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Report of
Geotechnical Engineering Services
Riverdale High School
9806 SW Terwilliger Boulevard
Portland, Oregon

CGT Project: G0101764

Prepared for

Mr. Jim Mabbott
Riverdale School District 51J
11733 SW Breyman Avenue
Portland, Oregon 97219-8409

May 10, 2001

9727 SW Terwilliger Blvd.

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INTRODUCTION

Carlson Geotechnical (CGT) is pleased to present the results of our geotechnical investigation for the proposed Riverdale High School project in Portland, Oregon. The site location is shown on the attached Figure 1. We have performed our work in general accordance with our proposal dated March 16, 2001. The purpose of our work was to explore subsurface conditions at the site in order to provide geotechnical recommendations for design and construction of the project. Because the project is part of a "special occupancy structure" (i.e., school), we will also provide a seismic hazard study in accordance with the requirements of Section 1802 of the 1997 Uniform Building Code and the 1998 Oregon Structural Specialty Code. Our scope of work included the following:

- Explore the near surface soil conditions at the site by excavating four test pits to a maximum depth of 10 feet using a track-mounted excavator.
- Explore the deeper subsurface conditions at the site by advancing two cone penetrometer probes to a maximum depth of 42 feet.
- Classify the materials encountered in the explorations by American Society for Testing and Materials (ASTM) Visual-Manual Method. A qualified member of CGT's staff will observe the explorations and maintain a detailed log of each test pit.
- Obtain soil samples at select depths in the explorations.
- Collect representative soil samples for laboratory testing and to verify our field classifications.
- Complete moisture content determinations on representative samples from our explorations.

- Complete one Atterberg limits test to determine plasticity characteristics of the site soils.
- Complete one sieve analysis to determine grain-size characteristics of the site soils.
- Provide recommendations for site preparation, grading and drainage, stripping depths, fill type for imported materials, compaction criteria, cut and fill slope criteria, trench excavation and backfill, use of on-site soils, and wet/dry weather earthwork.
- Provide recommendations for design and construction of shallow spread foundations, including allowable design bearing pressures, minimum footing depth and width, and estimates for settlement.
- Provide geotechnical engineering recommendations for the design and construction of concrete floor slabs, including an anticipated value for subgrade modulus.
- Provide recommendations for subsurface drainage of foundations, floor slabs, and pavements, if necessary.
- Provide recommendations for design of retaining walls, including lateral earth pressures, backfill, compaction, and drainage.
- Evaluate design pavement sections, including base course and asphalt concrete thicknesses for parking areas and access roads.
- Evaluate seismic hazards at the site, including liquefaction potential, and provide a discussion of seismic activity in the vicinity of the site and recommendations for the Uniform Building Code (UBC) site coefficient and seismic zone.
- Provide a written report summarizing the results of our geotechnical evaluation.

PROJECT INFORMATION AND SITE DESCRIPTION

Project Information

The site is currently developed with an existing school building. The southern half of the building will be demolished and a new addition will be constructed in its place. The new addition will consist of office, class, and media rooms, along with a new gymnasium. The addition will be a one-story, wood-framed building with either a concrete or masonry block gymnasium. Detailed structural information has not been provided; however, we assume that bearing wall loads and individual column loads will not exceed 3 kips per linear foot (klf) and 100 klf, respectively. We have also assumed that soil-supported ground floor loads will not exceed 150 pounds per square foot (psf). The maximum depth of cuts and fills are anticipated to be up to 5 feet. New driveways and parking areas for both passenger vehicles and buses, and associated utilities are anticipated to be included in the development.

Regional Geology

The site is located just northeast of Mt. Sylvania in the Willamette Valley geologic province in Portland, Oregon. The Willamette Valley was formed when the volcanic rocks of the Oregon Coast Range, originally formed as submarine islands, were added onto the North American Continent. The volcanic rocks slowly subsided forming a depression in which various types of marine sedimentary rocks accumulated. Approximately 15 million years ago, these marine sediments were covered by the Columbia River Basalts that flowed down the Columbia River Gorge as far south as Salem. Uplift and tilting of the Oregon Coast Range and the western Cascade Range formed the trough-like character of the Willamette Valley, and folded and faulted the Columbia River Basalts. Approximately 1.3 to 2.6 million years ago, a volcanic episode erupted the Boring Lavas in several localized vents, including Mt. Sylvania, Mt. Scott, and Mt. Tabor. Catastrophic floods washed into the Willamette Valley approximately 12,000 to 15,000 years ago and deposited silt and fine- to coarse-grained sands and gravels (Pleistocene Flood Deposits), including wind blow silt (loess).

<u>Earthquake Sources and Seismicity</u>

The site is located in a tectonically active area that may be affected by crustal earthquakes, intra-slab earthquakes, or large subduction zone earthquakes. Damaging crustal earthquakes in this region are derived from local sources such as the Portland Hills Fault Zone, and the Gales Creek-Newberg-Mt. Angel Structural Zone, and typically occur at depths ranging from 15 to 40 km (Geomatrix Consultants, 1995). Intra-slab earthquakes occur within the subducting Juan De Fuca oceanic plate at depths ranging from approximately 40 km to 70 km. Large subduction zone earthquakes in this region are derived from the Cascadia Subduction Zone (CSZ). Due to the lack of historical data on large subduction zone earthquakes, a typical depth for the occurrence of a subduction zone earthquake was inferred from models presented by Geomatrix Consultants in 1995, and is roughly 10 to 25 km. These three sources are capable of generating damaging earthquakes at this site.

<u>Portland Hills Fault Zone</u>

An inferred fault associated with the Portland Hills Fault Zone is mapped on the east side of the site. The Portland Hills Fault Zone is a series of northwest-trending faults that vertically displace the Columbia River Basalt Group by 1,130 feet, and appear to control thickness changes in late Pleistocene (approximately 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 has been mapped in the

Portland area as a series of inferred faults with no surface expression. Geomorphic lineaments suggestive of Pleistocene deformation have been identified within the fault zone, but none of the fault segments have been shown to cut Holocene (last 10,000 years) deposits (Balsillie and Benson, 1971; Cornforth and Geomatrix Consultants, 1992). The fact that the faults do not cut Holocene sediments is most likely a result of the faulting being related to a time of intense uplift of the Oregon Coast Range during Miocene time, and little to no movement along the faults during the Holocene. 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 northwest-trending shear plane located 1.3 miles east of the fault (Yelin, 1992). Beeson and others (1976, 1985,1989) interpreted this zone of faulting as a zone characterized by right-lateral strike-slip faulting with a subordinate normal slip component.

Gales Creek-Newberg-Mt. Angel Structural Zone

The Gales Creek-Newberg-Mt. Angel Structural zone is a 50-mile-long zone of discontinuous, northwest-trending faults located approximately 35 miles to the south of the site. In 1993, Unruh and others modeled the Gales Creek, Newberg, and Mt. Angel faults as separate faults rather than a long, continuous fault zone based on changes in sense of displacement, evidence for discontinuities in the subsurface, different deformation histories, and differences in geomorphic expression and seismicity. However, since the faults share a common orientation, and several other studies (Yeats and others, 1991; Nablek and others, 1991) have indicated that these faults may be part of a larger interconnected zone of deformation, we have considered these faults as an interconnected structural zone as well. The Gales Creek-Newberg-Mt. Angel Structural Zone is recognized in the subsurface by vertical separation of the Columbia River Basalt and offset seismic reflectors in overlying basin sediments (Yeats et al., 1991; Werner et al., 1992). A geologic study conducted for the Scoggins Dam site in the Tualatin Basin revealed no evidence of deformed geomorphic surfaces along the Gales Creek or Newberg Faults and no seismicity has been recorded on these faults (Unruh, 1994). In contrast, geomorphic surfaces that extend across the Mt. Angel Fault are warped such that they are consistent with uplift on the northeast side of the fault (Unruh et al., 1994). In 1990, a series of small earthquakes (<M3.5) occurred near the town of Woodburn, and in 1993, a M5.6 earthquake occurred near the town of Scotts Mills (Werner et al., 1992; Geomatrix Consultants, 1995). These seismic events are generally attributed to the Mt. Angel Fault.

Other Mapped and Unmapped Crustal Sources

Several other crustal sources including the Grant Butte and Damascus-Trickle Creek Fault Zones, The Lacamas Creek Fault, and the Oatfield Fault, are capable of producing damaging earthquakes in the region. However, due to their distance from the site, non-active classification, or their short fault segments, we did not elaborate on these sources for this study.

Several crustally derived seismic events have been recorded in areas where no faults are mapped. This fact is most likely a function of the heavy forestation of western Oregon preventing the direct observation of faults that may occur in those areas. Additionally, most faulting within the Portland area does not cut the Holocene sediments, and is thus difficult to define. Furthermore, the displacement of the Holocene sediments due to ongoing fault movement in recent geologic time is minor and difficult to observe. Additional geophysical studies may define these unmapped sources in the future.

Intra-Slab Source

Earthquakes derived from intra-slab sources occur within the subducting Juan De Fuca Plate at depths ranging from 20 to 40 miles (Geomatrix, 1995). Approximately 20 miles west of the current coast line is the Cascadia Subduction Zone where the subducting Juan De Fuca Plate moves eastward (relative to the North American Continent) beneath the North American plate dipping at an angle of 10 to 20 degrees. As the plate moves farther away from the CSZ, the curvature of the plate increases and causes normal faulting within the oceanic slab in response to the extensional forces of the down dipping plate. The region of maximum curvature of the slab is where large intra-slab earthquakes are expected to occur, and is located roughly 30 miles below the Oregon Coast Range. Historically, the seismicity rate within the Juan De Fuca Plate beneath Oregon is very low in northern Oregon and extremely low in southern and central Oregon (Geomatrix, 1993, 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/year (DeMets et al., 1990). Very little seismicity has occurred on the plate interface in historic time, and as a result, the seismic potential of the Cascadia Subduction Zone is a subject of scientific controversy. The lack of seismicity may be interpreted as a period of quiescent stress buildup between large magnitude earthquakes or as being characteristic of the long-term behavior of the subduction zone. A growing body of geologic evidence, however, strongly 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 is roughly coincident with the Oregon coastline and lies approximately 80 miles west of the site.

Earthquake Magnitude

Both deterministic and probabilistic methods are generally used to evaluate the seismic hazard at a specific site. The deterministic method considers the worst-case scenario based on the maximum credible earthquake (the largest earthquake that could be expected to occur) and is used for critical facilities like power plants, hospitals, and hazardous substance storage facilities. The probabilistic method considers the probability of earthquake occurrence during the lifetime of a particular facility, and is more appropriate for residential and commercial development. Both methods involve the choice of a design earthquake that is used to calculate the intensity of ground motion expected at the site.

Maximum Credible Earthquake

The primary means for estimating the maximum earthquake that a particular fault could generate are empirical relationships between earthquake magnitude and fault rupture length (Bonilla et al., 1984). Based on these relationships, the size of historical earthquakes, and the thickness of seismogenic crust in the Willamette Valley, the maximum earthquake magnitude expected from crustal sources is M6.0 to M6.6 (Geomatrix Consultants, 1995). Based on the likely thin nature of the Juan De Fuca plate and comparing the historic siesmicity along Cascadia with other margins, Geomatrix Consultants (1995) estimated the maximum magnitude earthquake for intra-slab sources is M7 to M7.5. Similarly, based on magnitude versus rupture area relationships for subduction zone earthquakes worldwide, the maximum magnitude of a Cascadia Subduction Zone earthquake is estimated to be M8.0 to M.9.0 (Geomatrix Consultants, 1995).

Maximum Probable Earthquake

Magnitude estimates for the maximum probable earthquake are based largely on the record of historical earthquakes in the region of interest. Table 1 lists earthquakes with magnitudes larger than M4.9 that have occurred in or near Oregon since 1873 (Wong and Bott, 1995).

Table 1 – Historical Earthquakes in Oregon with Magnitudes Greater than M4.9

Date	Magnitude	Maximum Modified Mercalli Intensity	Location
1873	M6.75*	VIII	Crescent City, CA
1877	M5.25*	VII	Portland
1892	M5.0*	VI	Portland
1936	M6.1	VII+	Milton-Freewater
1962	M5.5	VII	Vancouver-Portland
1968	M5.0	V	Adel
1993	M5.6	VII	Scotts Mills
1993	M6.0	VII-VIII	Klamath Falls

* Magnitude estimated from Modified Mercalli intensity.

Based on the historical record and crustal faulting models of the Willamette Valley region, the maximum probable earthquake for crustal sources in the vicinity of the subject site is estimated to be M5.75 (Geomatrix Consultants, 1995). Similarly, the maximum probable earthquake for an intra-slab source on the Cascadia Subduction Zone is estimated to be M7.5 to M7.7.

Seismic Shaking

A standard quantitative method of describing ground motion associated with propagating seismic waves is to specify peak ground accelerations (PGA) in bedrock. PGAs are average values based on empirical attenuation relationships of seismic wave energy with distance from the causative fault. PGAs are expressed as a fraction of the acceleration of gravity (i.e., a vertical PGA of >1.0 g would throw objects into the air). Table 2 shows the estimated PGA at the subject site for the maximum credible events on the listed faults, based on attenuation relationships developed by Sadigh (1984) and Geomatrix Consultants (1995), and on numerical models by Cohee et al. (1991) and Youngs et al. (1993).

Table 2 – Estimated Peak Ground Accelerations at "Rock Sites" Resulting from Maximum Credible Events on Known Faults

Earthquake Source	Moment Magnitude (M_w)	Epicentral Distance(miles)	Estimated Peak Ground Acceleration
Portland Hills Fault Zone	6.6	< 0.5	0.5 g
Gale Creek-Newberg-Mt. Angel Fault Zone	6.6	35	0.12 g
Intra-Slab	7.5	30	0.23 g
Cascadia Subduction Zone	8.5	60	0.07 g

A recent study commissioned by the Oregon Department of Transportation evaluated all known earthquake sources in Oregon, and formulated probabilistic assessments of expected seismic shaking based on maximum probable earthquake magnitudes (Geomatrix Consultants, 1995; Oregon Department of Geology and Mineral Industries, 1996). Table 3 presents the peak bedrock accelerations expected at the subject site (5% dampening), estimated recurrence intervals, and the corresponding probability of occurrence in the next 50 years.

Table 3 – Expected Ground Shaking at "Rock Sites" from Crustal, Plate-Interface, and Intra-slab Earthquake Sources

Modified Mercalli Intensity	Peak Ground Acceleration (% gravity)	Recurrence Interval	Chance of Occurrence in the Next 50 Years
VII+	0.20 g	500 years	10%
VIII	0.28 g	1,000 years	5%
VIII+	0.38 g	2,500 years	2%

Another method of describing the intensity of ground shaking associated with an earthquake is the Modified Mercalli intensity scale. This scale is a subjective measure of the affects experienced by people, man-made structures, and the earth surface. The two largest historical earthquakes in northwestern Oregon, the 1962 M5.5 earthquake near Portland and the 1993 M5.6 earthquake in Scotts Mills, generated maximum Modified Mercalli intensities of VII (Wong and Bott, 1995). The Modified Mercalli intensities predicted for the subject site due to occurrence of maximum probable events is shown in Table 3. An abridged portion of the Modified Mercalli intensity scale after Bolt (1993) is presented in Table 4.

Table 4 – Abridged Portion of the Modified Mercalli Intensity Scale

VII (0.10 to 0.15 g)	General alarm and everyone runs outdoors. Damage is negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable in poorly built or badly designed structures; some chimneys broken. Plaster and some stucco fall. Loosened brickwork and roof tiles shake down. Heavy furniture overturns. Stream and cut banks cave.
VIII (0.25 to 0.30 g)	General fright and alarm approaching panic. Damage is slight in specially designed structures; considerable in ordinary substantial buildings with partial collapse; great in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys, columns, and walls. Heavy furniture is overturned. Branches and tree trunks break off. Liquefied sand and mud erupts on ground surface.
IX (0.50 to 0.55 g)	General panic. Damage is considerable in specially designed structures; well designed frame structures thrown out of plumb; great in substantial buildings, with partial collapse. Buildings shifted off foundations. Conspicuous ground cracking. Underground pipes broken.

Site Geology

The available mapping (Open File Report 0-90-2) indicates that the site is underlain by less than 5 feet of loess deposits, which are in turn underlain by Columbia River Basalt. During our explorations, we observed loess deposits to the full depth of our test pit explorations (10 feet below ground surface [bgs]). We also observed unlithified, fine-grained sedimentary deposits to the full depth of our cone penetrometer testing (42 feet bgs). A northwest trending fault associated with the Portland Hills Fault zone is mapped on the east side of the site. Several other inferred faults are mapped within 5 miles of the site. All these faults offset the Columbia River Basalts, but they do not appear to cut the loess deposits.

Site Surface Conditions

The site is located at 9806 SW Terwilliger Boulevard in Portland Oregon. At the time of our explorations, an existing school with associated pavement and playground areas occupied the site. Moderate slopes are located on the north, south, and west borders of the site, where the slopes descend roughly 15 to 25 feet to residential areas. The site is bordered by residential development on the north, south, and west sides. SW Terwilliger Boulevard borders the site to the east.

Site Subsurface Conditions

Field Exploration

We excavated four test pits on April 24, 2001, to 10 feet bgs. The approximate test pit locations are shown on Figure 2. A member of CGT's staff logged the test pits, collected samples, and performed in-situ testing. Logs of the test pits are presented in the attached Figures 3 through 6. Our laboratory staff visually examined all samples returned to our laboratory in general accordance with the Unified Soil Classification System, in order to refine the field classifications. Subsurface Technologies advanced two cone penetrometer probes at the site on April 26, 2001, to a depth of 42 feet bgs. The cone logs are included as Figures 7 and 8.

Subsurface Materials

In general, we encountered approximately 1 foot of soft silt topsoil underlain by medium stiff to very stiff silt with weathered basalt fragments to the full depths explored. Test pit TP-3 encountered soft to medium stiff silt fill below the topsoil, likely associated with an abandoned drainpipe.

We did not encounter ground water seepage in the test pits; however, we anticipate that ground water levels will fluctuate due to seasonal variations in precipitation, changes in site utilization, or other factors. Additionally the site soils are conducive to the formation of a perched water table.

We advanced two cone penetrometer probes to a depth of approximately 42 feet bgs. The probe encountered medium stiff to very stiff silts and clays to the full depths of the exploration.

Liquefaction Analysis

A wide variety of slope and ground failures can occur in response to intense seismic shaking during large magnitude earthquakes. These failures are usually related to the phenomenon of liquefaction, the process by which water-saturated sediment changes from a solid to a liquid state. Since liquefied sediment may not support the overlying ground, or any structure built thereon, a variety of failures may occur including lateral spreading, landslides, ground settlement and cracking, sand boils, oscillation lurching, etc. The conditions necessary for liquefaction to occur are: (1) the presence of poorly consolidated, cohesionless sediment, (2) saturation of the sediment by ground water, and (3) an earthquake that produces intense seismic shaking (generally a Richter Magnitude greater than M5.0). In general, older, more consolidated sediment, clayey or gravely sediment, and sediment above the water table will not liquefy (Youd and Hoose, 1978). Field performance data and laboratory tests indicate that liquefaction occurs predominantly in well-sorted, loose to medium dense (SPT N-values of 0 to 20) sand or silty sand with a mean grain size between 0.8 mm and 0.08 mm (Lee and Fitton, 1968; Seed and Idriss, 1971). Existing maps indicate that the site has a very low potential for liquefaction settlement.

We used data obtained from the probe to analyze liquefaction potential at the site. Our analysis indicates liquefaction settlement at the site will likely be less than 1 inch during a design level earthquake.

CONCLUSIONS

General

Based on the results of our explorations and analyses, it is our opinion that the proposed structure with the assumed building loads can be supported on shallow spread footings bearing on the medium stiff to very stiff native silt or on structural fill that is properly installed during construction.

Seismic Hazards

Although there is no definitive evidence of recent activity, the Portland Hills Fault Zone is judged to be potentially active (Geomatrix Consultants, 1995). Additionally, based on possible deformation of Quaternary (last 1.6 million years) geomorphic surfaces and a spatial association with seismic activity, the Gales Creek-Newberg-Mt. Angel Structural zone is considered to be potentially active (Geomatrix Consultants, 1995). Relative earthquake hazard maps indicate that the site has a moderate potential for amplification hazards during a design level earthquake.

Liquefaction-Related Ground Failure

No free faces are present toward which lateral spreading could occur. Historic lateral spread failures which adversely affected man-made structures or resulted in large mass movements have tended to occur at sites where a nearby "free face" was present (such as river banks) towards which failure could occur (Seed and Idriss, 1971; Youd and Hoose, 1978). However, given the lack of free faces present at the site, the potential for damaging lateral spreading at the site is low.

Landsliding

In our opinion, the potential for seismically induced landsliding or slope instability at the site is low. In addition, relative earthquake hazard maps indicate that the site has a low potential for instability during a design level earthquake.

Tsunami or Seiche Inundation

The site is located more than several miles from any significant body of water; therefore, the potential for tsunami or seiche inundation of the site is low.

Fault Displacement and Subsidence

There is an inferred fault associated with the Portland Hills Fault Zone mapped across the east side of the site. While no historical seismicity is correlated with the mapped portion of the Portland Hills Fault Zone, it is generally considered to be active and, therefore, there is some potential for fault displacement at the site.

The following paragraphs present specific geotechnical recommendations for design and construction of the proposed project.

RECOMMENDATIONS

Seismic Design

We anticipate that the static equivalent lateral force procedure will be used to determine lateral base shear and dynamic design criteria for the proposed development. The site is located within Seismic Zone 3 of the Uniform Building Code. Based on our explorations, soil conditions correspond to an S_D soil profile, resulting in seismic coefficients of $C_a = 0.36$ and $C_v = 0.54$.

Although code only requires that the building be designed to Zone 3, little data exists as to the response of structures located in the near field during an earthquake (i.e., close to the earthquake source or fault). Because of the site's proximity to the Portland Hills Fault Zone and the sparse historic data regarding the fault zone's behavior, ground shaking at the site during a design level earthquake could exceed design expectations, resulting in greater damage than anticipated. This could be accounted for during design in a number of ways, but we suggest that the owner and design team consider designing the structure using the applicable provisions of the State of Oregon Structural Specialty Code for Zone 4. In our opinion, this would provide greater protection of the structure and safety of the occupants to account for some of the uncertainties associated with the potentially active Portland Hills Fault Zone.

Site Preparation

Where present, existing roots should be stripped and removed from proposed building and pavement locations, and for a 5-foot margin around such areas. Based on our explorations, the depth of stripping will be approximately 4 inches. A representative of CGT should provide recommendations for actual stripping/overexcavation depths, based on observations during site stripping. Stripped material should be transported off site for disposal or stockpiled for use in landscaped areas.

Silt fences, hay bales, buffer zones of natural growth, sedimentation ponds, and granular haul roads should be used as required to reduce sediment transport during construction to acceptable levels. Measures to reduce erosion should be implemented in accordance with Oregon Administrative Rules 340-41-006 and 340-41-455 and Multnomah County regulations regarding erosion control.

After site grading and prior to excavation for footings, a representative from CGT should observe a proof roll of the existing site subgrades to identify areas of excessive yielding. If areas of soft soil or excessive yielding are identified, the material should be excavated and replaced with compacted materials as recommended for structural fill. Areas that appear too soft and wet to support proof-rolling equipment should be prepared in accordance with recommendations for wet weather construction given below.

Wet Weather Considerations

The site soils contain silt and may be susceptible to disturbance during wet weather. Trafficability of the site soils may be difficult and significant damage to subgrade soils could occur if earthwork is undertaken without proper precautions at times when the exposed soils are more than a few percentage points above optimum moisture content.

For construction that occurs during the wet season, the site preparation activities may need to be accomplished using track-mounted equipment, loading removed material into trucks supported on granular haul roads, or other methods to limit soil disturbance. The subgrade should be evaluated during excavation by a qualified geotechnical engineer by probing rather than proof rolling. Soils that have been disturbed during site preparation activities, or soft or loose areas identified during probing, should be removed and replaced with structural fill.

Haul roads subjected to repeated heavy construction traffic will require a minimum of 18 inches of imported granular material. Twelve inches of imported granular material should be sufficient for light staging areas. The imported granular material should consist of crushed rock that is well-graded between coarse and fine, contains no unsuitable materials or particles larger than 4 inches, and has less than 5 percent by weight passing the U.S. Standard No. 200 Sieve. The imported granular material should be placed in one lift over the prepared, undisturbed subgrade and compacted using a smooth-drum, nonvibratory roller.

We recommend that a geotextile be placed as a barrier between the subgrade and imported fill in areas of repeated construction traffic. The geotextile should have a minimum Mullen burst strength of 250 pounds per square inch for puncture resistance and an apparent opening size (AOS) between the U.S. No. 70 and No. 100 Sieves.

Structural Fill

<u>On-site Soils</u>

Use of the on-site silts as structural fill may be difficult because the silt is sensitive to small changes in moisture content and is difficult, if not impossible, to adequately compact during wet weather. We anticipate that the moisture content of the on-site silts will be higher than the optimum moisture content for satisfactory compaction. Therefore, moisture conditioning (drying) should be expected in order to achieve adequate compaction. When used as structural fill, the on-site silts should be placed in lifts with a maximum thickness of 8 inches and compacted to not less than 92 percent of the maximum dry density as determined by ASTM D-1557.

If the on-site soils cannot be properly moisture-conditioned, we recommend using imported granular material for structural fill.

Imported Granular Material

Imported granular structural fill should consist of angular pit or quarry run rock, crushed rock, or crushed gravel and sand that is fairly well-graded between coarse and fine particle sizes. The fill should contain no organic matter or other deleterious materials, have a maximum particle size of 3 inches, and have less than 5 percent passing the U.S. No. 200 Sieve. The percentage of fines can be increased to 12 percent of the material passing the U.S. No. 200 Sieve, if placed during dry weather and provided the fill material is moisture-conditioned, as necessary, for proper compaction. The material should be placed in lifts with a maximum uncompacted thickness of 12 inches and be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D-1557. During the wet season or when wet subgrade conditions exist, the initial lift thickness should be increased to 24 inches and should be compacted by rolling with a smooth-drum, nonvibratory roller.

Shallow Foundations

We recommend that spread footings be founded on the medium stiff to very stiff native silt or on structural fill that is properly installed during construction. We recommend that all spread footings have a minimum width of 24 inches, and the base of the footings be founded at least 24 inches below the lowest adjacent grade. Continuous wall footings should have a minimum width of 18 inches and be founded a minimum of 18 inches below the lowest adjacent grade. Excavations near foundation footings should not extend within a 1H:1V (horizontal to vertical) plane projected from the bottom of the footings.

Bearing Pressure and Settlement

Footings founded as recommended should be proportioned for a maximum allowable soil bearing pressure of 2,500 psf. This bearing pressure is a net bearing pressures and applies to the total of dead and long-term live loads, and may be increased by 1/3 when considering seismic or wind loads.

For the recommended design bearing pressures, total settlement of footings is anticipated to be less than 1 inch. Differential settlements should not exceed ½-inch.

Lateral Capacity

We recommend using a passive pressure of 250 pounds per cubic foot for design, for footings confined by the medium stiff to very stiff native silt or on structural fill. In order to develop these capacities, concrete must be poured neat in excavations, the adjacent grade must be level, and the static ground water must remain below the base of the footing throughout the year. Adjacent floor slabs, pavements, or the upper 12-inch-depth of adjacent, unpaved areas should not be considered when calculating passive resistance.

A coefficient of friction equal to 0.32 may be used when calculating resistance to sliding.

Floor Slabs

Satisfactory subgrade support for building floor slabs supporting up to 350 psf areal loading can be obtained from the medium stiff to very stiff native silt or structural fill when prepared in accordance with the recommendations presented in the "Site Preparation" section of this report. A minimum 6-inch-thick layer of crushed rock should be placed over the prepared subgrade to assist as a capillary break. A subgrade modulus of 175 pounds per cubic inch can be used for the design of the floor slab. Floor slabs constructed as recommended will likely settle less than ½-inch. We recommend that slabs be jointed around columns and walls to permit slabs and foundations to settle differentially.

Drainage Considerations

We recommend that subsurface drains be connected to a tightline leading to the storm drain. Pavement surfaces and open space areas should be sloped such that the surface water runoff is collected and routed to suitable discharge points. We recommend that the ground and paved surfaces adjacent to the buildings be sloped to drain away from the buildings.

Utility Trenches

Utility Trench Excavation

Trench cuts should stand near vertical to a depth of approximately 4 feet in the gravel provided no ground water seepage is observed in the sidewalls. If seepage is encountered that undermines the stability of the trench, the sidewalls should be flattened or shored.

Trench dewatering may be required to maintain dry working conditions if the invert elevations of the proposed utilities are below the ground water level. Pumping from sumps located within the trench will likely be effective in removing water resulting from seepage. If groundwater is present at the base of utility excavations, we recommend placing trench stabilization material at the base of the excavation consisting of 1 foot of well-graded gravel, crushed gravel, or crushed rock with a minimum particle size of 4 inches and less than 5 percent passing the U.S. Standard No. 4 Sieve. The material should be free of organic matter and other deleterious material and should be placed in one lift and compacted until well-keyed.

While we have described certain approaches to the trench excavation, it is the contractor's responsibility to select the excavation and dewatering methods, to monitor the trench excavations for safety, and to provide any shoring required to protect personnel and adjacent improvements. All trench excavations should be in accordance with applicable OSHA and state regulations.

Trench Backfill Material

Trench backfill for the utility pipe base and pipe zone should consist of well-graded granular material containing no organic material or other deleterious material, have a maximum particle size of ¾-inch, and have less than 8 percent passing the U.S. Standard No. 200 Sieve.

Backfill for the pipe base and within the pipe zone should be placed in maximum 12-inch-thick lifts and compacted to not less than 90 percent of the maximum dry density, as determined by ASTM D-1557 or as recommended by the pipe manufacturer. Backfill above the pipe zone should be placed in maximum 12-inch-thick lifts and compacted to not less than 92 percent of the maximum dry density, as determined by ASTM D-1557. Trench backfill located within 2 feet of finish subgrade elevation should be placed in maximum 12-inch-thick lifts and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D-1557.

Permanent Slopes

Permanent slopes should not exceed 2H:1V. Adjacent on-site and off-site structure and surfacing should be located at least 5 feet from the top of slopes. Footings constructed within slopes should have a minimum of 5 feet between the face of the slope and the outer edge of the footing.

Pavements

We recommend a pavement section of 2.5 inches of asphalt concrete over 6 inches of aggregate base be used in paved areas that will be exposed to passenger car traffic only. For pavement areas that will be exposed to up to 10 trucks per day, we recommend a pavement section of 3.0 inches of asphalt concrete over 10 inches of base rock. The design of the recommended pavement section is based on the assumption that construction will be completed during an extended period of dry weather. Increased base rock sections may be required in wet conditions to support construction traffic and protect the subgrade. Asphalt concrete should conform to Section 00745 of the Standard Specifications for Highway Construction, Oregon State Highway Division, 1996 Edition, for light-duty asphalt concrete. Aggregate base should conform to Section 02630 of the same specifications. Place aggregate base in one lift and compact to not less than 95 percent of the maximum dry density, as determined by ASTM D-1557.

OBSERVATION OF CONSTRUCTION

Satisfactory pavement and earthwork performance depends to a large degree on the quality of construction. Sufficient monitoring of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. Subsurface conditions observed during construction should be compared with those encountered during subsurface explorations, and recognition of changed conditions often requires experience. We recommend that qualified personnel visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those observed to date and anticipated in this report.

We recommend that site stripping, rough grading, foundation and pavement subgrades, and placement of engineered fill are observed by the project geotechnical engineer, or their representative. Because observation is typically performed on an on-call basis, we recommend that the earthwork contractor be held contractually responsible for scheduling observation.

LIMITATIONS

We have prepared this report for use by the owner/developer and other members of the design and construction team for the proposed development. The opinions and recommendations contained within this report are not intended to be, nor should they be construed as, a warranty of subsurface conditions, but are forwarded to assist in the planning and design process.

We have made observations based on our explorations that indicate the soil conditions at only those specific locations and only to the depths penetrated. These observations do not necessarily reflect soil types, strata thickness, or water level variations that may exist between explorations. If subsurface conditions vary from those encountered in our site exploration, CGT should be alerted to the change in conditions so that we may provide additional geotechnical recommendations, if necessary. Observation by experienced geotechnical personnel should be considered an integral part of the construction process.

The owner/developer is responsible for insuring that the project designers and contractors implement our recommendations. When the design has been finalized, we recommend that the design and specifications be reviewed by our firm to see that our recommendations have been interpreted and implemented as intended. If design changes are made, we request that we be retained to review our conclusions and recommendations and to provide a written modification or verification.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with the generally accepted practices in this area at the time this report was prepared. No warranty or other conditions express or implied, should be understood.

We appreciate the opportunity to serve as your geotechnical consultant on this project. Please contact us if you have any questions.

Sincerely,

CARLSON GEOTECHNICAL

Brian C. Ranney, G.I.T.
Staff Geologist

REGISTERED PROFESSIONAL ENGINEER #15848 OREGON SEPT. 17, 1991 JEANNE M. NIEMER
EXPIRES: 12-31-02

Jeanne M. Niemer, P.E.
Principal Geotechnical Engineer

Attachments: Figures 1 through 8

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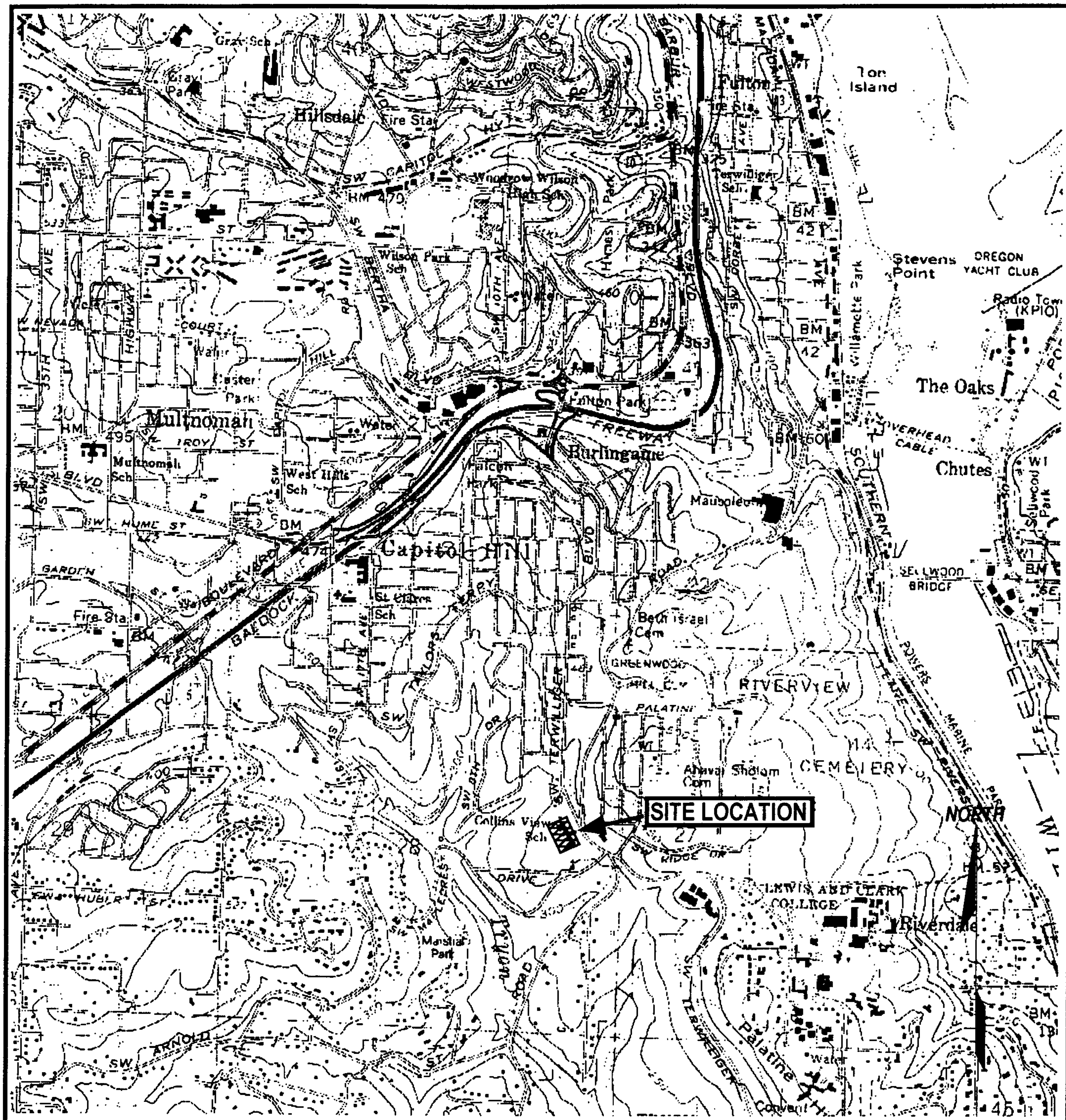
RIVERDALE HIGH SCHOOL

SITE LOCATION



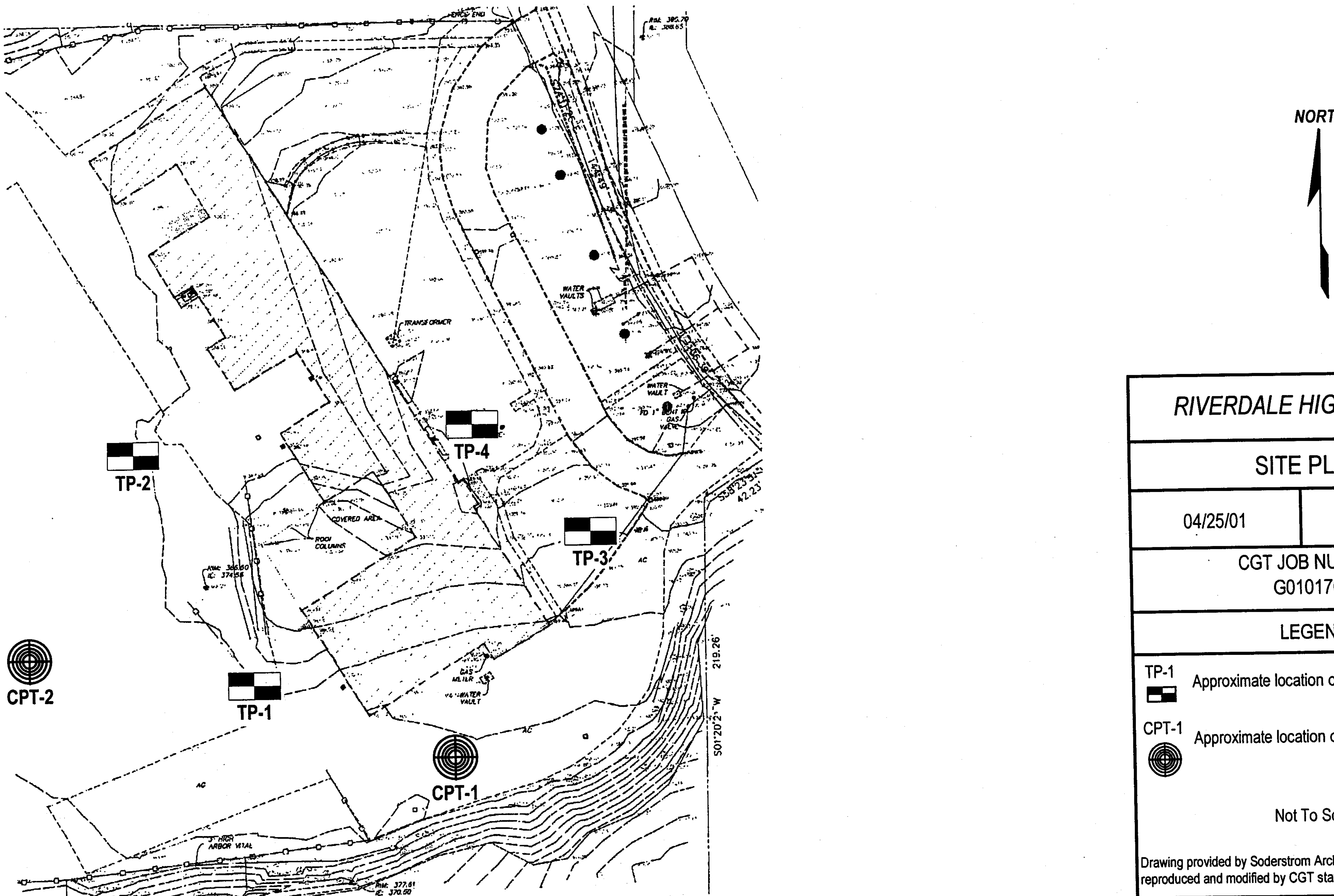
Base map from USGS 7.5 Minute Topographic Map Series, Lake Oswego, OR Quadrangle 1961, Photorevised 1984.

Scale 1 Inch = 2,000 feet

0 2000 4000

Sec.28, T.1S, R.1E of Willamette Meridian

Carlson Geotechnical *P.O. Box 23814* *Tigard, Oregon 97281*	CGT Job No. G0101764	FIGURE 1



Locations of all features are approximate.

RIVERDALE HIGH SCHOOL	
SITE PLAN	
04/25/01	FIGURE 2
CGT JOB NUMBER G0101764	
LEGEND	
TP-1	Approximate location of Test Pit
CPT-1	Approximate location of Cone Penetrometer test

Not To Scale

Drawing provided by Soderstrom Architechs. The drawing has been reproduced and modified by CGT staff.

CARLSON GEOTECHNICAL

A DIVISION OF CARLSON TESTING INC.

P.O. BOX 23814 TIGARD, OR. 97281

RIVERDALE HIGH SCHOOL

Logged by: Brian Ranney — Date Excavated: 04/24/01

Location: See Figure 2 — Surface Elevation:Unavailable

Depth (ft)	Pocket Penetrometer (tsf)	Sample Number	Sample Type	Moisture Content %	Groundwater	Unified Soil Classification	Material Description
	0.0					ML	Soft dark brown **SILT TOPSOIL** with gravel and sand; moist. 4 inch root zone.
1	1.0					ML	Medium stiff to stiff light brown clayey **SILT** with sand (sand is weathered basalt fragments); moist.
	1.0						
2	1.0	S-1					
	0.5						
3	0.5						
	0.5						
4	0.5						grades to orange-grey mottled and sandy with clay below 4 feet.
5		S-2					
6							grades to very stiff below 6 feet.
7							grades to sandy, no clay below 7 feet.
8						ML	Stiff light brown **SILT** with sand; moist.
9		S-3					
10							Test Pit terminated at 10 feet.
11							
12							
13							
14							
15							
16							NOTE: No ground water seepage or caving observed during excavation.
17							

Job No. G0101764	Log of Test Pit 1	Figure: 3

RIVERDALE HIGH SCHOOL

Logged by: Brian Ranney | Date Excavated: 04/24/01

Location: See Figure 2 | Surface Elevation:Unavailable

Depth (ft)	Pocket Penetrometer (tsf)	Sample Number	Sample Type	Moisture Content %	Groundwater	Unified Soil Classification	Material Description
	0.0					ML	Soft dark brown sandy **SILT TOPSOIL** with gravel; moist. 4 inch root zone.
1	0.0					ML/SM	Soft to medium stiff light brown **SILT** with sand; moist.
	0.5						
2	0.5						
	0.5						
3	0.5						
	0.5						
4	0.5	S-1					grades to medium stiff with clay below 4 feet.
5							
6							grades to stiff at 6 feet.
7							
8							
9							
10							Test Pit terminated at 10 feet.
11							
12							
13							
14							
15							
16							NOTE: No ground water seepage or caving observed during excavation.
17							

Job No. G0101764	Log of Test Pit 2	Figure: 4

RIVERDALE HIGH SCHOOL

Logged by: Brian Ranney | Date Excavated: 04/24/01

Location: See Figure 2 | Surface Elevation:Unavailable

Depth (ft)	Pocket Penetrometer (tsf)	Sample Number	Sample Type	Moisture Content %	Groundwater	Unified Soil Classification	Material Description
						ML	Soft dark brown **SILT TOPSOIL** with some gravel; moist.
	0.25						
1	0.5					ML	Soft to medium stiff brown sandy **SILT FILL** with some gravel;moist.
	0.5						terra cotta pipe at 1.5 feet.
2	0.5	S-1					fill over cobble at 2 feet.
	1.0						
3	1.5						
	1.5						
4	1.5					ML/ SM	Stiff light brown **SILT** with sand; moist.
5		S-2					
6							
7		S-3					
8							
9							
10							Test Pit terminated at 10 feet.
11							
12							
13							
14							
15							
16							NOTE: No ground water seepage or caving observed during excavation.
17							

Job No. G0101764	Log of Test Pit 3	Figure: 5

RIVERDALE HIGH SCHOOL

Logged by: Brian Ranney — Date Excavated: 04/24/01

Location: See Figure 2 — Surface Elevation:Unavailable

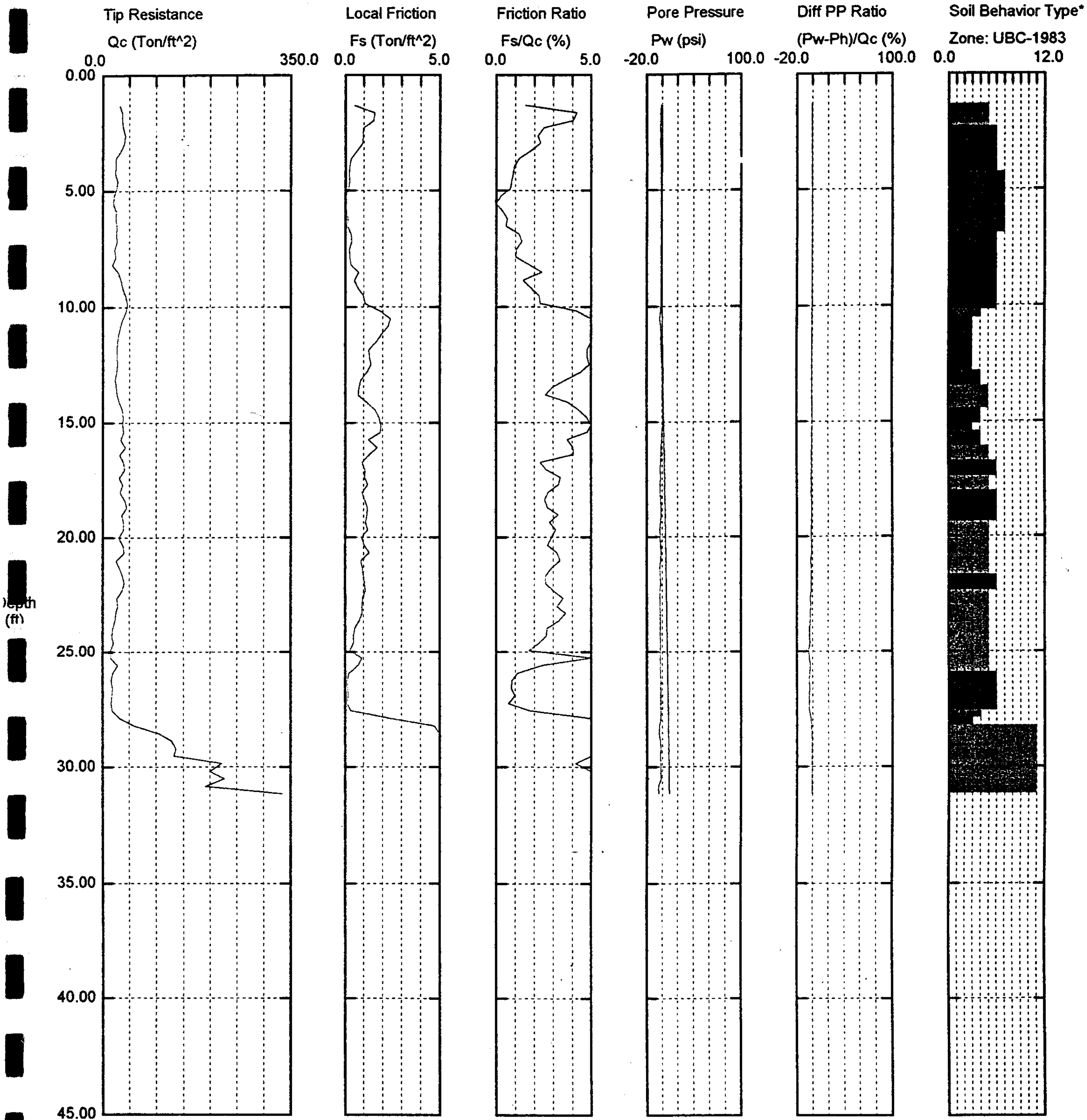
Depth (ft)	Pocket Penetrometer (tsf)	Sample Number	Sample Type	Moisture Content %	Groundwater	Unified Soil Classification	Material Description
						ML	Soft dark brown **SILT TOPSOIL** with sand; moist. 4 inch root zone.
0.5	0.0						
1	0.5					ML	Medium stiff to stiff light brown orange-grey mottled sandy **SILT** with clay; moist.
1.5	0.5						
2	1.5						
2.5	2.5						
3	3.5	S-1					plasticity index = 5.7
3.5	3.5						
4	2.0						
4.5						ML	Stiff light brown **SILT** with sand; moist.
5		S-2					
6							
7							
8							
9							
10							Test Pit terminated at 10 feet.
11							
12							
13							
14							
15							
16							NOTE: No ground water seepage or caving observed during excavation.
17							

Job No. G0101764	Log of Test Pit 4	Figure: 6

Subsurface Technologies

Operator: W.MCC / A.MEE
Sounding: SND252
Cone Used: 683 TC

CPT Date/Time: 04-26-01 12:18
Location: CPT-2 RIVERDALE
Job Number: G0101764



Maximum Depth = 31.17 feet

Depth Increment = 0.16 feet

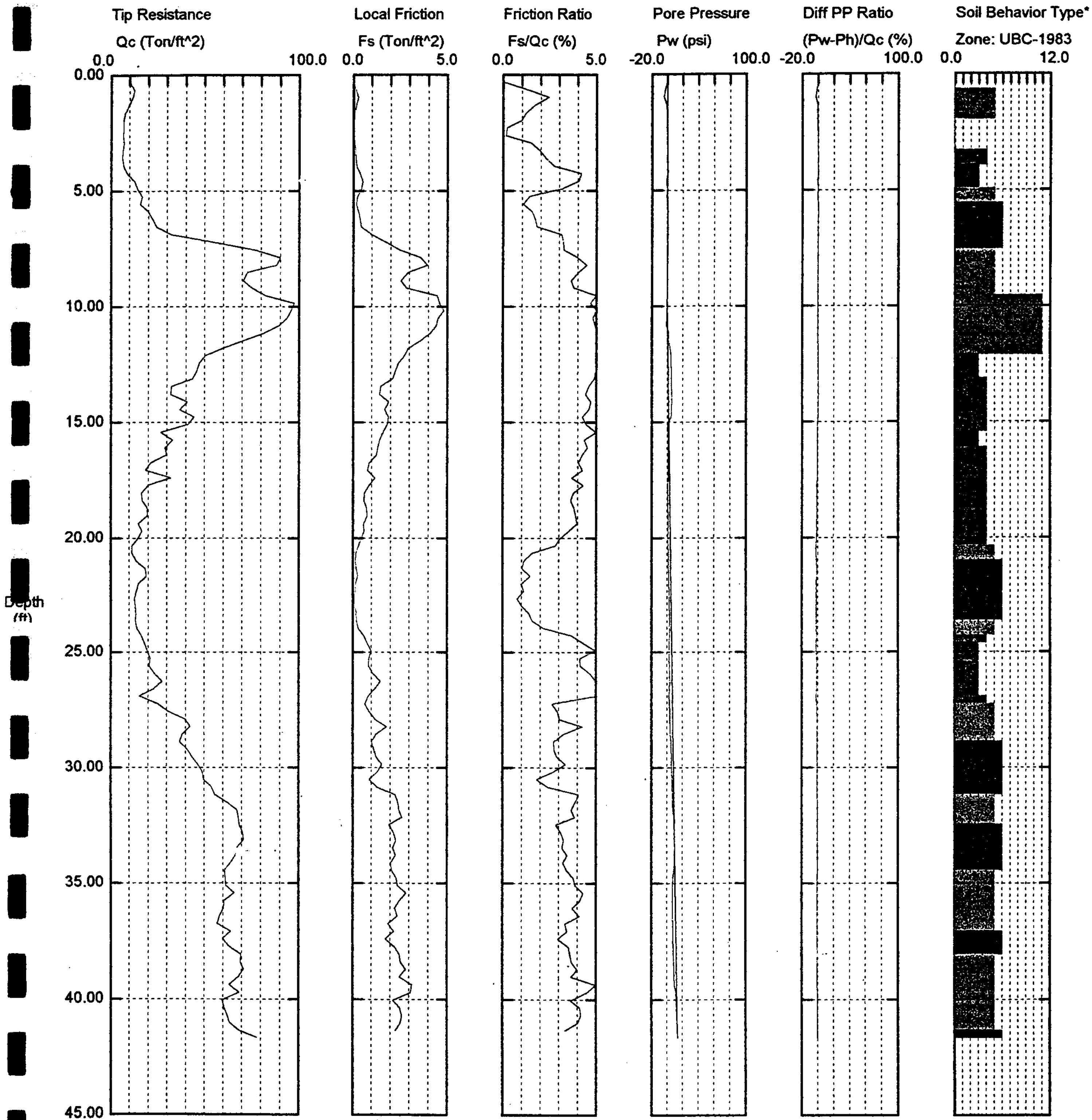
1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

Figure 7

Subsurface Technologies

Operator: W.MCC / A.MEE
Sounding: SND251
Cone Used: 683 TC

CPT Date/Time: 04-26-01 11:21
Location: CPT-1 RIVERDALE
Job Number: G0101764



Maximum Depth = 41.67 feet

Depth Increment = 0.16 feet

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

Figure 8