GeoPacific Engineering, Inc.

Real-World Geotechnical Solutions
Investigation • Design • Construction Support

Revised March 12, 2010
GeoPacific Project No. 09-1681

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Copy: Robert Mosier, William Wilson Architects

Subject: **GEOTECHNICAL ENGINEERING REPORT**
1984 NW PETTYGROVE STREET
PORTLAND, OREGON

Reference: *Preliminary Geotechnical Engineering Report, 1984 NW Pettygrove Street, Portland, Oregon*, GeoPacific Engineering, Inc. report dated February 20, 2009.

GeoPacific Engineering, Inc. (GeoPacific) performed a geotechnical engineering study at 1984 NW Pettygrove Street in the City of Portland, Oregon (see Vicinity Map, Figure 1). The general site layout and locations of subsurface explorations are shown on the Site and Exploration Plan, Figure 2. The purpose of the geotechnical study was to explore and evaluate the surface and subsurface conditions at the site, and based on the conditions observed, provide geotechnical recommendations for foundation design and construction of the currently proposed development.

This report includes results of additional borings and geotechnical evaluation of foundation requirements. It completely replaces our previous preliminary geotechnical report, referenced above.

PROJECT DESCRIPTION

The site encompasses approximately 0.34 acres, in a rectangular parcel bounded by NW 20th Avenue to the west and NW Pettygrove Street to the north (Figure 2). A parking area borders the site to the east and residential and commercial buildings to the south. The site is currently an open lot with no existing structures. A structure that formerly existed in the northwest portion of the site has been removed. The basement area of the former structure was backfilled using crushed rock, compacted as engineered fill.

The current design concept for the project includes a six-story, wood-framed residential structure, with the lowermost floor near existing grade. The lower floor is planned as a parking garage. The structure will be supported by conventional spread footings with a concrete slab-on-grade lower floor.

SCOPE OF WORK AND AUTHORIZATION

The initial scope of work for this geotechnical study was presented in our December 11, 2008 proposal, and consisted of site reconnaissance, exploratory drilling, geotechnical analyses, and preparation of this report. This scope of services and our *General Conditions for Geotechnical Services* were authorized by the client on

January 13, 2009. Supplemental borings were requested in February of 2010, and authorized by the client in an e-mail dated March 4, 2010.

FIELD EXPLORATION AND LABORATORY TESTING

Subsurface conditions were explored on February 12, 2008 by drilling two exploratory borings (designated B-1and B-2) to 20.5 feet using a 4 inch diameter solid stem auger. Three additional borings were conducted on March 5, 2010 to a maximum depth of 26.5 feet. Dan Fischer Excavating performed the borings under subcontract to GeoPacific. The boring locations are shown on the Site Plan, Figure 2. The explorations were located in the field by pacing or taping distances from property corners and other site features. As such, the locations of the explorations should be considered approximate.

At each boring location, SPT (Standard Penetration Test) sampling was performed in general accordance with ASTM D1586 using a 2-inch outside diameter split-spoon sampler and a 140-pound hammer equipped with a mechanically driven air clutch control cathead mechanism. During the test, a sample is obtained by driving the sampler 18 inches into the soil with the hammer free-falling 30 inches. The number of blows for each 6 inches of penetration is recorded. The Standard Penetration Resistance ("N-value") of the soil is calculated as the number of blows required for the final 12 inches of penetration. If 50 or more blows are recorded within a single 6-inch interval, the test is terminated, and the blow count is recorded as 50 blows for the number of inches driven. This resistance, or N-value, provides a measure of the relative density of granular soils and the relative consistency of cohesive soils.

Explorations were conducted under the full-time observation of GeoPacific personnel. Soil samples obtained from the boring were classified in the field and representative portions were placed in relatively air-tight plastic bags. These soil samples were then returned to the laboratory for further examination and fines content testing. Pertinent information including soil sample depths, stratigraphy, soil engineering characteristics, and groundwater occurrence was recorded. Soils were classified in general accordance with the Unified Soil Classification System.

REGIONAL GEOLOGY AND SEISMIC SETTING

The subject site lies within the Portland Basin, a broad structural depression situated between the Coast Range on the west and the Cascade Range on the east. The Portland Basin is a northwest-southwest trending structural basin produced by broad regional downwarping of the area. The Portland Basin is approximately 20 miles wide and 45 miles long and is filled with consolidated and unconsolidated sedimentary rocks of late Miocene, Pliocene and Pleistocene age.

The subject 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, the last of which occurred about 10,000 years ago (Madin, 1990). Underlying the project site, these deposits consist of horizontally layered, micaceous, silt to coarse sand forming distinct beds less than 3 feet thick. Based on our exploratory borings, the Willamette Formation in the vicinity of the subject site is estimated to be approximately 25 feet thick.

Soils of the Willamette Formation are underlain by the Tertiary-aged (2-65 million years ago) Troutdale formation - a partially cemented conglomerate and sandstone deposited by an ancestral Columbia River (Trimble, 1963). Regionally, the Troutdale Formation is informally divided into an upper and a lower member (Phillips, 1987). Lithologies in the upper member include lenticular layers of volcaniclastic (vitric) sand, quartzite-bearing gravel, fine-grained sand, silt and clay, micaceous quartz-rich sand, and conglomerate with a cumulative average thickness of 100 to 150 feet. The lower member consists primarily of laminated silty clay and sand with reported thicknesses in water well logs of up to 880 feet.

The site has a history of residential and commercial buildings that have come and gone since the late 1800s. Although significant thicknesses of undocumented fill soils were not encountered in the explorations, undocumented fill soils could be present in areas of the site not explored.

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

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 20 miles southwest of the subject site. 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). No seismicity has been recorded on the Gales Creek or Newberg Faults (the faults 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).

PORTLAND HILLS FAULT ZONE

The Portland Hills Fault Zone is a series of NW-trending faults that 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 fault zone extends along the eastern margin of the Portland Hills for a distance of 25 miles, and lies about 0.5 mile southwest of the subject site. 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 generally assumed to be potentially active (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 below the surface.

SUBSURFACE CONDITIONS

The following discussion is a summary of subsurface conditions encountered in our explorations. For more detailed information regarding subsurface conditions at specific exploration locations, refer to the attached boring logs. Also, please note that subsurface conditions can vary between exploration locations, as discussed in the *Uncertainty and Limitations* section below.

SOIL

Results of the exploration program indicate that the site is underlain by fill and interbedded silt and sand belonging to the Willamette Formation. The Troutdale Formation is anticipated to underlie the site at depths of about 60 to 90 feet. The observed conditions and soil properties are summarized below.

Undocumented Fill: Most of the site appears covered by fill associated with the former improvements, including a gravel driveway and parking areas. Based on results of the exploration program, fill thicknesses of 1 to 3 feet are anticipated. Although significant fill thicknesses were not encountered on site in the borings performed, such deposits may exist on site in areas beyond the explorations.

Engineered Fill: Boring B-3 was performed in the northwest corner of the site, in an area previously occupied by an approximately 40 by 30 foot structure. The engineered fill extended to a depth of approximately 7 feet and was the result of a recent basement demolition from the previously existing building. GeoPacific observed the excavation and performed periodic density tests on the backfill.

Willamette Formation: Underlying the gravel driveway fill and documented fill, the explorations encountered 7 to 9 feet of medium stiff to very stiff clayey silt underlain by interbedded layers of loose to medium dense silt and sand belonging to the Willamette Formation. In borings, soils belonging to the Willamette Formation were light brown with strong mottling and extended to a depth of approximately 25 feet below the ground surface (boring B-4).

Troutdale Formation: Underlying the Willamette Formation in boring B-4 was very dense gravel belonging to the Troutdale Formation, encountered at about 25 feet below ground surface. The well rounded gravel contained a matrix composed of silty sand and extended beyond the maximum depth of exploration in boring B-4.

GROUNDWATER

At the time of our explorations conducted on February 12, 2009, no groundwater or seepage was observed in borings B-1 and B-2. On March 5, 2010, groundwater was encountered in boring B-3 at a depth of 14 feet. No groundwater was observed in borings B-4 and B-5 on March 5, 2010. In our experience, it is common to encounter thin perched groundwater zones within the Willamette Formation in this area. Typically, such zones are less than 1 foot thick and found directly above or within a silt deposit.

The groundwater conditions reported above are for the specific dates and locations indicated, and therefore may not necessarily be indicative of other times and/or locations. Furthermore, it is anticipated that groundwater conditions will vary depending on the season, local subsurface conditions, changes in land use and other factors.

CONCLUSIONS AND RECOMMENDATIONS

Results of this study indicate that the proposed development is geotechnically feasible, provided that the recommendations of this report are incorporated into the design and construction phases of the project. During final design, GeoPacific should review proposed foundation plans and structural loading to verify the applicability of the geotechnical recommendations and revision/finalization of this report if necessary.

The proposed structure may be supported on a spread foundation system bearing on competent undisturbed native soils, designed and constructed as recommended in this report. Additional recommendations are presented below for site preparation, engineered fill, wet weather construction, seismic design, spread footing foundations, permanent below-grade walls, concrete slab-on-grade floors, excavating conditions and utility trenches, and erosion control.

SITE PREPARATION

All proposed structure and driveway areas to receive fill should first be cleared of vegetation, pavement, loose debris, and undocumented fill, and all debris from clearing should be removed from the site. Undocumented fill was not encountered during the subsurface exploration, but due to the past site use undocumented fill is likely present on site, as discussed above. The final depth of unsuitable soil removal should be determined on the basis of a site inspection during construction, and unsuitable soil removal operations should be observed and documented by GeoPacific.

Excavations within the building footprint should be made in a manner that will limit disturbance of the soils. Exposed subgrade soils should be evaluated by the geotechnical engineer. If soils are disturbed or softened by saturation, it may be necessary to process the upper 12 inches, moisture condition the soils, and compact them in-place prior to placement of the crushed aggregate working surface. Soft/loose soils identified during subgrade preparation should be compacted to a firm and unyielding condition or over-excavated and replaced with engineered fill, as described below. The depth of overexcavation, if required, should be evaluated by GeoPacific at the time of construction.

ENGINEERED FILL

On-site soils are considered suitable for use as engineered fill in dry weather conditions, provided they are relatively free of organics and are properly moisture conditioned for compaction. 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 90 percent of the maximum dry density determined by ASTM D1557 (Modified Proctor) or equivalent. On-site soils may be wet of optimum; therefore, we anticipate that aeration of native soil will be necessary for compaction operations performed during late spring to early summer.

Proper test frequency and earthwork documentation usually requires daily observation and testing during stripping, rough grading, and placement of engineered fill. Field density testing should conform to ASTM D2922 and D3017, or D1556. Engineered fill should be periodically observed and tested by the project geotechnical engineer or his representative. 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.

WET WEATHER EARTHWORK

The on-site soils are 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 fill 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, GeoPacific should be contacted for additional recommendations.

Under wet weather, the construction area will unavoidably become wet and the condition of fill or native soils exposed will degrade. To limit the impacts of wet weather on the finished building pad surface, consideration may be given to placement of a crushed aggregate pad. Where used, we recommend the working pad be constructed using 1½"–0 crushed aggregate, and should have minimum thickness of at least 12 inches. This thickness is considered adequate to support light construction traffic, but will not be sufficient to support heavy traffic such as loaded dump trucks or other heavy rubber-tired equipment.

As an alternative to the crushed rock working surface, consideration may be given to cement treating the upper 12 inches of subgrade soils in place. The objective of in-place cement treatment is to mix the subgrade soils with cement to reduce the water content and increase the structural strength of the materials to a level that enables adequate compaction, and provides a working surface for wet weather construction. This is an approach that has become relatively common in the Pacific Northwest. In our experience, 3 to 6 percent cement (by weight) is typically adequate for this application. We suggest that the contractor start cement treatment initially using 5 percent cement. A minimum 12-inch treatment depth is recommended. The 5 percent cement content is considered an appropriate starting point, although adjustment may be necessary based on the effectiveness of treatment as observed in the field.

The cement treatment process involves spreading the cement over the treated area in a uniform lift, and then tilling with a high-speed rotovator or similar equipment. Cement must be spread evenly across the treated surface, and mixing must be thorough to achieve the desired result.

SEISMIC DESIGN

Structures should be designed to resist earthquake loading in accordance with the methodology described in the 2006 International Building Code (IBC), with applicable 2007 Oregon Structural Specialty Code (OSSC) revisions. Design values determined for the site using the USGS (United States Geological Survey) Earthquake Ground Motion Parameters utility are summarized in Table 1.

Table 1. Recommended Earthquake Ground Motion Parameters (2006 IBC / 2007 OSSC)

Parameter	Value
Location (Lat, Long), degrees	45.533, -122.692
Site Class	D
Mapped Spectral Acceleration Values (MCE, Site Class B):	
Short Period, S_s	0.983 g
1.0 Sec Period, S_1	0.346 g
Soil Factors for Site Class D:	
F_a	1.107
F_v	1.707
$SD_s = 2/3 \times F_a \times S_s$	0.726 g
$SD_1 = 2/3 \times F_v \times S_1$	0.394 g

Potential seismic impacts also include secondary effects such as soil liquefaction, fault rupture potential, and other hazards as discussed below:

- **Soil Liquefaction Potential** – 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. On-site soils consist of generally medium stiff to stiff silt and medium dense silty sand not considered susceptible to liquefaction. It is our opinion that special design or construction measures are not required to mitigate the effects of liquefaction.

- **Fault Rupture Potential** – Based on our review of available geologic literature, we are not aware of any mapped active (demonstrating movement in the last 10,000 years) faults on the site. During our field investigation, we did not observe any evidence of surface rupture or recent faulting. Therefore, we conclude that the potential for fault rupture on site is low.

- **Seismic Induced Landslide** – Topography in the vicinity of the subject site is generally flat to gently sloping. The potential for slope instability and seismic induced landslide on site is considered low to very low.

- **Effects of Local Geology and Topography** – In our opinion, no additional seismic hazard will occur due to local geology or topography. The site is expected to have no greater seismic hazard than surrounding properties and the Portland area in general.

We understand that, given the size and height of the proposed building, the City of Portland will not require a site-specific seismic response spectrum for the project.

SPREAD FOOTING FOUNDATIONS

Based on our understanding of the proposed project and the results of our exploration program, and assuming our recommendations for site preparation are followed, a spread foundation system is considered suitable for support of the planned building.

Due to the compressibility of the native silt materials beneath the site, spread foundation bearing pressures will be limited by allowable settlement. Estimated settlements can be limited by overexcavation of native soils immediately beneath the footings and replacement with compacted crushed rock. We recommend an overexcavation depth of 24 inches below interior column footings and exterior wall foundations. The crushed rock material should have a maximum particle size of 1½ inch, with no more than 80 percent passing the No. 4 sieve and less than 5 percent fines (material passing the U.S. Standard No. 200 sieve). Crushed rock should be compacted to at least 92 percent of its maximum dry density as determined by ASTM D1557 or equivalent.

For footing subgrade soils prepared as recommended above, we recommend maximum allowable bearing pressures of 3,000 pounds per square foot (psf) for designing the footings of the below-grade portions of the structure. The recommended maximum allowable bearing pressures may be increased by 1/3 the above-recommended value, for short term transient conditions such as wind and seismic loading. All exterior and interior footings should be founded at least 24 inches below the lowest adjacent finished grade or below top of slab.

Wind, earthquakes, and unbalanced earth loads will subject the proposed structure to lateral forces. Lateral forces on a structure will be resisted by a combination of sliding resistance of its base or footing on the underlying soil and passive earth pressure against the buried portions of the structure. For use in design, a coefficient of friction of 0.50 may be assumed along the interface between the base of the footing and subgrade soils. Passive earth pressure for buried portions of structures may be calculated using an equivalent fluid weight of 390 pounds per cubic foot (pcf), assuming footings are cast against firm, natural soils or engineered fill. The recommended coefficient of friction and passive earth pressure values do not include a safety factor, and an appropriate safety factor should be included in design.

Assuming construction is accomplished as recommended herein, and for the foundation loads anticipated, we estimate total settlement of spread foundations of less than about 1 inch and differential settlement between two adjacent load-bearing components supported on competent soil of less than about ½ inch. We anticipate that settlements will occur relatively uniformly, and that the majority of the estimated settlement will occur during construction, as loads are applied.

If final loading conditions or structural configurations vary from those assumed herein, the estimated settlement would also change. GeoPacific should be contacted at an appropriate stage in the design process, to review the building loads and revise settlement calculations as necessary.

Footing excavations should be trimmed neat and the bottom of the excavation should be carefully prepared. All loose or softened soil should be removed from the footing excavation prior to placing reinforcing steel bars. We recommend that footing excavations be observed by GeoPacific prior to placing steel and concrete, to verify that the recommendations of this report have been followed, and that an appropriate bearing stratum has been exposed.

The above foundation recommendations are for dry weather conditions. Due to the high moisture sensitivity of engineered fill and native soils anticipated on site, additional thicknesses of crushed rock may be needed beneath footing foundations during wet weather. Appropriate recommendations can be made for wet weather foundation construction, if needed during construction.

PERMANENT BELOW-GRADE WALLS

Lateral earth pressures against below-grade retaining walls will depend upon the inclination of any adjacent slopes, type of backfill, degree of wall restraint, method of backfill placement, degree of backfill compaction, drainage provisions, and magnitude and location of any adjacent surcharge loads. At-rest soil pressure is exerted on a retaining wall when it is restrained against rotation. In contrast, active soil pressure will be exerted on a wall if its top is allowed to rotate or yield a distance of roughly 0.001 times its height or greater.

If the subject retaining walls will be free to rotate at the top, they should be designed for an active earth pressure equivalent to that generated by a fluid weighing 35 pcf for level backfill against the wall. For restrained wall, an at-reset equivalent fluid pressure of 55 pcf should be used in design, again assuming level backfill against the wall. These values assume that the recommended drainage provisions are incorporated, and hydrostatic pressures are not allowed to develop against the wall.

During a seismic event, lateral earth pressures acting on below-grade structural walls will increase by an incremental amount that corresponds to the earthquake loading. Based on the Mononobe-Okabe equation and peak horizontal accelerations appropriate for the site location, seismic loading should be modeled using the active or at-rest earth pressures recommended above, plus an incremental rectangular-shaped seismic load of magnitude 5.5H, where H is the total height of the wall.

We assume relatively level ground surface below the base of the walls. As such, we recommend passive earth pressure of 390 pcf for use in design, assuming wall footings are cast against competent native soils or engineered fill. If the ground surface slopes down and away from the base of any of the walls, a lower passive earth pressure should be used and GeoPacific should be contacted for additional recommendations.

A coefficient of friction of 0.5 may be assumed along the interface between the base of the wall footing and subgrade soils. The recommended coefficient of friction and passive earth pressure values do not include a safety factor, and an appropriate safety factor should be included in design. The upper 12 inches of soil should be neglected in passive pressure computations unless it is protected by pavement or slabs on grade.

The above recommendations for lateral earth pressures assume that the backfill behind the subsurface walls will consist of properly compacted structural fill, and no adjacent surcharge loading. If the walls will be subjected to the influence of surcharge loading within a horizontal distance equal to or less than the height of the wall, the walls should be designed for the additional horizontal pressure. For uniform surcharge pressures, a uniformly distributed lateral pressure of 0.3 times the surcharge pressure should be added. Traffic surcharges may be estimated using an additional vertical load of 250 psf (2 feet of additional fill), in accordance with local practice.

The recommended equivalent fluid densities assume a free-draining condition behind the walls so that hydrostatic pressures do not build-up. This can be accomplished by placing a 12- to 18-inch wide zone of sand and gravel containing less than 5 percent fines against the walls. A 3-inch minimum diameter perforated, plastic drain pipe should be installed at the base of the walls and connected to a suitable discharge point to remove water in this zone of sand and gravel. The drain pipe should be wrapped in filter fabric (Mirafi 140N or other as approved by the geotechnical engineer) to minimize clogging.

GeoPacific should be contacted during construction to verify subgrade strength in wall keyway excavations, to verify that backslope soils are in accordance with our assumptions, and to take density tests on the wall backfill materials.

Structures should be located a horizontal distance of at least 1.5H away from the back of the retaining wall, where H is the total height of the wall. GeoPacific should be contacted for additional foundation recommendations where structures are located closer than 1.5H to the top of any wall.

CONCRETE SLAB-ON-GRADE FLOORS

Preparation of areas beneath concrete slab-on-grade floors should be performed as recommended in the *Site Preparation* section. Care should be taken during excavation for foundations and floor slabs, to avoid disturbing subgrade soils. If subgrade soils have been adversely impacted by wet weather or otherwise disturbed, the surficial soils should be scarified to a minimum depth of 8 inches, moisture conditioned to within about 3 percent of optimum moisture content, and compacted to engineered fill specifications. Alternatively, disturbed soils may be removed and the removal zone backfilled with additional crushed rock.

For evaluation of the concrete slab-on-grade floors using the beam on elastic foundation method, a modulus of subgrade reaction of 200 kcf (115 pci) should be assumed for the stiff native silt soils anticipated at foundation depth. This value assumes the concrete slab system is designed and constructed as recommended herein, with a minimum thickness of crushed rock of 9 inches beneath the slab.

Interior slab-on-grade floors should be provided with an adequate moisture break. The capillary break material should consist of free-draining, crushed rock, with a maximum particle size of ¾ inch, no more than 80 percent passing the No. 4 sieve, and less than 5 percent fines (material passing the U.S. Standard No. 200 sieve). For dry-weather construction, the minimum recommended thickness of capillary break materials on re-compacted soil subgrade is 9 inches. The total thickness of crushed aggregate will be dependent on the subgrade conditions at the time of construction, and should be verified visually by proof-rolling. Under-slab aggregate should be compacted to at least 92% of its maximum dry density as determined by ASTM D1557 or equivalent.

In areas where moisture will be detrimental to floor coverings or equipment inside the proposed structure, appropriate vapor barrier and damp-proofing measures should be implemented. A commonly applied vapor barrier system consists of a 10-mil polyethylene vapor barrier placed directly over the capillary break material. With this type of system, an approximately 2-inch thick layer of sand is often placed over the vapor barrier to protect it from damage, to aid in curing of the concrete, and also to help prevent cement from bleeding down into the underlying capillary break materials. Other damp/vapor barrier systems may also be feasible. Appropriate design professionals should be consulted regarding vapor barrier and damp proofing systems, ventilation, building material selection and mold prevention issues, which are outside GeoPacific's area of expertise.

EXCAVATING CONDITIONS AND UTILITY TRENCHES

We anticipate that on-site soils can be excavated using conventional heavy equipment such as trackhoes. 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. All temporary cuts in excess of 4 feet in height should be sloped in accordance with U.S. Occupational Safety and Heath Administration (OSHA) regulations (29 CFR Part 1926), or be shored. The existing native soils classify as Type B and C Soils 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. Flatter temporary excavation slopes will be needed if groundwater is present, or if significant thicknesses of sandy soils are present in excavation sidewalls.

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 structural trench backfill be compacted to at least 90% of the maximum dry density obtained by Modified Proctor (ASTM D1557) or equivalent. Initial backfill lift thick nesses 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. Erosion at the site during construction can be minimized by implementing the project erosion control plan, which should include judicious use of straw bales and silt fences. If used, these erosion control devices should be in place and remain in place throughout site preparation and construction. Areas of exposed soil requiring immediate and/or temporary protection against exposure should be covered with either mulch or erosion control netting/blankets.

March 12, 2010
GeoPacific Project No. 09-1681

UNCERTAINTIES AND LIMITATIONS

We have prepared this report for the owner and his/her consultants for use in design of this project only. This report should be provided in its entirety to prospective contractors for bidding and estimating purposes; however, 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.

Sufficient geotechnical monitoring, testing and consultation should be provided during construction to confirm that the conditions encountered are consistent with those indicated by explorations. The checklist attached to this report outlines recommended geotechnical observations and testing for the project. Recommendations for design changes will be provided should conditions revealed during construction differ from those anticipated, and to verify that the geotechnical aspects of construction comply with the contract plans and specifications.

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, expressed 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
OREGON
SCOTT L. HARDMAN
3-12-10
EXPIRES: 06-30-2011

Beth K. Rapp, G.I.T.
Project Geologist

Scott L. Hardman, P.E.
Principal Engineer

Attachments: References
Checklist of Recommended Geotechnical Testing and Observation
Figure 1 – Vicinity Map
Figure 2 – Site and Exploration Plan
Borings Logs – B-1 through B-5

REFERENCES

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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	Pavement and fill removal	During removal	Soil Technician	
3	Density testing of engineered fill (90% of Modified Proctor)	During filling, tested every 2 vertical or 500 yd^3	Soil Technician	
4	Under-slab base rock (92% of Modified Proctor)	Prior to placing vapor barrier or steel	Soil Technician	
5	Foundation Excavations / Overexcavations	Prior to placement of crushed rock / setting forms	Geotechnical Engineer	
6	Density testing of trench backfill (90% of Modified Proctor)	During backfilling, tested every 4 vertical feet for every 200 lineal feet	Soil Technician	
7	Final Geotechnical Engineer's Letter	Completion of project	Geotechnical Engineer	

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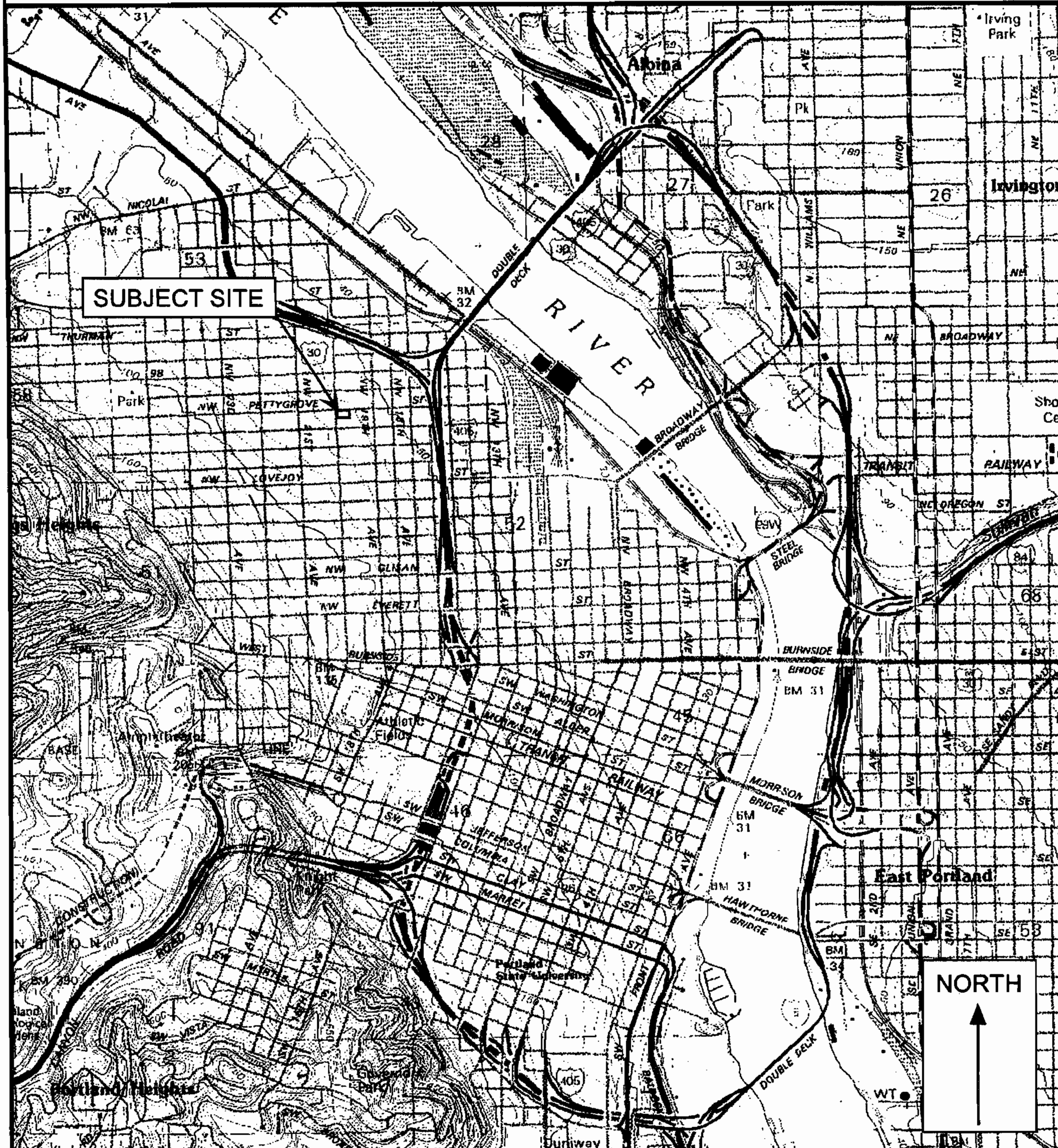
VICINITY MAP



Legend

Approximate Scale 1 in = 2,000 ft

Date: 3/11/10
Drawn by: TCW

Base map: U.S. Geological Survey 7.5 minute Topographic Map Series, Portland, Oregon-Washington Quadrangle, 1990

Project: 1984 NW Pettygrove Street Portland, Oregon	Project No. 09-1681	FIGURE 1

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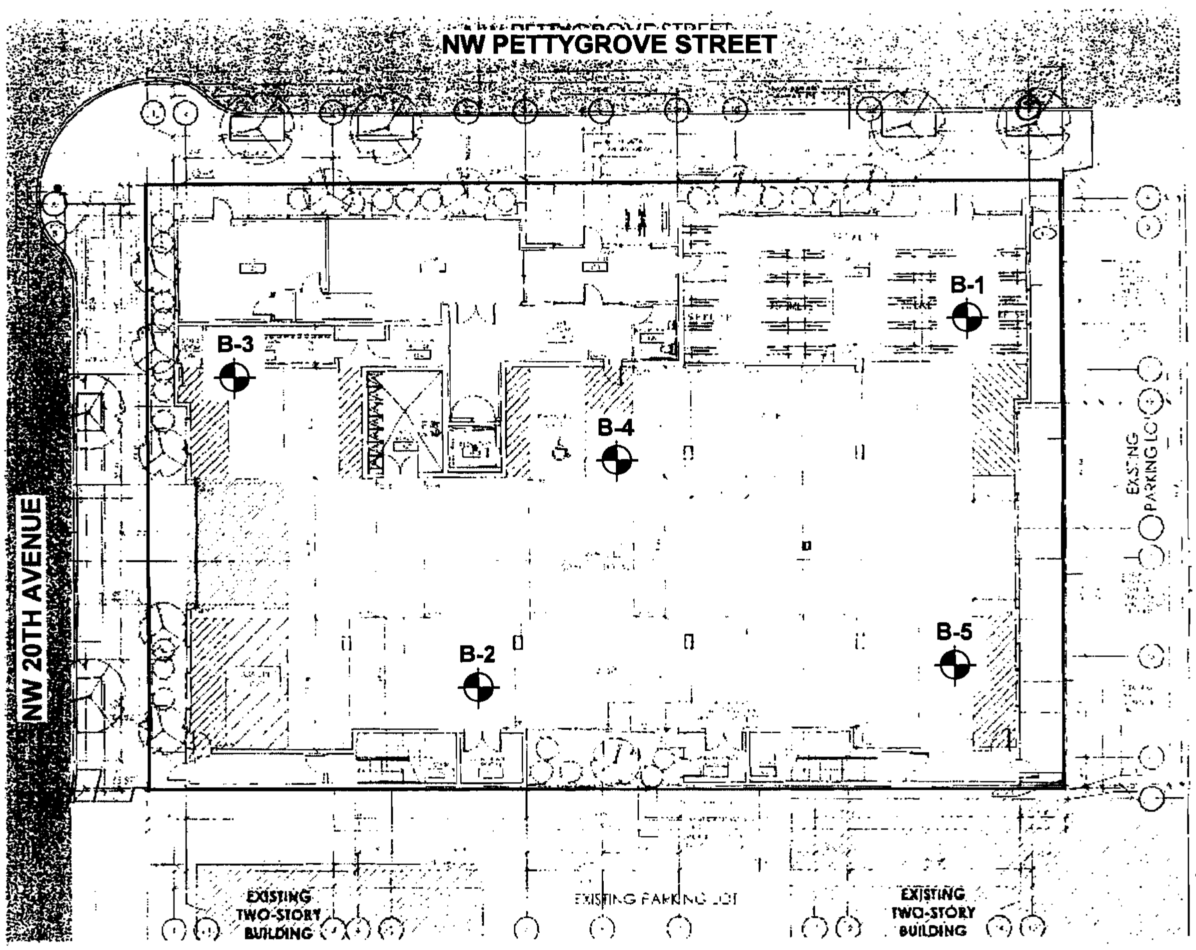
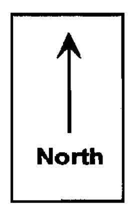
SITE AND EXPLORATION PLAN





Base map provided by William Wilson Architects PC dated 12/28/09

Legend

B-1
Boring Designation and Approximate Location

Date: 3/11/10
Drawn by: TCW

Project: 1984 Pettygrove Street, Portland, Oregon	Project No: 09-1681	FIGURE 2

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BORING LOG

Project: 1984 NW Pettygrove Street Portland, Oregon	Project No. 09-1682	Boring No. B-1

Depth (ft)	Sample Type	N-Value	Well Construction	Moisture Content (%)	Water Bearing Zone	Material Description
	Split-Spoon	10				Stiff to very stiff, clayey Silt (ML), trace sand, brown to light brown, subtle mottling, damp (Willamette Formation)
5	Split-Spoon	9				
	Split-Spoon	8				Becomes less mottled and micaceous
10	Split-Spoon	10				Stiff, micaceous, fine sandy SILT(ML), some clay, brown, damp (Willamette formation)
15	Split-Spoon	11				Medium dense, micaceous, silty fine SAND (SM), some clay, brown, damp (Willamette formation)
20	Split-Spoon	16				
						Boring terminated at 20.5 Feet.
25						No groundwater or seepage observed
30						
35						

LEGEND

100 to 1,000 g

Bag Sample | Split-Spoon | Shelby Tube Sample | Static Water Table at Drilling | 10-20-99 Static Water Table | Water Bearing Zone

Date Drilled: 2/12/09
Logged By: T. Wayland
Surface Elevation: 66 ft

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BORING LOG

Project: 1984 NW Pettygrove Street Portland, Oregon	Project No. 09-1682	Boring No. B-2

Depth (ft)	Sample Type	N-Value	Well Construction	Moisture Content (%)	Water Bearing Zone	Material Description
	Split-Spoon	27				Stiff to very stiff, clayey Silt (ML), trace sand, brown to light brown, subtle mottling, damp (Willamette Formation)
5	Split-Spoon	18				
	Split-Spoon	11				Becomes less mottled, micaceous
10	Split-Spoon	9				Medium dense, micaceous, silty fine to medium SAND (SM), brown, damp (Willamette formation)
15	Split-Spoon	10				
20	Split-Spoon	8				Medium stiff, micaceous, fine sandy SILT (ML), brown, damp (Willamette formation)
						Boring terminated at 20.5 Feet. No groundwater or seepage observed.
25						
30						
35						

LEGEND

Bag Sample (100 to 1,000 g) · Split-Spoon · Shelby Tube Sample · Static Water Table at Drilling · Static Water Table (10-20-99) · Water Bearing Zone

Date Drilled: 2/12/09
Logged By: T. Wayland
Surface Elevation: 66 ft

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BORING LOG

Project: 1984 NW Pettygrove Street Portland, Oregon	Project No. 09-1682	Boring No. B-3

Depth (ft)	Sample Type	N-Value	Well Construction	Moisture Content (%)	Water Bearing Zone	Material Description
	Split-Spoon	81				Dense to very dense, GRAVEL (GM) with sand and silt, gray to light brown, moist (Engineered Fill)
5	Split-Spoon	38				
	Split-Spoon	3				Very loose to medium dense, silty SAND (SM) with interbeds of sandy SILT (ML), light brown to gray, sand is fine to medium grained, subtle to strong orange and gray mottling, micaceous, trace black staining, moist to wet (Willamette Formation)
10	Split-Spoon	8				
					▼	
15	Split-Spoon	16				
20	Split-Spoon	22				
25						Boring Terminated at 21.5 Feet. Groundwater encountered at 14 Feet.
30						
35						

LEGEND

Bag Sample (100 to 1,000 g) · Split-Spoon · Shelby Tube Sample · Static Water Table at Drilling · Static Water Table (10-20-99) · Water Bearing Zone

Date Drilled: 3/05/10
Logged By: B. Rapp
Surface Elevation: 66 ft

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BORING LOG

Project: 1984 NW Pettygrove Street, Portland, Oregon | Project No. 09-1682 | Boring No. B-4

Depth (ft)	Sample Type	N-Value	Well Construction	Moisture Content (%)	Water Bearing Zone	Material Description
	Split-Spoon	12				Stiff, clayey SILT (ML), light brown, strong orange and gray mottling, black staining, micaceous, moist (Willamette Formation)
5	Split-Spoon	12				
	Split-Spoon	7				Loose to medium dense, silty SAND (SM) with interbeds of sandy SILT (ML), light brown to gray, sand is fine to medium grained, subtle to strong orange and gray mottling, micaceous, black staining, moist (Willamette Formation)
10	Split-Spoon	6				
15	Split-Spoon	7				
20	Split-Spoon	17				
25	Split-Spoon	50 for 5.5"				Very Dense, sandy GRAVEL (GP), trace silt, brown to gray, gravel is well rounded, sand is coarse grained, moist (Troutdale Formation)
	Split-Spoon	50 for 5.75"				Practical Refusal on rounded Cobbles at 26.5 Feet. No Groundwater or Seepage encountered.
30						
35						

LEGEND

Bag Sample (100 to 1,000 g) | Split-Spoon | Shelby Tube Sample | Static Water Table at Drilling | Static Water Table (10-20-99) | Water Bearing Zone

Date Drilled: 3/05/10
Logged By: B. Rapp
Surface Elevation: 66 ft

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BORING LOG

Project: 1984 NW Pettygrove Street Portland, Oregon	Project No. 09-1682	Boring No. B-5

Depth (ft)	Sample Type	N-Value	Well Construction	Moisture Content (%)	Water Bearing Zone	Material Description
	Split-Spoon	5				Medium stiff to stiff, SILT (ML), light brown, strong orange and gray mottling, black staining, moist (Willamette Formation)
5	Split-Spoon	8				
	Split-Spoon	8				Loose to medium dense, silty SAND (SM) with interbeds of sandy SILT (ML), light brown to gray, sand is fine to medium grained, subtle to strong orange and gray mottling, black staining, moist (Willamette Formation)
10	Split-Spoon	6				
15	Split-Spoon	9				
20	Split-Spoon	12				
25						Boring Terminated at 21.5 Feet. No Groundwater or Seepage encountered.
30						
35						

LEGEND

Bag Sample (100 to 1,000 g) · Split-Spoon · Shelby Tube Sample · Static Water Table at Drilling · Static Water Table (10-20-99) · Water Bearing Zone

Date Drilled: 3/05/10
Logged By: B. Rapp
Surface Elevation: 66 ft