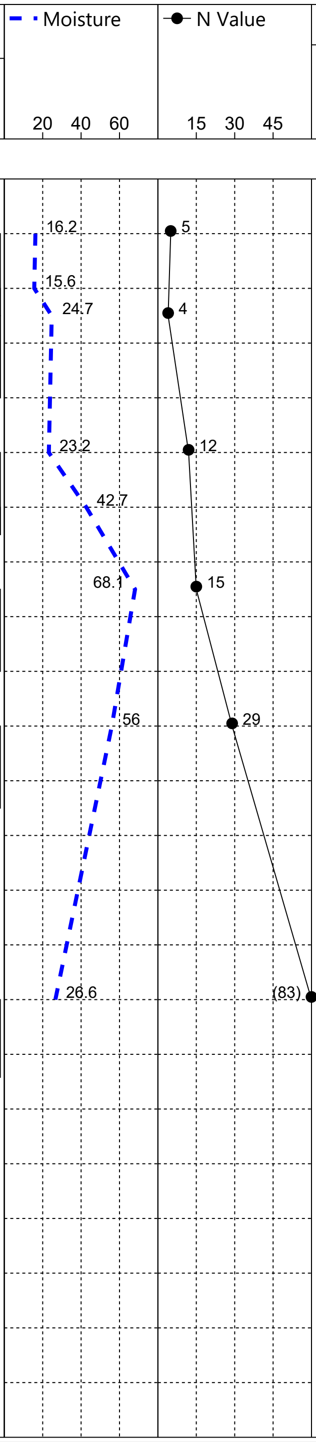
ADDRESS: 7390 South 6th Street, Klamath Falls, OR 97603

NOTES:

COORD.:42.19572, -121.70125

DEPTH TO WATER
∇ INITIAL: 4.5 ft
▼ FINAL: 6.5 ft

Lithologic Description	Samples No.	Samples Type	Gravel	Sand	Silt	Clay	Fines	ASTM D4318 LL (%)	ASTM D4318 PL (%)
Ground Surface									
Organic soil (OL), vegatation and root zone; dry 0.75 ft									
Medium dense, brown, Silt (ML), with sand and trace clay; dry to damp. 2.5 ft	S-1	▽							
Loose, brown, fine grained Sand (SP-SM), with silt and clay; damp to saturated. Fines content decreases with depth. Water content increases with depth. 6 ft	S-7 S-2	# ▽	0	76	13	11	24		
Stiff to hard, light brown, sandy, elastic elastic Silt (MH), with clay; moist.	S-3A S-3B	▽ ●							
	S-4	▽						71	58
13 ft	S-5	▽	0	30	58	12	70		
Very dense, brown with orange streaks, silty Sand (SM), with a little clay; moist to damp. SPT blow counts for S-6:12" - 18" were 50 in 5.0" 16.5 ft	S-6	▽							
Borehole B-1 Terminated at 16.5 ft									



Sample Legend: Bulk # Disturbed ● SPT Split Spoon ▽ Undisturbed Shelby Tube ☐

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PROJECT TITLE: New Child Learning Center Development-copy
PROJECT NO.: 02-4434-04
CLIENT: the-galli-group
SHEET: 1 of 1

BOREHOLE LOG:B-2

LOGGED BY: K.S. Pierce
START DATE: 2025-05-02
END DATE: 2025-05-02
CONTRACTOR:
DRILL EQUIPMENT:

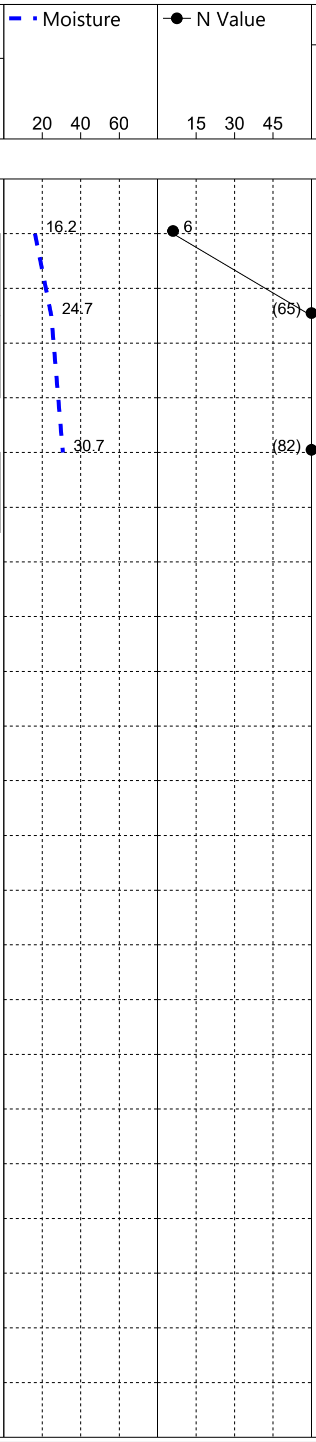
ADDRESS: 7390 South 6th Street, Klamath Falls, OR 97603
NOTES:
COORD.:42.19654, -121.70084

DEPTH TO WATER
INITIAL: N/A
FINAL: N/A

Ground Surface

Depth	Lithologic Description	Samples No.	Samples Type	Gravel	Sand	Silt	Clay	Fines	ASTM D4318 LL (%)	ASTM D4318 PL (%)
0 – 0.75 ft	Organic soil (OL), vegetation and root zone; dry.									
0.75 – 2.5 ft	Medium stiff, brown, sandy Silt (ML), with clay; damp.	S-1	▽							
2.5 – 6.5 ft	Very dense, brown, silty Sand (SM), with trace clay; damp. Some dark red sand lenses.	S-2	▽	0	64	31	5	36		
	SPT blow counts for S-3:12" - 18" were 50 in 5.5".	S-3	▽							

Borehole B-2 Terminated at 6.5 ft
Groundwater not encountered.



Depth: 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23

Sample Legend: Bulk # Disturbed ● SPT Split Spoon ▽ Undisturbed Shelby Tube ☐

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PROJECT TITLE: New Child Learning Center Development-copy
PROJECT NO.: 02-4434-04
CLIENT: the-galli-group
SHEET: 1 of 1

RSLog / TGG Borehole Soil Log with LABS / the-galli-group / admin / May 23, 2025 01:07 PM

APPENDIX B

LABORATORY TEST RESULTS

THE GALLI GROUP
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Grants Pass, Oregon 97526

Atterberg Limits Testing
ASTM D4318

Client: Klamath Community College
Project: KCC Child Learning Center
Job No. 4434-04
Date Sampled: 5/2/2025
Sample Location B1/4
Depth of Sample: 7.5-9.0'
Description of Soil: **Light brown elastic Silt**
Date Tested: 5/19/2025

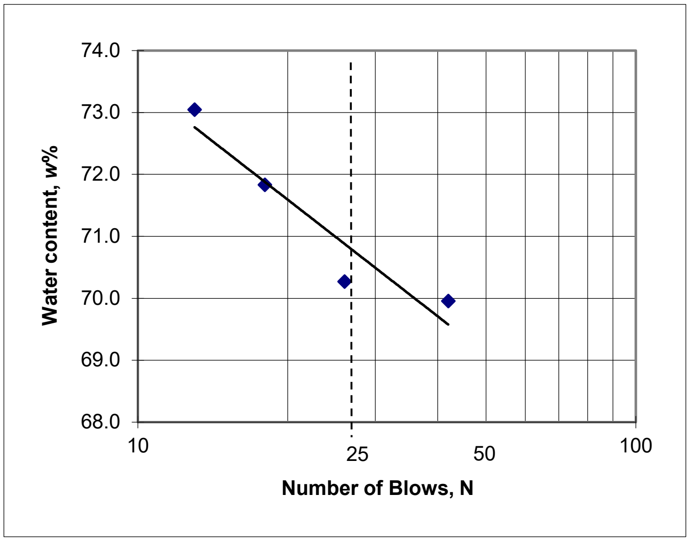
Liquid Limit Determination

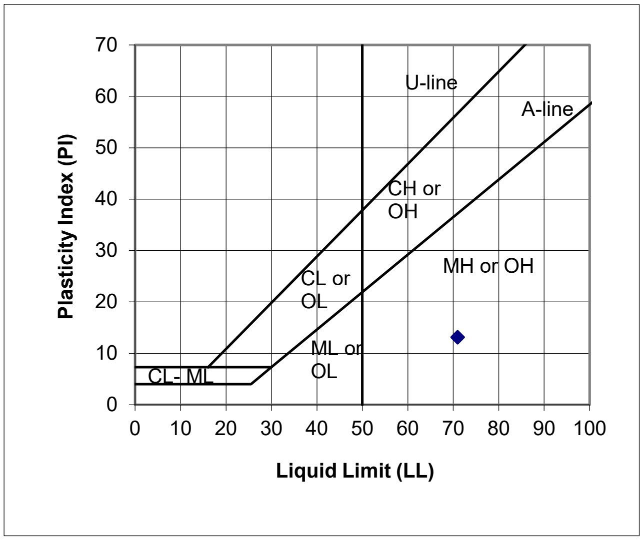
Can No.	M	Z	1	5
Wt. of wet soil + can (g)	24.24	26.72	25.48	25.73
Wt. of dry soil + can (g)	19.01	20.37	19.69	19.91
Wt. of can (g)	11.85	11.53	11.45	11.59
Wt. of dry soil (g)	7.16	8.84	8.24	8.32
Wt. of Moisture (g)	5.23	6.35	5.79	5.82
Water content, w%	73.0	71.8	70.3	70.0
No. of blows, N	13	18	26	42

Plastic Limit Determination

Can No.	L	J	6	202
Wt. of wet soil + can (g)	21.43	20.80	19.20	21.56
Wt. of dry soil + can (g)	17.90	17.52	16.40	17.91
Wt. of can (g)	11.72	11.90	11.57	11.63
Wt. of dry soil (g)	6.18	5.62	4.83	6.28
Wt. of Moisture (g)	3.53	3.28	2.80	3.65
Water content, w%	57.1	58.4	58.0	58.1

LIQUID LIMIT (LL)= 71
PLASTIC LIMIT (PL)= 58
PLASTICITY INDEX (PI)= 13





Tested by: Dakota Kinyon