EXHIBIT G

Geotechnical Report

Real-World Geotechnical Solutions
Investigation • Design • Construction Support

Geotechnical Engineering Report

The Riverlands Subdivision
1303 South River Street
Newberg, Oregon 97132

GeoPacific Engineering, Inc. Project No. 18-4860
April 17, 2018

14835 SW 72nd Avenue
Portland, Oregon 97224

Tel (503) 598-8445
Fax (503) 941-9281

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April 17, 2018
Project No. 18-4860

Katrina Little
The Riverlands, LLC
PO Box 3189
Newberg, Oregon 97132
Email: klittle@dbvcorp.com.com

SUBJECT: GEOTECHNICAL ENGINEERING REPORT
THE RIVERLANDS SUBDIVISION
1303 SOUTH RIVER STREET
NEWBERG, OREGON 97132

PROJECT INFORMATION

This report presents the results of a geotechnical engineering study conducted by GeoPacific Engineering, Inc. (GeoPacific) for the above-referenced project. The purpose of our investigation was to evaluate subsurface conditions at the site, and to provide geotechnical recommendations for site development. This geotechnical study was performed in accordance with GeoPacific Proposal No. P-6372, dated January 21, 2018, and your subsequent authorization of our proposal and *General Conditions for Geotechnical Services*.

SITE AND PROJECT DESCRIPTION

The subject property is located on the west side of S River Street in the City of Newberg, Yamhill County, Oregon. The property is approximately 1.5 acres in size and topography is flat to gently sloping. Ground elevations range from 169 to 172 feet above mean sea level. The site is currently occupied by one home, a garage, barn and several sheds on the east half of the property. Vegetation consists of numerous trees and grass lawn to the east and a grass field on the western portion. Some smaller areas of standing water were observed in the grass field at the western end of the site.

It is our understanding that proposed development includes 16 lots for single family homes, construction of a local public street, a storm water retention pond in the southwest corner and associated underground utilities. The existing structures will be removed. A grading plan was not provided for our review; however, we anticipate cuts and fill will be less than 4 feet.

REGIONAL GEOLOGIC SETTING

The subject site lies within the Willamette Valley/Puget Sound lowland, a broad structural depression situated between the Coast Range on the west and the Cascade Range on the east. A series of discontinuous faults subdivide the Willamette Valley into a mosaic of fault-bounded, structural blocks (Yeats et al., 1996). Uplifted structural blocks form bedrock highlands, while down-warped structural blocks form sedimentary basins.

The site is underlain by the Quaternary age (last 1.6 million years) Willamette Formation, a catastrophic flood deposit associated with repeated glacial outburst flooding of the Willamette Valley (Yeats et al., 1996). The last of these outburst floods occurred about 10,000 years ago. These deposits typically consist of horizontally layered, micaceous, silt to coarse sand forming poorly-defined to distinct beds less than 3 feet thick.

Underlying the Willamette Formation is an unnamed sequence of non-marine, fine-grained strata that consists of moderately to poorly lithified siltstone, sandstone, mudstone, and claystone with common wood fragments and minor volcanic ash and pumice (Yeats et al., 1996). These rocks are tentatively correlated with the Sandy River Mudstone, and the Troutdale and Helvetia Formations. The unnamed strata rest on Miocene (about 14.5 to 16.5 million years ago) Columbia River Basalt, a thick sequence of lava flows which forms the crystalline basement of the basin.

REGIONAL SEISMIC SETTING

At least three major fault zones capable of generating damaging earthquakes are thought to exist in the vicinity of the subject site. These include the Portland Hills Fault Zone, the Gales Creek-Newberg-Mt. Angel Structural Zone, and the Cascadia Subduction Zone.

Portland Hills Fault Zone

The Portland Hills Fault Zone is a series of NW-trending faults that include the central Portland Hills Fault, the western Oatfield Fault, and the eastern East Bank Fault. These faults occur in a northwest-trending zone that varies in width between 3.5 and 5.0 miles. The combined three faults vertically displace the Columbia River Basalt by 1,130 feet and appear to control thickness changes in late Pleistocene (approx. 780,000 years) sediment (Madin, 1990). The Portland Hills Fault occurs along the Willamette River at the base of the Portland Hills and is about 20 miles northeast of the site. The Oatfield Fault occurs along the western side of the Portland Hills and is about 18 miles northeast of the site. The East Bank Fault occurs along the eastern margin of the Willamette River, and is located approximately 23 miles northeast of the site. The accuracy of the fault mapping is stated to be within 500 meters (Wong, et al., 2000). No historical seismicity is correlated with the mapped portion of the Portland Hills Fault Zone, but in 1991 a M3.5 earthquake occurred on a NW-trending shear plane located 1.3 miles east of the fault (Yelin, 1992). Although there is no definitive evidence of recent activity, the Portland Hills Fault Zone is assumed to be potentially active (Geomatrix Consultants, 1995).

Gales Creek-Newberg-Mt. Angel Structural Zone

The Gales Creek-Newberg-Mt. Angel Structural Zone is a 50-mile-long zone of discontinuous, NW-trending faults that lies about 1/4 mile northeast of the subject site, as indicated on Figure 1. These faults are recognized in the subsurface by vertical separation of the Columbia River Basalt and offset seismic reflectors in the overlying basin sediment (Yeats et al., 1996; Werner et al., 1992). A geologic reconnaissance and photogeologic analysis study conducted for the Scoggins Dam site in the Tualatin Basin revealed no evidence of deformed geomorphic surfaces along the structural zone (Unruh et al., 1994). No seismicity has been recorded on the Gales Creek Fault or Newberg Fault (the fault closest to the subject site); however, these faults are considered to be potentially active because they may connect with the seismically active Mount Angel Fault and the rupture plane of the 1993 M5.6 Scotts Mills earthquake (Werner et al. 1992; Geomatrix Consultants, 1995).

Cascadia Subduction Zone

The Cascadia Subduction Zone is a 680-mile-long zone of active tectonic convergence where oceanic crust of the Juan de Fuca Plate is subducting beneath the North American continent at a rate of 4 cm per year (Goldfinger et al., 1996). A growing body of geologic evidence suggests that prehistoric subduction zone earthquakes have occurred (Atwater, 1992; Carver, 1992; Peterson et al., 1993; Geomatrix Consultants, 1995). This evidence includes: (1) buried tidal marshes recording episodic, sudden subsidence along the coast of northern California, Oregon, and Washington, (2) burial of subsided tidal marshes by tsunami wave deposits, (3) paleoliquefaction features, and (4) geodetic uplift patterns on the Oregon coast. Radiocarbon dates on buried tidal marshes indicate a recurrence interval for major subduction zone earthquakes of 250 to 650 years with the last event occurring 300 years ago (Atwater, 1992; Carver, 1992; Peterson et al., 1993; Geomatrix Consultants, 1995). The inferred seismogenic portion of the plate interface lies roughly along the Oregon coast at depths of between 20 and 40 miles.

FIELD EXPLORATION AND SUBSURFACE CONDITIONS

Our site-specific exploration for this report was conducted on March 19, 2018. A total of 3 exploratory test pits were excavated with a trackhoe to depths of 10.5 to 11 feet at the approximate locations indicated on Figure 2. It should be noted that test pit locations were located in the field by pacing or taping distances from apparent property corners and other site features shown on the plans provided. As such, the locations of the explorations should be considered approximate.

A GeoPacific geologist continuously monitored the excavations and logged the test pits. Soils observed in the explorations were classified in general accordance with the Unified Soil Classification System. During exploration geotechnical conditions such as soil consistency, moisture and groundwater conditions were noted. Logs of test pits are attached to this report. The following report sections are based on the conditions observed during our investigation and summarize subsurface materials encountered at the site.

Topsoil Horizon: Directly underlying the ground surface in all test pits was a topsoil horizon consisting of dark brown, highly organic SILT (OL-ML). The topsoil horizon was generally loose, contained fine roots throughout, and extended to depths of 8 to 10 inches. However, thicker topsoil extended to approximately 12 inches below the surface was observed in the vicinity of test pit TP-3, where several trees were located.

Undocumented Fill: A thick layer of topsoil, approximately 18 inches, mixed with aggregate and fragmented brick was observed in test pit TP-2. This undocumented fill appeared associated with the nearby barn and fill may be present in other area near existing structures.

Willamette Formation: Underlying the topsoil horizon in all test pits were fine-grained loess soils belonging to the Willamette Formation. From 0.5 to 3 feet below the surface, soils consisted of light gray, medium expansive FAT CLAY (CH) that was generally characterized by a soft to very stiff consistency. The fat clay was underlain by stiff to very stiff, SILT (ML) that was light brown with strong gray and rust colored mottling. Field pocket penetrometer measurements of Willamette Formation soils indicate an approximate unconfined compressive strength of 0.5 to 4.0 tons/ft^2.

Laboratory testing conducted on representative samples obtained within test pits TP-1 at 2 feet, indicated 95 percent by weight passing the U.S. No. 200 sieve, and a moisture content ranging from 34.5 percent. Atterberg limit testing indicated a liquid limit of 85, and a plasticity index of 60. The soil type classified as fat CLAY (CH) according to the USCS soil classification system, and as A-7-6(66) according to AASHTO standards.

Laboratory testing conducted on representative samples obtained within test pits TP-1 at 9 feet, indicated 81 percent by weight passing the U.S. No. 200 sieve, and a moisture content ranging from 39 percent. Atterberg limit testing indicated a liquid limit of 43, and a plasticity index of 16. The soil type classified as SILT (ML) with sand according to the USCS soil classification system, and as A-7-6(14) according to AASHTO standards.

Groundwater and Soil Moisture

On March 19, 2018, perched groundwater seepage was encountered in test pit TP-2 at a depth of 10.5 feet. Discharge was visually estimated at 1/8 gallon per minute. It is anticipated that groundwater conditions will vary depending on the season, local subsurface conditions, changes in site utilization, and other factors. Perched groundwater may be encountered in localized areas and small ponding was observed in the surface soils of the western portion of the site. Seeps and springs may exist in areas not explored and may become evident during site grading.

Infiltration Testing

The stand pipe and open hole method of infiltration testing were performed at test pit TP-1. The stand pipe method was performed at 7 feet below the ground surface in test pit TP-1 and a second pit for the open hole method was excavated several feet to the north. Soils were pre-saturated for a period of over 1 hour. Following the soil saturation, the infiltration tests were conducted. The water level was measured to the nearest sixteenth of an inch with reference to the ground surface.

Tests were conducted at half hour intervals and continued until two successive measurements did not vary by more than 1/16th of an inch. The total test period was 4 hours. Table 1 presents the results of our falling head infiltration tests.

Table 1. Summary of Infiltration Test Results

Exploration Designation	Depth (feet)	Soil Type	Infiltration Rate(in/hr)	Hydraulic Head Range (inches)
TP-1 Stand Pipe	7	SILT (ML)	0.0	12
TP-1 Open Hole	8.3	SILT (ML)	0.0	9

The results of our infiltration testing indicate the soils exhibit low permeability with a high probability of silting up over time.

CONCLUSIONS AND RECOMMENDATIONS

Our investigation indicates that the proposed development is geotechnically feasible, provided that the recommendations of this report are incorporated into the design and sufficient geotechnical monitoring is incorporated into the construction phases of the project. The primary geotechnical concerns for the proposed development are the presence of weak soils in the upper 2 feet and low permeability soils. No infiltration was observed during infiltration testing in test pit TP-1 at 8.3 feet below existing ground surface.

Site Preparation Recommendations

Areas of proposed buildings, new streets, and areas to receive fill should be cleared of vegetation and any organic and inorganic debris. Existing buried structures, should be demolished and any cavities structurally backfilled. Inorganic debris and organic materials from clearing should be removed from the site.

Existing fill and any organic-rich topsoil should then be stripped from construction areas of the site or where engineered fill is to be placed. The estimated depth necessary for removal of topsoil is approximately 8 to 10 inches – deeper stripping may be necessary to remove large tree roots in isolated areas or disturbed soils. One such location was identified in test pit TP-2, where approximately 18 inches of undocumented fill consisting of topsoil mixed with aggregate and brick fragments was identified to the west of the existing barn. The upper 2 feet can be removed and replaced with engineered fill to improve week soils. The final depth of soil removal will be determined on the basis of a site inspection after the stripping/ excavation has been performed. Stripped topsoil should preferably be removed from the site. Any remaining topsoil should be stockpiled only in designated areas and stripping operations should be observed and documented by the geotechnical engineer or his representative.

Any remaining undocumented fills and subsurface structures (tile drains, basements, driveway and landscaping fill, old utility lines, septic leach fields, etc.) should be removed and the excavations backfilled with engineered fill.

Once stripping of a particular area is approved, the area must be ripped or tilled to a depth of 12 inches, moisture conditioned, root-picked, and compacted in-place prior to the placement of engineered fill or crushed aggregate base for pavement. Exposed subgrade soils should be evaluated by the geotechnical engineer. For large areas, this evaluation is normally performed by proof-rolling the exposed subgrade with a fully loaded scraper or dump truck. For smaller areas where access is restricted, the subgrade should be evaluated by probing the soil with a steel probe. Soft/loose soils identified during subgrade preparation should be compacted to a firm and unyielding condition, over-excavated and replaced with engineered fill (as described below) or stabilized with rock prior to placement of engineered fill. The depth of over-excavation, if required, should be evaluated by the geotechnical engineer at the time of construction.

Engineered Fill

All grading for the proposed development should be performed as engineered grading in accordance with the applicable building code at time of construction with the exceptions and additions noted herein. Proper test frequency and earthwork documentation usually requires daily observation and testing during stripping, rough grading, and placement of engineered fill. Imported fill material must be approved by the geotechnical engineer prior to being imported to the site. Oversize material greater than 6 inches in size should not be used within 3 feet of foundation footings, and material greater than 12 inches in diameter should not be used in engineered fill.

Engineered fill should be compacted in horizontal lifts not exceeding 8 inches using standard compaction equipment. We recommend that engineered fill be compacted to at least 95% of the maximum dry density determined by ASTM D698 (Standard Proctor) or equivalent. Field density testing should conform to ASTM D2922 and D3017, or D1556. All engineered fill should be observed and tested by the project geotechnical engineer or his representative. Rocky fill may need to be evaluated by proofrolling and should be placed wet of optimum moisture content. Typically, one density test is performed for at least every 2 vertical feet of fill placed or every 500 yd^3, whichever requires more testing. Because testing is performed on an on-call basis, we recommend that the earthwork contractor be held contractually responsible for test scheduling and frequency.

Site earthwork will be impacted by soil moisture and shallow groundwater conditions. Earthwork in wet weather would likely require extensive use of cement or lime treatment, or other special measures, at considerable additional cost compared to earthwork performed under dry-weather conditions.

Excavating Conditions and Utility Trench Backfill

All temporary cuts in excess of 4 feet in height should be sloped in accordance with U.S. Occupational Safety and Health Administration (OSHA) regulations (29 CFR Part 1926) or be shored. The existing native soils classify as Type B Soil and temporary excavation side slope inclinations as steep as 1H:1V may be assumed for planning purposes. This cut slope inclination is applicable to excavations above the water table only. Maintenance of safe working conditions, including temporary excavation stability, is the responsibility of the contractor. Actual slope inclinations at the time of construction should be determined based on safety requirements and actual soil and groundwater conditions.

Saturated soils and groundwater may be encountered in utility trenches, particularly during the wet season. We anticipate that dewatering systems consisting of ditches, sumps and pumps would be adequate for control of perched groundwater. Regardless of the dewatering system used, it should be installed and operated such that in-place soils are prevented from being removed along with the groundwater.

Vibrations created by traffic and construction equipment may cause some caving and raveling of excavation walls. In such an event, lateral support for the excavation walls should be provided by the contractor to prevent loss of ground support and possible distress to existing or previously constructed structural improvements.

PVC pipe should be installed in accordance with the procedures specified in ASTM D2321. We recommend that trench backfill be compacted to at least 95% of the maximum dry density obtained by Modified Proctor ASTM D1557 or equivalent. Initial backfill lift thickness for a ¾"-0 crushed aggregate base may need to be as great as 4 feet to reduce the risk of flattening underlying flexible pipe. Subsequent lift thickness should not exceed 1 foot. If imported granular fill material is used, then the lifts for large vibrating plate-compaction equipment (e.g. hoe compactor attachments) may be up to 2 feet, provided that proper compaction is being achieved and each lift is tested. Use of large vibrating compaction equipment should be carefully monitored near existing structures and improvements due to the potential for vibration-induced damage.

Adequate density testing should be performed during construction to verify that the recommended relative compaction is achieved. Typically, one density test is taken for every 4 vertical feet of backfill on each 200-lineal-foot section of trench.

Erosion Control Considerations

During our field exploration program, we did not observe soil types that would be considered highly susceptible to erosion. In our opinion, the primary concern regarding erosion potential will occur during construction, in areas that have been stripped of vegetation. Erosion at the site during construction can be minimized by implementing the project erosion control plan, which should include judicious use of straw wattles and silt fences. If used, these erosion control devices should be in place and remain in place throughout site preparation and construction.

Erosion and sedimentation of exposed soils can also be minimized by quickly re-vegetating exposed areas of soil, and by staging construction such that large areas of the project site are not denuded and exposed at the same time. Areas of exposed soil requiring immediate and/or temporary protection against exposure should be covered with either mulch or erosion control netting/blankets. Areas of exposed soil requiring permanent stabilization should be seeded with an approved grass seed mixture, or hydroseeded with an approved seed-mulch-fertilizer mixture.

Wet Weather Earthwork

Soils underlying the site are likely to be moisture sensitive and may be difficult to handle or traverse with construction equipment during periods of wet weather. Earthwork is typically most economical when performed under dry weather conditions. Earthwork performed during the wet-weather season will probably require expensive measures such as cement treatment or imported granular material to compact areas where fill may be proposed to the recommended engineering specifications. If earthwork is to be performed or fill is to be placed in wet weather or under wet conditions when soil moisture content is difficult to control, the following recommendations should be incorporated into the contract specifications.

- Earthwork should be performed in small areas to minimize exposure to wet weather. Excavation or the removal of unsuitable soils should be followed promptly by the placement and compaction of clean engineered fill. The size and type of construction equipment used may have to be limited to prevent soil disturbance. Under some circumstances, it may be necessary to excavate soils with a backhoe to minimize subgrade disturbance caused by equipment traffic;
- The ground surface within the construction area should be graded to promote run-off of surface water and to prevent the ponding of water;
- Material used as engineered fill should consist of clean, granular soil containing less than 5 percent passing the No. 200 sieve. The fines should be non-plastic. Alternatively, cement treatment of on-site soils may be performed to facilitate wet weather placement;
- The ground surface within the construction area should be sealed by a smooth drum vibratory roller, or equivalent, and under no circumstances should be left uncompacted and exposed to moisture. Soils which become too wet for compaction should be removed and replaced with clean granular materials;
- Excavation and placement of fill should be observed by the geotechnical engineer to verify that all unsuitable materials are removed, and suitable compaction and site drainage is achieved; and
- Geotextile silt fences, straw wattles, and fiber rolls should be strategically located to control erosion.

If cement or lime treatment is used to facilitate wet weather construction, GeoPacific should be contacted to provide additional recommendations and field monitoring.

Spread Foundations

The proposed residential structures may be supported on shallow foundations bearing on competent native soils and/or engineered fill placed and compacted over competent native soils, appropriately designed and constructed as recommended in this report.

Foundation design, construction, and setback requirements should conform to the applicable building code at the time of construction. For maximization of bearing strength and protection against frost heave, spread footings should be embedded at a minimum depth of 12 inches below exterior grade. Foundations should be designed by a licensed structural engineer.

The anticipated allowable soil bearing pressure is 1,500 lbs/ft^2 for footings bearing on competent, native soil below a depth of 2 feet and/or engineered fill. A maximum chimney and column load of 30 kips is recommended for the site. The recommended maximum allowable bearing pressure may be increased by 1/3 for short-term transient conditions such as wind and seismic loading. For heavier loads, the geotechnical engineer should be consulted. The coefficient of friction between on-site soil and poured-in-place concrete may be taken as 0.42, which includes no factor of safety. The maximum anticipated total and differential footing movements (generally from soil expansion and/or settlement) are 1 inch and ¾ inch over a span of 20 feet, respectively. We anticipate that the majority of the estimated settlement will occur during construction, as loads are applied. Excavations near structural footings should not extend within a 1H:1V plane projected downward from the bottom edge of footings.

Footing excavations should penetrate through topsoil and any loose soil to competent subgrade that is suitable for bearing support. All footing excavations should be trimmed neat, and all loose or softened soil should be removed from the excavation bottom prior to placing reinforcing steel bars. Due to the moisture sensitivity of on-site native soils, foundations constructed during the wet weather season may require over-excavation of footings and backfill with compacted, crushed aggregate.

Our recommendations are for house construction incorporating raised wood floors and conventional spread footing foundations. If living space of the structures will incorporate basements, a geotechnical engineer should be consulted to make additional recommendations for retaining walls, water-proofing, underslab drainage and wall subdrains. After site development, a Final Soil Engineer's Report should either confirm or modify the above recommendations.

Drainage

The upslope edge of perimeter footings may be provided with a drainage system consisting of 3-inch diameter, slotted, plastic pipe embedded in a minimum of 1 ft^3 per lineal foot of clean, free-draining gravel or uncompacted 3/4"-0 rock. Water collected from the footing drains should be directed into the local storm drain system or another suitable outlet. A minimum 0.5 percent fall should be maintained throughout the drain and non-perforated pipe outlet. Down spouts and roof drains should not be connected to the foundation drains in order to reduce the potential for clogging. The footing drains should include clean-outs to allow periodic maintenance and inspection. Grades around the proposed structure should be sloped such that surface water drains away from the building. Footing drain recommendations are given to prevent detrimental effects of groundwater on foundations and should not be expected to eliminate all potential sources of water entering a crawlspace. An adequate grade to a low point outlet drain in the crawlspace is required by code.

Flexible Pavement Design: Local Public Street

We understand that development at the site will included the construction of a new public street to provide access to the new homes and support a 75,000-gross vehicle weight emergency vehicle. We assume that traffic will primarily consist of light duty residential cars, weekly trash and recycling pickups, and occasional fire trucks. The new street is designed with a cul-de-sac at the western end, preventing any through traffic. We assumed an 18-kip ESAL count of 53,248 over 20 years, accounting for projected population growth. Table 1 presents our flexible pavement design input factors.

Table 1 – Flexible Pavement Section Design Input Factors for Local Public Street

Input Parameter	Design Value
18-kip ESAL Initial Performance Period (20 Years)	53,248
Initial Serviceability	4.2
Terminal Serviceability	2.5
Reliability Level	85 Percent
Overall Standard Deviation	0.5
Roadbed Soil Resilient Modulus (PSI)	7,5000
Structural Number	**2.09**

Table 2 presents our recommended minimum dry-weather pavement section with estimated structural coefficients. Pavement design calculations are attached to this report.

Table 2 – Recommended Minimum Dry-Weather Pavement Section for Local Public Street

Material Layer	Section Thickness (in.)	Structural Coefficient	Compaction Standard
Asphaltic Concrete (AC)	3	0.42	91%/ 92% of Rice Density AASHTO T-209
Crushed Aggregate Base ¾"-0 (leveling course)	2	0.10	95% of Modified Proctor AASHTO T-180
Crushed Aggregate Base 1½"-0	8	0.10	95% of Modified Proctor AASHTO T-180
Subgrade	12	7,500 PSI	95% of Standard Proctor AASHTO T-99 or equivalent
Calculated Structural Number			**2.26**

The subgrade should be ripped or tilled to a depth of 12 inches, moisture conditioned, root-picked, and compacted in-place prior to the placement of crushed aggregate base for pavement. Any pockets of organic debris or loose fill encountered during ripping or tilling should be removed and replaced with engineered fill (see *Site Preparation* section). In order to verify subgrade strength, we recommend proof-rolling directly on subgrade with a loaded dump truck during dry weather and on top of base course in wet weather. Soft areas that pump, rut, or weave should be stabilized prior to paving.

If pavement areas are to be constructed during wet weather, the subgrade and construction plan should be reviewed by the project geotechnical engineer at the time of construction so that condition specific recommendations can be provided. The moisture sensitive subgrade soils make the site a difficult wet weather construction project. General recommendations for wet weather pavement sections are provided below.

During placement of pavement section materials, density testing should be performed to verify compliance with project specifications. Generally, one subgrade, one base course, and one asphalt compaction test is performed for every 100 to 200 linear feet of paving.

Wet Weather Construction Pavement Section

This section presents our recommendations for wet weather pavement section and construction for new pavement sections at the project. These wet weather pavement section recommendations are intended for use in situations where it is not feasible to compact the subgrade soils, due to wet subgrade soil conditions, and/or construction during wet weather.

Based on our site review, we recommend a wet weather section with a minimum subgrade deepening of 6 inches to accommodate a working subbase of additional 1½"-0 crushed rock. Geotextile fabric, Mirafi 500x or equivalent, should be placed on subgrade soils prior to placement of base rock.

In some instances, it may be preferable to use Special Treated Base (STB) in combination with overexcavation and increasing the thickness of the rock section. GeoPacific should be consulted for additional recommendations regarding use of STB in wet weather pavement sections if it is desired to pursue this alternative. Cement treatment of the subgrade may also be considered instead of overexcavation. For planning purposes, we anticipate that treatment of the onsite soils would involve mixing cement powder to approximately 6 percent cement content and a mixing depth on the order of 12 to 18 inches.

With implementation of the above recommendations, it is our opinion that the resulting pavement section will provide equivalent or greater structural strength than the dry weather pavement section currently planned. However, it should be noted that construction in wet weather is risky and the performance of pavement subgrades depend on a number of factors including the weather conditions, the contractor's methods, and the amount of traffic the road is subjected to. There is a potential that soft spots may develop even with implementation of the wet weather provisions recommended in this letter. If soft spots in the subgrade are identified during roadway excavation, or develop prior to paving, the soft spots should be overexcavated and backfilled with additional crushed rock.

During subgrade excavation, care should be taken to avoid disturbing the subgrade soils. Removals should be performed using an excavator with a smooth-bladed bucket. Truck traffic should be limited until an adequate working surface has been established. We suggest that the crushed rock be spread using bulldozer equipment rather than dump trucks, to reduce the amount of traffic and potential disturbance of subgrade soils.

Care should be taken to avoid overcompaction of the base course materials, which could create pumping, unstable subgrade soil conditions. Heavy and/or vibratory compaction efforts should be applied with caution. Following placement and compaction of the crushed rock to project specifications (95 percent of Modified Proctor), a finish proof-roll should be performed before paving.

The above recommendations are subject to field verification. GeoPacific should be on-site during construction to verify subgrade strength and to take density tests on the engineered fill, base rock and asphaltic pavement materials.

Seismic Design and Soil Liquefaction

The Oregon Department of Geology and Mineral Industries (Dogami), Oregon HazVu: 2018 Statewide GeoHazards Viewer indicates that the site is in an area where *very strong* ground shaking is anticipated during an earthquake (Dogami HazVu, 2018). Structures should be designed to resist earthquake loading in accordance with the methodology described in the 2015 International Building Code (IBC) with applicable Oregon Structural Specialty Code (OSSC) revisions (current 2014). We recommend Site Class D be used for design per the OSSC, Table 1613.5.2 and as defined in ASCE 7, Chapter 20, Table 20.3-1. Design values determined for the site using the USGS (United States Geological Survey) 2018 Seismic Design Maps Summary Report are summarized in Table 3 and are based upon existing soil conditions.

Table 3 - Recommended Earthquake Ground Motion Factors (2010 ASCE-7)

Parameter	Value
Location (Lat, Long), degrees	45.289026, -122.969614
Probabilistic Ground Motion Values, 2% Probability of Exceedance in 50 yrs	
Mean Peak Ground Acceleration	0.457 g
Short Period, S_s	0.952 g
1.0 Sec Period, S_1	0.436 g
Soil Factors for Site Class D:	
F_a	1.119
F_v	1.564
$SD_s = 2/3 \times F_a \times S_s$	0.710 g
$SD_1 = 2/3 \times F_v \times S_1$	0.454 g
Seismic Design Category	D

Soil liquefaction is a phenomenon wherein saturated soil deposits temporarily lose strength and behave as a liquid in response to earthquake shaking. Soil liquefaction is generally limited to loose, granular soils located below the water table. According to the Oregon HazVu: Statewide

Geohazards Viewer, the subject site is regionally characterized as having a *moderate* risk of soil liquefaction (DOGAMI:HazVu, 2018).

For construction of single family structures, special design or construction measures are not required by code to mitigate the effects of liquefaction. However, GeoPacific may be consulted to perform further study of seismic hazards on the site if desired. If multi-family residential, high occupancy, or critical structures were to be incorporated into plans for site development, further study and evaluation of seismic hazards would be required by code to more fully evaluate the potential adverse effects due to liquefaction. We anticipate that our additional explorations on the site for the purpose of evaluating seismic hazards would include at least two cone penetrometer tests.

UNCERTAINTIES AND LIMITATIONS

We have prepared this report for the owner and his/her consultants for use in design of this project only. The conclusions and interpretations presented in this report should not be construed as a warranty of the subsurface conditions. Experience has shown that soil and groundwater conditions can vary significantly over small distances. Inconsistent conditions can occur between explorations that may not be detected by a geotechnical study. If, during future site operations, subsurface conditions are encountered which vary appreciably from those described herein, GeoPacific should be notified for review of the recommendations of this report, and revision of such if necessary.

Within the limitations of scope, schedule and budget, GeoPacific executed these services in accordance with generally accepted professional principles and practices in the fields of geotechnical engineering and engineering geology at the time the report was prepared. No warranty, express or implied, is made. The scope of our work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous or toxic substances in the soil, surface water, or groundwater at this site.

We appreciate this opportunity to be of service.

Sincerely,

GEOPACIFIC ENGINEERING, INC.

REGISTERED PROFESSIONAL ENGINEER
14743PE
OREGON
JAN. 23, 1990
JAMES D. IMBRIE

EXPIRES: 06/30/2019

Michael T. Baker
Geotechnical Staff

James D. Imbrie, G.E., C.E.G.
Principal Geotechnical Engineer

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CHECKLIST OF RECOMMENDED GEOTECHNICAL TESTING AND OBSERVATION

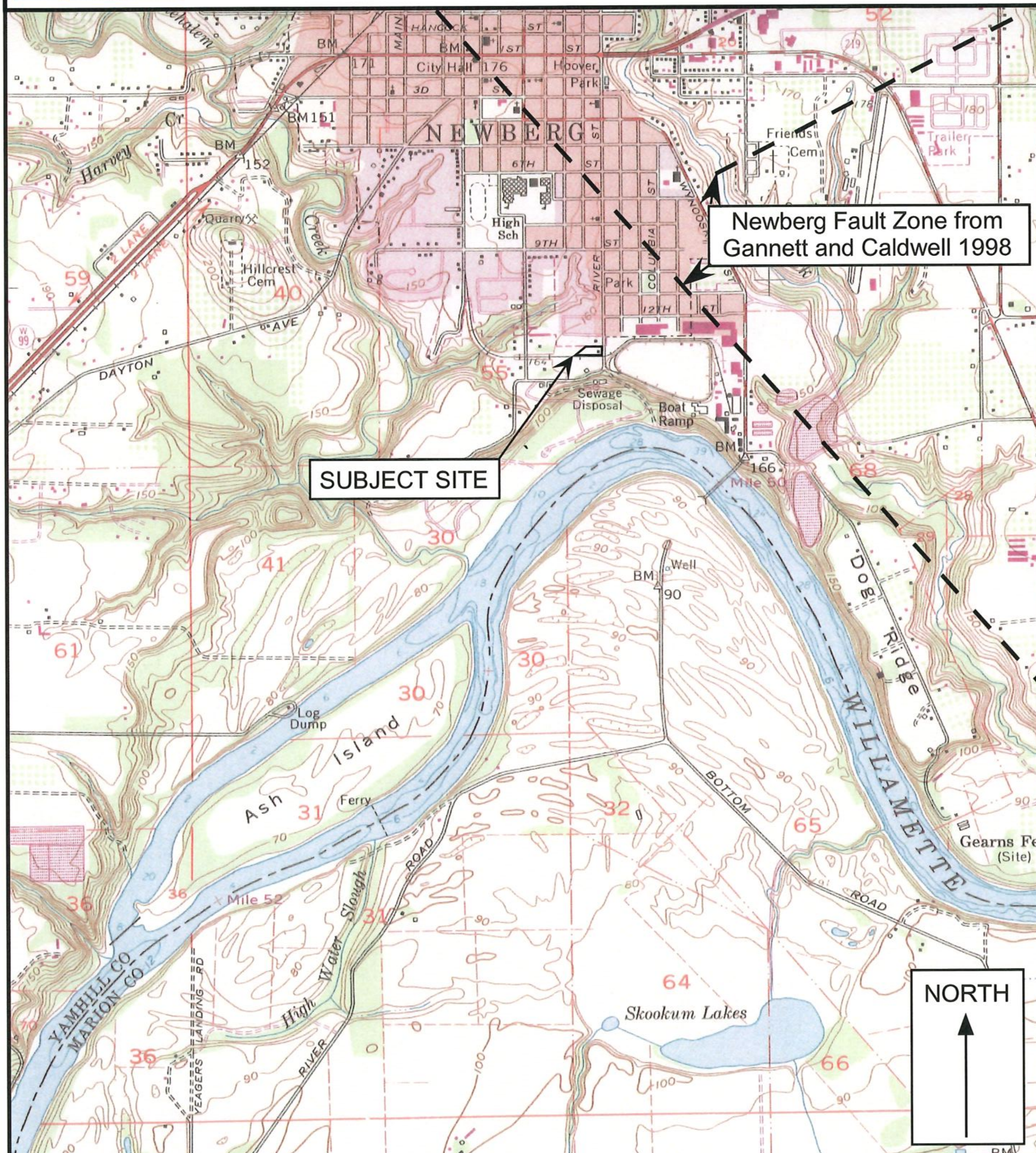
Item No.	Procedure	Timing	By Whom	Done
1	Preconstruction meeting	Prior to beginning site work	Contractor, Developer, Civil and Geotechnical Engineers	
2	Fill removal from site or sorting and stockpiling	Prior to mass stripping	Soil Technician/ Geotechnical Engineer	
3	Stripping, aeration, and root-picking operations	During stripping	Soil Technician	
4	Compaction testing of engineered fill (95% of Standard Proctor)	During filling, tested every 2 vertical feet	Soil Technician	
5	Compaction testing of trench backfill (95% of Standard Proctor)	During backfilling, tested every 4 vertical feet for every 200 lineal feet	Soil Technician	
6	Street Subgrade Inspection	Prior to placing base course	Soil Technician	
7	Base course compaction (95% of Modified Proctor)	Prior to paving, tested every 200 lineal feet	Soil Technician	
8	Footing Subgrade Inspection	Prior to placement of forms	Soil Technician/ Geotechnical Engineer	
9	Final Geotechnical Engineer's Report	Completion of project	Geotechnical Engineer	

FIGURES

GeoPacific Engineering, Inc.

14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

VICINITY MAP



Legend

Approximate Scale 1 in = 2,000 ft

Date: 3.20.2018
Drawn by: MTB

Base map: U.S. Geological Survey 7.5 minute Topographic Map Series, Newberg, Oregon Quadrangle, 1961 (Photorevised in 1985).

Project: The Riverlands Subdivision Newberg, Oregon	Project No. 18-4860	FIGURE 1

GeoPacific Engineering, Inc.
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SITE PLAN AND EXPLORATION LOCATIONS

Legend

TP-1 Test Pit Designation and Approximate Location

Site Boundary

0 50'
APPROXIMATE SCALE 1"=50'

Date: 3.20.2018
Drawn by: MTB

Project: The Riverlands Subdivision Newberg, Oregon	Project No. 18-4860	FIGURE 2

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EXPLORATION LOGS

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Tel: (503) 598-8445 Fax: (503) 941-9281

TEST PIT LOG

Project: The Riverlands Subdivision Newberg, Oregon	Project No. 18-4860	Boring No. **TP-1**

Depth (ft)	Sample Type	tons/sq.ft.	Moisture Content (%)	Water Bearing Zone	Material Description
0–0.5					Soft, highly organic SILT (OL), dark brown, grass roots and plant litter, moist to wet [Topsoil]
0.5–3.0	Bag Sample (100 to 1,000 g) at 2	0.5 (at 1); 1.5 (at 2)			Soft to stiff, FAT CLAY (CH), light gray, faint orange mottling, high plasticity, medium expansivity, homogenous, moist [Willamette Formation]
3.0–4.5	Bag Sample (100 to 1,000 g) at 4	4.0 (at 3); 2.5 (at 4)			Very stiff, lean CLAY (CL), light brown, moderate plasticity, homogenous, moist [Willamette Formation]
4.5–8.0					Very stiff, SILT (ML), trace sand, light brown, low plasticity, friable, homogenous, small holes less than 1/8 inch in diameter, damp [Willamette Formation] Infiltration test IT-1 conducted at -8.3 feet bgs. Measured hydraulic conductivity (k) = 0 inches per hour.
8.0–10.5	Bag Sample (100 to 1,000 g) at 9				Very stiff, sandy SILT (SM), fine to medium sand, light brown, low plasticity, homogenous, small holes less than 1/8 inch in diameter, moist [Willamette Formation]
10.5–14					Test Pit terminated at 10.5 feet. No groundwater or seepage encountered in excavation.

LEGEND: Bag Sample (100 to 1,000 g); Split-Spoon; Shelby Tube Sample; Seepage; Static Water Table; Water Bearing Zone

Date Excavated: 3.19.2018
Logged By: MTB
Surface Elevation: 169 Feet

GeoPacific Engineering, Inc.
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TEST PIT LOG

Project: The Riverlands Subdivision, Newberg, Oregon	Project No. 18-4860	Boring No. TP-2

Depth (ft)	Sample Type	tons/sq.ft.	Moisture Content (%)	Water Bearing Zone	Material Description
0–1.5		0.25			Soft, highly organic SILT (OL), dark brown, grass roots and plant litter, crushed aggregate and brick debris, moist to wet [Topsoil/Fill]
1.5–3		2.0			Medium stiff to stiff, FAT CLAY (CH), light gray, faint orange mottling, high plasticity, medium expansivity, homogenous, moist [Willamette Formation]
3–5.5		3.0 3.5			Very stiff, lean CLAY (CL), light brown, moderate plasticity, homogenous, moist [Willamette Formation]
5.5–8					Very stiff, SILT (ML), trace sand, light brown, low plasticity, friable, homogenous, small holes less than 1/8 inch in diameter, damp [Willamette Formation]
8–11				Seepage (10.5)	Very stiff, sandy SILT (SM), fine to medium sand, light brown, low plasticity, homogenous, small holes less than 1/8 inch in diameter, moist [Willamette Formation]
11					Test Pit terminated at 11 feet. Perched groundwater seepage encountered in excavation at 10.5 Feet, Discharge visually estimated at less than 1/8 gallon per minute.

LEGEND

Bag Sample (100 to 1,000 g) · Split-Spoon · Shelby Tube Sample · Seepage · Static Water Table · Water Bearing Zone

Date Excavated: 3.19.2018
Logged By: MTB
Surface Elevation: 171 Feet

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TEST PIT LOG

Project: The Riverlands Subdivision, Newberg, Oregon	Project No. 18-4860	Boring No. TP-3

Depth (ft)	Sample Type	tons/sq.ft.	Moisture Content (%)	Water Bearing Zone	Material Description
0–1.5		0.25 (at 1)			Soft, highly organic SILT (OL), dark brown, grass roots and plant litter, crushed aggregate and brick debris, moist to wet [Topsoil]
1.5–3		1.5 (at 2)			Medium stiff to stiff, FAT CLAY (CH), light gray, faint orange mottling, high plasticity, medium expansivity, homogenous, moist [Willamette Formation]
3–5.5		2.5 (at 3); 2.5 (at 4)			Very stiff, lean CLAY (CL), light brown, moderate plasticity, homogenous, moist [Willamette Formation]
5.5–8					Very stiff, SILT (ML), trace sand, light brown, low plasticity, friable, homogenous, small holes less than 1/8 inch in diameter, damp [Willamette Formation]
8–11					Very stiff, sandy SILT (SM), fine to medium sand, light brown, low plasticity, homogenous, small holes less than 1/8 inch in diameter, moist [Willamette Formation]
11					Test Pit terminated at 11 feet. No groundwater or seepage encountered in excavation.

LEGEND

Bag Sample (100 to 1,000 g) | Split-Spoon | Shelby Tube Sample | Seepage | Static Water Table | Water Bearing Zone

Date Excavated: 3.19.2018
Logged By: MTB
Surface Elevation: 171 Feet

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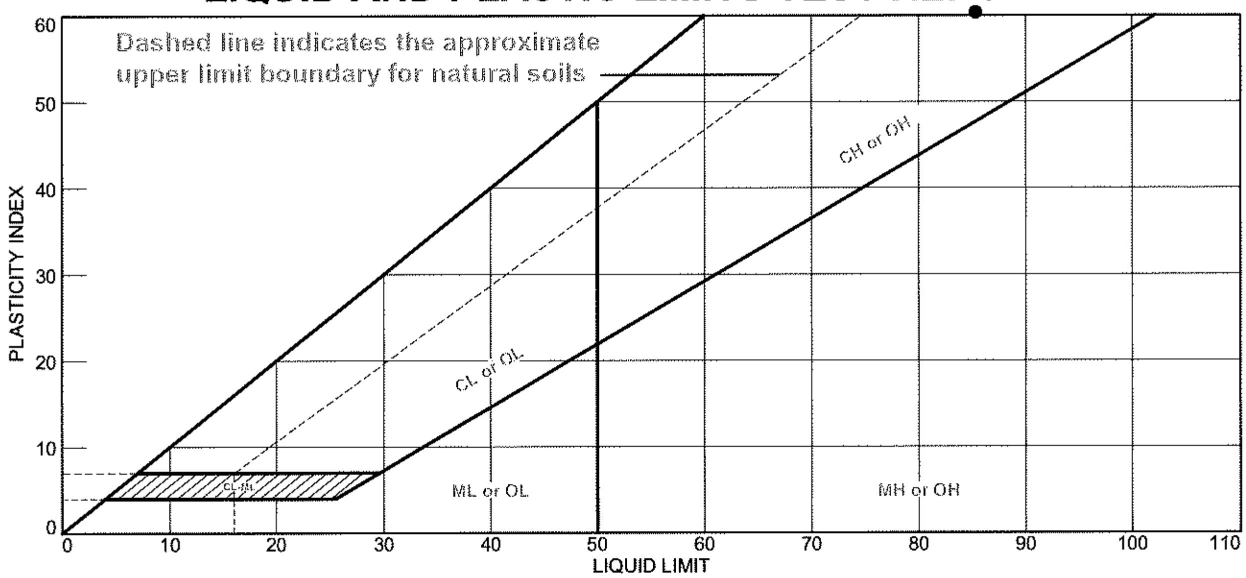
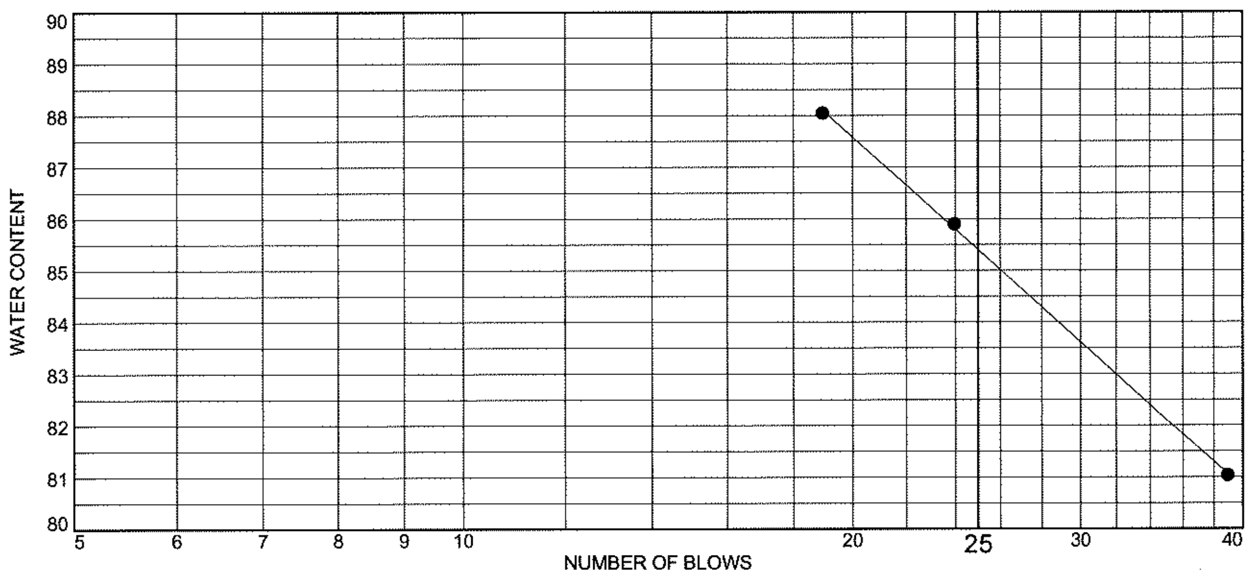
LABORATORY TEST RESULTS

LIQUID AND PLASTIC LIMITS TEST REPORT



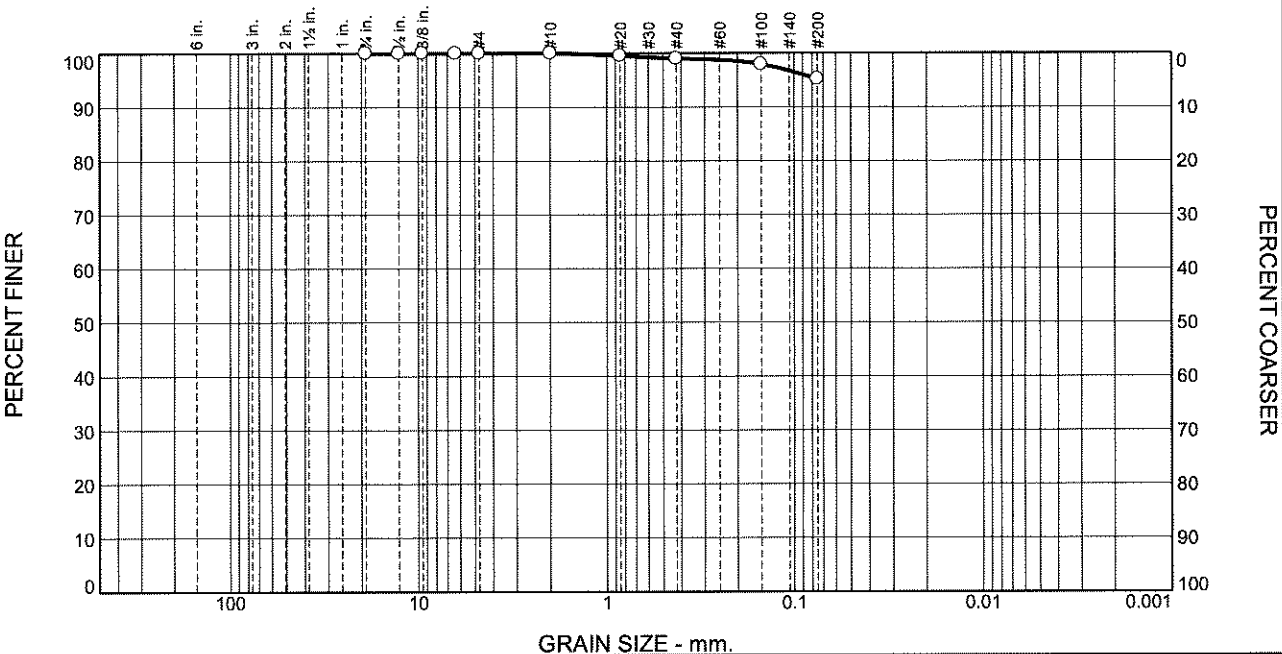


	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Fat Clay	85.4	25.1	60.3	99.0	95.2	CH

Project No. 18-4860 **Client:** Riverlands, LLC **Project:** The Riverlands Subdivision **Location:** TP-1 **Sample Number:** S18-081 **Depth:** 2'	**Remarks:**
GEOPACIFIC ENGINEERING, INC.	**Figure**

Tested By: SJC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	1.0	3.8	95.2	

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
.75	100.0		
.5	100.0		
.375	100.0		
.25	100.0		
#4	100.0		
#10	100.0		
#20	99.5		
#40	99.0		
#100	97.9		
#200	95.2		

* (no specification provided)

Material Description

Fat Clay

Atterberg Limits (ASTM D 4318)

PL= 25.1 **LL=** 85.4 **PI=** 60.3

Classification

USCS (D 2487)= CH **AASHTO (M 145)=** A-7-6(66)

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Moisture 34.5%

Date Received: **Date Tested:** 3/23/2018

Tested By: SJC

Checked By:

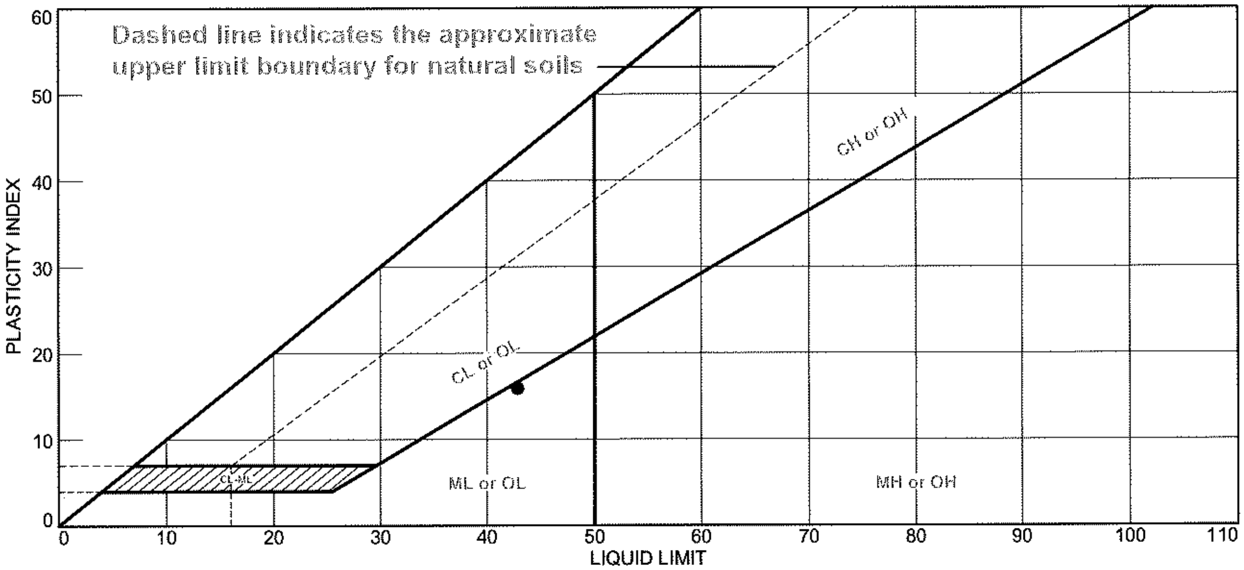
Title:

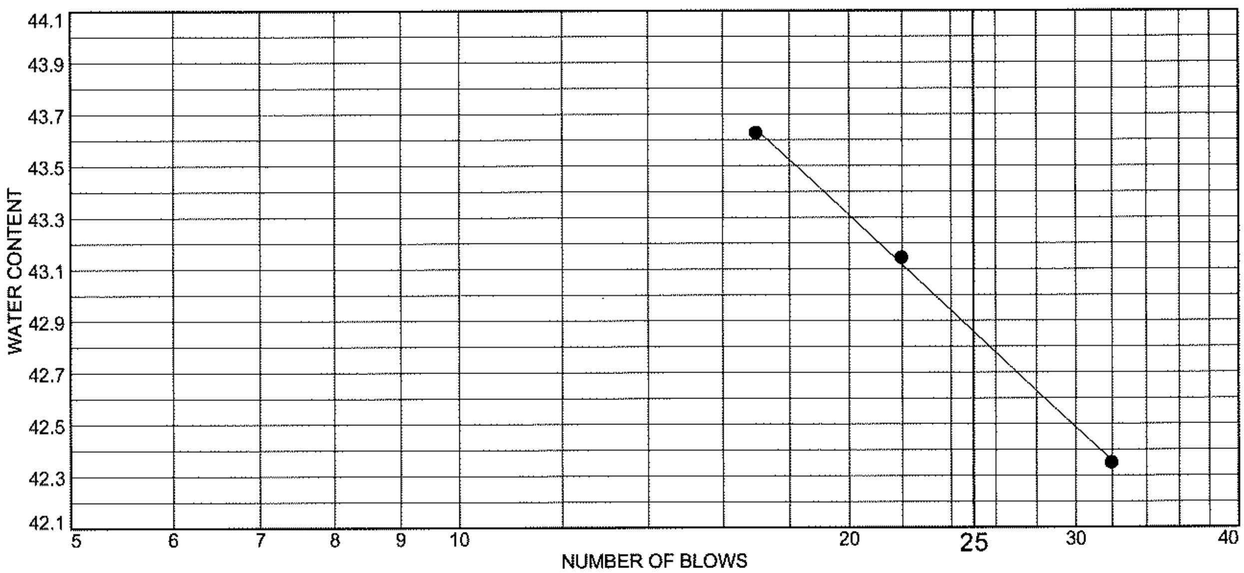
Location: TP-1
Sample Number: S18-081 **Depth:** 2'
Date Sampled: 3/19/2018

GEOPACIFIC ENGINEERING, INC.

Client: Riverlands, LLC
Project: The Riverlands Subdivision
Project No: 18-4860 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Silt with Sand	42.9	27.1	15.8	99.2	81.4	ML

Project No. 18-4860 **Client:** Riverlands, LLC

Project: The Riverlands Subdivision

Location: TP-1

Sample Number: S18-082 **Depth:** 9'

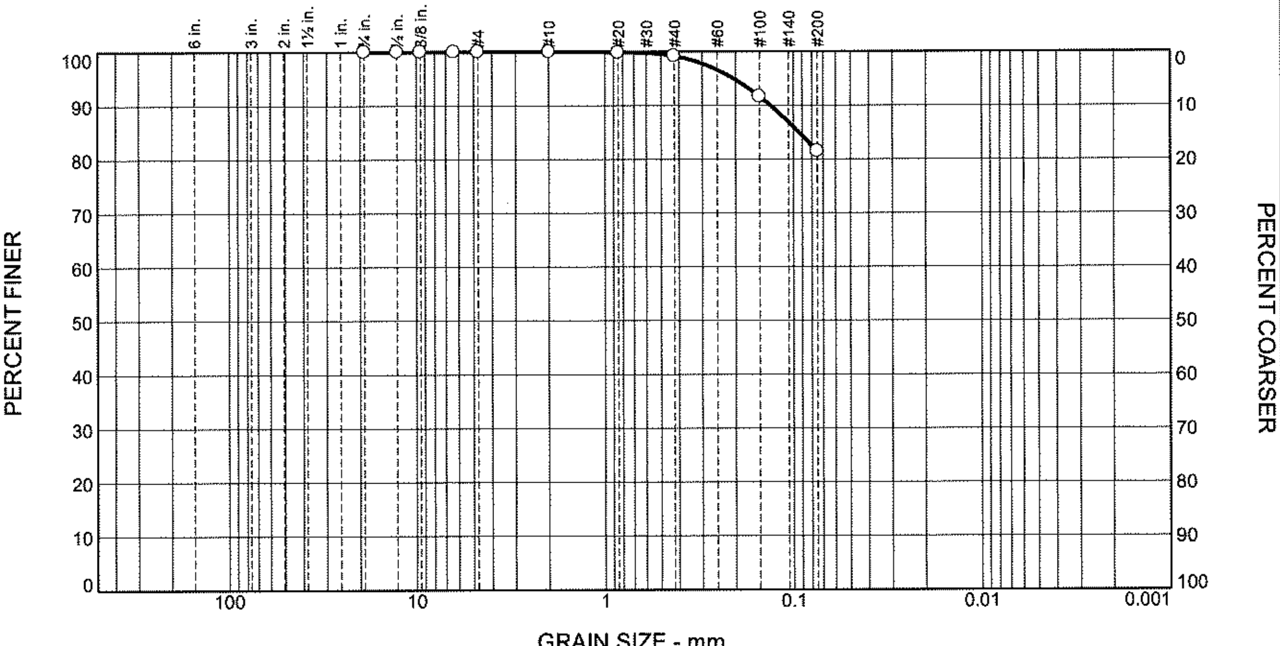
Remarks:

GEOPACIFIC ENGINEERING, INC.

Figure

Tested By: SJC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.8	17.8	81.4	

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
.75	100.0		
.5	100.0		
.375	100.0		
.25	100.0		
#4	100.0		
#10	100.0		
#20	99.9		
#40	99.2		
#100	91.6		
#200	81.4		

* (no specification provided)

Material Description

Silt with Sand

Atterberg Limits (ASTM D 4318)

PL= 27.1 LL= 42.9 PI= 15.8

Classification

USCS (D 2487)= ML AASHTO (M 145)= A-7-6(14)

Coefficients

D_{90}= 0.1321 D_{85}= 0.0940 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Moisture 38.8%

Date Received: **Date Tested:** 3/23/2018
Tested By: SJC
Checked By:
Title:

Location: TP-1
Sample Number: S18-082 **Depth:** 9'
Date Sampled: 3/19/2018

GEOPACIFIC ENGINEERING, INC.

Client: Riverlands, LLC
Project: The Riverlands Subdivision
Project No: 18-4860 **Figure**

Project Name:	The Riverlands
Project #: 18-4860	Sample ID: S18-081 Depth: 2'
Material Type:	Fat Clay
Material Source:	TP-1

EXPANSION INDEX ASTM D4829

Initial Height (0.001 in.)	1.000
Initial Moisture Content (0.1%)	16.2
Initial Dry Unit Weight (0.1 lbf/cu.ft.)	91.0
Initial Degree of Saturation (50.0+/-2%)	51.4
Initial Dial Reading (0.001 in.)	0.2489
Final Dial Reading (0.001 in.)	0.1665
Final Moisture Content (0.1%)	0.4
Expansion Index	**82**

Sampled By:	MTB	Tested By:	SJC
Sample Date:	3/15/2018	Tested Date:	3/28/2018

Expansion Index, EI	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
>130	Very High

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FLEXIBLE PAVEMENT DESIGN

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A Proprietary AASHTOWARE(tm)
Computer Software Product

Flexible Structural Design Module

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Portland, OR 97224
Michael T Baker

Project Description

18-4860 The Riverlands Subdivision Local Public Street within Western Cul-de-sac New Pavement Section

Flexible Structural Design Module Data

18-kip ESALs Over Initial Performance Period: 53,248
Initial Serviceability: 4.2
Terminal Serviceability: 2.5
Reliability Level (%): 85
Overall Standard Deviation: .5
Roadbed Soil Resilient Modulus (PSI): 7,500
Stage Construction: 1

Calculated Structural Number: 2.09

Specified Layer Design

Layer: 1
Material Description: 1/2-0 Lv 2 HMAC
Structural Coefficient (Ai): .42
Drainage Coefficient (Mi): 1
Layer Thickness (Di) (in): 3.00
Calculated Layer SN: 1.26

Layer: 2
Material Description: 3/4-0 Crushed Rock
Structural Coefficient (Ai): .1
Drainage Coefficient (Mi): 1
Layer Thickness (Di) (in): 2.00
Calculated Layer SN: .20

Layer: 3
Material Description: 1.5-0 Crushed Rock
Structural Coefficient (Ai): .1
Drainage Coefficient (Mi): 1
Layer Thickness (Di) (in): 8.00
Calculated Layer SN: .80

Total Thickness (in): 13.00
Total Calculated SN: 2.26

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Flexible Structural Design Module

GeoPacific Engineering, Inc.
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18-4860 The Riverlands Subdivision Local Public Street within Western Cul-de-sac   New Pavement Section

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Reliability Level (%):	85
Overall Standard Deviation:	.5
Roadbed Soil Resilient Modulus (PSI):	7,500
Stage Construction:	1
Calculated Structural Number:	2.09

Simple ESAL Calculation

Initial Performance Period (years):	20
Initial Two-Way Daily Traffic (ADT):	200
% Heavy Trucks (of ADT) FHWA Class 5 or Greater:	3
Number of Lanes In Design Direction:	2
Percent of All Trucks In Design Lane (%):	100
Percent Trucks In Design Direction (%):	50
Average Initial Truck Factor (ESALs/truck):	2
Annual Truck Factor Growth Rate (%):	0
Annual Truck Volume Growth Rate (%):	2
Growth:	Compound
Total Calculated Cumulative Esals:	53,248