Appendix B

Geotechnical Report
November 2007

URS

Preliminary Geotechnical Report
Saddleback Wind Energy Project
White Salmon, Washington

Prepared for

SDS Lumber Company

November 2007

Prepared by

URS Corporation
URS Job #33758687

URS

URS

November 5, 2007

SDS Lumber Company
PO Box 266
Bingen, WA 98605

Attn: Jason S. Spadaro, President

Re: Preliminary Geotechnical Report
Saddleback Wind Energy Project
SDS Lumber Company
White Salmon, Washington
URS Job No: 33758687

Dear Mr. Spadaro:

We are pleased to submit herewith our report entitled "Preliminary Geotechnical Report – Saddleback Wind Energy Project, White Salmon, Washington." This report presents our findings, conclusions, and recommendations regarding the proposed project.

It has been our pleasure to assist you with this project. Should you have any questions regarding the contents of this report, please call us at your convenience.

Yours very truly,

URS CORPORATION

Dan B. Meier, LEG
Senior Engineering Geologist

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1 INTRODUCTION

1.1 GENERAL

The purpose of this report is to provide preliminary geotechnical feasibility and recommendations regarding design of wind turbine tower-foundations and approach roads for the Saddleback Wind Energy Project (SWEP). This is at site located approximately 7 miles west of the town of White Salmon, Washington, and approximately 2 miles east of the Little White Salmon River. The location of the site is shown on Figure 1, Vicinity Map.

The project area is located on private land immediately north of the Columbia River Gorge National Scenic Area boundary. The area of the proposed project is approximately 3.2 square miles (2000 acres). The Project site is located on a series of north trending ridges that range in elevation from approximately 2100 to 2300 feet above mean sea level (msl). The land west of the proposed Project site drops sharply to a narrow river terrace and then to an elevation of less than 800 feet above msl in the Little White Salmon River valley. The topography to the northeast of the site drops gradually toward the White Salmon River or climbs gently up the northeast flank of Underwood Mountain (2,728 ft above msl). To the south, the topography drops to the Columbia River.

1.2 PROPOSED CONSTRUCTION

The SWEP project involves the installation of approximately forty eight wind energy turbines at its White Salmon, Washington site. As of the date of this report, the tower designer and tower locations have not been finalized. Because of this, the exact bearing capacity of the foundations required to support the Wind Turbine Generators (WTG) is not known. URS received information from a proposed turbine construction contractor, D.H. Blattner & Sons, Inc. which preliminarily assumed installation of 80-meter high GE 1.5 WTG which will be supported by 30 foot-deep concrete foundations. Each turbine tower will be coupled to the foundation with 128 rock anchor (consistent with GE towers). Final foundation design will be developed after detailed investigation and when designs are finalized.

Construction and Maintenance access to the proposed tower locations will be achieved by improving existing roadways that have historically been used primarily in support of logging activity and for access to existing BPA transmission lines. Modification of the roadways will be necessary to support the long and heavy loads that will be required for delivery of the wind turbine systems.

1.3 SCOPE OF WORK

To complete this preliminary report, URS has completed the following scope of work.

County Roads - Pavement Engineering – Upon speaking with the County personnel URS recommends that best pavement approach at this time would be to develop a "pavement assessment and design" strategy rather than a comprehensive field effort – as the field conditions will likely change over the next couple of years. The plan entails the following and is located in Appendix C:

- Anticipated routes for the project
- Existing pavement sections along the route – as they are best known (field verification of the actual sections later)
- Estimate axle weights and number of trips for trucks
- Strategy and design parameters for the haul roads on SDS property
- Strategy for pavement assessment just prior to heavy hauls, and strategy for pavement assessment upon completion of the heavy hauls

Landslide Hazards – URS completed a landslide assessment which should be repeated just prior to site development so that we would be able to note changed conditions. This work is presented in a Appendix D of this report and entails the following:

- Review of existing published geologic and geologic hazards literature for the site.
- Review of Sections of the County Code that address Geologically Hazardous Areas;
- Review of aerial photographs if they are available. URS will also review public groundwater records for the immediate area, as available.
- Conduct a site visit to evaluate site conditions. This will include evaluating the geologic conditions and existing slopes as well as geologic activities (slides, faulting, rupture, etc.) that may have influenced geologic hazards at the site. URS anticipates this will take approximately one day of field work.

Preliminary Geotechnical Engineering – URS has developed a preliminary foundation design based on the encountered foundation conditions. To complete this, URS did the following:

- Reviewed of the URS / Dames & Moore project files for information pertaining to this project. URS will review geologic maps of the area to further understand the soils prior to investigation.
- URS personnel met with the Owner's site representative just prior to investigation. **The tower designer has not been identified for the project site, and the exact locations of the towers have not been identified. Therefore, URS will not perform any deep subsurface explorations at the site.** We will, however, perform up to twelve 15-foot deep test pits that will be excavated by a SDS Employee. The test pits will be backfilled with the excavated soils.

- Along the roadway areas, URS assessed the near surface capacity and modulus of the onsite soils by performing a total of 30 dynamic cone penetrometer tests (DCP) located selectively across the existing roads. Each DCP probe was advanced to approximately 3 feet, or refusal, whichever is greater, below existing grade or the pavement surface to accurately determine the *insitu* subgrade characteristics. The DCP will furthermore clearly delineate between layers of weak and strong subgrade soils. The DCP is an inexpensive, manually driven exploration device designed for pavement assessments with limited subgrade exposure.
- Five field resistivity tests to assess the electrical resistivity of the near surface soils. We will use the 4 point "Wenner" method with electrodes spaced at 5, 10, 20 and 30 feet. The design engineers, to assess cathodic protection and grounding grid requirements, will use this data.
- URS will be doing limited laboratory testing for the project. We anticipate moisture contests, visual manual identification and atterberg limits. All will be done to applicable ASTM standards.

- ***Preliminary Recommendations for Spread Footings, Mat Foundations,*** - Allowable soil bearing pressure, stated as net or gross pressure at the underside of foundation level. State how allowable pressure changes with depth.
  - Vertical Subgrade Reaction Modulus "k" for slab on grade design.
  - Lateral sliding friction coefficients between soil and concrete.
  - Active and at-rest equivalent fluid pressures for the design of earth retaining structures.
  - Passive equivalent fluid pressures used to resist horizontal forces.
  - Soil unit weights dry and saturated.
  - Backfill material specifications and installation specifications for subgrade and base material, including Preliminary Recommendations for preparation, placement, and compaction. Comment on the use of native materials for backfill material.
  - Settlement values (total and differential, including long term settlements over period of several years for the reciprocating machinery).
  - Minimum depth to bottom of footing based on frost depth and bearing values.
  - Comment on the appropriateness of increased allowable soil bearing and lateral resistance for short duration loads.
  - Anchorage requirements for the shallow foundations.

- ***Preliminary Seismic Considerations***
  - Discuss past seismic activity, including known faults, and potential for a future event.
  - Discuss potential for liquefaction during a seismic event.
  - Define the Site Class and Soil Profile per 2006 edition of the IBC.

2 FIELD AND LABORATORY INVESTIGATIONS

2.1 SUBSURFACE INVESTIGATIONS

The field exploration program was conducted between September 18 and September 25, 2007. The subsurface investigation included the completion of twelve test pits to assess near surface soil and rock characteristics, and thirty dynamic cone penetration (DCP) tests along the roadway alignments to evaluate the near surface bearing capacity and moduli of onsite soils.

2.1.1 Test Pits

The test pit exploration program was conducted between September 18 and September 19, 2007. The program consisted of 12 test pits excavated to depths of 7 to 16 feet below ground surface (bgs). Approximate test pit locations are shown on the Figure 2. Test pit logs are presented in Appendix A.

A representative from URS maintained a log of conditions observed in the test pits, visually classified the soils encountered according to the Unified Soil Classification System and obtained representative bulk samples at selected intervals. The test pits were backfilled with excavated materials and compacted using the bucket of the backhoe. The stratigraphic contacts indicated within the test pit logs represent the approximate boundaries between soil and rock types; actual transitions may be more gradual and indistinct. The subsurface conditions depicted are only for the specific locations reported, and therefore, are not necessarily representative of other locations. The sample intervals are shown on the test pit logs attached in Appendix A.

2.1.2 Dynamic Cone Penetration Test

Roadway subgrade testing was conducted using the Dynamic Cone Penetrometer (DCP). A total of 30 DCP tests were performed for the proposed site access roads to depths of between 1 and 3 feet bgs. The DCP is a widely used device to determine *in situ* strength properties of base materials and subgrade soils. The four main components of the DCP include the cone, rod, anvil, and hammer. The cone is attached to one end of the DCP rod while the anvil and hammer are attached to the other end. Energy is applied to the cone tip through the rod by dropping the 17.64-lb hammer a distance of 22.6-inches against the anvil. The diameter of the cone is 0.1575-inch larger that the rod to ensure that only tip resistance is measured. The number of blows required to advance the cone into the subsurface materials is recorded. The DCP index is the ratio of the depth of penetration to the number of blows of the hammer. This can then be correlated to a variety of material properties, including California Bearing Ratio (CBR) and

Resilient Modulus, both of which are used in pavement design. Logs of the DCP test results are included in Appendix C to this report. Approximate DCP test locations are shown on Figure 2.

2.1.3 Field Soil Electrical Resistivity Testing

Resistivity tests for the proposed Wind Energy Project site were performed on September 24, 2007 to assess the electrical resistivity of the near surface soil. The locations of the tests are noted on Figure 2, Site Plan. Test locations were selected to correspond with the sites of the proposed Wind Turbine Generators. A Strata-Scout Model R-40CY resistivity meter was used to measure the resistance by the 4-point (Wenner) configuration at equal spacing of 5, 10, and 20 feet for tests R-1 through R-5. A Nilsson Model 400 meter was used in the same configuration at equal spacing of 5, 15, and 20 feet for tests R-1 through R-5. The resistance and spacing were used to calculate the resistivity at each location. Results of the resistivity tests are presented in Appendix B. These indicators can be used for determinations regarding corrosion potential and grounding grid design.

2.2 LABORATORY TESTING

Upon completion of the field investigation, samples obtained from test pits were transported to our Portland, Oregon laboratory for further examination and testing. The laboratory tests included the following:

- Visual soil classification performed in general accordance with ASTM D 2487;
- Moisture content performed in general accordance with D 2216;
- Grain size analysis performed in general accordance with ASTM D 422 and D 1140;
- Atterberg Limits in general accordance with ASTM D4318..

Complete individual laboratory test results are shown in Appendix B of this report.

3 SITE DESCRIPTION

3.1 REGIONAL GEOLOGY

The White Salmon, Washington area is located within the Cascade Range and the Columbia Intermontane Physiographic Province. The project area is located just within the western boundary of the Columbia Plateau, which is located at the western edge of the Columbia Intermontane Physiographic Province (Freeman et al, 1945). This lowland province is surrounded on all sides by mountain ranges and highlands, and covers a vast area of eastern Washington and parts of northeastern Oregon and western Idaho. The Columbia Plateau is underlain by a series of layered basalt flows extruded from vents (located mainly in southeastern Washington and northeastern Oregon) during the Miocene epoch (between 5.3 and 23.8 million years before present [B.P.]). Collectively, these basalt flows are known as the Columbia River Basalt Group (CRBG). Individual basalt flows range in thickness from a few millimeters to as much as 300 feet. Where significant time elapsed between successive flows, interflow zones developed. The interflow zones are characterized by the presence of highly weathered basalt and paleosols. These interflow zones are generally significantly weaker than the surrounding basalt and sometimes form basal failure surfaces for large landslide complexes within the CRBG.

A variety of younger volcanic rocks and sedimentary materials that range from Pliocene (1.8 to 5.3 million years B.P.) to Holocene (less than 10,000 years B.P. in age) overlie the CRBG in the project area. Sedimentary rocks are generally thought to underlie the basalts in the Project area.

3.2 SITE GEOLOGY

The proposed Project site is located within the northern boundary of the structural Hood River Valley, which extends a few miles into southern Washington. In general, the geology of the area consists of basalt flows extruded from local vents, layered with conglomerate, tuff, tuff breccias, and other volcanoclastic deposits. These formations are typically overlain by silt and clay soil of varying thickness in the Project vicinity.

The bedrock underlying the proposed Project site consists of Grande Ronde Basalt of the CRBG and Quaternary basalt of Underwood Mountain - a shield volcano that lies approximately midway between the lower reaches of the Little White Salmon and White Salmon Rivers. Its southern slopes drain to the Columbia River. Site geology is presented on Figure 3.

Underwood Mountain Basalt Unit: The Pleistocene-epoch (1.8 million years to 10,000 years B.P.) basalts and cinders erupted from the Underwood Mountain vents and overlie the Tertiary

CRBG Grande Ronde and Wanapum basalts. Public records of wells located within the Underwood volcanic field indicate a 310-foot thick repetitive sequence of thin lava flows (2 to 8 feet thick), cinders and silty-clays overlying a productive confined aquifer consisting of intensely fractured Grande Ronde basalt (Yinger, 2000 and 2001). The Miocene-epoch Grand Ronde Basalt consists of multiple basalt flows that are a subgroup of the CRBG, and has been described to have a thickness of up to 1000 feet, although the thickness in the Project vicinity is not known.

Field observations of rock outcrop and test pits excavated during a geotechnical investigation at the proposed site indicate that the near-surface rock consists of yellow-gray volcanoclastic rocks, medium to dark gray, fine-grained to medium-grained basalt and andesite, which is fractured into angular gravels, cobbles, and boulders. The basalt observed in the test pits was most commonly vesicular, very soft to moderately hard, and decomposed to slightly weathered. Some zones displayed non-vesicular characteristics and were generally harder. In most exposures the basalt was moderately to highly weathered, with fractures and vesicles filled by clayey residual soil. In most of the test pits excavated in this basalt, the rock is weathered into varying layers of residual (clay) soil, and clayey gravelly cobble-sized basalt. The residual soil layers often exhibit remnant rock structure.

Unconsolidated Deposits: Unconsolidated deposits are thin to absent in the Project vicinity. Based on observations made during field reconnaissance, the surficial materials consisted primarily of a thin veneer of brown, silty topsoil that is likely derived from forest duff and wind-blown deposits. The thickness of this material varied across the site from a few inches to three feet, based on test pit observations. In several areas bedrock and talus were observed at the ground surface.

Landslide Deposits: Regional Geologic maps indicate the presence of Quaternary-age mass wasting landslide deposits located to the north of Underwood Mountain (Korosec, 1987, excerpted in Figure 3, this report). These deposits are mapped as a large landslide, estimated to be approximately 1/3 square mile in area and almost a mile long. A URS Engineering Geologist reviewed stereo aerial photographs that were flown specifically for this project in 2007 and performed a one day site reconnaissance. There is no obvious evidence, based on the review, to suggest the presence of a landslide as mapped on the 1:100,000 scale geologic map. If landslide deposits are present, they are so old that most or all of the geomorphic evidence has been removed by erosion. A separate Landslide Hazard Report for the project is presented as Appendix C to this report.

Faults: No faults are mapped within the footprint of the proposed Project area. However, faults are mapped approximately 1.5 miles southwest and northeast of the proposed Project area. Many of these faults are inferred and shown as dotted lines buried by younger surficial deposits. The activity of the area faults is unknown. However, a review of aerial photography shows no indication of recent movement along the trace of the inferred faults.

3.3 SUBSURFACE CONDITIONS

The following is a general summary of the soil conditions encountered in the explorations conducted at the site to date. More detailed description of the soils encountered in the test pits are provided on the logs included in Appendix A.

Based on the current test pits and field observations, we anticipate that unconsolidated soils extend up to 3 feet below ground surface (bgs). The surficial soils are primarily characterized as soft, moist sandy SILT [ML] to CLAY with sand [CL], and clayey SAND [SC]. Immediately beneath the unconsolidated soils, rock with variable strength and weathering properties is present. The test pit data is limited to depths no greater than 16 feet bgs. It is anticipated that rock quality of the basalts will improve with depth but that weaker interflow zones consisting of volcaniclastic material and paleosols are possible at any depth. Prior to final design of the tower foundations, additional subsurface investigations (boreholes) will be required to provide geotechnical data at foundation and anchor depths.

The United States Department of Agriculture National Resources Conservation Service (NRCS) describes the soils in the project vicinity as follows (USDA, 2003):

- **Chemawa Series**: The Chemawa series consists of very deep soils (up to 5 feet) formed in alluvium from volcanic ash and basalt. The soils exist on terraces, footslopes and backslopes at elevations between 800 and 2500 feet in southeast Skamania County and southwest Klickitat County. Chemawa Soils are well drained with slow to medium runoff and moderate permeability.

- **McElroy Series**: The McElroy series consists of very deep soils (up to 5 feet) formed in colluvium and residuum from basalt with a mantle of volcanic ash that influences soils in the top 9 to 13 inches. The soils exist on the footslopes and backslopes of mountains on slopes from 5 to 90 percent at elevations from 400 to 2600 feet in eastern Skamania County and western Klickitat County. McElroy Soils are well drained with medium to rapid runoff and moderate permeability. The series was established in 1981 following the introduction of volcanic ash from the eruption of Mt. St. Helens.

- **Timberhead Series**: The Timberhead series consists of very deep soils (up to 5 feet) formed in residuum and colluvium from basalt mixed with volcanic ash. The soils exist on mountain slopes between 5 and 65 percent at elevations from 2000 to 3600 feet in Skamania County and western Klickitat County. McElroy Soils are well drained with medium to rapid runoff and moderate permeability. The series was established in 1981 following the introduction of volcanic ash from the eruption of Mt. St. Helens.

- **Underwood Series**: The Underwood series consists of very deep soils (5 feet or more) formed in residuum and colluvium from basalt and andesite with a thin mantle of volcanic ash. The soils exist on benches, backslopes, and footslopes of mountains with slopes between 2 and 50 percent at elevations between 500 and 2700 feet in southeast Skamania County and west Klickitat County. Chemawa Soils are well drained with slow to medium runoff and moderately slow permeability.

- **Undusk Series**: The Undusk series consists of very deep soils (5 feet or more) formed in residuum and colluvium from basalt and andesite with a thin mantle of volcanic ash. The soils exist on benches, backslopes, and footslopes of mountains with slopes between 2 and 50 percent at elevations between 500 and 2700 feet in southeast Skamania County and west Klickitat County. Chemawa Soils are well drained with slow to medium runoff and moderately slow permeability.

3.4 GEOLOGIC HAZARDS

3.4.1 Description of Geologic Hazards

In general, geologic hazards are geologic processes or geological conditions that constitute a threat to human safety, improved property, and the natural environment. For the purposes of this report, the focus is on geologic hazards associated with the construction and operation of the proposed wind energy project. The primary geologic hazards present in the project area can be divided into three categories; landslides, seismic, and volcanic. Landslide hazards include rotational-translational slides, earthflows, debris slides, and debris flows. Seismic hazards can include ground shaking, fault surface rupture, settlement, liquefaction, and lateral spreading. Volcanic hazards at the project site are generally limited to ash fall from any of three nearby Cascade volcanoes. By identifying areas prone to specific geologic hazards, design and construction details can be modified to avoid dangers to human safety, improved property, and environmentally sensitive areas from such hazards as a result of project construction and operation.

3.4.2 Landslide Hazards

The most common types of landslides in the Pacific Northwest include rock falls, topples, rotational-translational slides, earthflows, debris slides, and debris flows. Most slope failures are complex combinations of these distinct types, but the generalized groupings enable the investigator to communicate the types of hazards anticipated and observed.

Landslides can be initiated in marginally stable slopes by a number of natural and human disturbances. Processes and conditions that can trigger slope failure include earthquake shaking, volcanic eruption, deforestation, intense rainfall, and rapid snowmelt. Two of the most common triggering events in southern Washington are intense precipitation and human alterations.

The Pacific Northwest is subject to severe rainfall storm events, particularly in the wet winter and spring months of November through April. These relatively high-precipitation storm events can trigger slope failures through a number of mechanisms. Water infiltration into zones of weakness can trigger failures by reducing the frictional resistance to sliding, increasing pore pressures within slope masses and adding weight acting downslope. Typically, all three mechanisms combine during longer duration, heavy precipitation or rain on snow events to trigger slope stability problems.

Landslide hazards were assessed as part of the public document review, aerial photograph investigation, and field reconnaissance. Results of the landslide hazard review are presented as a separate technical report in Appendix D to this report.

3.4.3 Seismic Hazards

Liquefaction is a phenomenon whereby soils undergo significant loss of strength and stiffness when they are subjected to vibration or large cyclic ground motions produced by earthquakes. Typically, cyclic loading of saturated soils leads to the build up of excess pore-water pressure as a result of soil particles being rearranged with a tendency toward denser packing. Under undrained conditions (such as during earthquake shaking), loads are transferred from the soil skeleton to the pore-water with consequent reduction in the soils' shear strength.

Saturated granular soils without cohesive fines (i.e. gravels, sands and silts) are most susceptible to liquefaction. Other factors affecting the potential for liquefaction in soils are density, amplitude of loading, confining pressure, past stress history, age of soil deposit, the size, shape and gradation of particles, and the soil fabric structure. Liquefaction-induced ground settlement and lateral spreading have been the primary cause for extensive damage to aboveground structures, foundations and pipelines during many earthquakes.

Test pits excavated at the project site encountered shallow bedrock covered with a combination of cohesive and cohesionless soil. No groundwater was observed in any of the test pits. Based on the soils encountered during the field explorations, it is URS' opinion that the potential for liquefaction is very low at this site.

The risk of seismically inducted settlement and lateral spreading is low due to the low liquefaction potential. It is URS' opinion settlements and lateral spread induced by a seismic event will be minimal.

Coseismic surface rupture occurs when a fault breaks to the land surface during an earthquake. Surface rupture is usually associated with moderate to large earthquakes (magnitude 6.5 or greater) or, rarely during smaller, very shallow events. There are no mapped faults crossing the site. Therefore, the potential for coseismic primary surface rupture at the proposed project site is small.

3.4.4 Volcanic Hazards

Within the region of the site, the USGS recognizes three volcanoes as either active or potentially active: Mount Hood, Mount Adams, and Mount St. Helens. In the last 200 years, only Mount St. Helens has erupted more than once (USGS, 2002b). Impacts in the geographic region surrounding the Project site from volcanic activity can be either direct or indirect. Direct impacts include the effects of lava flows, blast, ash fall, and avalanches of volcanic products. Indirect effects include mudflows, flooding, and sedimentation. Data accumulated as a result of the 1980 Mount St. Helens eruption indicate that there could be ash fallout in the geographic region surrounding the Project site if one of the three regional volcanoes were to erupt.

In the event that a volcanic eruption would damage or impact Project facilities, the Project facilities would be shut down until safe operating conditions return. If an eruption occurred during construction, a temporary shut-down would most likely be required to protect human health and equipment.

3.5 GROUNDWATER CONDITIONS

During the current subsurface exploration ground water was not encountered in the site up to a depth of 16 feet bgs. It should be noted that these observations reflect groundwater levels at the time of the field investigation and actual groundwater levels may fluctuate significantly in response to seasonal effects, regional rainfall, and other factors not observed during this investigation. There may be regional or perched water tables at greater depth. Prior to final design of the tower foundations, additional subsurface investigations (boreholes) will be required to provide geotechnical data at foundation and anchor depths. Future deep foundation investigations will include observation of groundwater, if encountered.

4 SEISMIC DESIGN

4.1 REGIONAL SEISMICITY

The Pacific Northwest has four types of seismic sources due to the presence of the Cascadia subduction zone. These sources include (1) the subduction zone megathrust, which represents the boundary (interface) between the subducting Juan de Fuca plate and the overriding North American plate; (2) faults located within the Juan de Fuca plate (referred to as the intraplate or intraslab region); (3) crustal faults principally in the North American plate; and (4) volcanic sources beneath the Cascade Range (Wong and Silva, 1998). Each of these events has different causes, and therefore produces earthquakes with different characteristics (that is, peak ground accelerations, response spectra, and duration of strong shaking).

Because of their proximity, crustal faults are possibly the most significant seismic sources to inland sites. Studies by Pezzopane (1993) and Geomatrix Consultants (1995) show that at least 70 crustal faults having earthquake potential exist in southwest Washington and northwest Oregon. Many of these faults were unknown or not recognized as being seismogenic a decade ago. Although the largest known crustal earthquake in south-west Washington and western Oregon is only about M_W 6 (Wong and Bott, 1995), potential exists for events of M_W 6½ or greater along several recognized faults.

4.2 2006 IBC SEISMIC DESIGN

We recommend that all structures on the site be built in accordance with the seismic design provisions presented in the 2006 version of the IBC, and ASCE/SEI 07-05. At this time, and without unconfined compressive strengths of the bedrock, URS best describes these soils as Soft Rock (Soil Site Class B) Based on the site location and site conditions described above, we recommend that the values listed in Table 4-1 be used for seismic design of the project in accordance with Section 1613.5.3 of the 2006 IBC. The occupancy category of the proposed structure is assumed III as per Section 1613.5.6 of the 2006 IBC.

Table 4-1: 2006 IBC Seismic Design Values

Parameter	Value	2006 IBC/ASCE 7-05 Reference
Soil Profile Site Class	B	Table 1613.5.2
0.2 Second Spectral Acceleration S_s	0.513 g	Figure 1613.5 (1)
1.0 Second Spectral Acceleration S_1	0.193 g	Figure 1613.5 (2)
Peak Ground Acceleration ($0.4S_{Ds}$)	0.136 g	ASCE 7-05 equation 11.4-5
Site Coefficient F_a	1	Table 1613.5.3 (1)
Site Coefficient F_v	1	Table 1613.5.3 (2)
Seismic Design Category[1]	B	Tables 1613.5.6 (1) & (2)

1. Assumes Seismic Use Group III

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

In support of this report, URS has conducted a limited site investigation, including several test pits, to determine the near-surface properties of the soil and rock. Rock, with varying strength and weathering characteristics, was encountered at shallow depths (ranging between 3 to 12 feet bgs). Because of the presence of relatively shallow rock, and the high potential overturning loads anticipated for the turbine towers, URS recommends rock anchored mat-slab foundations to support the turbines for this preliminary assessment (see Figure 4). However, a drilled shaft foundation concept may prove to be more beneficial in the future, as the design advances. The viability of this foundation concept will be determined with the final geotechnical engineering report for this site.

5.2 WIND TURBINE GENERATOR FOUNDATIONS

As of the date of this report, the tower designer and tower locations have not been finalized. Because of this, the exact loads on the foundations required to support the turbines is not known. One proposed turbine construction contractor, D.H. Blattner & Sons, Inc. has preliminarily assumed installation of 80-meter high GE 1.5 Wind Turbine Generators (WTG) which will be supported by 30 foot-deep concrete foundations. Each turbine tower will be coupled to the foundation with 128 anchor bolts (consistent with GE towers). Based on the soil conditions present at the site, URS preliminarily recommends that mat-slab foundations be used to support the proposed wind turbine tower structures. Once the final exploration is completed, URS may also recommend that the foundation system be drilled shafts.

Due to the anticipated high axial and lateral loads exerted by the towers, URS recommends that the base of the foundation excavation be established in competent rock and be properly leveled to provide uniformity of support. Following excavation, the bearing surfaces should be thoroughly cleaned of loosened or disturbed rock, by hand if necessary. This removed rock should be A URS inspector should manually probe the area within the excavation bottom for confirmation of the bearing surface and to identify any soft areas. Any soft or unsuitable rock encountered at the base of foundation excavations should be removed. URS anticipates that the excavation base would irregular, but if completed properly, the allowable bearing capacity on the excavated base would be on the order of 30,000 psf with an allowable increase of 1/3 to this value for temporary loading conditions. This would have to be verified with additional borings and laboratory testing of the cored rock.

5.2.1 Settlement of WTG Foundation

The WTG foundations are anticipated to be established in rock. Because of this, elastic settlement of the foundation is anticipated to occur as the loads are applied, however, once the tower is constructed, additional settlements would be negligible.

5.2.2 Earth Pressure and Friction Factors

Passive earth pressures acting against the toe of the shallow foundations and friction on the base of the foundations may be considered to provide resistance to lateral forces tending to cause translational sliding. These structural members should be considered for counteracting lateral forces only if the member is placed in direct contact with tested and approved soils. If the foundation is constructed by using forms, lean concrete may be placed between the foundation and the undisturbed wall of the adjacent excavation in order to provide the direct contact required to consider passive pressure for counteracting lateral movement. The lean concrete should have a minimum 28-day compressive strength of 1,500 psi. An allowable passive pressure of 8,000 psf may be used for the underlying bedrock, for the foundation face located more than one-foot below the adjacent elevation of the bedrock. This is based on a factor of safety of two and requires confirmation with the final deisng..

An ultimate friction factor of 0.5 for mass concrete on the compacted granular fill can be used for design for those portions of the foundations with full positive pressure on the base of the foundation. Only long-term dead loads should be considered in calculating the available friction on the foundation base.

5.2.3 Rock Anchor Design and Installation

We anticipate that rock anchors are used in the final foundation designs to resist overturning loads and provide lateral stability, we recommend nominally prestressed anchors consisting of high-strength reinforcing bars grouted into the rock using polyester resin grout such as Fasloc or Celtite. These commercially available, two-component grouts are contained in plastic cartridges. The grout cartridges are placed in the hole and the bar is driven into the hole while being rotated in order to expose and mix the grout. The use of the concrete grout is an acceptable alternative, but will generally require longer anchors and a larger diameter drilled anchor hole for a given bar diameter. Due to the possible fractured and vesicular nature (presence of small voids within the rock mass) of the basalt (the extent of which needs to be determined with a final geotechnical report), the concrete grout alternative may be necessary to prevent reduction of anchor strength due to loss of resin grout into voids. Prior to final design of the tower foundations, additional subsurface investigations (boreholes) will be required to provide geotechnical data at foundation and anchor depths. Core samples from the borehole investigations will provide information on the prevalence of voids in the rock mass, which will allow URS to develop recommendations for rock anchor construction.

5.2.3.1 Rock Anchor Capacity

The load bearing capacity of each anchor depends on the "fixed anchor length" (L), the spacing of the anchors, and engineering properties of the grout and rock. For this project, we anticipate that rock anchors will be generally be used singly, or in single rows, where anchor spacing is greater than 0.25L. If anchor spacing is greater than 0.25L or consisting of multiple rows, we should be contacted to evaluate the effects of a single anchor capacity. Preliminary anchor design charts have been prepared (See Figures 5 and 6) to assist in the selection of anchor length to achieve a designed anchor capacity. The charts have been prepared based upon an analysis of the possible failure mode of the anchors, including the following:

- Bond failure between bar and grout;
- Bond failure between grout and rock;
- Pullout of inverted cone of rock surrounding the anchored tendon;
- Tensile yield of the tendon.

Based upon the properties of the rock and grout, and if the anchors meet the spacing and the distance requirements specified above, failure at the grout/rock interface will control the allowable anchor capacity up to 60% of the bar yield strength. Anchors used to resist lateral movements should not be placed at an angle shallower than 45 degrees to the surface.

Figure 5 presents anchor design of polyester resin grout with unconfined compressive strength of approximately 13,000 psi. Figure 6 presents alternative anchor design curve if 15,000-psi compressive strength concrete grout is used instead of the resin grout. The concrete grout may be placed by gravity flow prior to inserting the steel bar. Each bar should be fitted with centralizers that center the bar in the hole.

Holes may be drilled using an air track or similar rotary percussion device. Hole diameters should be at least ½-inches greater than the bar thread diameter if concrete grout is used. All holes must be thoroughly cleaned of debris prior to placement of the anchor.

In order to attain the higher anchor capacities required for the project, we recommend he use of steel alloy anchor bars with an ultimate tensile strength of 150 kips per square inch (ksi) conforming to ASTM standard A-722. Threaded and deformed bars of this type are available from DYWIDAG Systems International, Inc. The anchor should be nominally prestressed to approximately 10 percent of the design load to take up slack in the system and provide an appropriately stiff anchorage. A portion of this prestress will bleed off due to creep and to downward deformation of the foundation when the dead and live loads are applied. A sufficient residual prestress is expected to remain to ensure that excessive anchor system deformation during uplift does not occur before full uplift resistance is mobilized. Direct-pull hydraulic jacks or a torque wrench may be used to apply prestress. Following prestressing, the free anchor length may be grouted with low-strength concrete to protect against corrosion. Overall, based on the resistivity determinations, the site is not corrosive.

5.2.3.2 Rock Anchor Testing

It is important that each anchor performs satisfactorily, therefore, we recommend that each anchor be tested using either performance testing or proof testing criteria. At a minimum, the ten percent of the at each site should be performance tested. Following satisfactory completion of performance testing, we recommend that each remaining rock anchor be proof tested. Proof testing evaluates the as-built anchor capacity. Performances and proof testing should be completed by incrementally loading the anchors in accordance with the schedule below. At each increment, the movement of the anchor should be recorded to the nearest of 0.001 inch with reference to an independently fixed anchor point. The jack load should be monitored with a properly calibrated pressure gauge or load cell. The test load sequences presented in Table 5-1 should be employed.

Each load should be held at each increment just long enough to obtain the movement reading, but not more than one minute. The testing should not exceed the 80% of the bar yield strength. The 1.33P test load should be held for 10 minutes. Total movements should be recorded at 1, 2, 3, 4, 5, 6, and 10 minutes.

Performance and proof test acceptance criteria should be developed in conjunction with URS when details of the anchor design and materials are known. A URS representative should monitor all rock anchorage installation and testing to determine whether the rock layer has been sufficiently penetrated and to monitor the proof and performance tests.

Table 5-1: Rock Anchor Test Recommendations

Terminology
P= Design Load
AL= Alignment Load (2-10 percent of Design Load)
Performance Test Load Sequence*
AL
.25P, AL
.25P, .50P, AL
.25P, .50P, .75P, AL
.25P, .50P, .75P, 1.00P, AL
.25P, .50P, 1.00P, 1.20P, AL
.25P, .50P, 1.00P, 1.20P, 1.33P (Test Load), Adjust to Lock-off Load
Proof Test Load Sequence*
AL, .25P, .50P, .75P, 1.00P, 1.20P, 1.33P (Test Load), Adjust to Lock-off Load
*1986, Post-Tensioning Institute, "Recommendations for Prestressed Rock and Soil Anchors," Phoenix, Arizona.

5.3 RETAINING WALLS

Following are typical design parameters for wall types that we believe represent the range of systems that may be constructed at this site. Walls may be required to provide a level working pad at the tower pads or for the embankment modifications. Please contact us if any additional design values or wall types need to be addressed.

5.3.1 Retaining Wall Design Parameters

Lateral soil pressures on a retaining wall depend on several factors including retained soil type, wall fixity, drainage provisions and the influence of surface loads imposed behind the wall. We have provided typical design parameters for wall types that we believe represent the range of retaining wall systems that are likely to be constructed at this site. Our recommendations are based on the following assumptions:

- Retaining walls will be designed to restrain both existing soils and constructed fills.

- Retaining walls will be backfilled with free draining crushed rock, in accordance with Section 5.3.3 (Retaining Wall backfill) of this report.
- Adequate subsurface drainage will be provided.

5.3.2 Equivalent Fluid Densities (Soil)

Unrestrained walls have no fixity at the top and are free to rotate about their base through tilting or translation. Most cantilever retaining walls fall into this category (unless they are attached to buildings or other structures). A lateral movement of 0.005 times the height of the retaining wall may be required to achieve this active pressure. For these walls, we recommend that a lateral equivalent fluid density of 40 pcf be used for design. If the retaining walls are used to restrain sloping backfill, URS should be contacted for additional designs.

Restrained walls are rigid structures where essentially no relative movement occurs between the structure and the soil. Most basement walls and other rigid walls that are restrained by buildings, parking decks, floor slabs or other perpendicular walls fall into the category of restrained walls. For restrained walls, we recommend that a lateral equivalent fluid density of 60 pcf be used for design. If the retaining walls are used to restrain sloping backfill, URS should be contacted for additional designs.

5.3.3 Retaining Wall Backfill

Backfill within 3 feet of retaining walls should consist of free draining crushed rock, free of organics and debris. This material should meet the requirements of the 2006 WSDOT Standard Specifications for Road, Bridge and Municipal Construction, Section 9-03.14(1). Backfill beyond 3 feet from the wall should meet requirements described in Section 5.6.5 (Structural Fill Material). We recommend that all fill be compacted to 95% of the maximum dry density as determined by the Modified Proctor test (ASTM 1557). Additionally, we recommend that any backfill that is placed within 5 feet of the wall (measured horizontally) be compacted with lightweight, hand operated compaction equipment. Over-compaction of this fill can increase wall pressures.

We recommend the placement of a 4-inch diameter slotted PVC pipe wrapped in non-woven geotextile fabric at the base of the wall backfill to facilitate drainage of this area depending on the final elevation of the basement slab. These pipes should be drained to a collection point and sumped.

5.4 TOWER FOUNDATION EXCAVATION

5.4.1 Temporary Shoring

It is the responsibility of the contractor to deal with the temporary construction excavation and site safety including overseeing the means, methods, and sequencing of construction operations. URS does not assume any responsibility for the contractor's activities or construction site safety for the information provided in this section. In this site the rock layer is relatively at shallow depth (3 feet to 10 feet bgs) and there is sufficient space left for providing the adequate slope as per OSHA requirement for the basement excavations, and hence, the temporary shoring is not recommended. URS does anticipate, however, that the rock will be able to be constructed with a near vertical face, possibly using shallow anchor bolts and gunite to seal its surface. This will be determined with the final construction.

5.4.2 Dewatering

During the excavation of test pits at the site, no ground water was encountered up to a depth of 16 feet bgs. Hence it is anticipated that seepage of ground water will not be a problem within foundation excavation at the site. Presence of groundwater will be determined during subsequent geotechnical investigations to be performed in support of final foundation design.

5.4.3 Rock Excavation

It should be anticipated that hard rock will be encountered during excavation work. Machinery capable of removing these this large intact rock, such as heavy duty backhoes with rock ripping teeth, hydraulic thumbs or pneumatic rock breaking equipment, should be anticipated for this work. There is also a possibility that the rock will have to be blasted, pre-split or utilize expansive materials prior to excavation, depending on the final depth of excavation. The actual methods will be developed with final design.

5.5 SHALLOW FOUNDATIONS (ANCILLIARY STRUCTURES)

We understand that new footings may be utilized to support ancillary structures such as the transformer pads. For footings that bear on shallow, undisturbed native soils, we recommend a net allowable bearing pressure of 2,500 pounds per square foot (psf). This bearing capacity is based on a settlement limit of 0.5 inches.

Allowable bearing pressures may be increased by one-third when considering load cases that include transient loads such as wind and seismic forces. We recommend that a unit weight of 115 pcf be used to calculate the reduction of overburden pressure due to excavation. Backfill soils will be slightly heavier than excavated soils but not enough to significantly influence the bearing pressure.

Exterior footings could be turned down footings from the slab and should be founded at least 18 inches beneath the lowest exterior grade to provide frost protection. Continuous wall footings should have a minimum width of 18 inches and isolated column footing should have a minimum plan dimension of 24 inches.

For foundations designed and constructed as specified in this report, we estimate settlements on the order of 0.5-inches. We anticipate the majority of the settlement will occur during construction, essentially as the loads are applied. The remainder of the settlement will likely occur within three weeks following the application of the load.

5.5.1 Passive Loads and Friction Factor

Passive earth pressures acting on the sides of shallow foundations and friction on the base of the foundations may be considered to provide resistance to lateral forces tending to cause translational sliding. These structural members should be considered for counteracting lateral forces only if the member is placed in direct contact with tested and approved soils. If the foundation is constructed by using forms, lean concrete may be placed between the footing and the undisturbed soil of the adjacent excavation in order to provide the direct contact required to consider passive pressure for counteracting lateral movement. The lean concrete should have a minimum 28-day compressive strength of at least 1,500 psi. An allowable passive pressure having an equivalent fluid pressure of 250 pcf may be used for design. This is based on a factor of safety of two.

An ultimate friction factor of 0.3 for mass concrete on compacted tested and approved native subgrade can be used for design for those portions of the foundations with full positive pressure on the base of the foundation. Only long-term dead loads should be considered in calculating the available friction on the foundation base.

5.5.2 Slabs on Grade

The subgrade under all floor slab areas should be prepared in accordance with Section 5.5.1 We recommend that floor slabs be underlain by a granular base course at least 6-inch thick to provide uniformity of support and to act as a capillary break against moisture migration through the slab. The granular base course should consist of well-graded gravel or crushed rock with a maximum nominal size of ¾ inch and having less than 5 percent by weight passing the No. 200 sieve. The base course should be compacted to at least 95 percent of its maximum dry density as measured by the modified Proctor test (ASTM Standard D 1557). We recommend a modulus of subgrade reaction of 200 pounds per cubic inch (pci) for the base course.

Even with a capillary break as outlined above, there is the possibility of some floor moisture or dampness. If floor moisture is a critical consideration due to storage of materials directly on the floor slab, or because of the use of glued-down impervious floor coverings such as tile or linoleum, we recommend the use of an under-slab impermeable membrane placed directly below the slab. To maximize water tightness, the membrane must be installed in accordance with the manufacturer's recommendations.

5.6 CONSTRUCTION CONSIDERATIONS

5.6.1 Site Work Preparation

Prior to construction of any new foundations, all areas that will receive fill, base rock, or structures should be stripped of all surface vegetation, organic topsoil, and any deleterious materials that might be encountered. Any soft or unsuitable soils encountered during stripping or excavation should be removed and replaced with structural fill meeting the requirements described in Section 5.6.4 (Wet Weather Earthwork). All subgrades should be approved by the Engineer prior to the placement of any materials or foundation elements.

5.6.2 Shallow Foundation Excavation

We recommend that excavations for foundations in soil and weathered rock be accomplished with a straight-edged grading bucket to minimize disturbance of the bearing surfaces. Following excavation, the bearing surfaces should be thoroughly cleaned of loosened or disturbed soil, by hand if necessary. Any soft or unsuitable soils encountered at the base of foundation excavations should be removed and replaced with compacted structural fill meeting the requirements described in Section 5.6.4 (Wet Weather Earthwork).

5.6.3 Dry Weather Earthwork

After areas are stripped or excavated to design elevations, we recommend scarification of the resulting subgrade in all areas that will receive fill or structures to a depth of 8 inches. The scarified soil should be compacted to at least 95% of its maximum dry density as determined by the standard proctor test, ASTM D698.

5.6.4 Wet Weather Earthwork

We anticipate that the native soils found at the site will be sensitive to moisture and erosion. Therefore, during or after wet weather, it may be necessary to import granular materials for structural fill to protect open subgrade materials. It may also be necessary to install a granular working pad to support construction equipment. Delays in site earthwork activities should be anticipated during periods of heavy rainfall. Additionally, site clearing and stripping activities may expose subgrade material that may be damaged if subjected to disturbance from construction traffic. During wet weather, we recommend that site stripping and excavation be performed using an excavator with a straight-edged bucket that does not traverse the final subgrade.

When a granular working base is used to protect open subgrade material and construction equipment, the base should consist of a suitable thickness of crushed rock or ballast placed by end-dumping off an advancing pad of rock fill. Because construction practices can greatly affect the amount of rock required, we recommend that if conditions require the installation of a granular working blanket, the design, installation, and maintenance be made the responsibility of the contractor. After installation, the working blanket should be compacted with a minimum of 4 passes with a smooth-drum roller.

We recommend that the contractor minimize soil exposure during the rainy season by proper timing of grading and construction activities and be prepared to shut down all earthwork if heavy precipitation occurs. We recommend that water runoff be diverted from foundations and equipment pads, and that all runoff water be directed to proper drainage areas and not be allowed to pond.

5.6.5 Structural Fill Material

We recommend that all fills intended to support structures be placed in horizontal lifts not exceeding about 8 inches in loose thickness and be compacted to at least 95 percent of the maximum dry density as determined by the Modified Proctor method (ASTM D 1557), unless where specified above.

Imported structural fill should be clean, well-graded granular material, free of organics and debris and meeting the requirements of the 2006 WSDOT Standard Specifications for Road, Bridge and Municipal Construction, Section 9-03.14(1). The procedure to achieve proper density of a compacted fill depends on the size and type of compacting equipment, the number of passes, thickness of the layer being compacted, and certain soil properties. When the size of the excavation restricts the use of heavy equipment, smaller equipment can be used, and the soil must be placed in lifts thin enough to achieve the required compaction. We recommend that methods of compaction be left to the discretion of the contractor, with compaction testing provided by URS.

We do not recommend the use of on-site soils for structural fill. This material may be used for miscellaneous fill and landscaping applications provided these areas are not intended to support structures. On-site soils should be compacted to at least 95% of the maximum dry density as determined by the standard proctor test, ASTM D698.

5.6.6 Embankment Design

Once the required roadway improvements on the SDS property are determined for the construction traffic equipment, URS could develop a formal embankment design that will have a static Factor of Safety (FOS) of 1.3. URS anticipates the majority of embankments will be relatively small and constructed over level ground. URS envisions that the general embankment section will consist of two zones and have finished slopes of 2H:1V. The final designs for the embankments will come with the final geotechnical engineering report.

5.6.7 Temporary Slopes

Depending on the Contractor's proposed excavation and shoring plan, temporary cut slopes or shoring may be required during construction. If open cuts are utilized, maximum slope inclinations must be made in accordance with regulations established by OSHA. In accordance with OSHA, the sands and silty sand overburden soils encountered on site are classified as Type C. The maximum allowable temporary slope for Type C soils is 1½ horizontal to 1 vertical (1½H: 1V) if fully dewatered. Flatter slopes will be required if dewatering provisions are not considered for the site. The slopes should be inspected and maintained as required by OSHA. For the excavations into the underlying bedrock, URS will provide site specific designs from the borings to be advanced for the final report.

6 CONSTRUCTION QUALITY ASSURANCE

We recommend that URS be retained to provide construction monitoring and testing services during foundation construction. The purpose of our field monitoring services is to confirm that site conditions are as anticipated, to provide field recommendations as required based on conditions encountered, and to document the activities of the contractor to assess compliance with the project recommendations provided by URS. We also recommend that URS review and comment on the foundation plans. The purpose of this review would be to identify any potential problem areas and to provide cost saving or efficiency improving suggestions, if possible.

7 LIMITATIONS

This report presents recommendations pertaining to the proposed structures as represented to URS, as described herein. The findings and recommendations presented in this report are based upon soil conditions observed the available subsurface explorations, interpolation of the soil conditions between test pits, and extrapolation of these conditions throughout the proposed site area. They are further based on the assumption that the subsurface conditions do not deviate appreciably from those reported and those assumed. However, the possibility of different conditions cannot be discounted.

In the event that changes in design loads or structural characteristics described in this report are made, URS should be retained to review our design recommendations and their applicability to the revised design plans. In this way, any required supplemental recommendations can be made in a timely manner.

This report has been prepared for the specific project, purpose, and client stated in the report; the report may not be adequate for other uses. The use of the recommendations of this report for other projects or purposes or by other parties is not authorized.

Although URS has endeavored to characterize the surface and subsurface conditions at the site, URS is not as able to assess potential construction difficulties as is a contractor specializing in the work to be performed. Consequently, the Contractor is responsible, and URS is not, for final evaluation of potential construction difficulties.

This report has been prepared in accordance with the care and skill generally exercised at the present time by reputable professionals in the field of geotechnical engineering, under similar circumstances, for projects in the project locality. No other warranty, either expressed or implied, is made as to the professional advice presented herein.

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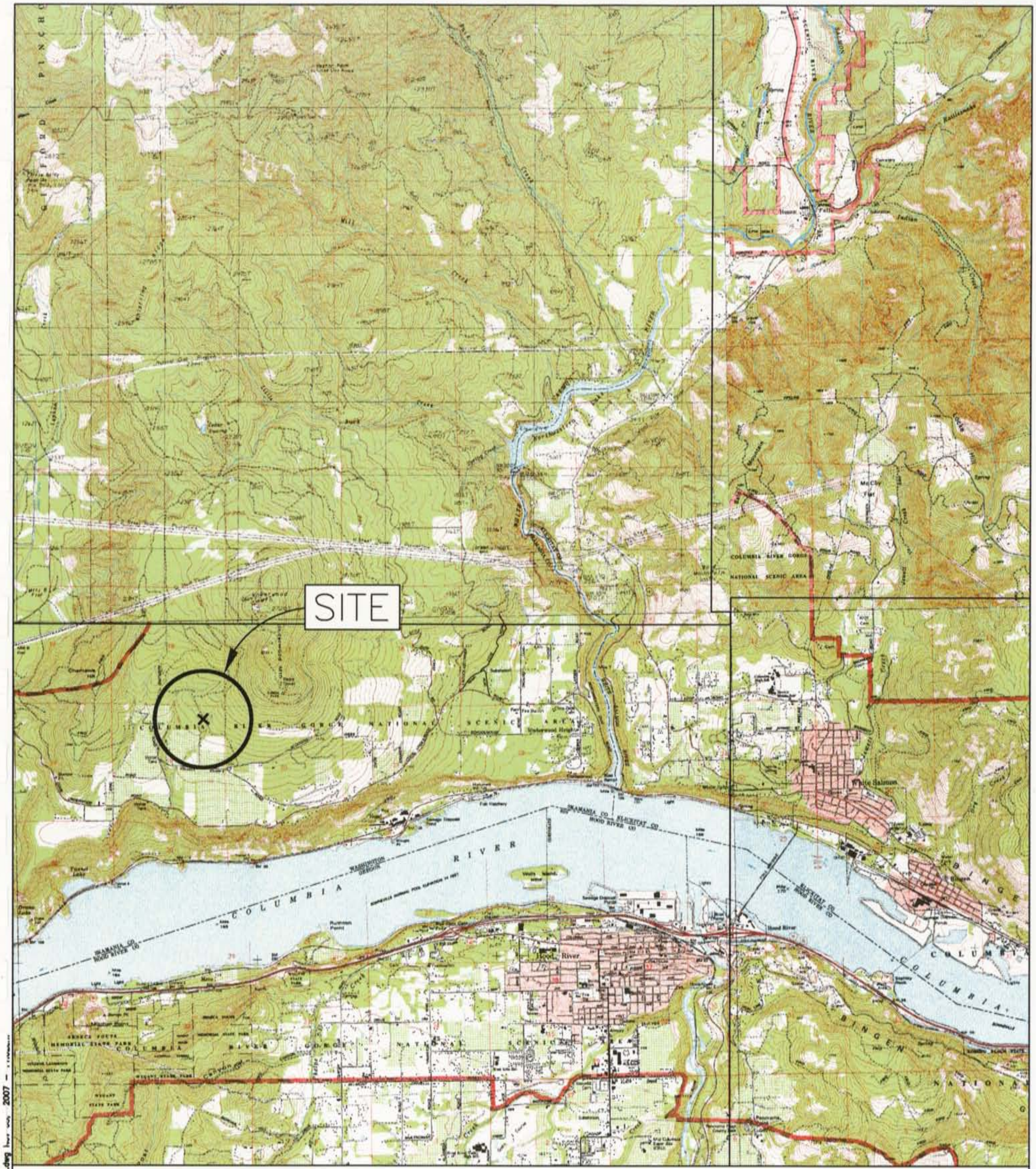
FIGURES

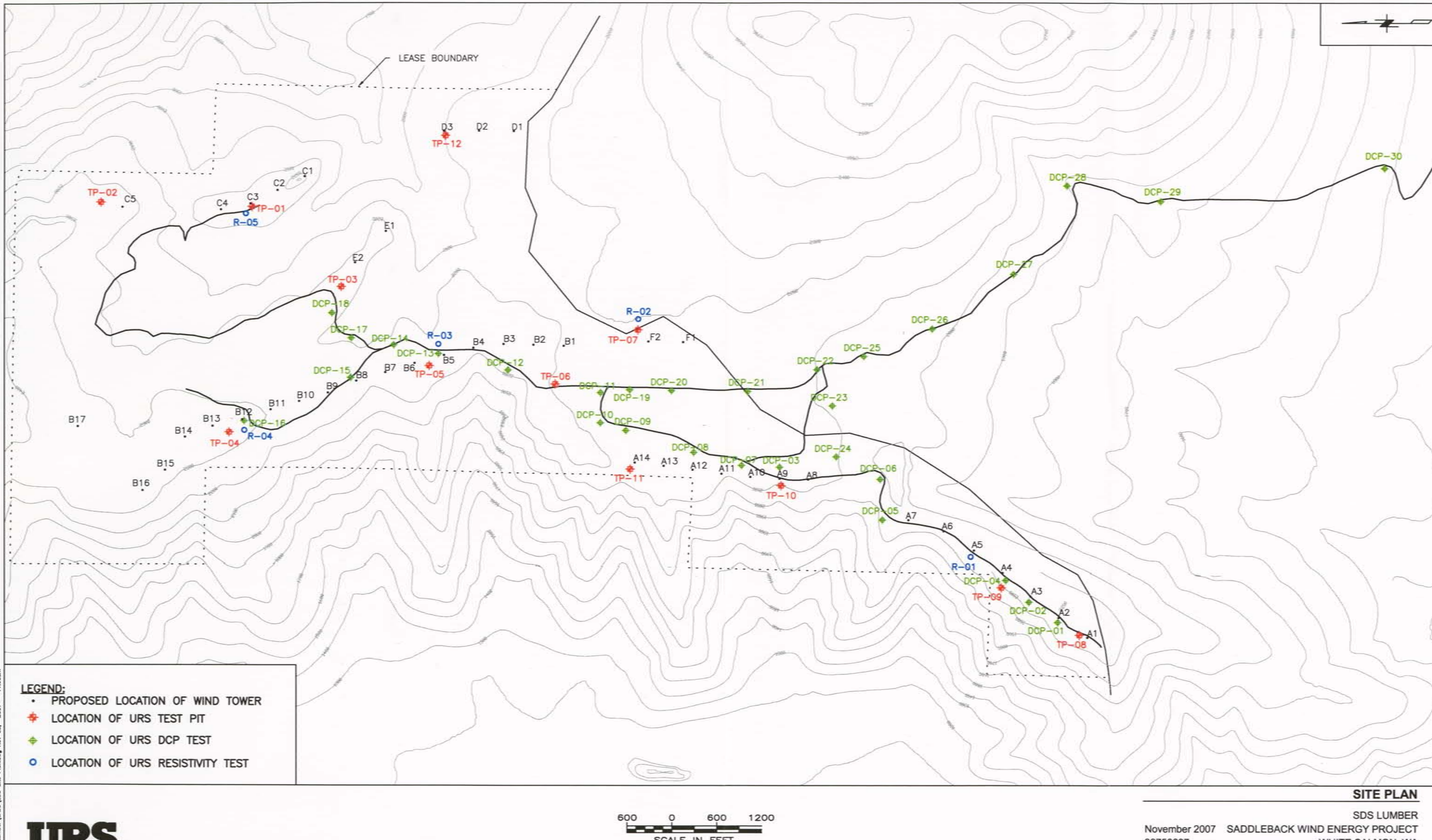


VICINITY MAP

SDS LUMBER
November 2007 SADDLEBACK WIND ENERGY PROJECT
33758687 WHITE SALMON, WA

URS

Figure 1



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SITE PLAN

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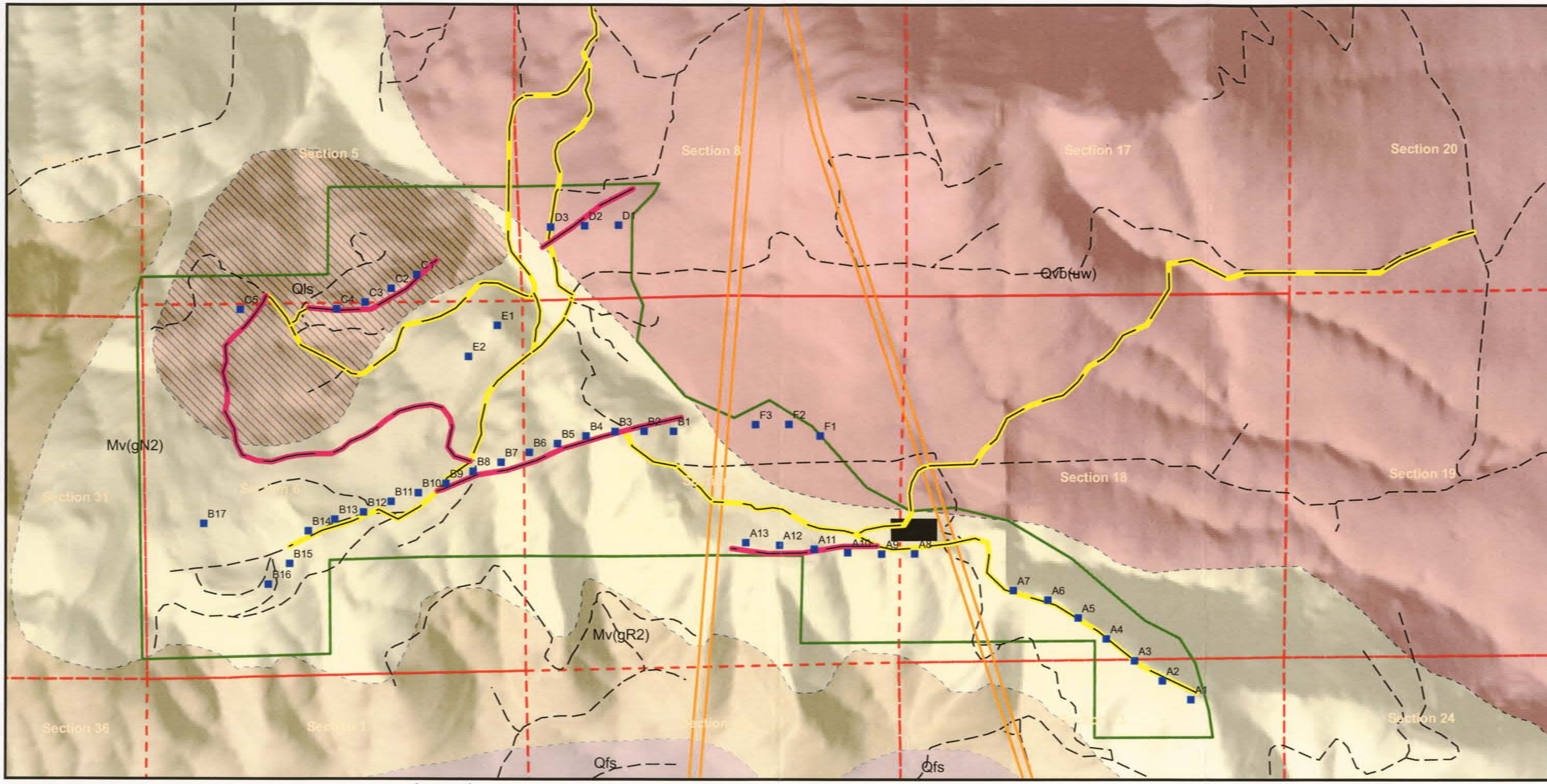
November 2007 SADDLEBACK WIND ENERGY PROJECT

33758687 WHITE SALMON, WA

Figure 2



Legend

- Proposed Turbine Locations
- Proposed Access Road
- Proposed Improved Road
- Existing Transmission Lines
- Local Road
- LeaseBoundary
- Proposed Substation
- Section Line

Geologic Rock Units (Legend)

- Mv(gN2)--Basalt Flows (Grande Ronde Basalt, upper flows of norm.mag.pol.)
- Mv(gR2)--Basalt Flows (Grande Ronde Basalt, upper flows of rev.mag.pol.)
- Qfs--Outburst Flood Deposits, Sand and Silt, Late Wisconsin
- Qls--Mass-Wasting Deposits, Mostly Landslides
- Qvb(uw)--Basalt Flows

E N S W

0 5001,000 2,000 3,000 4,000 Feet

Figure 3, Site Geology
Saddleback Wind Project

URS SDS Lumber

GeoDataScape

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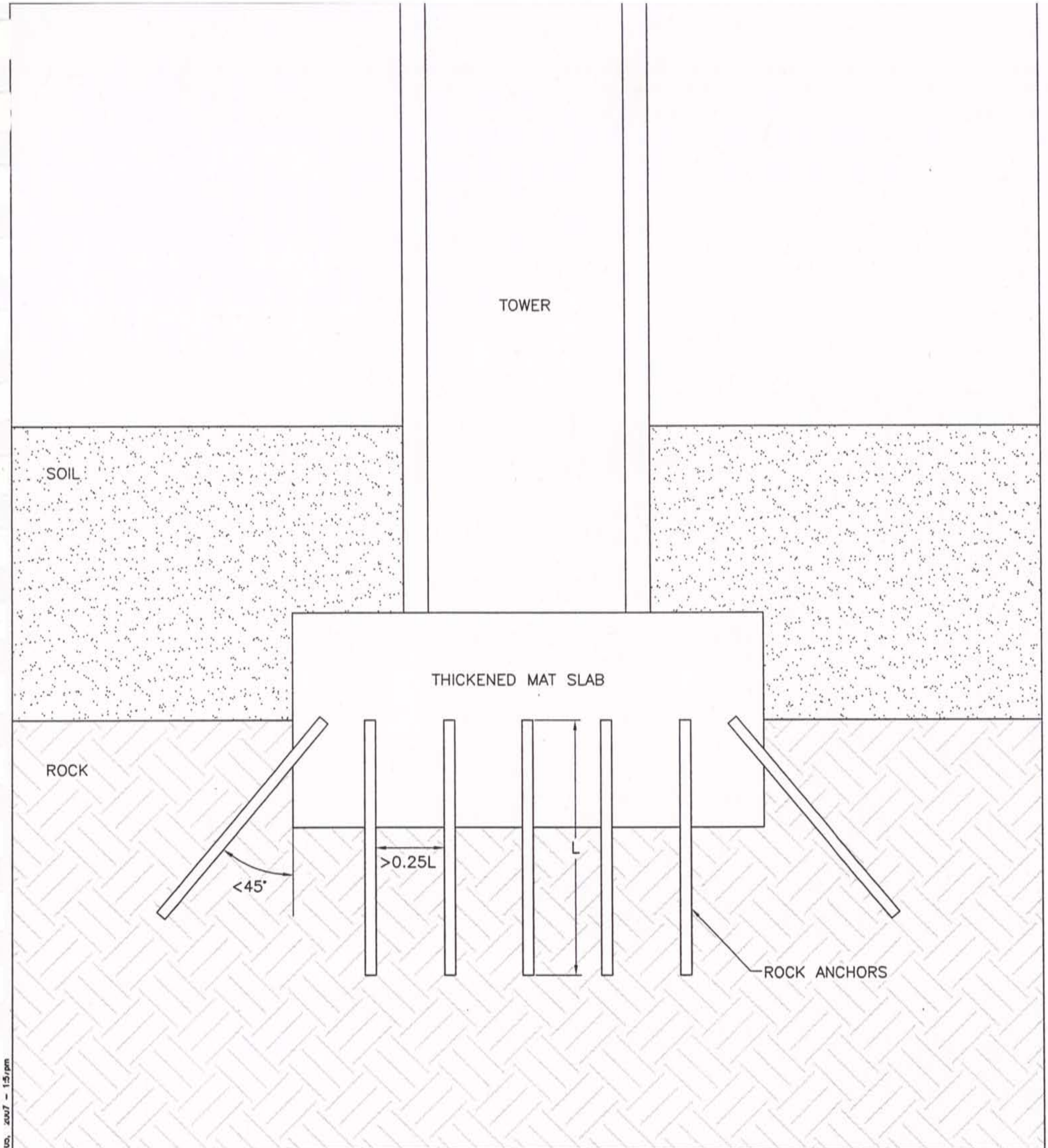

MAT-SLAB FOUNDATION CONCEPT

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WHITE SALMON, OREGON

FIGURE 4

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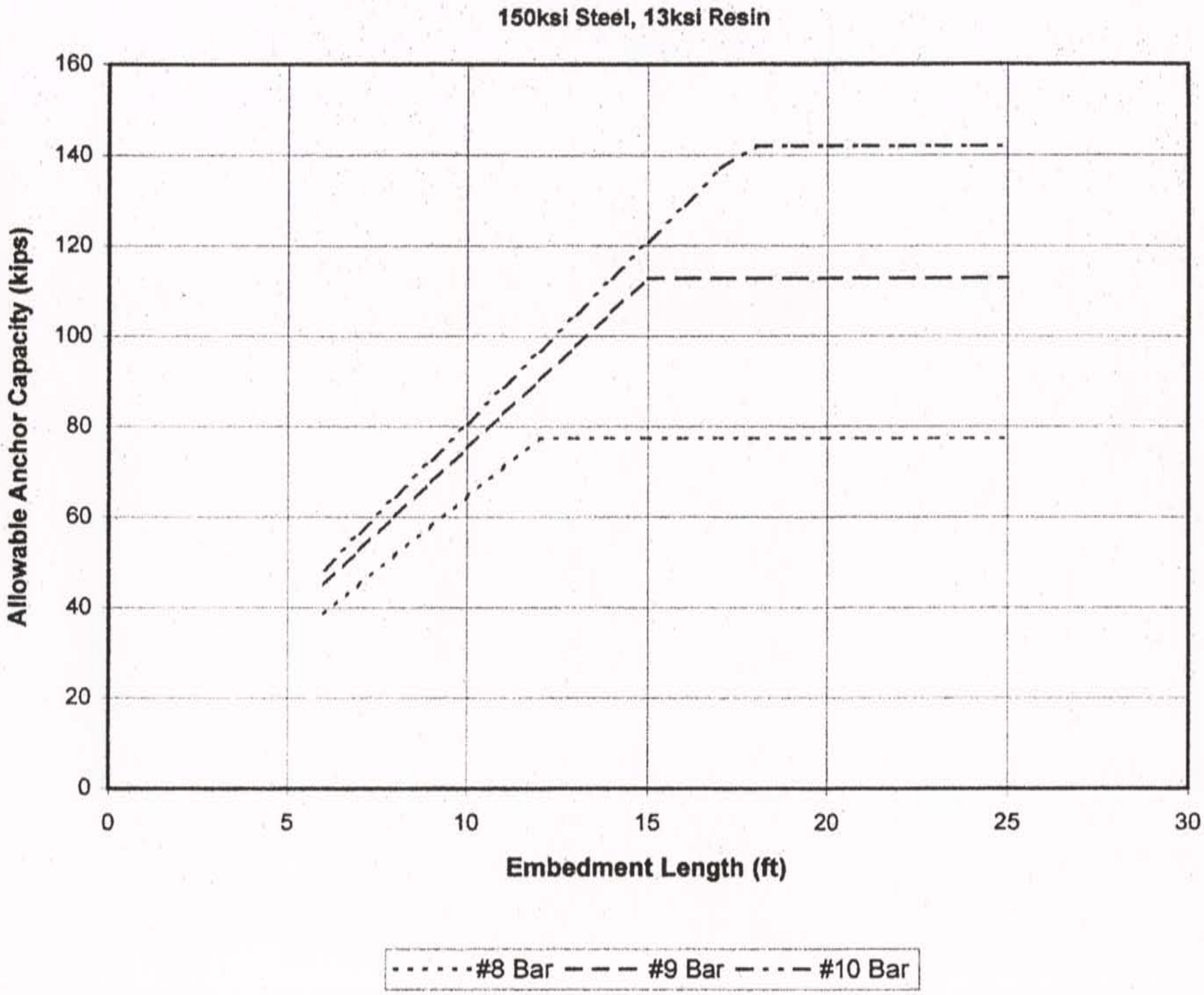

PRELIMINARY ROCK ANCHOR CAPACITY - RESIN

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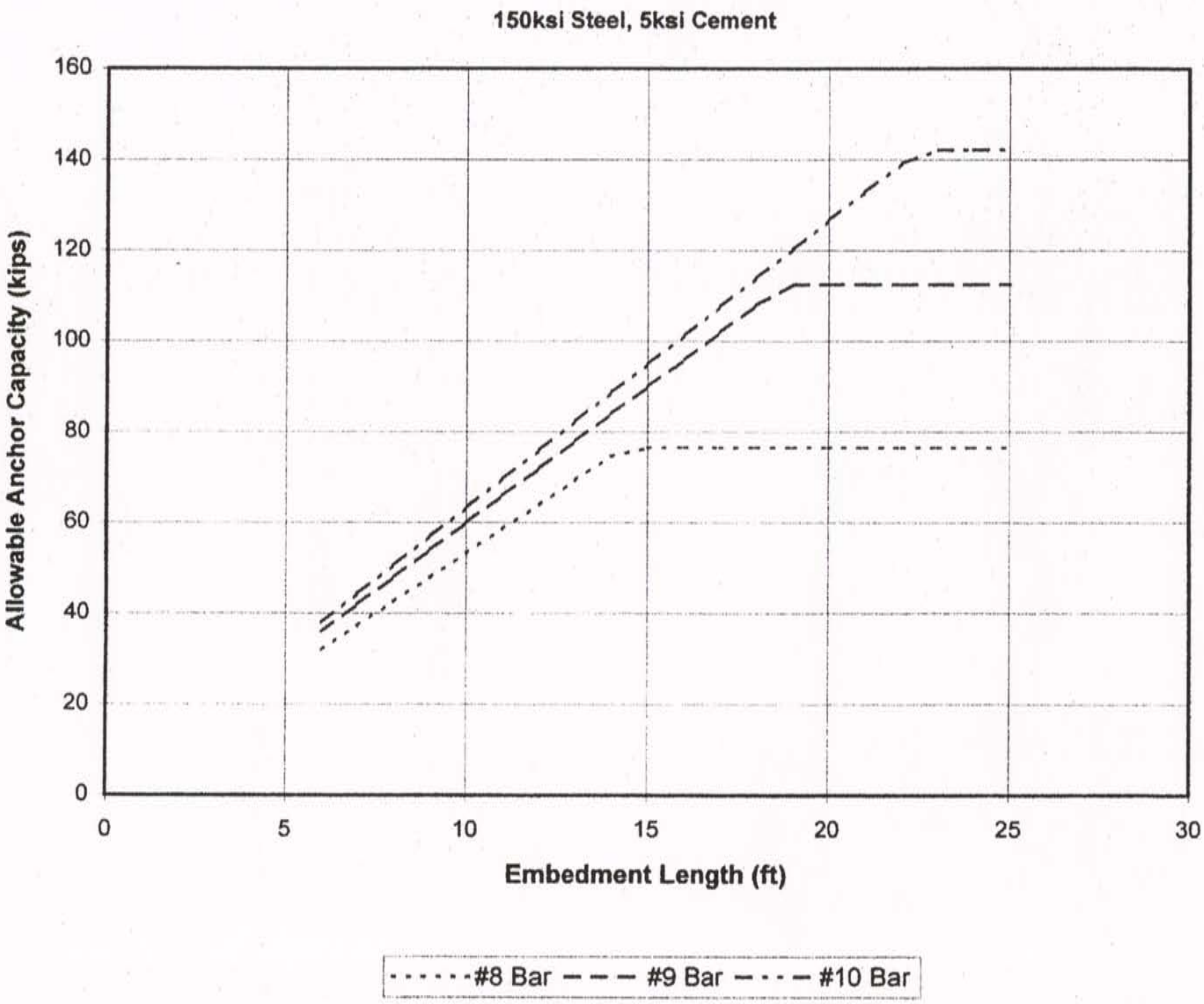
SADDLEBACK WIND ENERGY PROJECT
WHITE SALMON, WA

FIGURE 5



PRELIMINARY ROCK ANCHOR CAPACITY - CEMENT

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SDS LUMBER
SADDLEBACK WIND ENERGY PROJECT
WHITE SALMON, WA

FIGURE 6

APPENDIX A
LOGS OF TEST PITS

Project: Saddleback Wind Energy Project **Project Location:** White Salmon, Washington **Project Number:** 33758687	# Log of Test Pit TP-01

Date(s) Excavated	9/18/2007	Logged By	EAM	Checked By	DBM
Length of Excavation	~10 feet	Width of Excavation	~5 feet	Depth of Excavation	13.0 feet
Excavation Equipment	Track Mounted Excavator	Excavation Contractor	SDS Lumber	Approximate Surface Elevation	2134 feet MSL
Water Observations	N/A			Weather	Sunny, 60's
Location	See site map			Surface Condition	

Elevation feet	Depth, feet	Sample Type	Sample Number	Graphic Log	Lithologic Log (USCS Code)	MATERIAL DESCRIPTION	Fines (%)	Moisture (%)	Pocket Penetrometer (tsf)	REMARKS AND OTHER TESTS
2134	0				ML	**SANDY SILT [ML]**, brown, moist, soft, contains roots and rock [Topsoil/Disturbed Material]				
	1				CH	**SANDY FAT CLAY [CH]**, orange-brown, moist, medium stiff, mottled with yellow-brown, rock fragments, blocky, occasional roots. [Residuum] *Becomes gray-brown*				
2132	2		1				57.2	52.1	3.5	
					SC	**CLAYEY SAND [SC]**, yellow-brown, possible decomposed bedrock				
	3					**BEDROCK**, orange-brown, gray-black mottled, highly to moderately weathered, very soft to soft, fine grained, slightly vesicular, black mineral coatings in vesicles, spheroidal weathering observed, grades to moderately weathered with depth			2.5	
2130	4									
	5									
2128	6									
	7									
2126	8		2							
	9									
2124	10									Difficult excavating
	11									
2122	12									
	13					End test pit at 13.0 feet bgs on 9/18/2007 due to refusal. Backfilled with excavated soils upon completion.				
2120	14									
	15									
2118	16									
	17									
2116	18									
	19									
2114	20									

URS

Project: Saddleback Wind Energy Project **Project Location:** White Salmon, Washington **Project Number:** 33758687	**Log of Test Pit TP-02**

Date(s) Excavated	9/18/2007	Logged By	EAM	Checked By	DBM
Length of Excavation	~10 feet	Width of Excavation	~5 feet	Depth of Excavation	14.0 feet
Excavation Equipment	Track Mounted Excavator	Excavation Contractor	SDS Lumber	Approximate Surface Elevation	2208 feet MSL
Water Observations	N/A			Weather	Sunny, 60's
Location	See site map			Surface Condition	

Elevation feet	Depth, feet	Sample Type	Sample Number	Graphic Log	Lithologic Log (USCS Code)	MATERIAL DESCRIPTION	Fines (%)	Moisture (%)	Pocket Penetrometer (tsf)	REMARKS AND OTHER TESTS
2208	0				ML	**SANDY SILT [ML]**, brown, moist, soft, contains roots [Topsoil]				
	1					**BEDROCK**, light orange-brown, tuffaceous, highly weathered, black staining on fracture surfaces, very soft, large boulders of fresh rock, boulders are very soft to soft, subrounded				
2206	2									
	3		1							
2204	4									
	5					*Becomes dark orange-brown*				
2202	6									
	7					*Becomes light gray mottled with orange and black, highly to moderately weathered, highly fractured*				
2200	8									
	9									
2198	10									
	11									
2196	12									
	13									
2194	14					End test pit at 14.0 feet bgs on 9/18/2007 due to extent of excavator. Backfilled with excavated soils upon completion.				
	15									
2192	16									
	17									
2190	18									
	19									
2188	20									

Project: Saddleback Wind Energy Project **Project Location:** White Salmon, Washington **Project Number:** 33758687	# Log of Test Pit TP-03

Date(s) Excavated	9/18/2007	Logged By	EAM	Checked By	DBM
Length of Excavation	~10 feet	Width of Excavation	~5 feet	Depth of Excavation	12.0 feet
Excavation Equipment	Track Mounted Excavator	Excavation Contractor	SDS Lumber	Approximate Surface Elevation	2173 feet MSL
Water Observations	N/A			Weather	Sunny, 60's
Location	See site map			Surface Condition	

Elevation feet	Depth, feet	Sample Type / Sample Number	Lithologic Log (USCS Code)	MATERIAL DESCRIPTION	Fines (%)	Moisture (%)	Pocket Penetrometer (tsf)	REMARKS AND OTHER TESTS
	0		ML	**SANDY SILT [ML]**, brown, moist, soft, occasional rock fragments [Topsoil]				
2172	1		SM	**SILTY SAND [SM]**, yellow-brown, moist, medium dense. [Colluvium]				
	2						4.5	
2170	3		SM	**SILTY SAND [SM]**, white and light orange mottled, dense [Decomposed Bedrock]			4.5+	
	4							
2168	5							
	6							
2166	7							
	8	1		*Contains breccia, angular, medium gray pieces of tuff in red-orange silty clay matrix*				
2164	9							
	10							
2162	11			Some structure and bedding observed				
	12			End test pit at 12.0 feet bgs on 9/18/2007. Backfilled with excavated soils upon completion.				
2160	13							
	14							
2158	15							
	16							
2156	17							
	18							
2154	19							
	20							

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Project: Saddleback Wind Energy Project **Project Location:** White Salmon, Washington **Project Number:** 33758687	**Log of Test Pit TP-04**

Date(s) Excavated	9/18/2007	Logged By	EAM	Checked By	DBM
Length of Excavation	~10 feet	Width of Excavation	~5 feet	Depth of Excavation	14.0 feet
Excavation Equipment	Track Mounted Excavator	Excavation Contractor	SDS Lumber	Approximate Surface Elevation	2302 feet MSL
Water Observations	N/A			Weather	Sunny, 60's
Location	See site map			Surface Condition	

Elevation feet	Depth, feet	Sample Type / Sample Number	Graphic Log	Lithologic Log (USCS Code)	MATERIAL DESCRIPTION	Fines (%)	Moisture (%)	Pocket Penetrometer (tsf)	REMARKS AND OTHER TESTS
2302	0			ML	**SANDY SILT [ML]**, brown, moist, soft [Topsoil]				
	1				**BEDROCK**, yellow-brown to gray mottled, tuffaceous, decomposed to highly weathered, pummice, rock fragments in dark orange soil, very small vesicles				
2300	2								
	3	1							
2298	4								
	5				*Becomes gray or salt and peppered, black staining on fracture surfaces, highly weathered, very soft*				
2296	6								
	7								
2294	8								
	9				*Becomes gray with orange-white specks, highly to moderately weathered, soft, spheroidal weathering patterns observed, large rounded boulders observed (~2.5 feet diameter)*				
2292	10								
	11								
2290	12								
	13								
2288	14				End test pit at 14.0 feet bgs on 9/18/2007. Backfilled with excavated soils upon completion.				
	15								
2286	16								
	17								
2284	18								
	19								
2282	20								

Project: Saddleback Wind Energy Project	Log of Test Pit TP-05
Project Location: White Salmon, Washington	
Project Number: 33758687	

Date(s) Excavated	9/19/2007	Logged By	EAM	Checked By	DBM
Length of Excavation	~10 feet	Width of Excavation	~5 feet	Depth of Excavation	14.0 feet
Excavation Equipment	Track Mounted Excavator	Excavation Contractor	SDS Lumber	Approximate Surface Elevation	2205 feet MSL
Water Observations	N/A			Weather	Sunny, 60's
Location	See site map			Surface Condition	

Elevation feet	Depth, feet	Sample Type	Sample Number	Graphic Log	Lithologic Log (USCS Code)	MATERIAL DESCRIPTION	Fines (%)	Moisture (%)	Pocket Penetrometer (tsf)	REMARKS AND OTHER TESTS
	0				ML	**SANDY SILT [ML]**, brown, moist, soft [Topsoil]				
2204	1		1		SM	**SILTY SAND [SM]**, moist, brown, dense. [Colluvium]	48.6	16.4	4.5+	
	2									
2202	3					**BEDROCK**, gray with orange specks, tuffaceous, highly weathered to decomposed, very soft, spheroidal weathering patterns observed				
	4									
2200	5									
	6									
2198	7									
	8									
2196	9									
	10					*Becomes highly to moderately weathered, black mineral deposits on fractures*				
2194	11									
	12									
2192	13		2							
	14					End test pit at 14.0 feet bgs on 9/18/2007. Backfilled with excavated soils upon completion.				
2190	15									
	16									
2188	17									
	18									
2186	19									
	20									

Report: PORT_TP_MC-PP_200_USCS; File: WINDFARM.GPJ; 10/22/2007 TP-05

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Project: Saddleback Wind Energy Project **Project Location:** White Salmon, Washington **Project Number:** 33758687	**Log of Test Pit TP-06**

Date(s) Excavated	9/19/2007	Logged By	EAM	Checked By	DBM
Length of Excavation	~10 feet	Width of Excavation	~5 feet	Depth of Excavation	14.0 feet
Excavation Equipment	Track Mounted Excavator	Excavation Contractor	SDS Lumber	Approximate Surface Elevation	2142 feet MSL
Water Observations	N/A			Weather	Sunny, 60's
Location	See site map			Surface Condition	

Elevation feet	Depth, feet	Sample Type / Sample Number	Graphic Log	Lithologic Log (USCS Code)	MATERIAL DESCRIPTION	Fines (%)	Moisture (%)	Pocket Penetrometer (tsf)	REMARKS AND OTHER TESTS
2142	0			ML	**SANDY SILT [ML]**, brown, moist, soft [Topsoil]				
	1			ML	**SANDY SILT [ML]**, brown, moist, medium dense, occasional rock fragments [Colluvium]				
								3.0	
2140	2								
								4.5	
	3	1				60.6	25.7		
2138	4								
	5								
2136	6								
	7								
2134	8								
	9								
2132	10								
	11	2							
					BEDROCK, brown, black mottled, highly weathered to decomposed, very to extremely soft, no fractures or structure observed (Decomposed Volcanics/Volcaniclastics)				
2130	12								
	13								
2128	14				End test pit at 14.0 feet bgs on 9/18/2007. Backfilled with excavated soils upon completion.				
	15								
2126	16								
	17								
2124	18								
	19								
2122	20								

Project: Saddleback Wind Energy Project	Log of Test Pit TP-07
Project Location: White Salmon, Washington	
Project Number: 33758687	

Date(s) Excavated	9/19/2007	Logged By	EAM	Checked By	DBM
Length of Excavation	~10 feet	Width of Excavation	~5 feet	Depth of Excavation	8.0 feet
Excavation Equipment	Track Mounted Excavator	Excavation Contractor	SDS Lumber	Approximate Surface Elevation	2184 feet MSL
Water Observations	N/A			Weather	Sunny, 60's
Location	See site map			Surface Condition	

Elevation feet	Depth, feet	Sample Type / Sample Number	Graphic Log	Lithologic Log (USCS Code)	MATERIAL DESCRIPTION	Fines (%)	Moisture (%)	Pocket Penetrometer (tsf)	REMARKS AND OTHER TESTS
2184	0			SM	**SILTY SAND [SM]**, brown, moist, medium dense, contains highly weathered to decomposed rock fragments [Colluvium]				
	1							3.5	
2182	2	1							
	3				**BASALT**, gray-brown, highly weathered				
2180	4								
	5								
2178	6	2			*Becomes gray, yellow staining on fractures, moderate to slightly weathered, hard, vesicular*				Difficult excavating
	7								
2176	8	3			End test pit at 8.0 feet bgs on 9/18/2007 due to refusal. Backfilled with excavated soils upon completion.				
	9								
2174	10								
	11								
2172	12								
	13								
2170	14								
	15								
2168	16								
	17								
2166	18								
	19								
2164	20								

Report: PORT_TP_MC-PP_200_USCS; File: WINDFARM.GPJ; 10/22/2007 TP-07

Project: Saddleback Wind Energy Project Project Location: White Salmon, Washington Project Number: 33758687	# Log of Test Pit TP-08

Date(s) Excavated	9/19/2007	Logged By	EAM	Checked By	DBM
Length of Excavation	~10 feet	Width of Excavation	~5 feet	Depth of Excavation	7.0 feet
Excavation Equipment	Track Mounted Excavator	Excavation Contractor	SDS Lumber	Approximate Surface Elevation	2214 feet MSL
Water Observations	N/A			Weather	Sunny, 60's
Location	See site map			Surface Condition	

Elevation feet	Depth, feet	Sample Type	Sample Number	Graphic Log	Lithologic Log (USCS Code)	MATERIAL DESCRIPTION	Fines (%)	Moisture (%)	Pocket Penetrometer (tsf)	REMARKS AND OTHER TESTS
2214	0				ML	**SANDY SILT [ML]**, brown, moist, soft [Topsoil]				
	1					**BEDROCK**, medium gray with black and orange specks, orange black staining on surfaces, tuffaceous, highly to moderately weathered, very soft to soft, vesicles present				
2212	2									
	3		1							
2210	4					*Becomes moderately weathered, soft to medium hard*				
	5									
2208	6									Difficult excavating
	7					End test pit at 7.0 feet bgs on 9/19/2007 due to refusal. Backfilled with excavated soils upon completion.				
2206	8									
	9									
2204	10									
	11									
2202	12									
	13									
2200	14									
	15									
2198	16									
	17									
2196	18									
	19									
2194	20									

Project: Saddleback Wind Energy Project
Project Location: White Salmon, Washington
Project Number: 33758687

Log of Test Pit TP-09

Date(s) Excavated	9/19/2007	Logged By	EAM	Checked By	DBM
Length of Excavation	~10 feet	Width of Excavation	~5 feet	Depth of Excavation	11.0 feet
Excavation Equipment	Track Mounted Excavator	Excavation Contractor	SDS Lumber	Approximate Surface Elevation	2176 feet MSL
Water Observations	N/A			Weather	Sunny, 60's
Location	See site map			Surface Condition	

Elevation feet	Depth, feet	Sample Type	Sample Number	Graphic Log	Lithologic Log (USCS Code)	MATERIAL DESCRIPTION	Fines (%)	Moisture (%)	Pocket Penetrometer (tsf)	REMARKS AND OTHER TESTS
2176	0				ML	**SANDY SILT [ML]**, brown, mosit, soft [Topsoil]				
	1					**BEDROCK**, light gray to black, salt and pepper, some yellow-brown, orange and black staining on surfaces, tuffaceous, highly to moderately weathered, very soft, spheroidal weathering patterns observed				
2174	2									
	3		1							
2172	4									
	5									
2170	6					*Becomes dark gray, moderately weathered, soft, traces of mica?*				Excavator notes material encountered at 6.0 feet bgs would be used for road bed
	7									
2168	8		2			*Becomes green-gray*				
	9									
2166	10									
	11					End test pit at 11.0 feet bgs on 9/19/2007. Backfilled with excavated soils upon completion.				
2164	12									
	13									
2162	14									
	15									
2160	16									
	17									
2158	18									
	19									
2156	20									

Project: Saddleback Wind Energy Project
Project Location: White Salmon, Washington
Project Number: 33758687

Log of Test Pit TP-10

Date(s) Excavated	9/18/2007	Logged By	EAM	Checked By	DBM
Length of Excavation	~10 feet	Width of Excavation	~5 feet	Depth of Excavation	8.0 feet
Excavation Equipment	Track Mounted Excavator	Excavation Contractor	SDS Lumber	Approximate Surface Elevation	2181 feet MSL
Water Observations	N/A			Weather	Sunny, 60's
Location	See site map			Surface Condition	

Elevation feet	Depth, feet	Sample Type	Sample Number	Graphic Log	Lithologic Log (USCS Code)	MATERIAL DESCRIPTION	Fines (%)	Moisture (%)	Pocket Penetrometer (tsf)	REMARKS AND OTHER TESTS
	0				ML	**SANDY SILT [ML]**, brown, moist, soft, rock fragments observed [Topsoil]				
2180	1									
	2					**BEDROCK**, meidum gray to brown with black and yellow specks, tuffaceous, highly weathered, very soft with hard core stones				
2178	3		1							
	4					*Becomes black and orange stained on fracture surfaces, highly to moderately weathered, spheroidal weather patterns observed*				
2176	5									
	6					*Becomes gray with black staining, soft to hard*				
2174	7									
	8					End test pit at 8.0 feet bgs on 9/18/2007 due to refusal. Backfilled with excavated soils upon completion.				
2172	9									
	10									
2170	11									
	12									
2168	13									
	14									
2166	15									
	16									
2164	17									
	18									
2162	19									
	20									

Project: Saddleback Wind Energy Project **Project Location:** White Salmon, Washington **Project Number:** 33758687	# Log of Test Pit TP-11

Date(s) Excavated	9/18/2007	Logged By	EAM	Checked By	DBM
Length of Excavation	~10 feet	Width of Excavation	~5 feet	Depth of Excavation	16.0 feet
Excavation Equipment	Track Mounted Excavator	Excavation Contractor	SDS Lumber	Approximate Surface Elevation	2119 feet MSL
Water Observations	N/A			Weather	Sunny, 60's
Location	See site map			Surface Condition	

Elevation feet	Depth, feet	Sample Type	Sample Number	Lithologic Log (USCS Code)	MATERIAL DESCRIPTION	Fines (%)	Moisture (%)	Pocket Penetrometer (tsf)	REMARKS AND OTHER TESTS
	0			ML	**SANDY SILT [ML]**, brown, moist, soft [Topsoil]				
2118	1		1	SM	**SILTY SAND [SM]**, brown, yellow and gray, moist, rock fragments. [Colluvium]	48.9	30.6	3.0	
	2			ML	**SANDY SILT [ML]**, reddish brown with medium gray core stones, black and orange on fracture surfaces, sandy silt, some structure observed including remnant vesicles [Decomposed Bedrock]				
2116	3		2						
	4							4.5	
2114	5								
	6				*Becomes highly weathered to decomposed*				
2112	7								
	8								
2110	9								
	10				**BEDROCK**, brown to gray, orange-black staining on fracture surfaces, tuffaceous, highly weathered to decomposed				
2108	11								
	12								
2106	13								
	14								
2104	15		3						
	16				End test pit at 16.0 feet bgs on 9/19/2007. Backfilled with excavated soils upon completion.				
2102	17								
	18								
2100	19								
	20								

Project: Saddleback Wind Energy Project **Project Location:** White Salmon, Washington **Project Number:** 33758687	**Log of Test Pit TP-12**

Date(s) Excavated	9/18/2007	Logged By	EAM	Checked By	DBM
Length of Excavation	~10 feet	Width of Excavation	~5 feet	Depth of Excavation	8.0 feet
Excavation Equipment	Track Mounted Excavator	Excavation Contractor	SDS Lumber	Approximate Surface Elevation	2070 feet MSL
Water Observations	N/A			Weather	Sunny, 60's
Location	See site map			Surface Condition	

Elevation feet	Depth, feet	Sample Type	Sample Number	Graphic Log	Lithologic Log (USCS Code)	MATERIAL DESCRIPTION	Fines (%)	Moisture (%)	Pocket Penetrometer (tsf)	REMARKS AND OTHER TESTS
2070	0				ML	**SANDY SILT [ML]**, brown, moist, soft [Topsoil]				
	1				SM	**SILTY SAND [SM]**, brown, moist to dry, loose, occasional rock fragments [Colluvium]				
			1				38.7	15.3		
2068	2					**BEDROCK**, rock fragments in silty sand matrix, ~20% rock fragments, fragments are gray, decomposed to highly weathered, soft, porous [Decomposed Basalt]				
	3									
2066	4									
	5									
2064	6					*Rock becomes light gray with black mottles and yellow coatings, moderately to highly weathered, large boulders encountered (2-3 feet diameter)*				
	7									Difficult excavating
			2							
2062	8					End test pit at 8.0 feet bgs on 9/18/2007 due to refusal. Backfilled with excavated soils upon completion.				
	9									
2060	10									
	11									
2058	12									
	13									
2056	14									
	15									
2054	16									
	17									
2052	18									
	19									
2050	20									

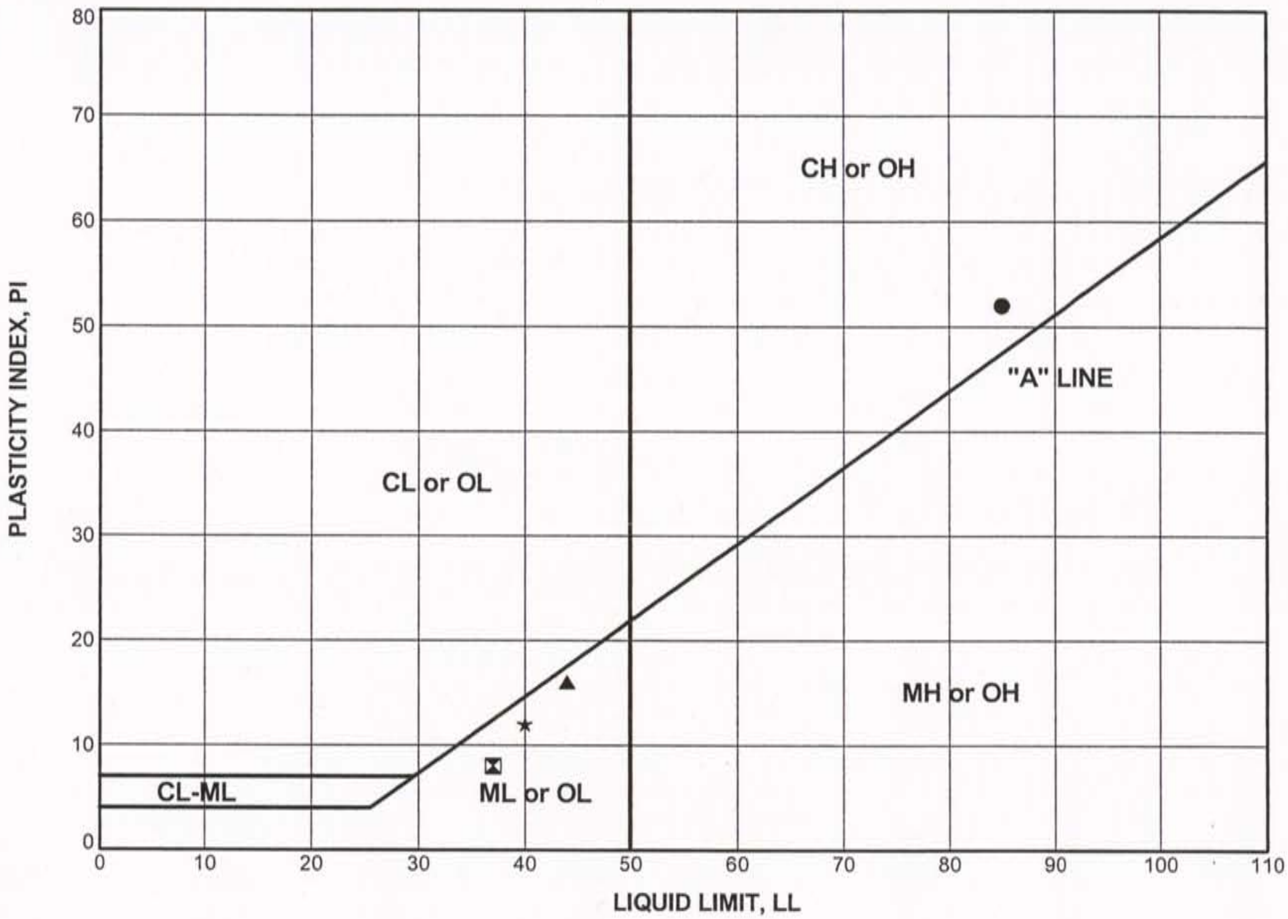
APPENDIX B
FIELD SOIL RESISTIVITY AND LABORATORY TESTING

Resistivity tests for the proposed Wind Energy Project site were performed on September 24, 2007 to assess the resistivity of the near surface soil. Table B-1 presents the results of these measurements. The soil resistivity at locations R-1, R-2, R-3, and R-5 are shown graphically in Figure B-1. Soil resistivity at location R-4 is shown in Figure B-2.

Table B-1: Measured Resistivity (ohm-cm)

Date Completed	Location	Spacing (ft)	Effective Depth (ft)	Reading	Multiplier	Resistivity (Ω cm)
9/24/2007	**R-1**	5.0	10.0	7.1	10	**67983**
9/24/2007	**R-1**	10.0	20.0	1.6	10	**30640**
9/24/2007	**R-1**	20.0	40.0	4.2	1	**16086**
9/24/2007	**R-2**	5.0	10.0	3.8	100	**363850**
9/24/2007	**R-2**	10.0	20.0	2.4	100	**459600**
9/24/2007	**R-2**	20.0	40.0	1.4	100	**536200**
9/24/2007	**R-3**	5.0	10.0	0.9	100	**86175**
9/24/2007	**R-3**	10.0	20.0	0.4	100	**76600**
9/24/2007	**R-3**	20.0	40.0	2.6	10	**99580**
9/24/2007	**R-4**	5.0	10.0	11.0	100000	*
9/24/2007	**R-4**	10.0	20.0	10.6	1000	*
9/24/2007	**R-4**	20.0	40.0	11.0	100000	*
9/24/2007	**R-5**	5.0	10.0	3.0	10	**28725**
9/24/2007	**R-5**	10.0	20.0	2.0	10	**38300**
9/24/2007	**R-5**	20.0	40.0	1.2	10	**45960**

* Field Resistivity found to be in excess of 1,000,000 ohm-cm.



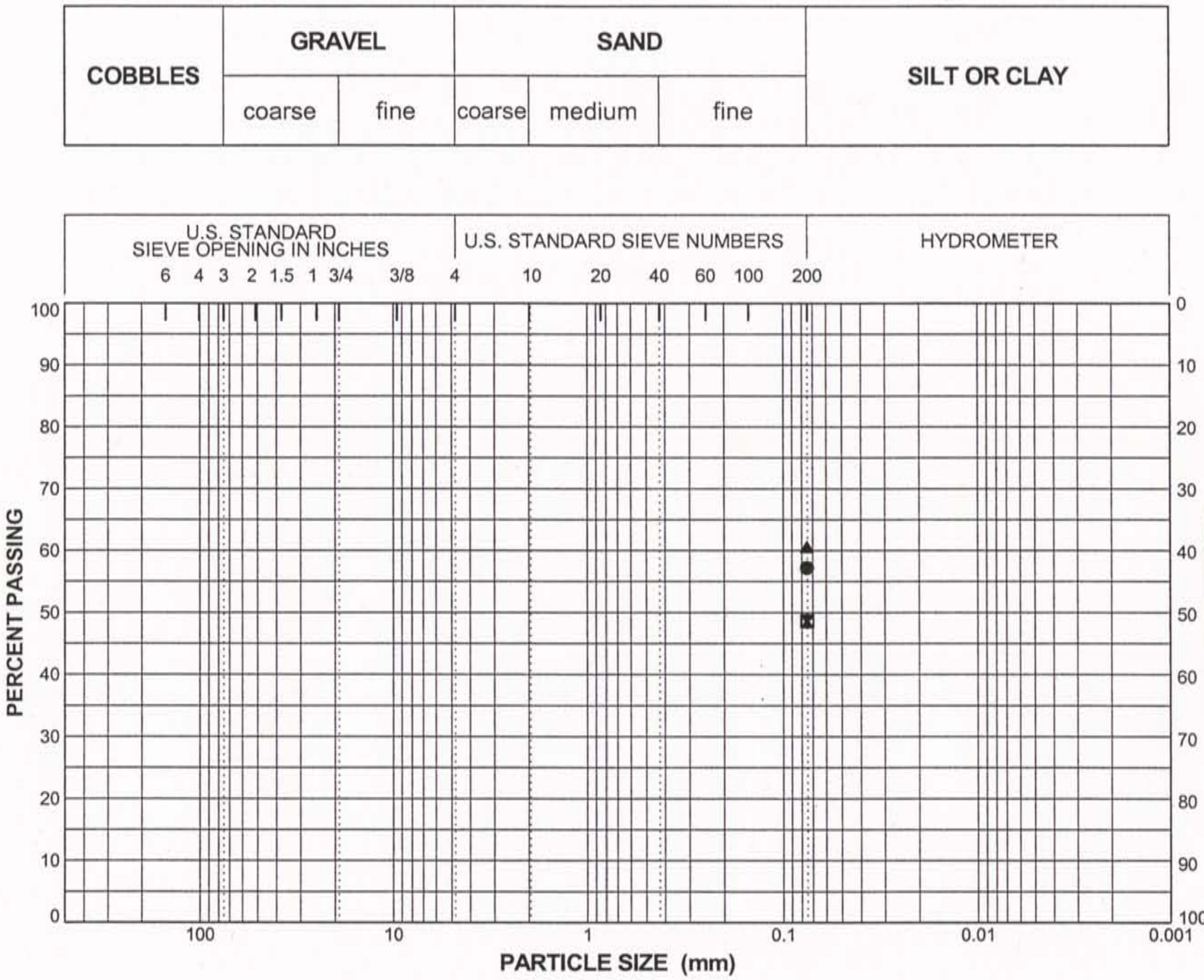
Boring ID	Sample #	Depth (feet)	Symbol	Sample Moisture %	LL	PL	PI	Classification
TP-01	1	2.0-2.5	●	52.1	85	33	52	Sandy Fat Clay [CH]
TP-05	1	1.0-1.5	⧗	16.4	37	29	8	Silty Sand [SM]
TP-06	1	2.5-3.0	▲	25.7	44	28	16	Sandy Silt [ML]
TP-11	1	1.0-1.5	★	30.6	40	28	12	Silty Sand [SM]

Report: ATTERBERG_PLOT_12 PTS; File: WINDFARM.GPJ; 11/5/2007 TP-11

Saddleback Wind Energy Project
White Salmon, Washington
33758687

PLASTICITY CHART

URS



Boring ID	Sample #	Depth (feet)	Sample Moisture (%)	Dry Density (lbs/ft³)	Symbol	LL	PI	% G	% S	% F	Classification
TP-01	1	2.0-2.5	52.1		●	85	52			57.2	Sandy Fat Clay [CH]
TP-05	1	1.0-1.5	16.4		⧗	37	8			48.6	Silty Sand [SM]
TP-06	1	2.5-3.0	25.7		▲	44	16			60.6	Sandy Silt [ML]
TP-11	1	1.0-1.5	30.6		★	40	12			48.9	Silty Sand [SM]

Report: SIEVE_4_PORT; File: WINDFARM.GPJ; 10/23/2007 TP-11

Saddleback Wind Energy Project
White Salmon, Washington
33758687

PARTICLE SIZE DISTRIBUTION CURVES

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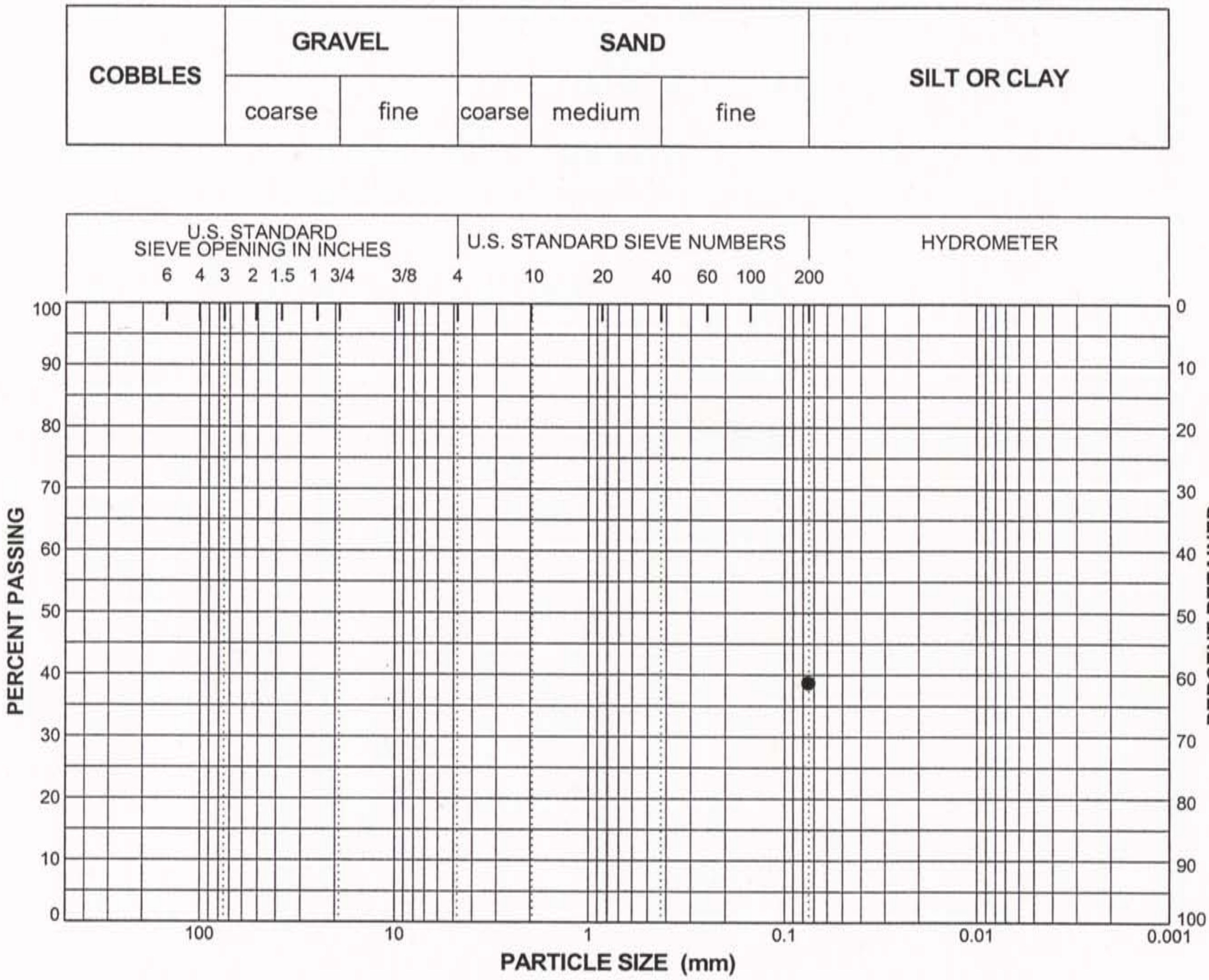

Boring ID	Sample #	Depth (feet)	Sample Moisture (%)	Dry Density (lbs/ft³)	Symbol	LL	PI	% G	% S	% F	Classification
TP-12	1	1.5-2.0	15.3		●					38.7	Silty Sand [SM]

Report: SIEVE_4_PORT; File: WINDFARM.GPJ; 10/23/2007 TP-12

APPENDIX C
PAVEMENT ENGINEERING

PAVEMENT ENGINEERING REPORT

Anticipated Haul Route

URS observed the anticipated haul route from the SDS Lumber facility to the Saddleback site during a site visit in September, 2007. We anticipate the route to run along SR 14 to Cook-Underwood Road, up Cook-Underwood Road to Kollock-Knapp Road, up Kollock-Knapp Road to a sharp right hand turn onto Scoggins Road, and then a left onto the logging road on SDS property (CG-2930) to the Saddleback site.

Anticipated Haul Routes

Very little information is available about the existing pavement and base thicknesses along the haul route. From observations made while driving along the haul route, the pavements appear to be generally in good condition however, we understand that the pavements along Cook-Underwood Road had been recently improved with a chip seal and/or minor overlay. Some areas of the Kollock-Knapp Road showed signs of distress.

Construction for the proposed project is not anticipated to start for about three years. Therefore, three years of local traffic will traverse the pavement prior to the initiation of construction traffic. URS recommends that just prior to the initiation of construction for the project a comprehensive pavement assessment program be undertaken to establish the condition of the existing pavements. This would then be coupled with a mechanistic-empirical approach for establishment of remaining pavement life just prior to construction traffic. Once this is done, the potential effects of construction traffic on the pavements current state could be captured. For the proposed project URS recommends a comprehensive investigation of the existing pavements including: a visual pavement condition survey; Falling Weight Deflectometer testing; pavement coring; laboratory testing; and Dynamic Cone Penetrometer testing. This program will give a baseline metric for the pavement, base and subgrade conditions prior to the construction traffic.

No information was available at the time of this report regarding the existing average daily traffic (ADT) volumes along the proposed haul route. This information will be necessary during the analysis portion of the work, just prior to the construction traffic. If this information is not readily available, a traffic survey should be performed to determine the ADT and type of vehicle traffic along the proposed haul route. This information will be used to determine the remaining life of the existing pavement section and the pavement section that will be required post construction.

Truck Hauling Information

In order to mobilize the sections of wind tower equipment to the site, it is anticipated that they will arrive on rail to the SDS Lumber facility, be off loaded onto specialized trucks, and then transported up hill to the Saddleback site. URS understands that they could possibly arrive by barge also, but this is to be determined. The final vendor for the wind towers has not been selected at the time of this report. URS has made some general assumptions regarding typical tower configuration for the purpose of this report. URS understands that at the time of this report, the towers will be about 250 feet in height, with blades that are approximately 135 feet in length. We assume that each wind tower consist of the tower section, hub, blades and a nacelle. Heavy and oversized pieces, such as the tower sections and blades will be trucked to the site individually, while other components will be bundled together for transport from SDS Lumber.

Tubular towers are typically transported in 60 to 90-foot long sections that weight from 42 to 59 tons. Three rotor blades are required for each tower and are about 135 feet in length and about 7 tons each. The entire rotor assembly weighs approximately 35 tons. The nacelle is approximately 30 feet long, 12 feet wide by 12 feet high and weighs approximately 57 tons. Specialized trucking equipment is required to transport the tower pieces from SDS Lumber to the Saddleback site. Truck size and axle loading depends on the piece of equipment being transported and any anticipated restrictions that will be encountered along the haul route such as low overhead conditions, uneven traveling surfaces and load restrictions. URS anticipates that approximately 8-10 truck hauls will be required for each tower installed. Therefore, more than 500 heavy haul trips will be required over the county roads for the towers only, in addition to construction equipment. This quantity does not include delivery of construction materials such as concrete required for the foundation, grading equipment to construct roads and prepare the site or other construction traffic associated with neighborhood construction and deliveries.

URS is able to obtain information from Anderson Trucking Services, Inc. (ATS), who specialize in wind turbine transportation, on typical axle loading information for the 80-meter high, GE 1.5 Wind Turbine Generators. URS will include this information in our final report.

Strategy and Design Parameters for Haul Roads On SDS Property

URS drove and observed the haul roads on SDS property during our September, 2007 site visit. The existing logging road (CG-2930) to the Saddleback site has primarily been used for accessing stands of timber for harvesting and exporting timber from the site. The dirt road is currently surfaced with soil and rock and is in poor condition. In its current state the road is not suitable for the trucks that will be carrying the wind tower equipment.

URS will analyze the existing topography and work within the equipment limitations of the haul trucks that will be transporting the equipment to the site. Likely this will include rebuilding large sections of the existing road and surfacing with rock. For areas with steep slopes, we anticipate flattening and rebuilding the slopes and placing asphalt in select areas to allow access by the hauling equipment. The asphalt could remain in place or be removed at the end of the project. URS will develop design parameters for the pavement suitable to protect the section through construction.

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Pre and Post Heavy Haul Strategy for Pavement Assessments

URS will implement a thorough investigation program to capture the existing pavement, aggregate base and subgrade conditions along the alignment prior to heavy hauls. This program will consist of the following:

- A visual pavement condition survey and report of the surface of the existing pavements along the haul route to quantify weak or deteriorated areas that may need repair
- Pavement core holes to obtain the pavement thicknesses along the haul route
- Sampling of the near surface soils for classification
- Laboratory testing, as necessary, of the soils to further refine their classification and strength parameters

- Dynamic Cone Penetrometer (DCP) testing in selected locations where pavement widening may need to occur to determine the strength of the near surface subgrade soils

- Falling Weight Deflectometer (FWD) testing along the haul route to assess the *insitu* strength of the asphalt, aggregate base and subgrade soils

URS will use the results of this investigation to determine the capacity of the existing pavement section and what, if any, modification to the pavement section may be required to support the heavy haul trucks. URS concern is that the heavy loading may significantly deteriorate the existing pavement section, which was likely not designed for such heavy loads. URS will provide design recommendations for improving the pavement section, as necessary, prior to the beginning of the hauling program.

Design parameters critical to this analysis include determining the number of equivalent single axle loads (ESALs), which are based on trailer loading, number of axles and their configuration, and the thickness and resilient modulus of the subgrade, aggregate base and pavement materials. These factors combined with recommended values for initial serviceability, standard deviation, reliability, and a terminal serviceability as outlined by the American Association of State Highway and Transportation Officials (AASHTO) make up the main components of this analysis. Using these values, URS will determine minimum acceptable asphalt and aggregate base thicknesses to support the proposed loading.

At the completion of the hauling program and construction, URS proposes to perform a visual assessment of the surface conditions of the pavement, similar to what was performed before the construction began. The visual assessment will identify weak or deteriorated areas along the haul route which may require mitigation. Depending on the outcome of the preconstruction pavement analysis and the visual observation of post construction conditions, URS may prepare a mitigation design to repair the pavements to pre-construction conditions or better. 20 years is the typical pavement design life.

DCP TEST DATA

DCP-01

Project: Saddleback Windfarm
Location: White Salmon, WA

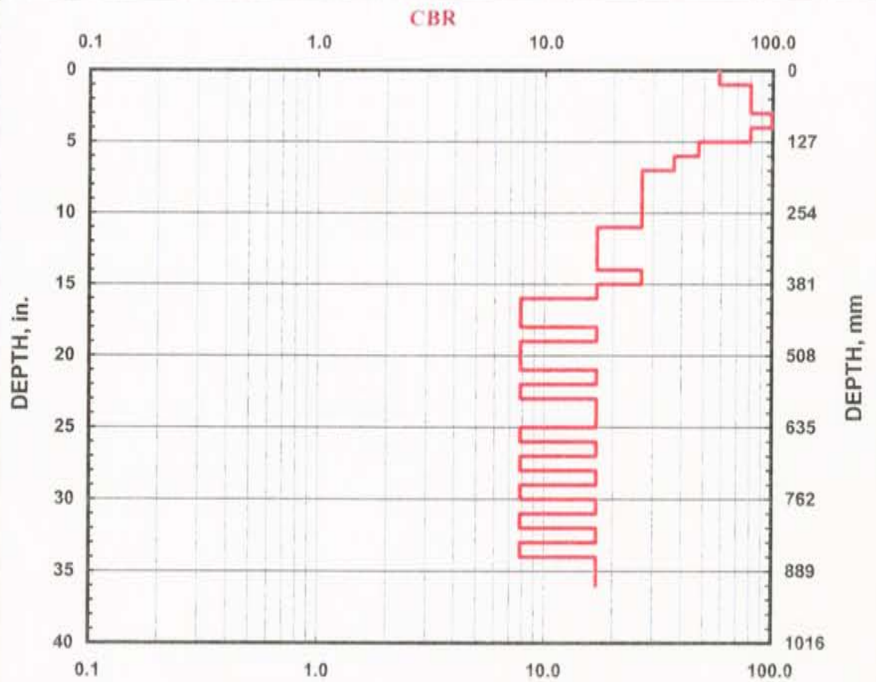
Date: 9/19/2007
Project No.: 33758687
Start: 0 feet bgs

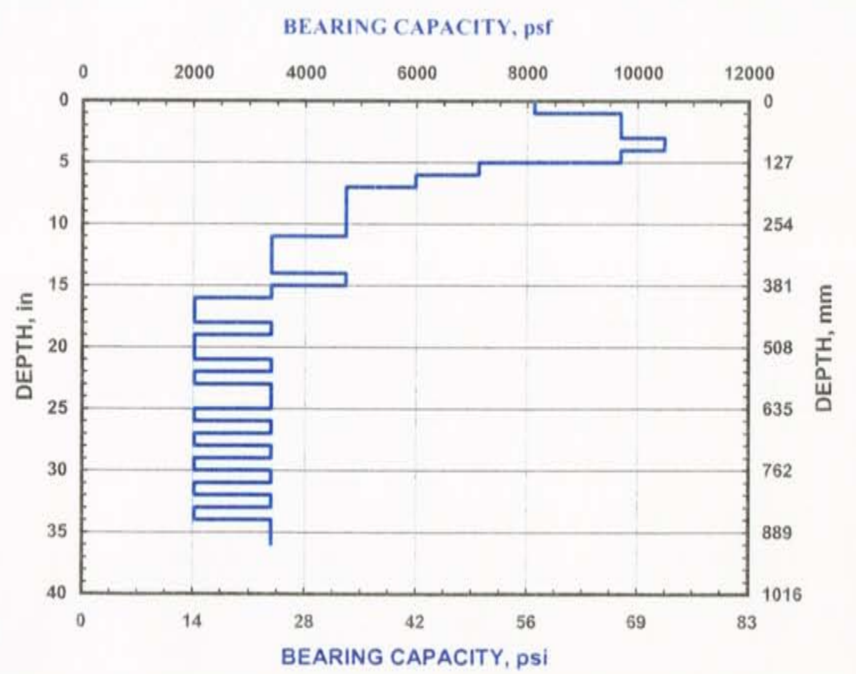
Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
6	25	1	1.0
8	51	1	2.0
8	76	1	3.0
10	102	1	4.0
8	127	1	5.0
5	152	1	6.0
4	178	1	7.0
3	203	1	8.0
3	229	1	9.0
3	254	1	10.0
3	279	1	11.0
2	305	1	12.0
2	330	1	13.0
2	356	1	14.0
3	381	1	15.0
2	406	1	16.0
1	432	1	17.0
1	457	1	18.0
2	483	1	19.0
1	508	1	20.0
1	533	1	21.0
2	559	1	22.0
1	584	1	23.0
2	610	1	24.0
2	635	1	25.0
1	660	1	26.0
2	686	1	27.0
1	711	1	28.0
2	737	1	29.0
1	762	1	30.0
2	787	1	31.0
1	813	1	32.0
2	838	1	33.0
1	864	1	34.0
2	889	1	35.0
2	914	1	36.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





DCP TEST DATA

DCP-02

Project: Saddleback Windfarm
Location: White Salmon, WA

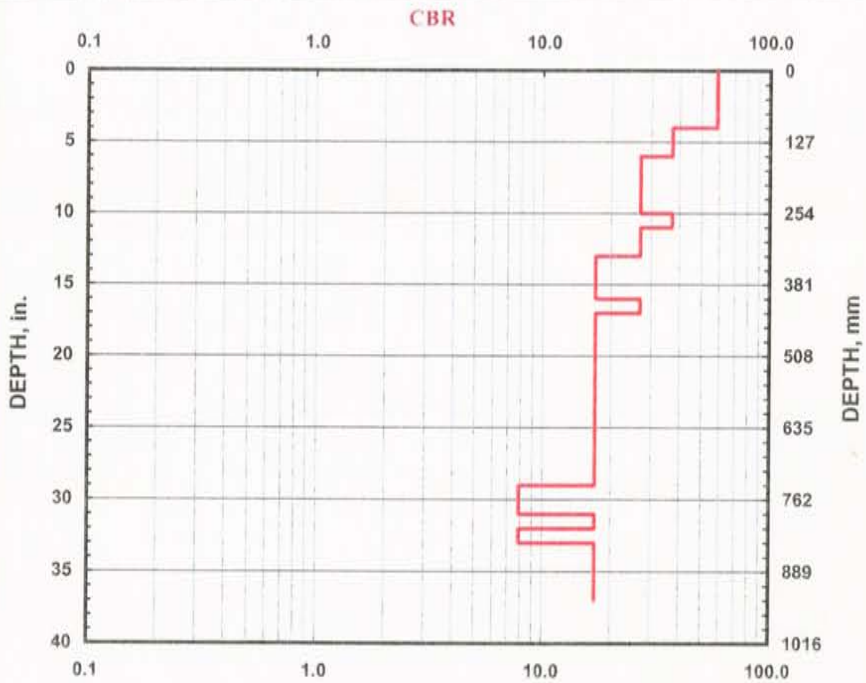
Date: 9/19/2007
Project No.: 33758687
Start: 0 feet bgs

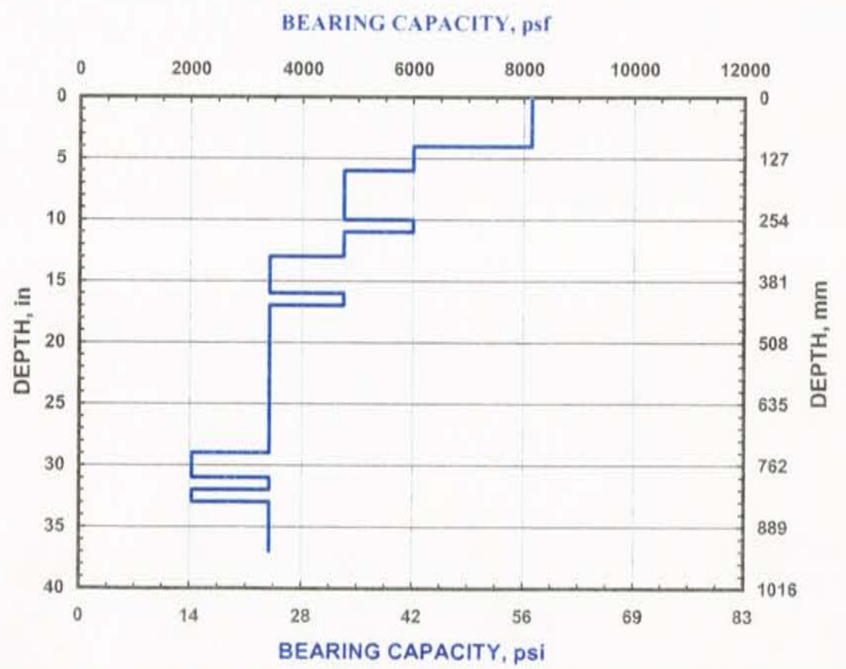
Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
6	25	1	1.0
6	51	1	2.0
6	76	1	3.0
6	102	1	4.0
4	127	1	5.0
4	152	1	6.0
3	178	1	7.0
3	203	1	8.0
3	229	1	9.0
3	254	1	10.0
4	279	1	11.0
3	305	1	12.0
3	330	1	13.0
2	356	1	14.0
2	381	1	15.0
2	406	1	16.0
3	432	1	17.0
2	457	1	18.0
2	483	1	19.0
2	508	1	20.0
2	533	1	21.0
2	559	1	22.0
2	584	1	23.0
2	610	1	24.0
2	635	1	25.0
2	660	1	26.0
2	686	1	27.0
2	711	1	28.0
2	737	1	29.0
1	762	1	30.0
1	787	1	31.0
2	813	1	32.0
1	838	1	33.0
2	864	1	34.0
2	889	1	35.0
2	914	1	36.0
2	940	1	37.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





DCP TEST DATA

DCP-03

Project: *Saddleback Windfarm*
Location: White Salmon, WA

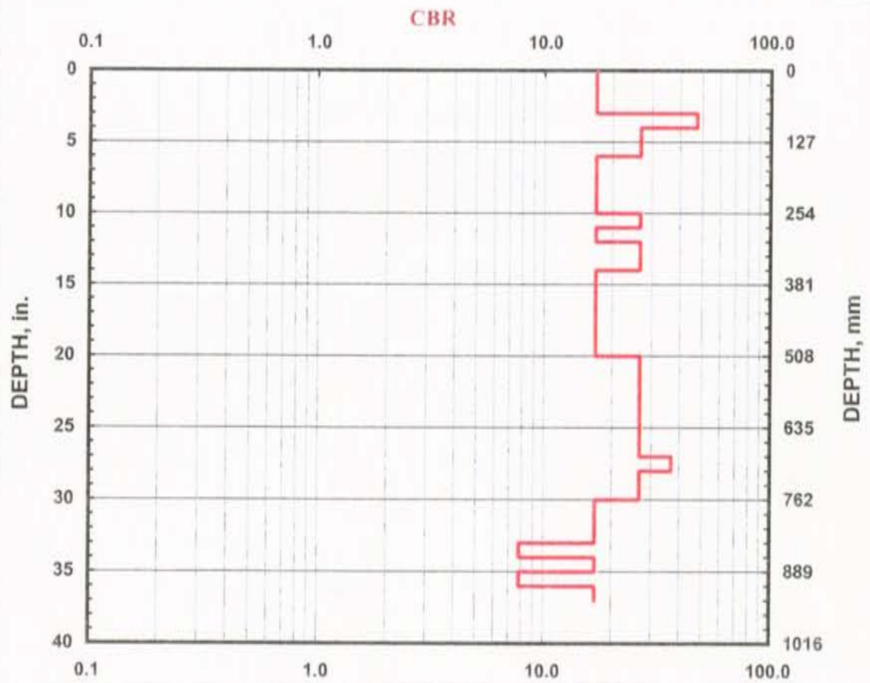
Date: 9/19/2007
Project No.: 33758687
Start: 0 feet bgs

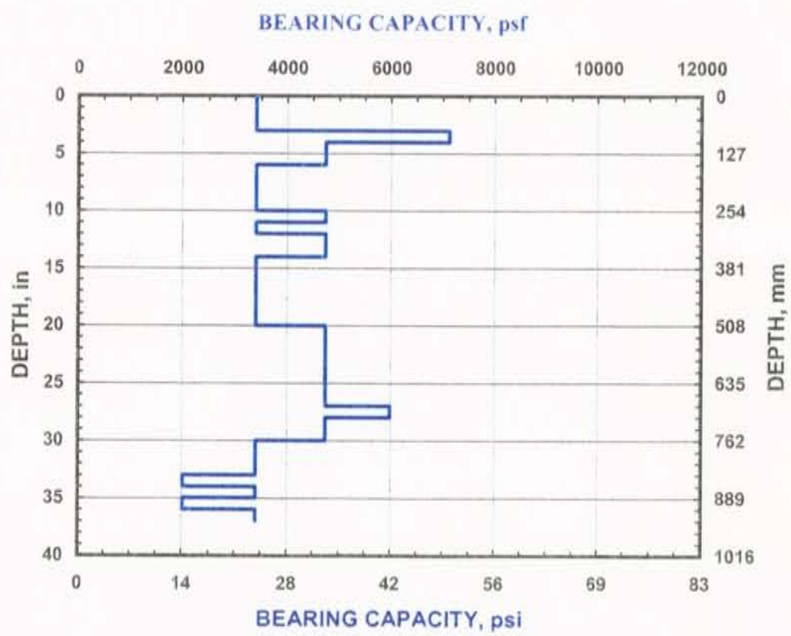
Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
2	25	1	1.0
2	51	1	2.0
2	76	1	3.0
5	102	1	4.0
3	127	1	5.0
3	152	1	6.0
2	178	1	7.0
2	203	1	8.0
2	229	1	9.0
2	254	1	10.0
3	279	1	11.0
2	305	1	12.0
3	330	1	13.0
3	356	1	14.0
2	381	1	15.0
2	406	1	16.0
2	432	1	17.0
2	457	1	18.0
2	483	1	19.0
2	508	1	20.0
3	533	1	21.0
3	559	1	22.0
3	584	1	23.0
3	610	1	24.0
3	635	1	25.0
3	660	1	26.0
3	686	1	27.0
4	711	1	28.0
3	737	1	29.0
3	762	1	30.0
2	787	1	31.0
2	813	1	32.0
2	838	1	33.0
1	864	1	34.0
2	889	1	35.0
1	914	1	36.0
2	940	1	37.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





DCP TEST DATA

DCP-04

Project: *Saddleback Windfarm*
Location: White Salmon, WA

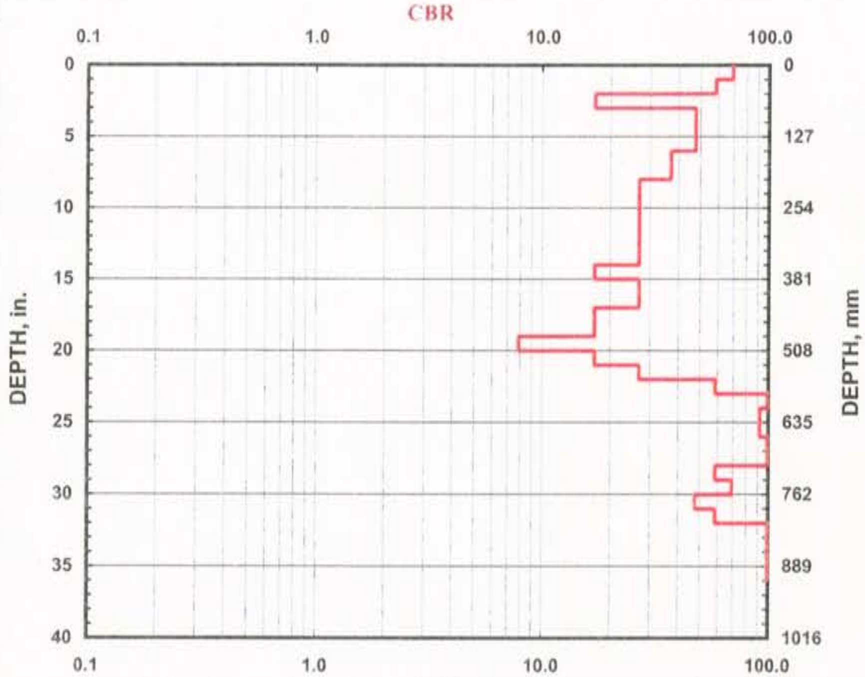
Date: 9/24/2007
Project No.: 33758687
Start: 0 feet bgs

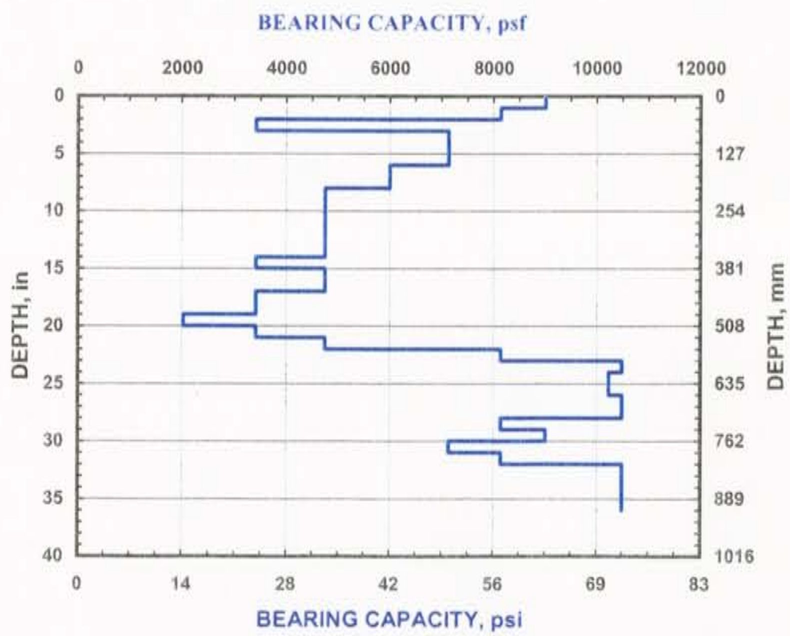
Hammer
○ 10.1 lbs.
◉ 17.6 lbs.
○ Both hammers used

Soil Type
○ CH
○ CL
◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
7	25	1	1.0
6	51	1	2.0
2	76	1	3.0
5	102	1	4.0
5	127	1	5.0
5	152	1	6.0
4	178	1	7.0
4	203	1	8.0
3	229	1	9.0
3	254	1	10.0
3	279	1	11.0
3	305	1	12.0
3	330	1	13.0
3	356	1	14.0
2	381	1	15.0
3	406	1	16.0
3	432	1	17.0
2	457	1	18.0
2	483	1	19.0
1	508	1	20.0
2	533	1	21.0
3	559	1	22.0
6	584	1	23.0
10	610	1	24.0
9	635	1	25.0
9	660	1	26.0
11	686	1	27.0
10	711	1	28.0
6	737	1	29.0
7	762	1	30.0
5	787	1	31.0
6	813	1	32.0
10	838	1	33.0
23	864	1	34.0
19	889	1	35.0
12	914	1	36.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





DCP TEST DATA

DCP-05

Project: *Saddleback Windfarm*
Location: White Salmon, WA

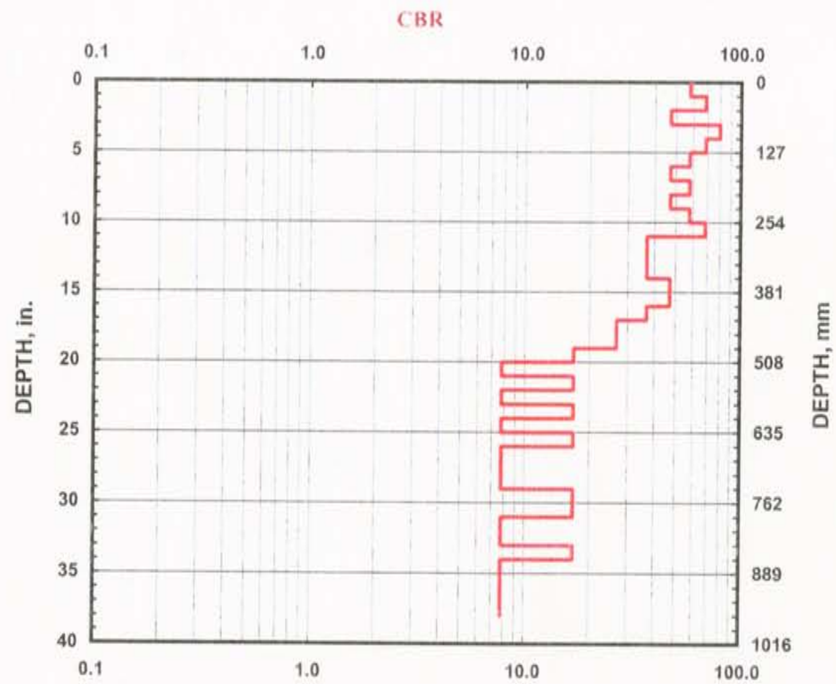
Date: 9/24/2007
Project No.: 33758687
Start: 0 feet bgs

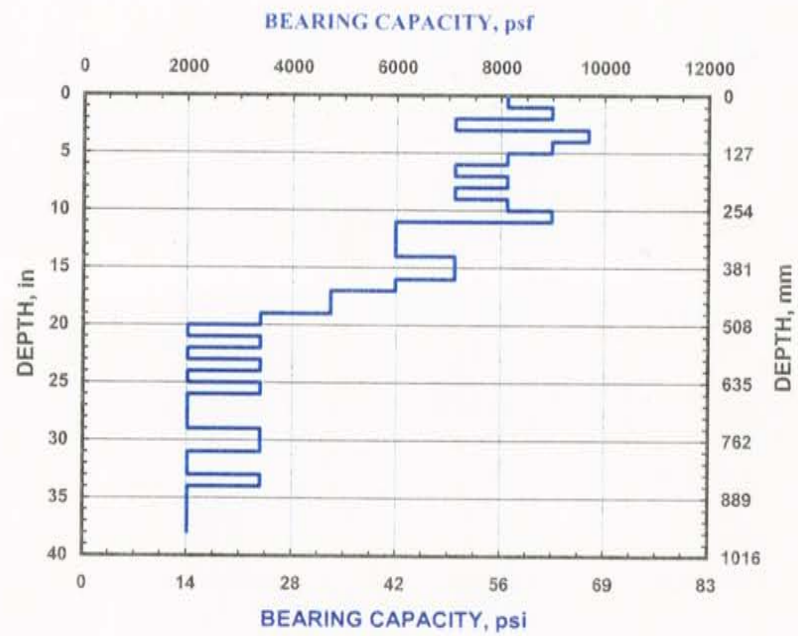
Hammer
○ 10.1 lbs.
◉ 17.6 lbs.
○ Both hammers used

Soil Type
○ CH
○ CL
◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
6	25	1	1.0
7	51	1	2.0
5	76	1	3.0
8	102	1	4.0
7	127	1	5.0
6	152	1	6.0
5	178	1	7.0
6	203	1	8.0
5	229	1	9.0
6	254	1	10.0
7	279	1	11.0
4	305	1	12.0
4	330	1	13.0
4	356	1	14.0
5	381	1	15.0
5	406	1	16.0
4	432	1	17.0
3	457	1	18.0
3	483	1	19.0
2	508	1	20.0
1	533	1	21.0
2	559	1	22.0
1	584	1	23.0
2	610	1	24.0
1	635	1	25.0
2	660	1	26.0
1	686	1	27.0
1	711	1	28.0
1	737	1	29.0
2	762	1	30.0
2	787	1	31.0
1	813	1	32.0
1	838	1	33.0
2	864	1	34.0
1	889	1	35.0
1	914	1	36.0
1	940	1	37.0
1	965	1	38.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





DCP TEST DATA

DCP-06

Project: Saddleback Windfarm
Location: White Salmon, WA

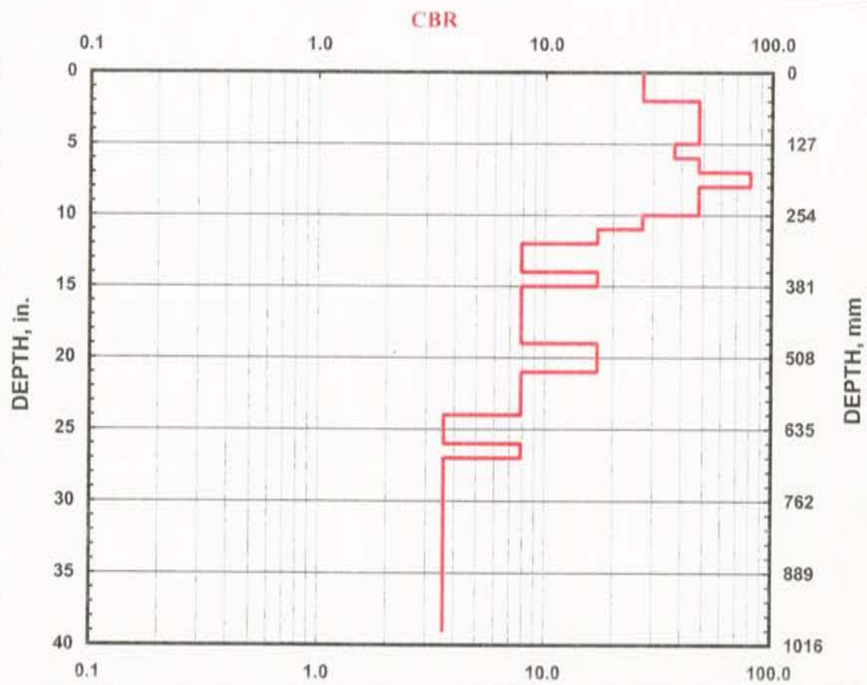
Date: 9/24/2007
Project No.: 33758687
Start: 0 feet bgs

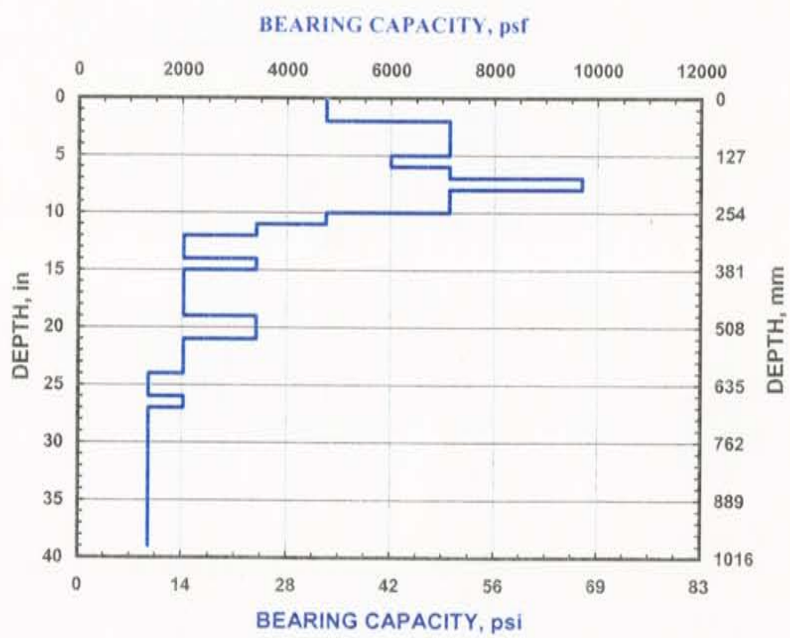
Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
3	25	1	1.0
3	51	1	2.0
5	76	1	3.0
5	102	1	4.0
5	127	1	5.0
4	152	1	6.0
5	178	1	7.0
8	203	1	8.0
5	229	1	9.0
5	254	1	10.0
3	279	1	11.0
2	305	1	12.0
1	330	1	13.0
1	356	1	14.0
2	381	1	15.0
1	406	1	16.0
1	432	1	17.0
1	457	1	18.0
1	483	1	19.0
2	508	1	20.0
2	533	1	21.0
1	559	1	22.0
1	584	1	23.0
1	610	1	24.0
1	660	1	26.0
1	686	1	27.0
1	737	1	29.0
1	787	1	31.0
1	838	1	33.0
1	889	1	35.0
1	940	1	37.0
1	991	1	39.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





DCP TEST DATA

DCP-07

Project: Saddleback Windfarm
Location: White Salmon, WA

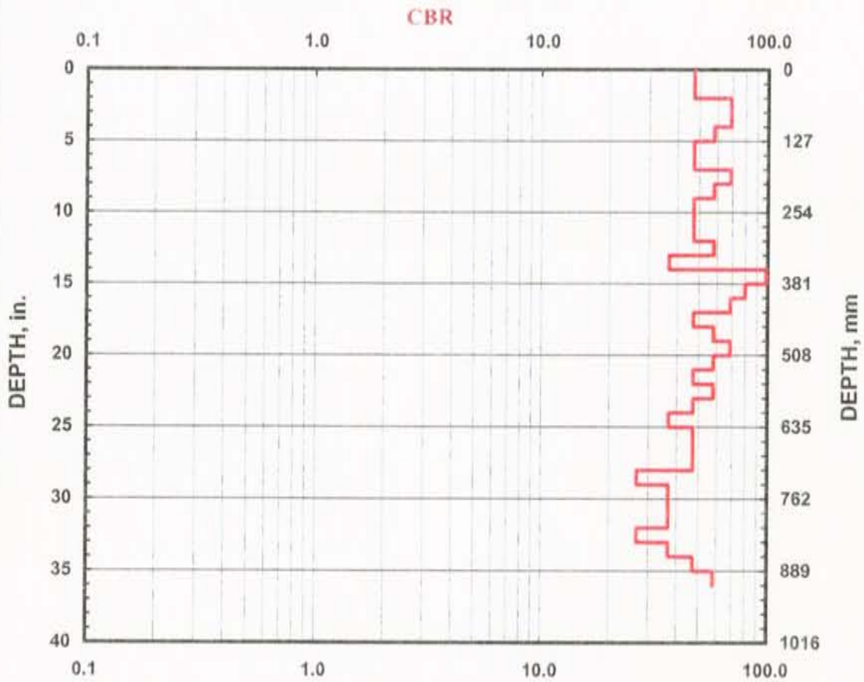
Date: 9/24/2007
Project No.: 33758687
Start: 0 feet bgs

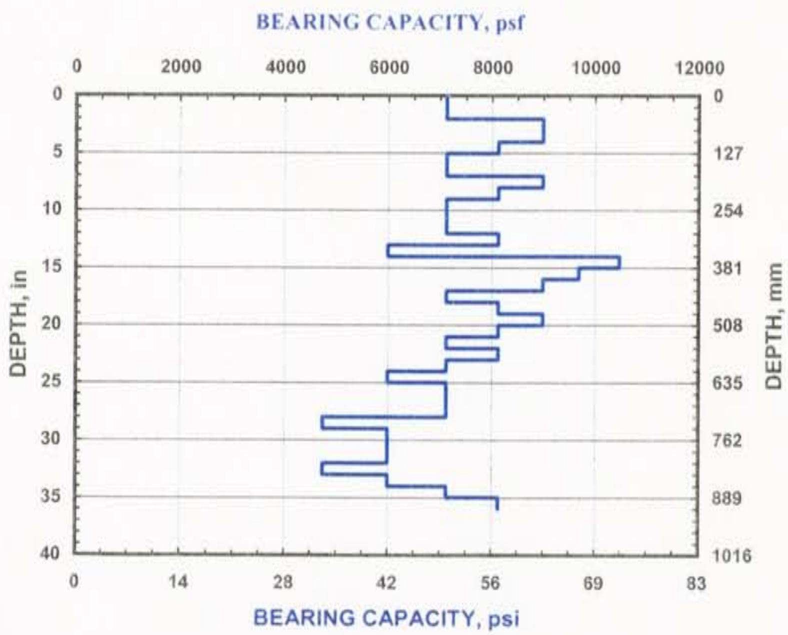
Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
5	25	1	1.0
5	51	1	2.0
7	76	1	3.0
7	102	1	4.0
6	127	1	5.0
5	152	1	6.0
5	178	1	7.0
7	203	1	8.0
6	229	1	9.0
5	254	1	10.0
5	279	1	11.0
5	305	1	12.0
6	330	1	13.0
4	356	1	14.0
10	381	1	15.0
8	406	1	16.0
7	432	1	17.0
5	457	1	18.0
6	483	1	19.0
7	508	1	20.0
6	533	1	21.0
5	559	1	22.0
6	584	1	23.0
5	610	1	24.0
4	635	1	25.0
5	660	1	26.0
5	686	1	27.0
5	711	1	28.0
3	737	1	29.0
4	762	1	30.0
4	787	1	31.0
4	813	1	32.0
3	838	1	33.0
4	864	1	34.0
5	889	1	35.0
6	914	1	36.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





DCP TEST DATA

DCP-08

Project: Saddleback Windfarm
Location: White Salmon, WA

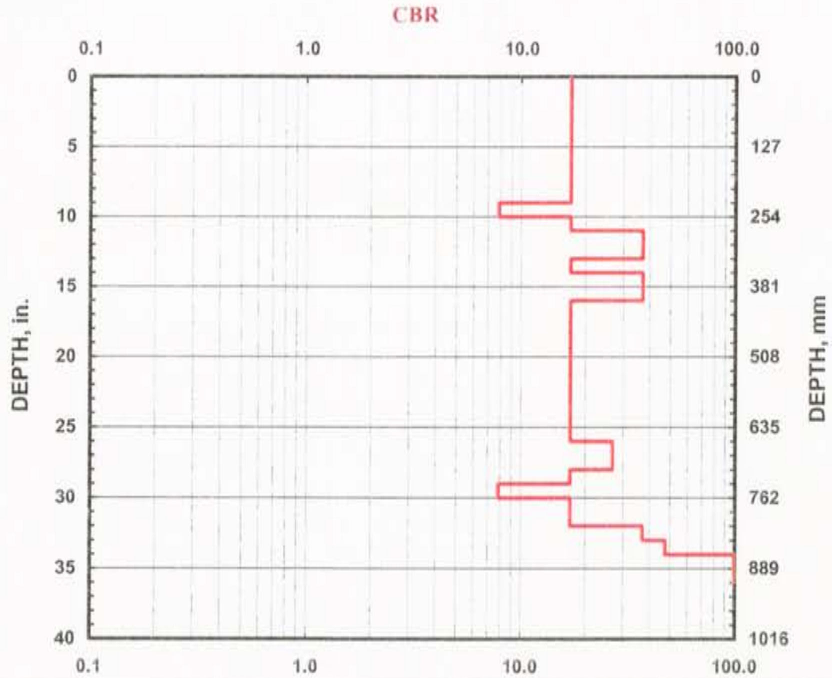
Date: 9/24/2007
Project No.: 33758687
Start: 0 feet bgs

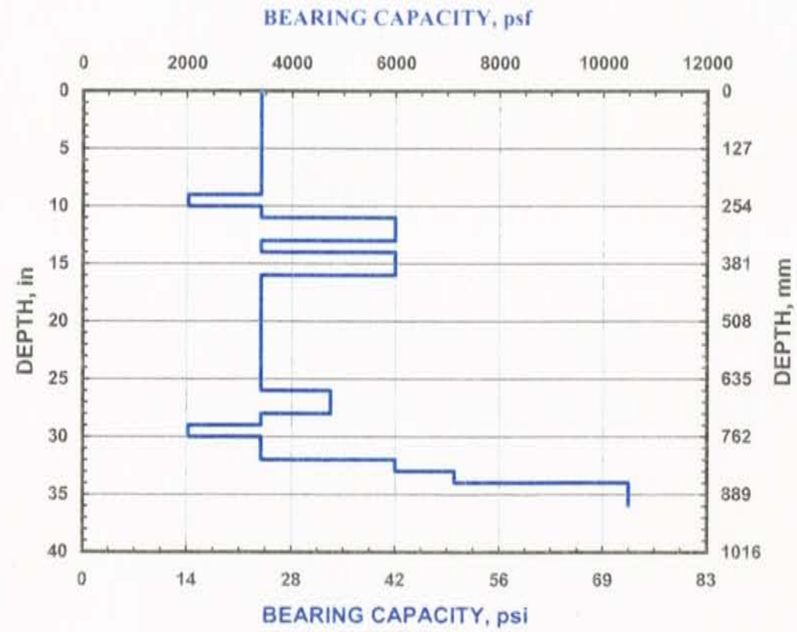
Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
2	25	1	1.0
2	51	1	2.0
2	76	1	3.0
2	102	1	4.0
2	127	1	5.0
2	152	1	6.0
2	178	1	7.0
2	203	1	8.0
2	229	1	9.0
1	254	1	10.0
2	279	1	11.0
4	305	1	12.0
4	330	1	13.0
2	356	1	14.0
4	381	1	15.0
4	406	1	16.0
2	432	1	17.0
2	457	1	18.0
2	483	1	19.0
2	508	1	20.0
2	533	1	21.0
2	559	1	22.0
2	584	1	23.0
2	610	1	24.0
2	635	1	25.0
2	660	1	26.0
3	686	1	27.0
3	711	1	28.0
2	737	1	29.0
1	762	1	30.0
2	787	1	31.0
2	813	1	32.0
4	838	1	33.0
5	864	1	34.0
12	889	1	35.0
11	914	1	36.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





DCP TEST DATA

DCP-09

Project: Saddleback Windfarm
Location: White Salmon, WA

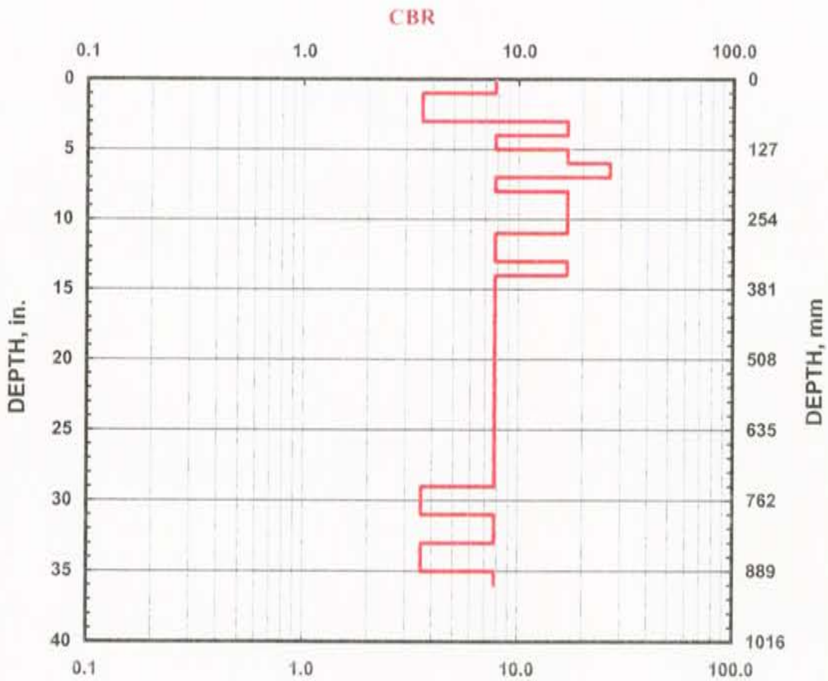
Date: 9/24/2007
Project No.: 33758687
Start: 0 feet bgs

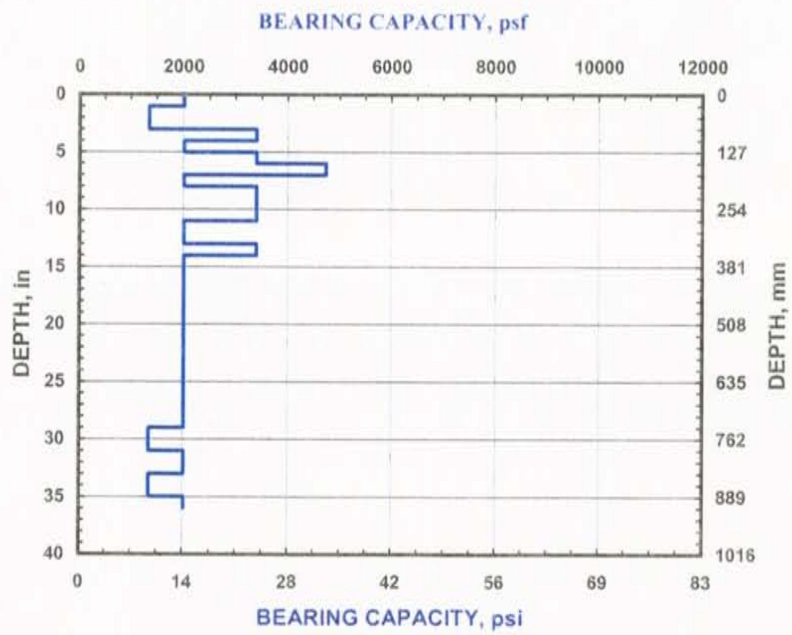
Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
1	25	1	1.0
1	76	1	3.0
2	102	1	4.0
1	127	1	5.0
2	152	1	6.0
3	178	1	7.0
1	203	1	8.0
2	229	1	9.0
2	254	1	10.0
2	279	1	11.0
1	305	1	12.0
1	330	1	13.0
2	356	1	14.0
1	381	1	15.0
1	406	1	16.0
1	432	1	17.0
1	457	1	18.0
1	483	1	19.0
1	508	1	20.0
1	533	1	21.0
1	559	1	22.0
1	584	1	23.0
1	610	1	24.0
1	635	1	25.0
1	660	1	26.0
1	686	1	27.0
1	711	1	28.0
1	737	1	29.0
1	787	1	31.0
1	813	1	32.0
1	838	1	33.0
1	889	1	35.0
1	914	1	36.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





DCP TEST DATA

DCP-10

Project: Saddleback Windfarm
Location: White Salmon, WA

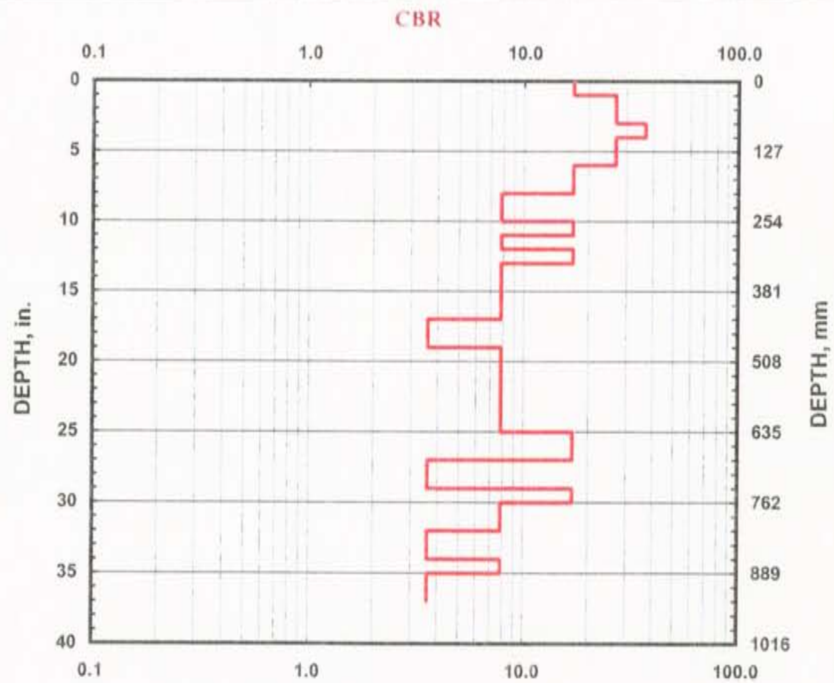
Date: 9/24/2007
Project No.: 33758687
Start: 0 feet bgs

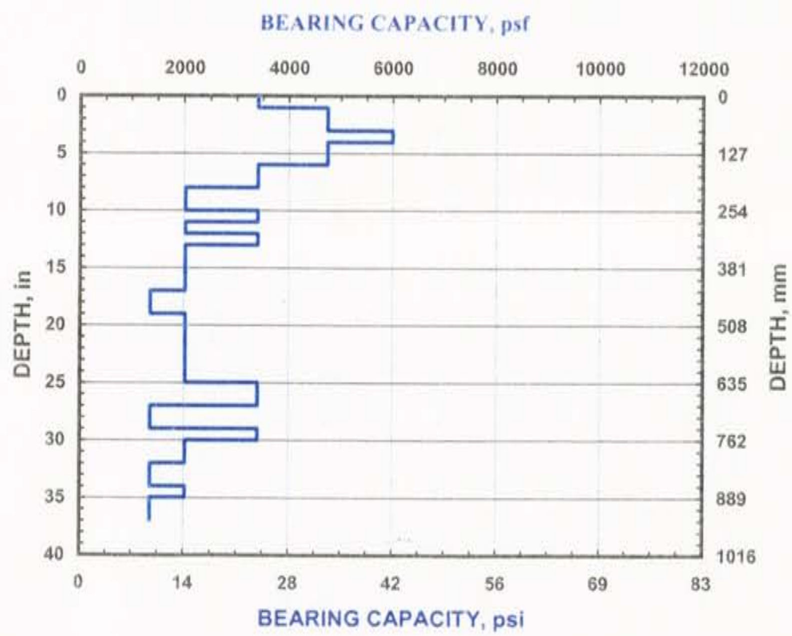
Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
2	25	1	1.0
3	51	1	2.0
3	76	1	3.0
4	102	1	4.0
3	127	1	5.0
3	152	1	6.0
2	178	1	7.0
2	203	1	8.0
1	229	1	9.0
1	254	1	10.0
2	279	1	11.0
1	305	1	12.0
2	330	1	13.0
1	356	1	14.0
1	381	1	15.0
1	406	1	16.0
1	432	1	17.0
1	483	1	19.0
1	508	1	20.0
1	533	1	21.0
1	559	1	22.0
1	584	1	23.0
1	610	1	24.0
1	635	1	25.0
2	660	1	26.0
2	686	1	27.0
1	737	1	29.0
2	762	1	30.0
1	787	1	31.0
1	813	1	32.0
1	864	1	34.0
1	889	1	35.0
1	940	1	37.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





DCP TEST DATA

DCP-11

Project: Saddleback Windfarm
Location: White Salmon, WA

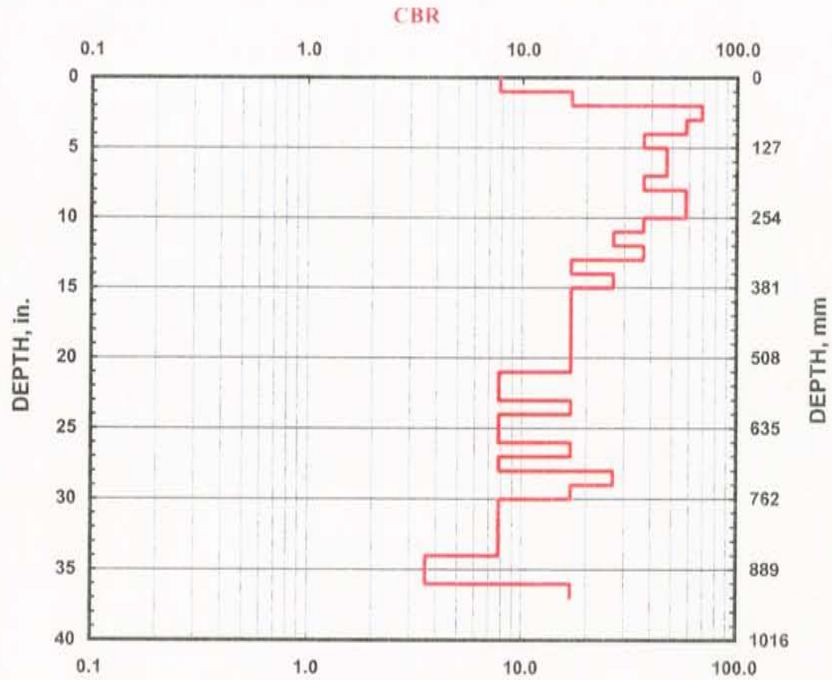
Date: 9/24/2007
Project No.: 33758687
Start: 0 feet bgs

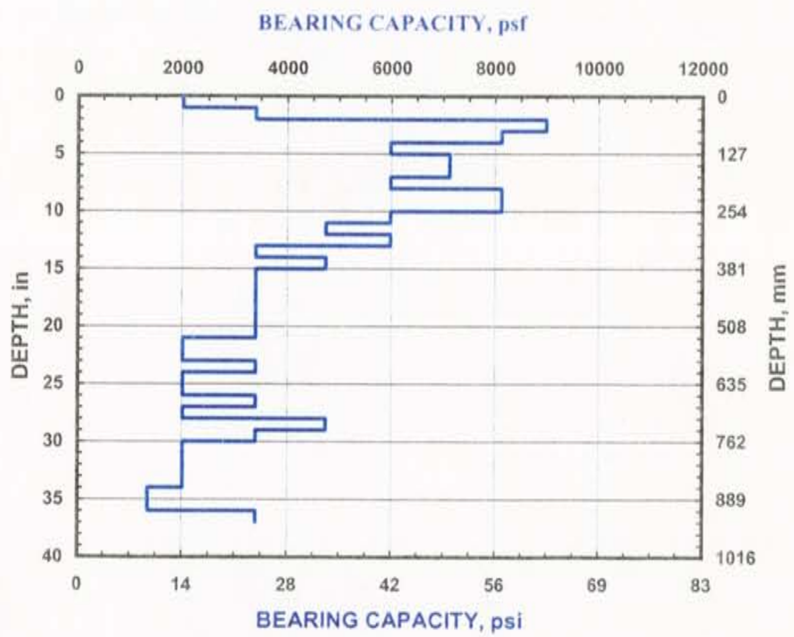
Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
1	25	1	1.0
2	51	1	2.0
7	76	1	3.0
6	102	1	4.0
4	127	1	5.0
5	152	1	6.0
5	178	1	7.0
4	203	1	8.0
6	229	1	9.0
6	254	1	10.0
4	279	1	11.0
3	305	1	12.0
4	330	1	13.0
2	356	1	14.0
3	381	1	15.0
2	406	1	16.0
2	432	1	17.0
2	457	1	18.0
2	483	1	19.0
2	508	1	20.0
2	533	1	21.0
1	559	1	22.0
1	584	1	23.0
2	610	1	24.0
1	635	1	25.0
1	660	1	26.0
2	686	1	27.0
1	711	1	28.0
3	737	1	29.0
2	762	1	30.0
1	787	1	31.0
1	813	1	32.0
1	838	1	33.0
1	864	1	34.0
1	914	1	36.0
2	940	1	37.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





DCP TEST DATA

DCP-12

Project: *Saddleback Windfarm*
Location: White Salmon, WA

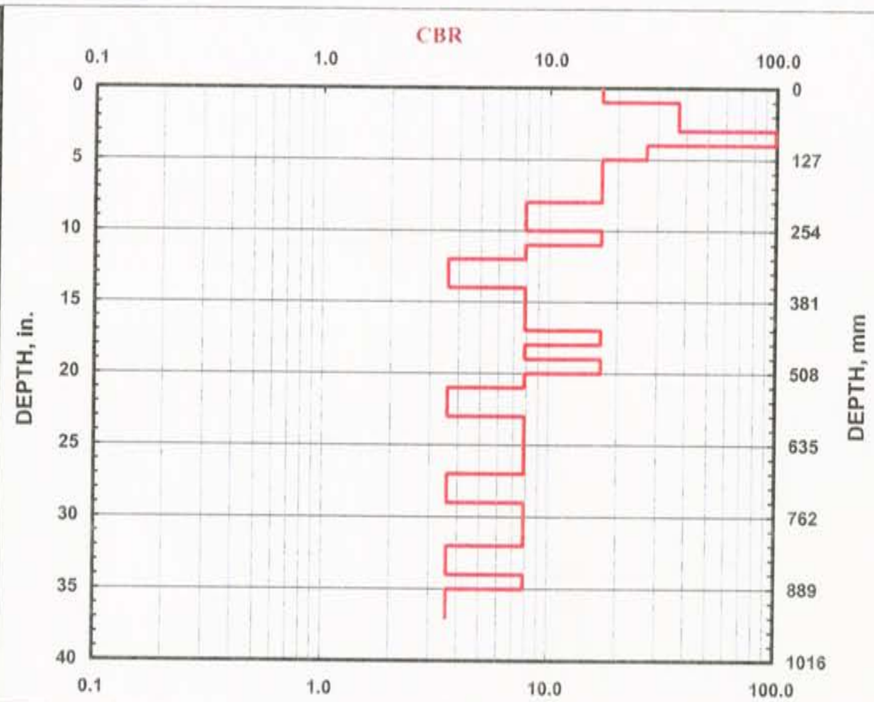
Date: 9/24/2007
Project No.: 33758687
Start: 0 feet bgs

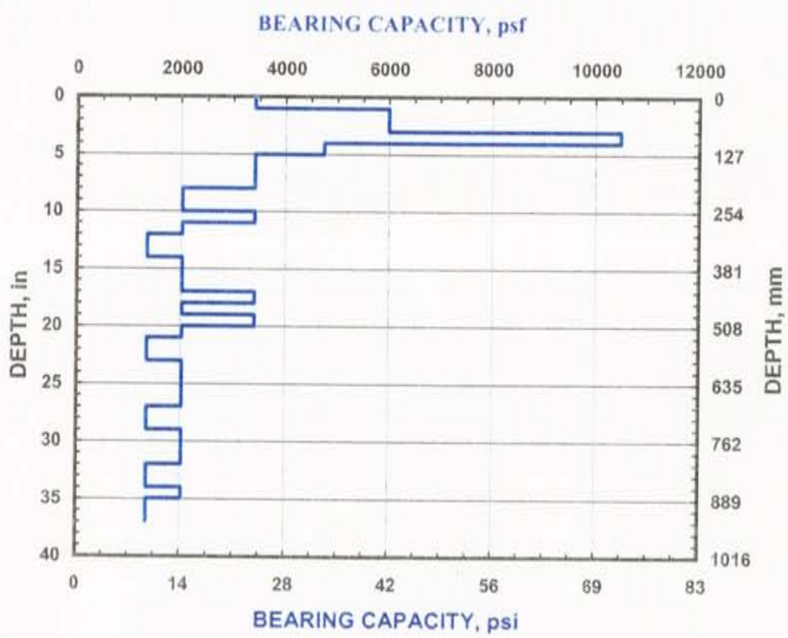
Hammer
○ 10.1 lbs.
◉ 17.6 lbs.
○ Both hammers used

Soil Type
○ CH
○ CL
◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
2	25	1	1.0
4	51	1	2.0
4	76	1	3.0
15	102	1	4.0
3	127	1	5.0
2	152	1	6.0
2	178	1	7.0
2	203	1	8.0
1	229	1	9.0
1	254	1	10.0
2	279	1	11.0
1	305	1	12.0
1	356	1	14.0
1	381	1	15.0
1	406	1	16.0
1	432	1	17.0
2	457	1	18.0
1	483	1	19.0
2	508	1	20.0
1	533	1	21.0
1	584	1	23.0
1	610	1	24.0
1	635	1	25.0
1	660	1	26.0
1	686	1	27.0
1	737	1	29.0
1	762	1	30.0
1	787	1	31.0
1	813	1	32.0
1	864	1	34.0
1	889	1	35.0
1	940	1	37.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





DCP TEST DATA

DCP-13

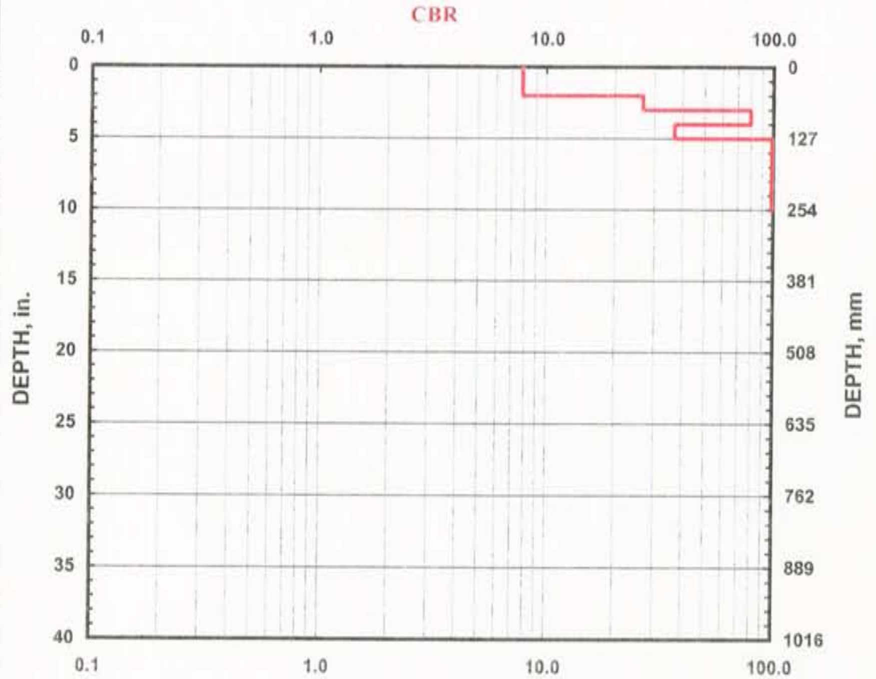
Project: *Saddleback Windfarm*
Location: White Salmon, WA

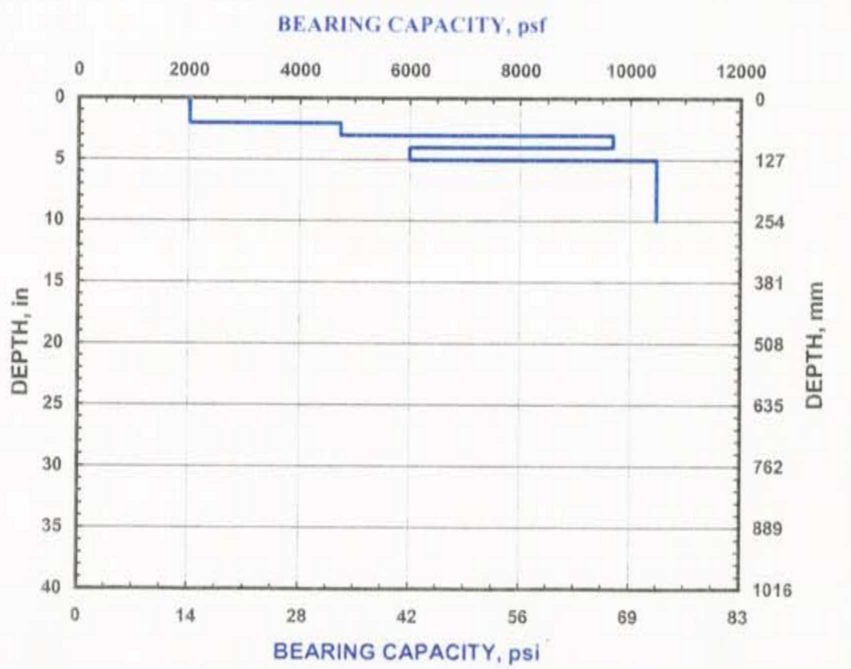
Date: 9/24/2007
Project No.: 33758687
Start: 0 feet bgs

Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
1	25	1	1.0
1	51	1	2.0
3	76	1	3.0
8	102	1	4.0
4	127	1	5.0
17	152	1	6.0
14	178	1	7.0
20	203	1	8.0
15	229	1	9.0
16	254	1	10.0





Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)

DCP TEST DATA

DCP-14

Project: *Saddleback Windfarm*
Location: White Salmon, WA

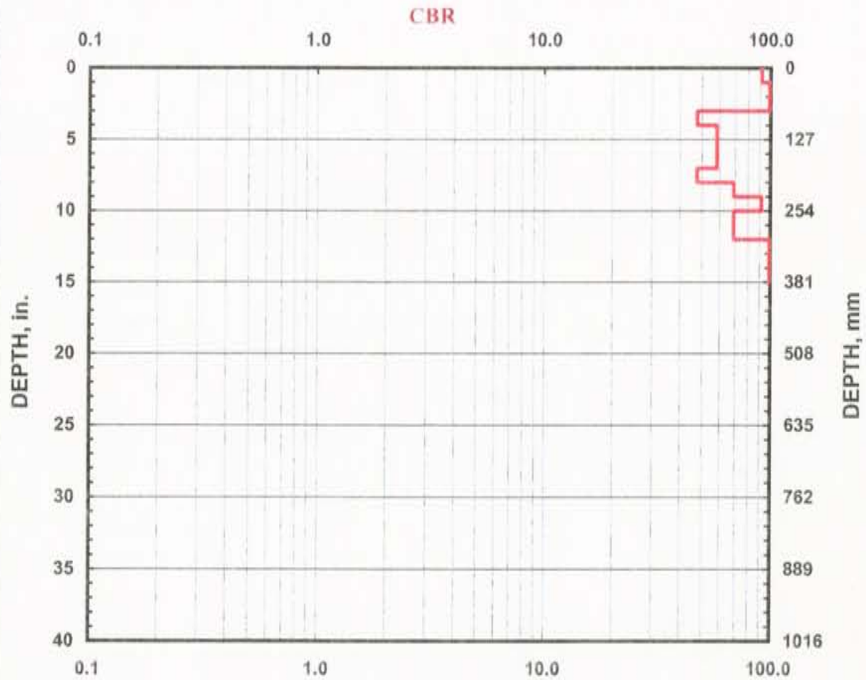
Date: 9/24/2007
Project No.: 33758687
Start: 0 feet bgs

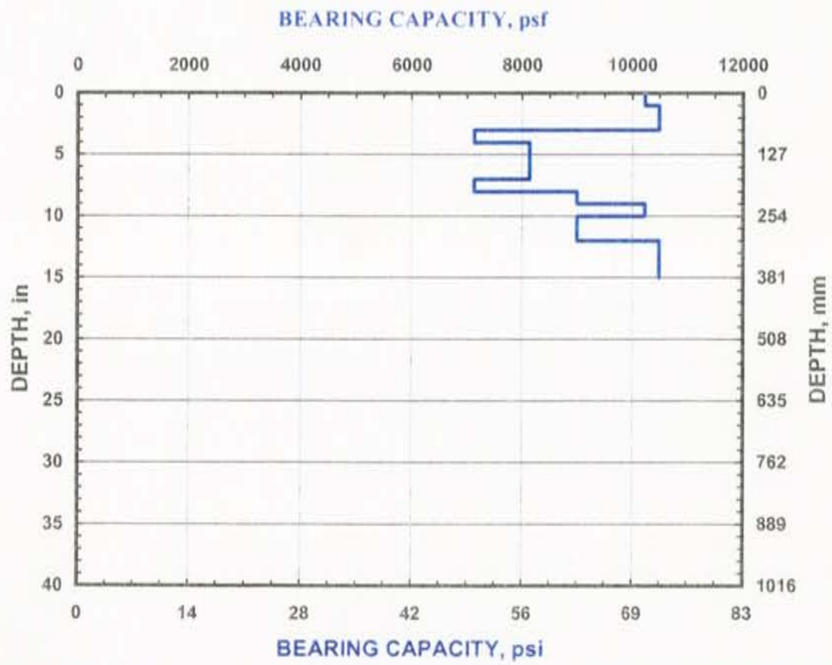
Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
9	25	1	1.0
23	51	1	2.0
11	76	1	3.0
5	102	1	4.0
6	127	1	5.0
6	152	1	6.0
6	178	1	7.0
5	203	1	8.0
7	229	1	9.0
9	254	1	10.0
7	279	1	11.0
7	305	1	12.0
10	330	1	13.0
28	356	1	14.0
50	381	1	15.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





DCP TEST DATA

DCP-15

Project: Saddleback Windfarm
Location: White Salmon, WA

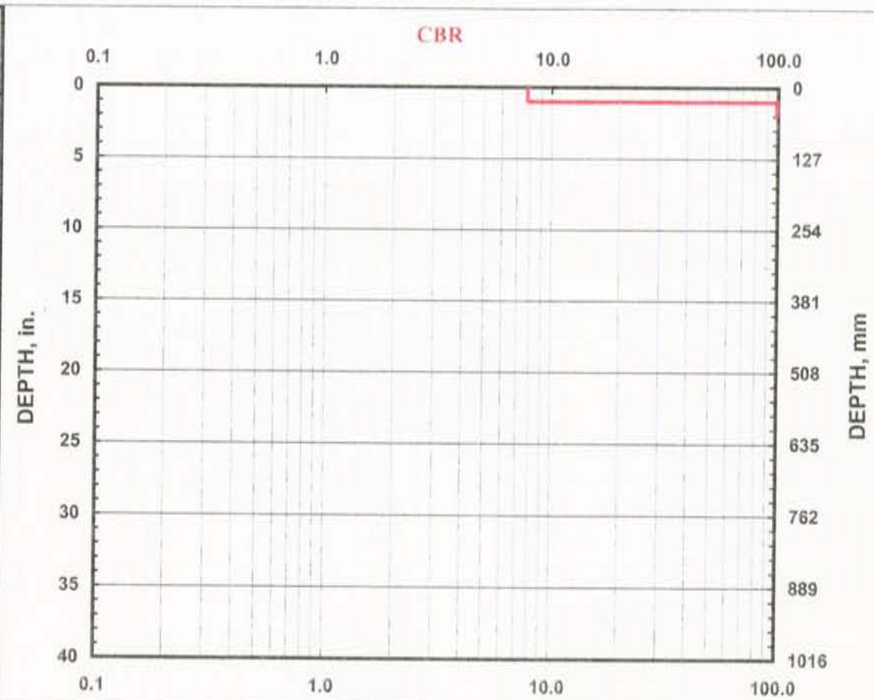
Date: 9/24/2007
Project No.: 33758687
Start: 0 feet bgs

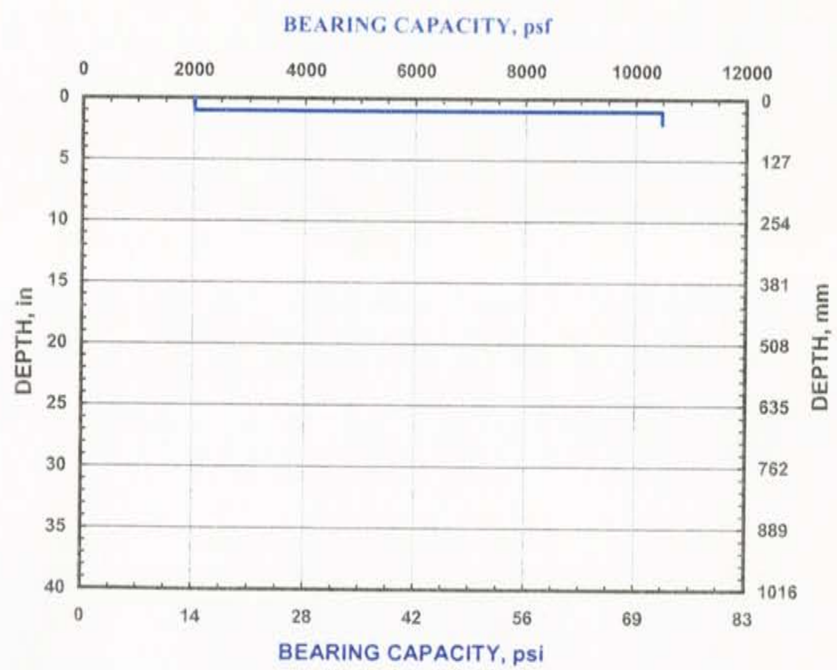
Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
1	25	1	1.0
50	51	1	2.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





DCP TEST DATA

DCP-16

Project: *Saddleback Windfarm*
Location: White Salmon, WA

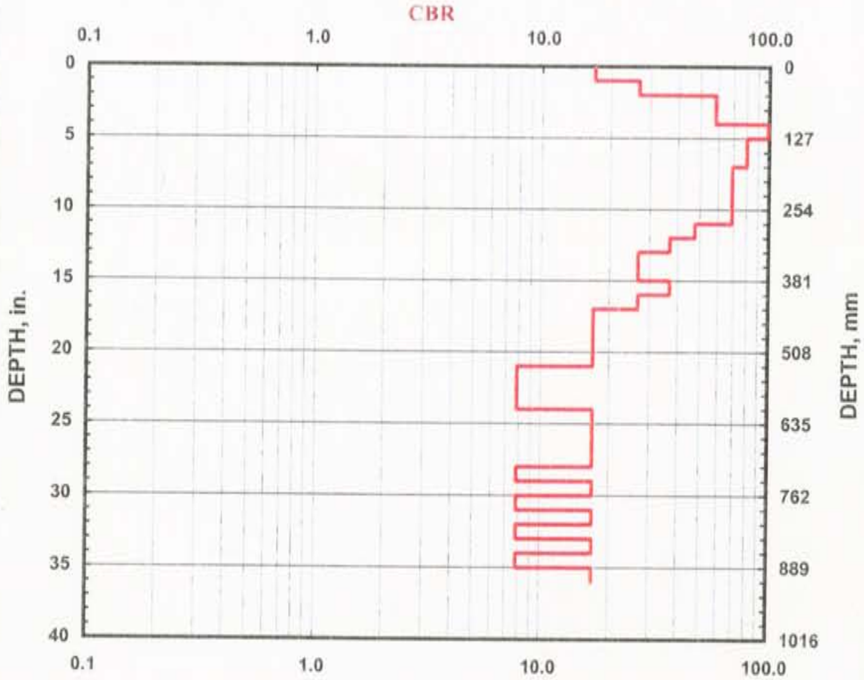
Date: 9/25/2007
Project No.: 33758687
Start: 0 feet bgs

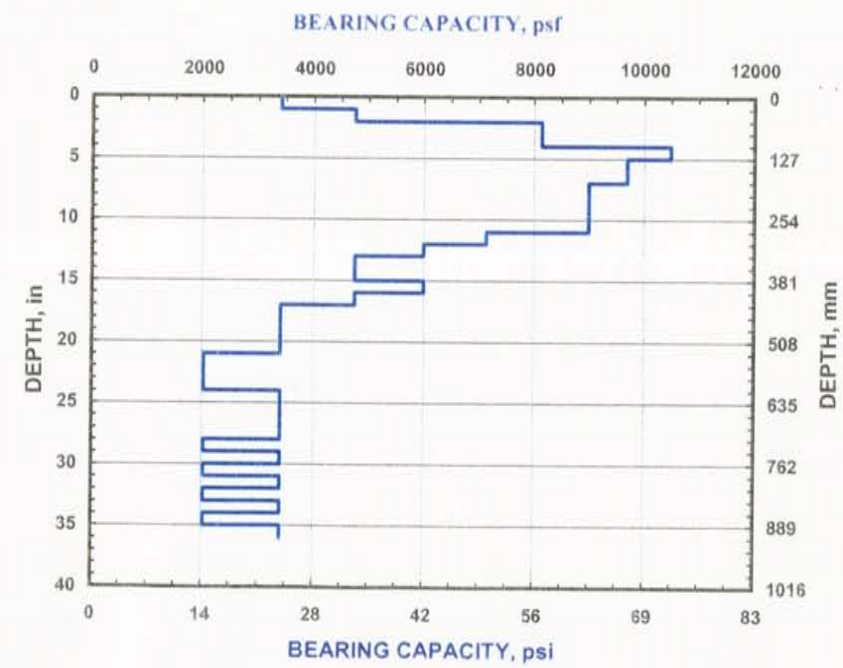
Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
2	25	1	1.0
3	51	1	2.0
6	76	1	3.0
6	102	1	4.0
10	127	1	5.0
8	152	1	6.0
8	178	1	7.0
7	203	1	8.0
7	229	1	9.0
7	254	1	10.0
7	279	1	11.0
5	305	1	12.0
4	330	1	13.0
3	356	1	14.0
3	381	1	15.0
4	406	1	16.0
3	432	1	17.0
2	457	1	18.0
2	483	1	19.0
2	508	1	20.0
2	533	1	21.0
1	559	1	22.0
1	584	1	23.0
1	610	1	24.0
2	635	1	25.0
2	660	1	26.0
2	686	1	27.0
2	711	1	28.0
1	737	1	29.0
2	762	1	30.0
1	787	1	31.0
2	813	1	32.0
1	838	1	33.0
2	864	1	34.0
1	889	1	35.0
2	914	1	36.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





DCP TEST DATA

DCP-17

Project: Saddleback Windfarm
Location: White Salmon, WA

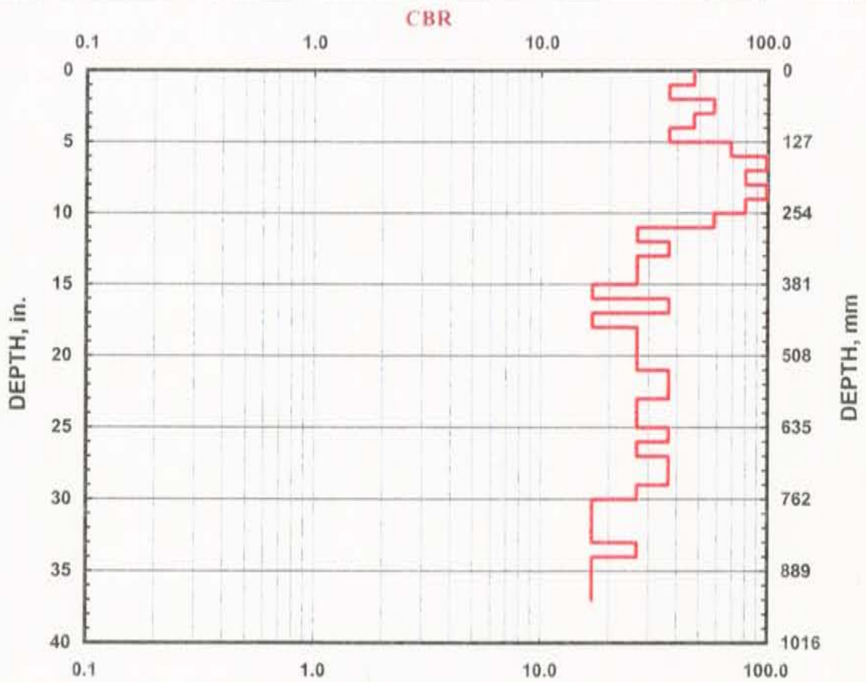
Date: 9/25/2007
Project No.: 33758687
Start: 0 feet bgs

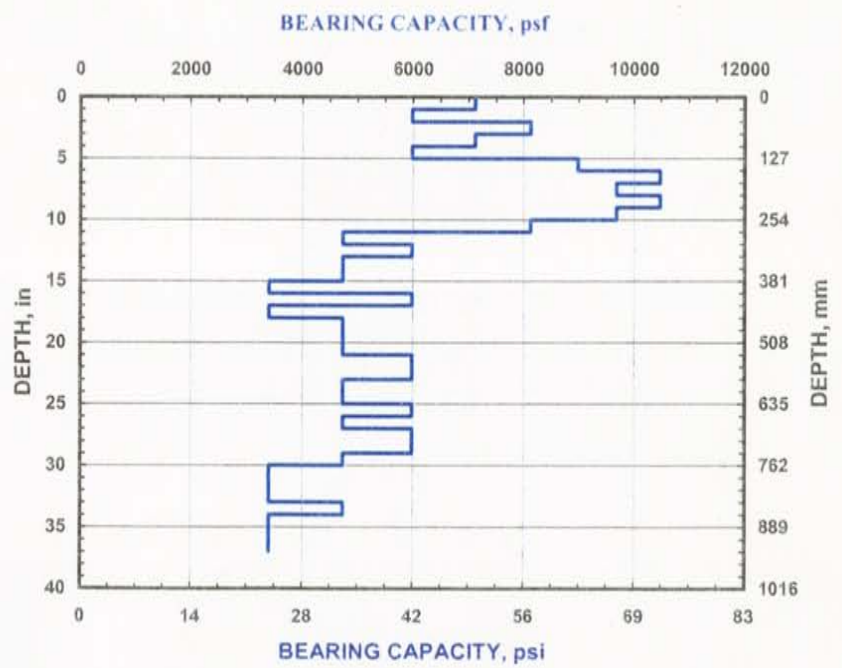
Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
5	25	1	1.0
4	51	1	2.0
6	76	1	3.0
5	102	1	4.0
4	127	1	5.0
7	152	1	6.0
10	178	1	7.0
8	203	1	8.0
20	229	1	9.0
8	254	1	10.0
6	279	1	11.0
3	305	1	12.0
4	330	1	13.0
3	356	1	14.0
3	381	1	15.0
2	406	1	16.0
4	432	1	17.0
2	457	1	18.0
3	483	1	19.0
3	508	1	20.0
3	533	1	21.0
4	559	1	22.0
4	584	1	23.0
3	610	1	24.0
3	635	1	25.0
4	660	1	26.0
3	686	1	27.0
4	711	1	28.0
4	737	1	29.0
3	762	1	30.0
2	787	1	31.0
2	813	1	32.0
2	838	1	33.0
3	864	1	34.0
2	889	1	35.0
2	914	1	36.0
2	940	1	37.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





DCP TEST DATA

DCP-18

Project: Saddleback Windfarm
Location: White Salmon, WA

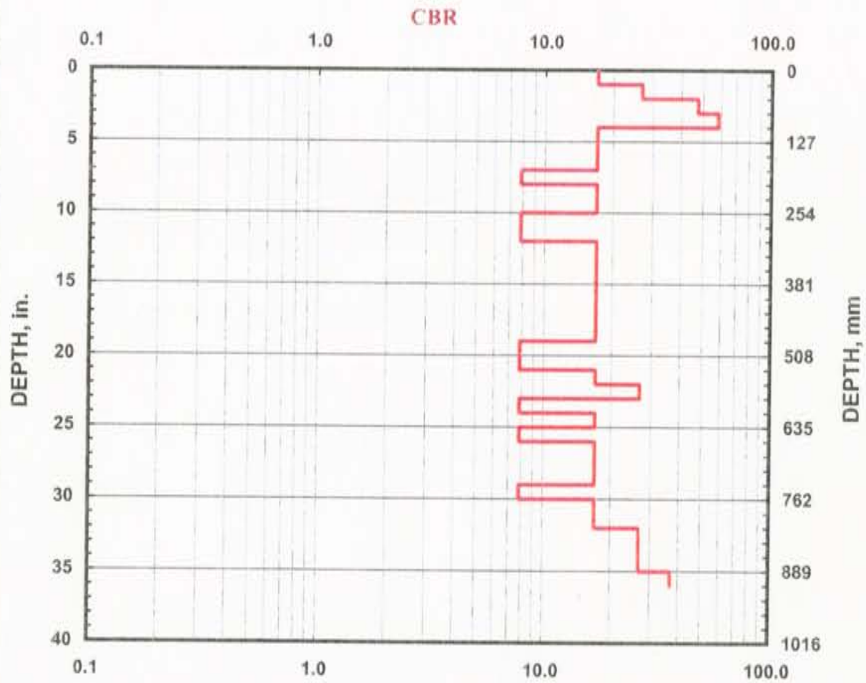
Date: 9/25/2007
Project No.: 33758687
Start: 0 feet bgs

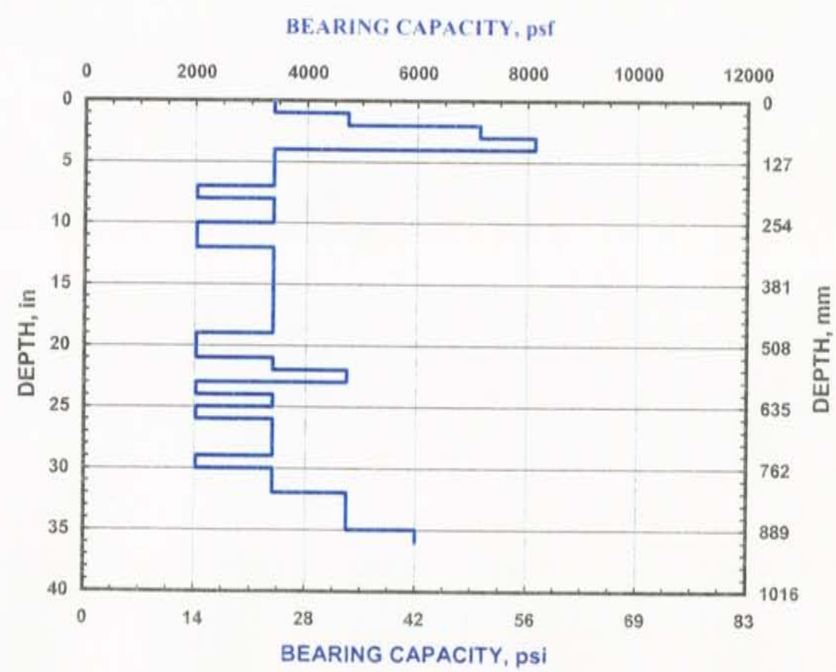
Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
2	25	1	1.0
3	51	1	2.0
5	76	1	3.0
6	102	1	4.0
2	127	1	5.0
2	152	1	6.0
2	178	1	7.0
1	203	1	8.0
2	229	1	9.0
2	254	1	10.0
1	279	1	11.0
1	305	1	12.0
2	330	1	13.0
2	356	1	14.0
2	381	1	15.0
2	406	1	16.0
2	432	1	17.0
2	457	1	18.0
2	483	1	19.0
1	508	1	20.0
1	533	1	21.0
2	559	1	22.0
3	584	1	23.0
1	610	1	24.0
2	635	1	25.0
1	660	1	26.0
2	686	1	27.0
2	711	1	28.0
2	737	1	29.0
1	762	1	30.0
2	787	1	31.0
2	813	1	32.0
3	838	1	33.0
3	864	1	34.0
3	889	1	35.0
4	914	1	36.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





DCP TEST DATA

DCP-19

Project: *Saddleback Windfarm*
Location: White Salmon, WA

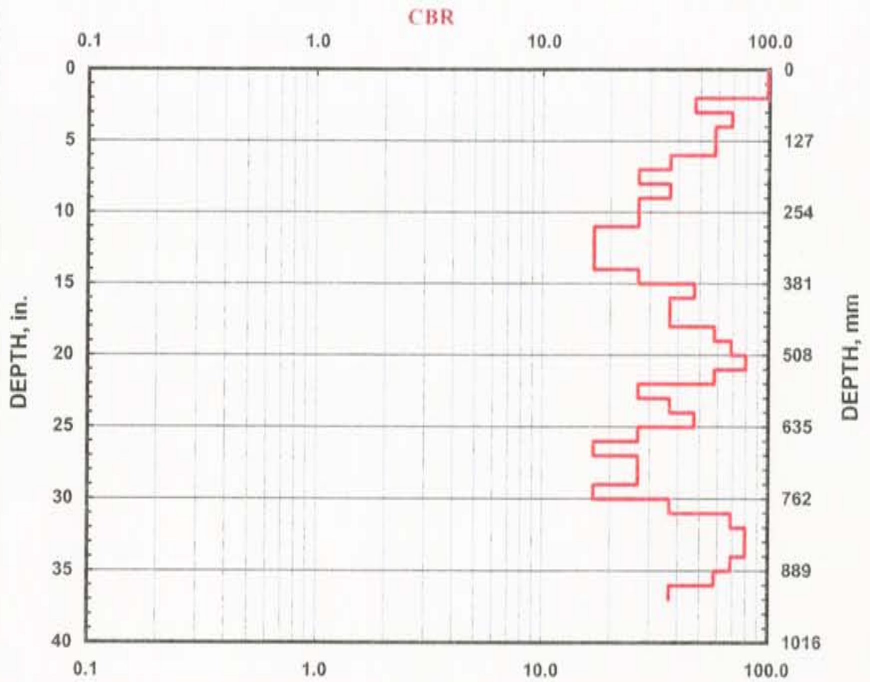
Date: 7/3/2007
Project No.: 33758687
Start: 0 feet bgs

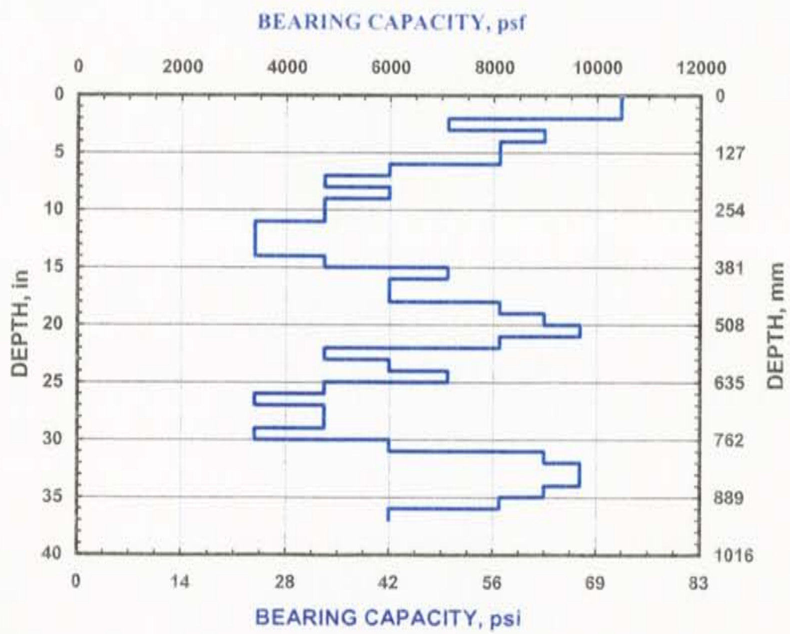
Hammer
○ 10.1 lbs.
◉ 17.6 lbs.
○ Both hammers used

Soil Type
○ CH
○ CL
◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
12	25	1	1.0
10	51	1	2.0
5	76	1	3.0
7	102	1	4.0
6	127	1	5.0
6	152	1	6.0
4	178	1	7.0
3	203	1	8.0
4	229	1	9.0
3	254	1	10.0
3	279	1	11.0
2	305	1	12.0
2	330	1	13.0
2	356	1	14.0
3	381	1	15.0
5	406	1	16.0
4	432	1	17.0
4	457	1	18.0
6	483	1	19.0
7	508	1	20.0
8	533	1	21.0
6	559	1	22.0
3	584	1	23.0
4	610	1	24.0
5	635	1	25.0
3	660	1	26.0
2	686	1	27.0
3	711	1	28.0
3	737	1	29.0
2	762	1	30.0
4	787	1	31.0
7	813	1	32.0
8	838	1	33.0
8	864	1	34.0
7	889	1	35.0
6	914	1	36.0
4	940	1	37.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





DCP TEST DATA

DCP-20

Project: Saddleback Windfarm
Location: White Salmon, WA

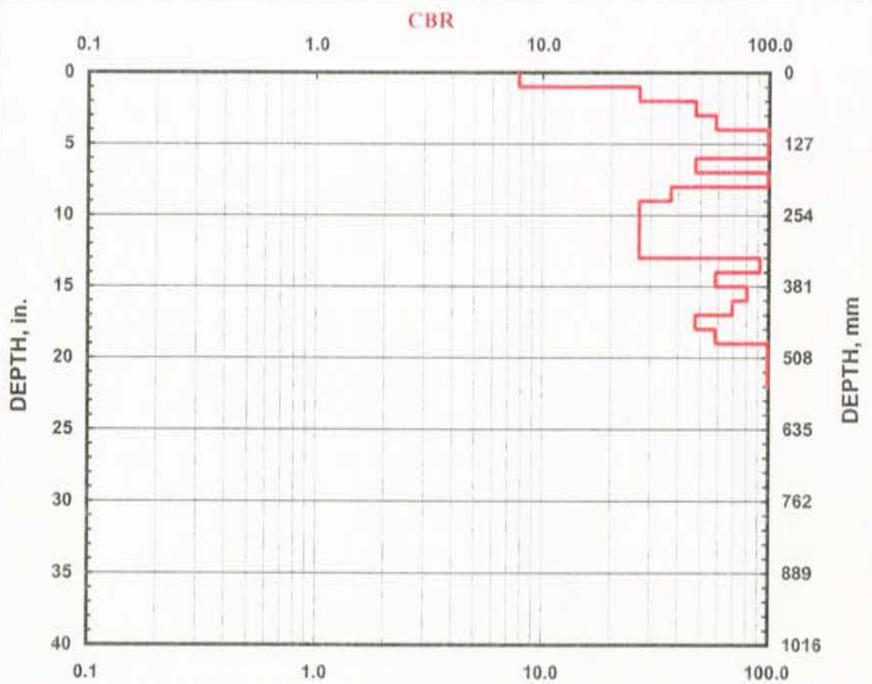
Date: 7/3/2007
Project No.: 33758687
Start: 0 feet bgs

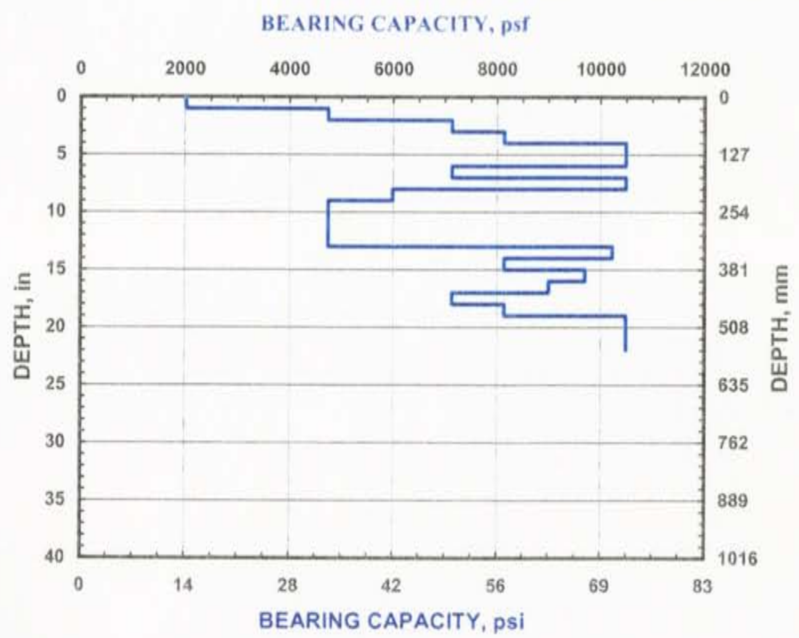
Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
1	25	1	1.0
3	51	1	2.0
5	76	1	3.0
6	102	1	4.0
15	127	1	5.0
13	152	1	6.0
5	178	1	7.0
10	203	1	8.0
4	229	1	9.0
3	254	1	10.0
3	279	1	11.0
3	305	1	12.0
3	330	1	13.0
9	356	1	14.0
6	381	1	15.0
8	406	1	16.0
7	432	1	17.0
5	457	1	18.0
6	483	1	19.0
10	508	1	20.0
20	533	1	21.0
50	559	1	22.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





DCP TEST DATA

DCP-21

Project: Saddleback Windfarm
Location: White Salmon, WA

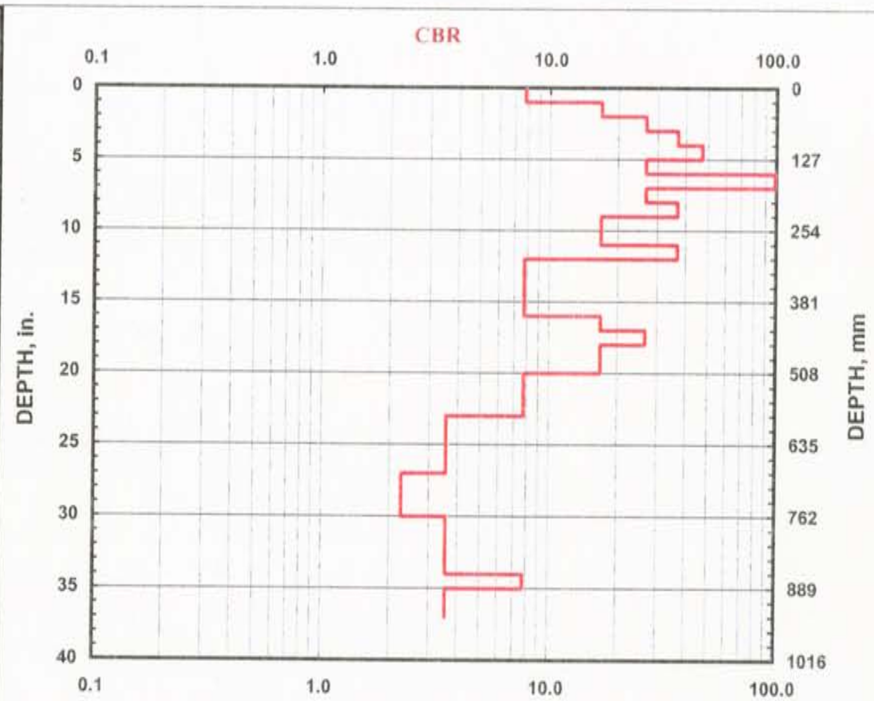
Hammer: ○ 10.1 lbs. ◉ 17.6 lbs. ○ Both hammers used

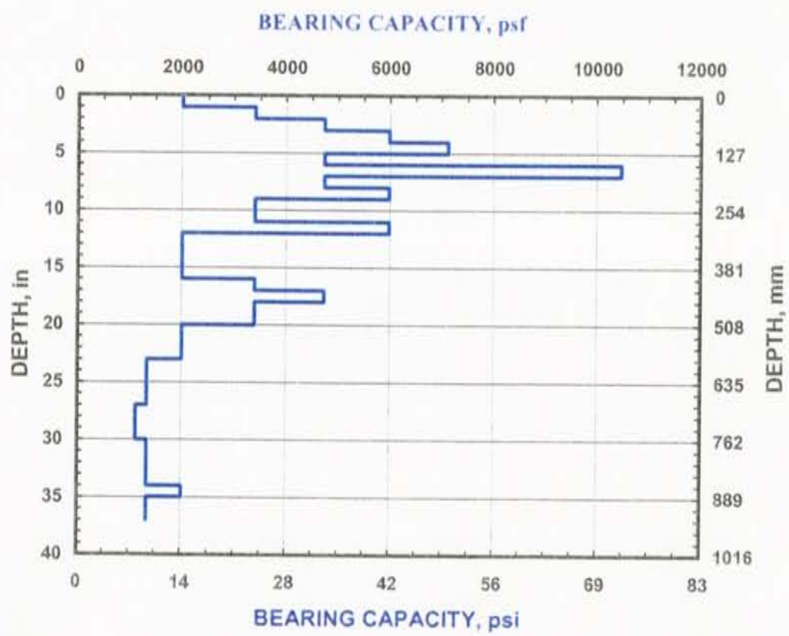
Soil Type: ○ CH ○ CL ◉ All other soils

Date: 9/25/2007
Project No.: 33758687
Start: 0 feet bgs

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
1	25	1	1.0
2	51	1	2.0
3	76	1	3.0
4	102	1	4.0
5	127	1	5.0
3	152	1	6.0
11	178	1	7.0
3	203	1	8.0
4	229	1	9.0
2	254	1	10.0
2	279	1	11.0
4	305	1	12.0
1	330	1	13.0
1	356	1	14.0
1	381	1	15.0
1	406	1	16.0
2	432	1	17.0
3	457	1	18.0
2	483	1	19.0
2	508	1	20.0
1	533	1	21.0
1	559	1	22.0
1	584	1	23.0
1	635	1	25.0
1	686	1	27.0
1	762	1	30.0
1	813	1	32.0
1	864	1	34.0
1	889	1	35.0
1	940	1	37.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





DCP TEST DATA

DCP-22

Project: *Saddleback Windfarm*
Location: White Salmon, WA

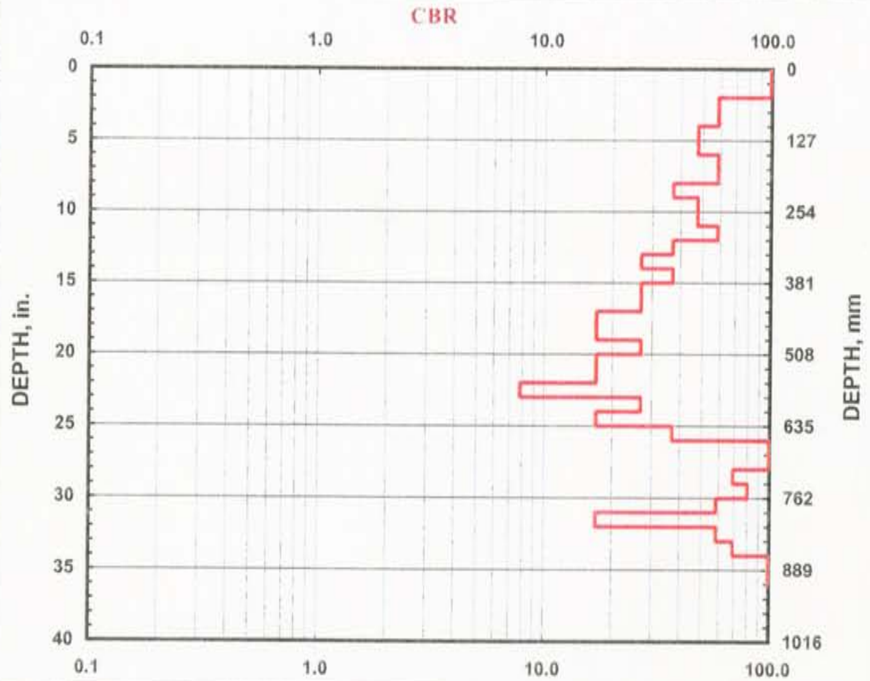
Date: 9/25/2007
Project No.: 33758687
Start: 0 feet bgs

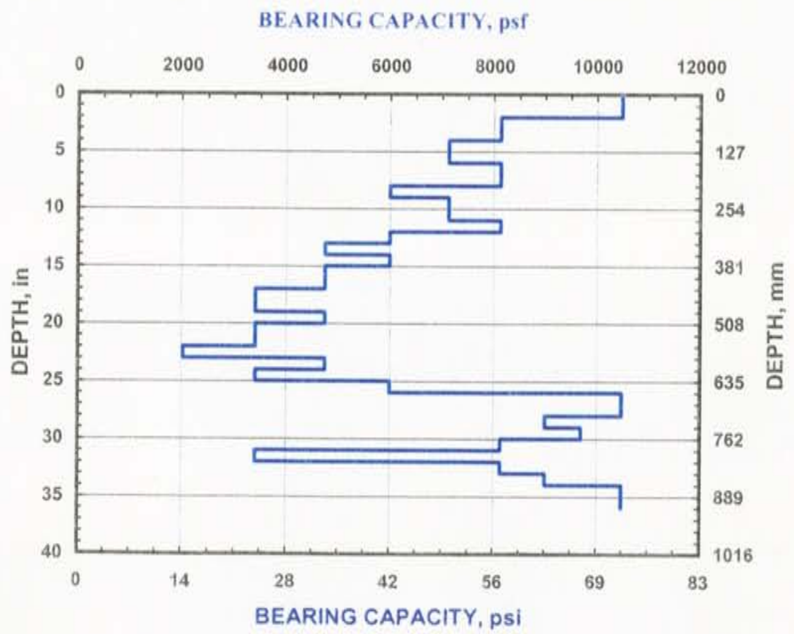
Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
15	25	1	1.0
10	51	1	2.0
6	76	1	3.0
6	102	1	4.0
5	127	1	5.0
5	152	1	6.0
6	178	1	7.0
6	203	1	8.0
4	229	1	9.0
5	254	1	10.0
5	279	1	11.0
6	305	1	12.0
4	330	1	13.0
3	356	1	14.0
4	381	1	15.0
3	406	1	16.0
3	432	1	17.0
2	457	1	18.0
2	483	1	19.0
3	508	1	20.0
2	533	1	21.0
2	559	1	22.0
1	584	1	23.0
3	610	1	24.0
2	635	1	25.0
4	660	1	26.0
12	686	1	27.0
12	711	1	28.0
7	737	1	29.0
8	762	1	30.0
6	787	1	31.0
2	813	1	32.0
6	838	1	33.0
7	864	1	34.0
15	889	1	35.0
15	914	1	36.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





DCP TEST DATA

DCP-23

Project: *Saddleback Windfarm*
Location: White Salmon, WA

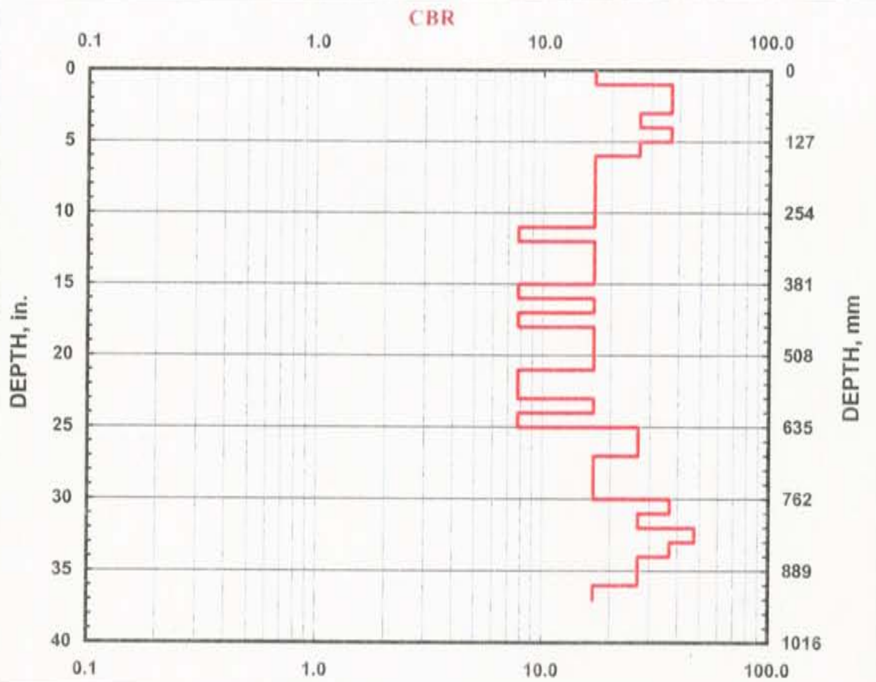
Date: 9/25/2007
Project No.: 33758687
Start: 0 feet bgs

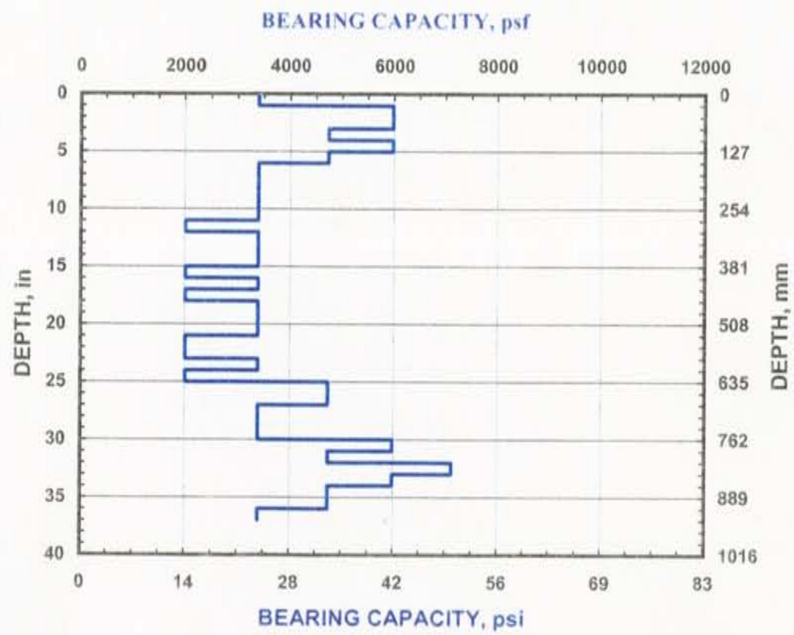
Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
2	25	1	1.0
4	51	1	2.0
4	76	1	3.0
3	102	1	4.0
4	127	1	5.0
3	152	1	6.0
2	178	1	7.0
2	203	1	8.0
2	229	1	9.0
2	254	1	10.0
2	279	1	11.0
1	305	1	12.0
2	330	1	13.0
2	356	1	14.0
2	381	1	15.0
1	406	1	16.0
2	432	1	17.0
1	457	1	18.0
2	483	1	19.0
2	508	1	20.0
2	533	1	21.0
1	559	1	22.0
1	584	1	23.0
2	610	1	24.0
1	635	1	25.0
3	660	1	26.0
3	686	1	27.0
2	711	1	28.0
2	737	1	29.0
2	762	1	30.0
4	787	1	31.0
3	813	1	32.0
5	838	1	33.0
4	864	1	34.0
3	889	1	35.0
3	914	1	36.0
2	940	1	37.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





DCP TEST DATA

DCP-24

Project: Saddleback Windfarm
Location: White Salmon, WA

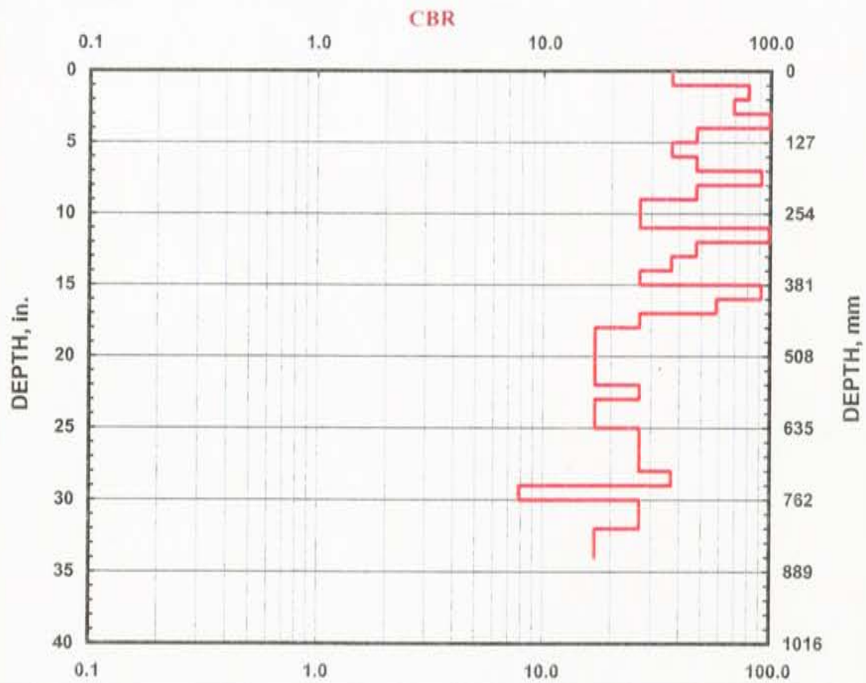
Date: 9/25/2007
Project No.: 33758687
Start: 0 feet bgs

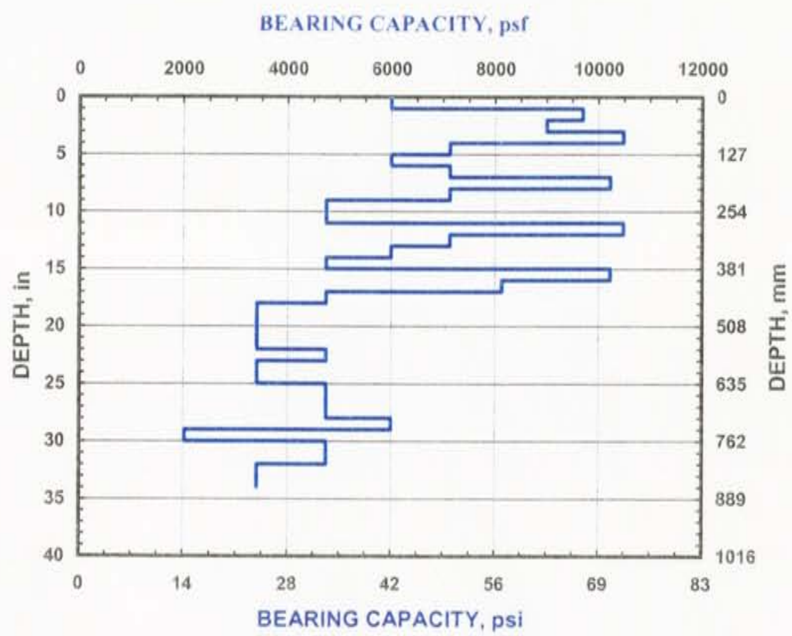
Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
4	25	1	1.0
8	51	1	2.0
7	76	1	3.0
10	102	1	4.0
5	127	1	5.0
4	152	1	6.0
5	178	1	7.0
9	203	1	8.0
5	229	1	9.0
3	254	1	10.0
3	279	1	11.0
10	305	1	12.0
5	330	1	13.0
4	356	1	14.0
3	381	1	15.0
9	406	1	16.0
6	432	1	17.0
3	457	1	18.0
2	483	1	19.0
2	508	1	20.0
2	533	1	21.0
2	559	1	22.0
3	584	1	23.0
2	610	1	24.0
2	635	1	25.0
3	660	1	26.0
3	686	1	27.0
3	711	1	28.0
4	737	1	29.0
1	762	1	30.0
3	787	1	31.0
3	813	1	32.0
2	838	1	33.0
2	864	1	34.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





DCP TEST DATA

DCP-25

Project: *Saddleback Windfarm*
Location: White Salmon, WA

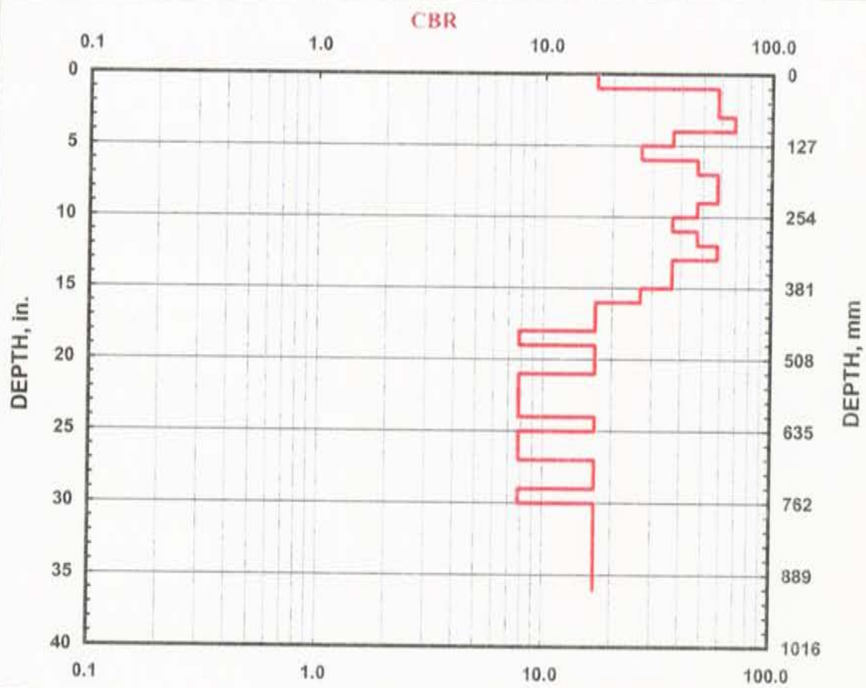
Date: 9/25/2007
Project No.: 33758687
Start: 0 feet bgs

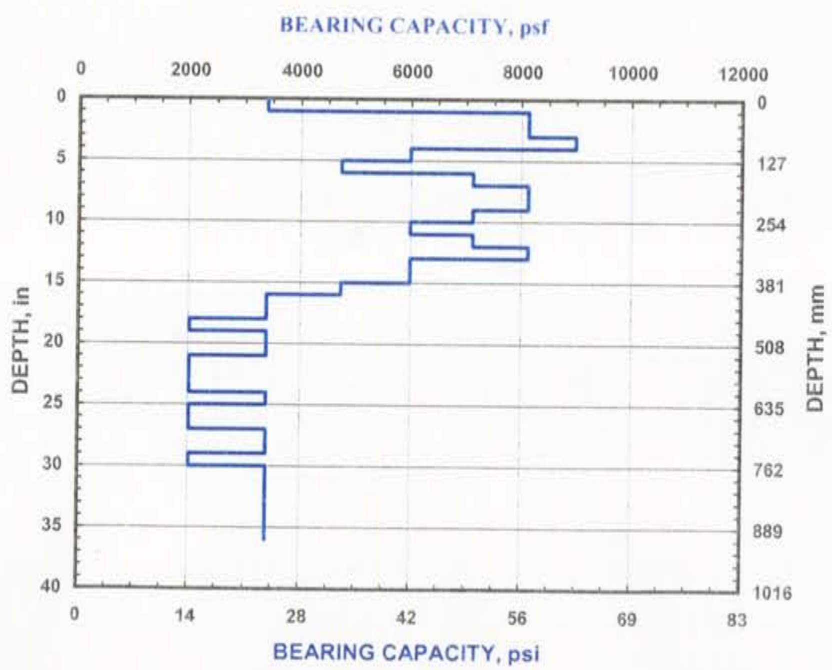
Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
2	25	1	1.0
6	51	1	2.0
6	76	1	3.0
7	102	1	4.0
4	127	1	5.0
3	152	1	6.0
5	178	1	7.0
6	203	1	8.0
6	229	1	9.0
5	254	1	10.0
4	279	1	11.0
5	305	1	12.0
6	330	1	13.0
4	356	1	14.0
4	381	1	15.0
3	406	1	16.0
2	432	1	17.0
2	457	1	18.0
1	483	1	19.0
2	508	1	20.0
2	533	1	21.0
1	559	1	22.0
1	584	1	23.0
1	610	1	24.0
2	635	1	25.0
1	660	1	26.0
1	686	1	27.0
2	711	1	28.0
2	737	1	29.0
1	762	1	30.0
2	787	1	31.0
2	813	1	32.0
2	838	1	33.0
2	864	1	34.0
2	889	1	35.0
2	914	1	36.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





DCP TEST DATA

DCP-26

Project: Saddleback Windfarm
Location: White Salmon, WA

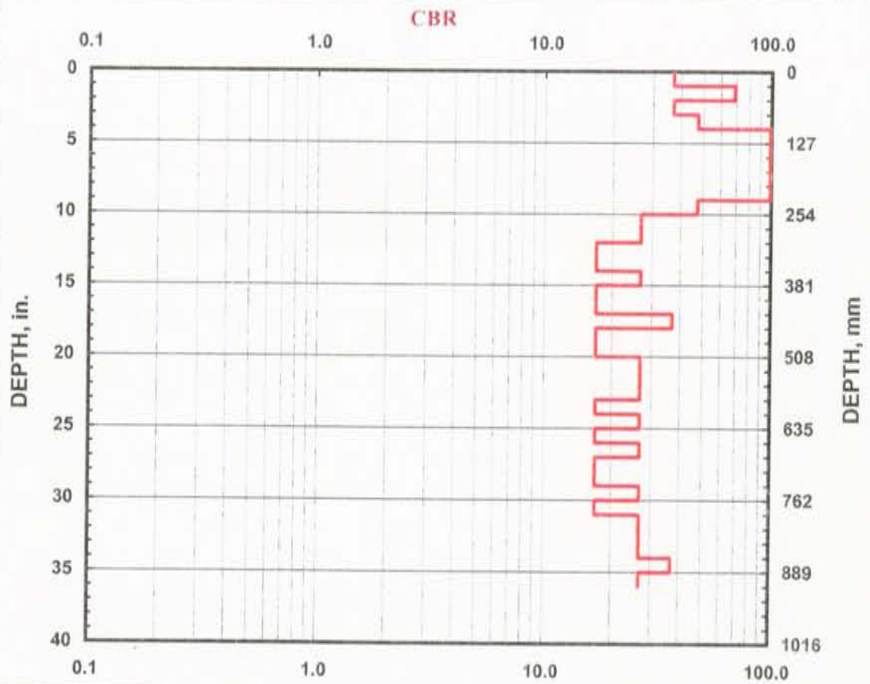
Date: 9/25/2007
Project No.: 33758687
Start: 0 feet bgs

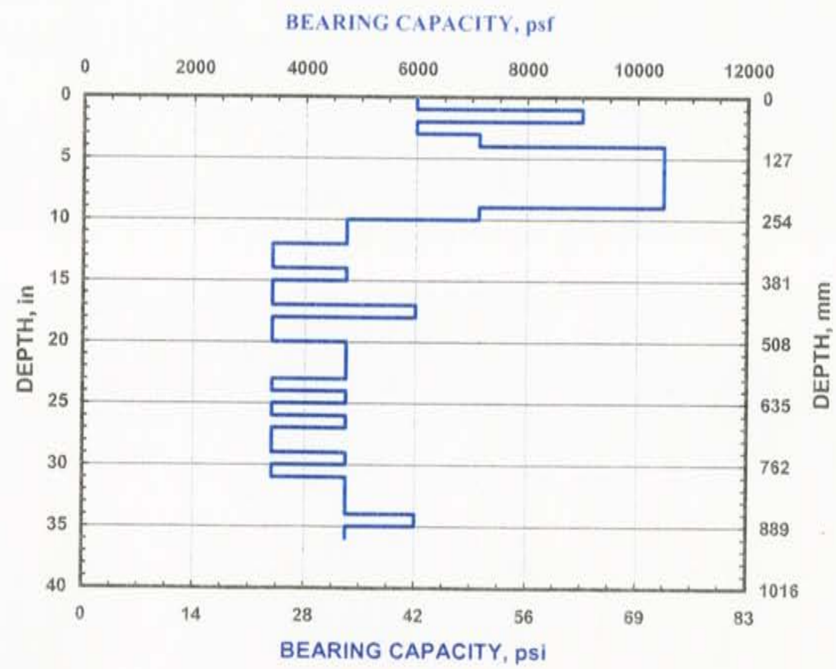
Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
4	25	1	1.0
7	51	1	2.0
4	76	1	3.0
5	102	1	4.0
21	127	1	5.0
18	152	1	6.0
21	178	1	7.0
19	203	1	8.0
12	229	1	9.0
5	254	1	10.0
3	279	1	11.0
3	305	1	12.0
2	330	1	13.0
2	356	1	14.0
3	381	1	15.0
2	406	1	16.0
2	432	1	17.0
4	457	1	18.0
2	483	1	19.0
2	508	1	20.0
3	533	1	21.0
3	559	1	22.0
3	584	1	23.0
2	610	1	24.0
3	635	1	25.0
2	660	1	26.0
3	686	1	27.0
2	711	1	28.0
2	737	1	29.0
3	762	1	30.0
2	787	1	31.0
3	813	1	32.0
3	838	1	33.0
3	864	1	34.0
4	889	1	35.0
3	914	1	36.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





DCP TEST DATA

DCP-27

Project: Saddleback Windfarm
Location: White Salmon, WA

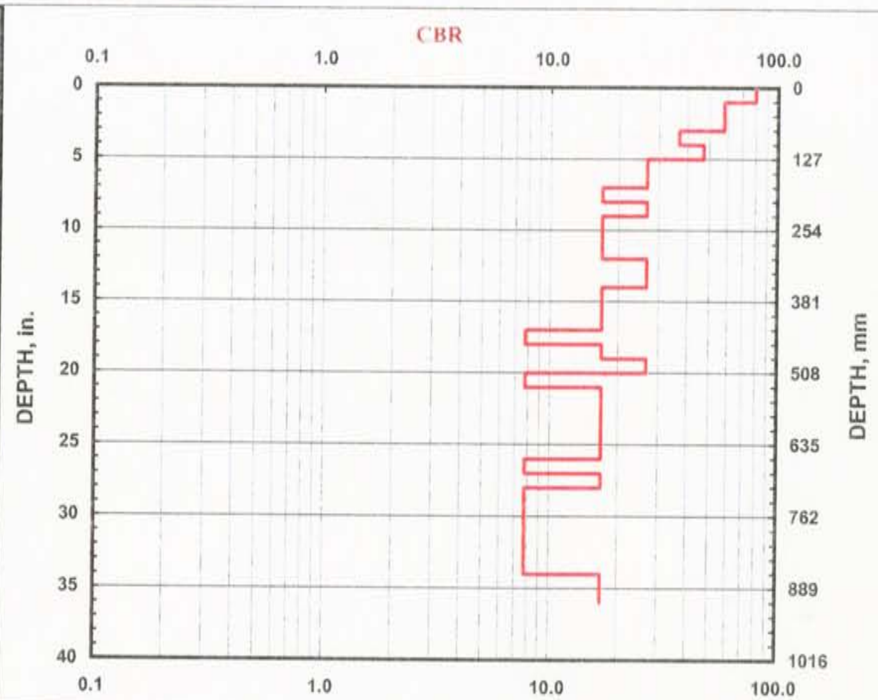
Date: 9/25/2007
Project No.: 33758687
Start: 0 feet bgs

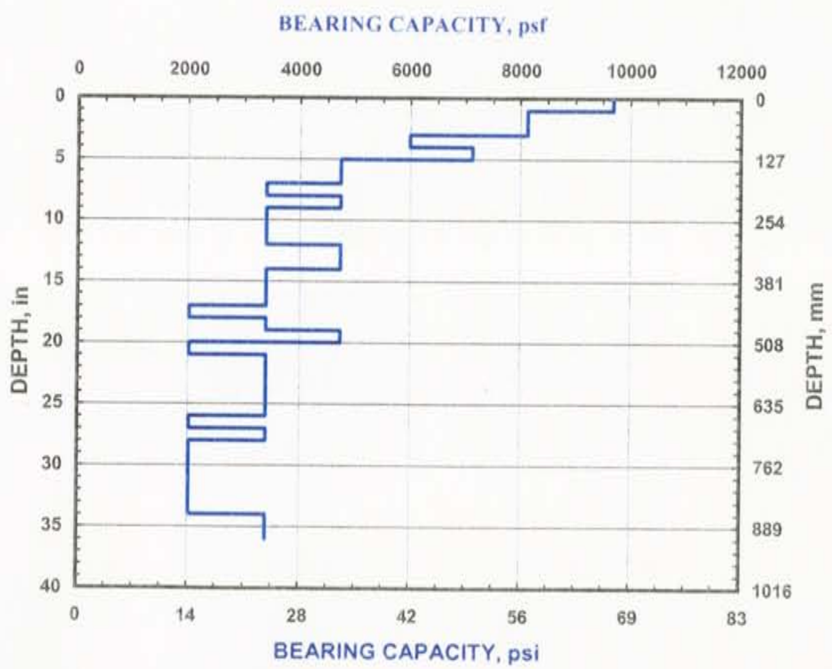
Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
8	25	1	1.0
6	51	1	2.0
6	76	1	3.0
4	102	1	4.0
5	127	1	5.0
3	152	1	6.0
3	178	1	7.0
2	203	1	8.0
3	229	1	9.0
2	254	1	10.0
2	279	1	11.0
2	305	1	12.0
3	330	1	13.0
3	356	1	14.0
2	381	1	15.0
2	406	1	16.0
2	432	1	17.0
1	457	1	18.0
2	483	1	19.0
3	508	1	20.0
1	533	1	21.0
2	559	1	22.0
2	584	1	23.0
2	610	1	24.0
2	635	1	25.0
2	660	1	26.0
1	686	1	27.0
2	711	1	28.0
1	737	1	29.0
1	762	1	30.0
1	787	1	31.0
1	813	1	32.0
1	838	1	33.0
1	864	1	34.0
2	889	1	35.0
2	914	1	36.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





DCP TEST DATA

DCP-28

Project: *Saddleback Windfarm*
Location: White Salmon, WA

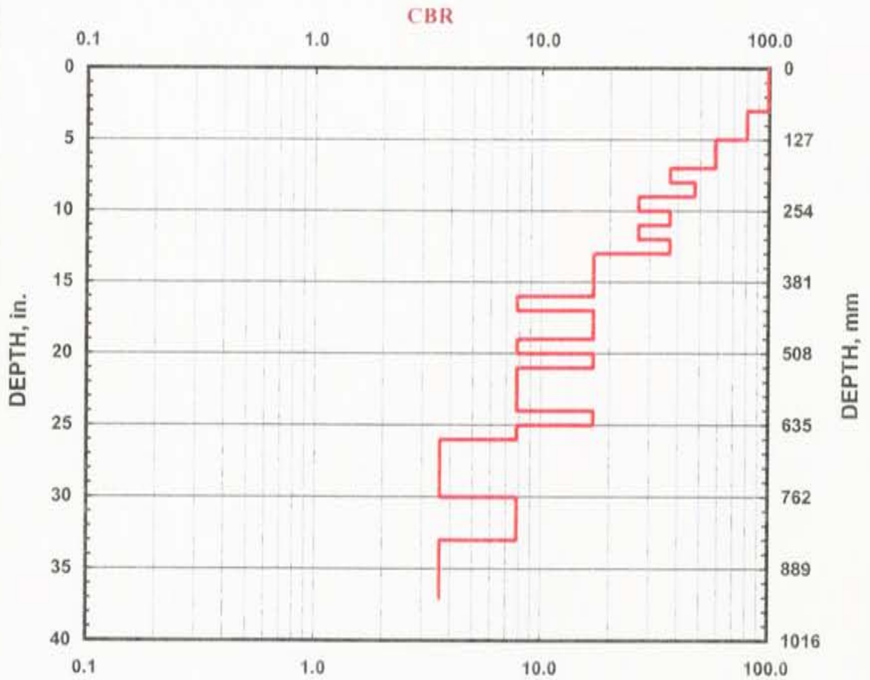
Date: 9/25/2007
Project No.: 33758687
Start: 0 feet bgs

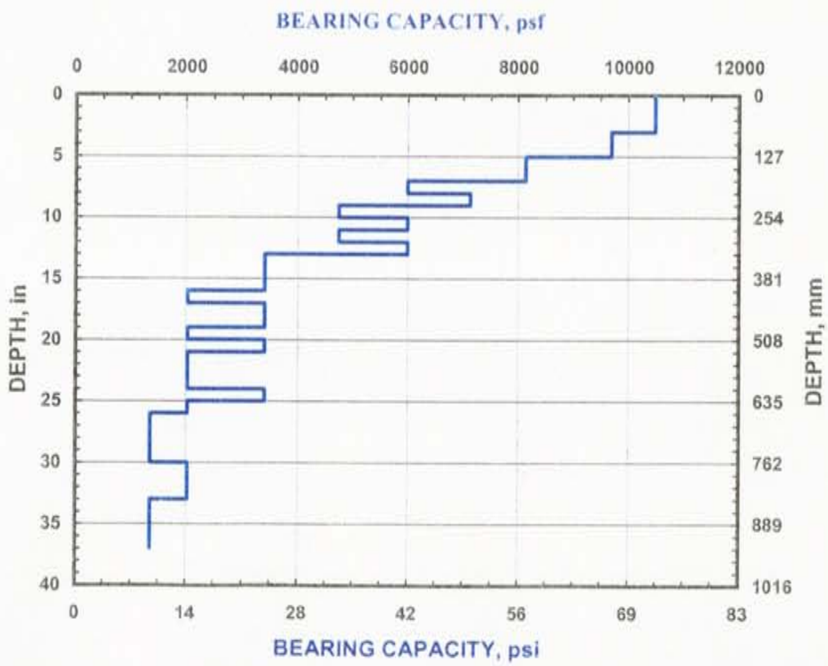
Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
12	25	1	1.0
14	51	1	2.0
15	76	1	3.0
8	102	1	4.0
8	127	1	5.0
6	152	1	6.0
6	178	1	7.0
4	203	1	8.0
5	229	1	9.0
3	254	1	10.0
4	279	1	11.0
3	305	1	12.0
4	330	1	13.0
2	356	1	14.0
2	381	1	15.0
2	406	1	16.0
1	432	1	17.0
2	457	1	18.0
2	483	1	19.0
1	508	1	20.0
2	533	1	21.0
1	559	1	22.0
1	584	1	23.0
1	610	1	24.0
2	635	1	25.0
1	660	1	26.0
1	711	1	28.0
1	762	1	30.0
1	787	1	31.0
1	813	1	32.0
1	838	1	33.0
1	889	1	35.0
1	940	1	37.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





DCP TEST DATA

DCP-29

Project: *Saddleback Windfarm*
Location: White Salmon, WA

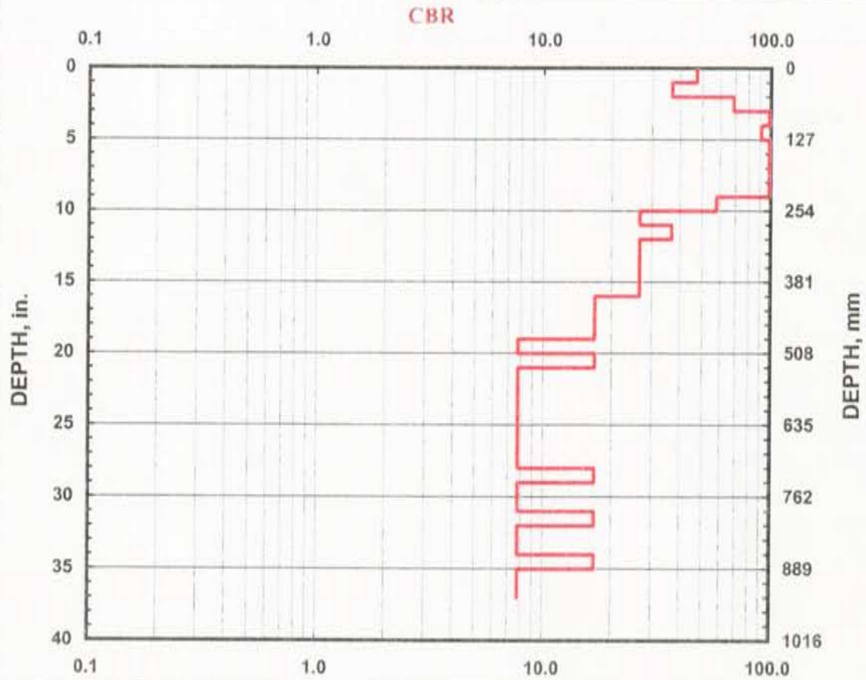
Date: 9/25/2007
Project No.: 33758687
Start: 0 feet bgs

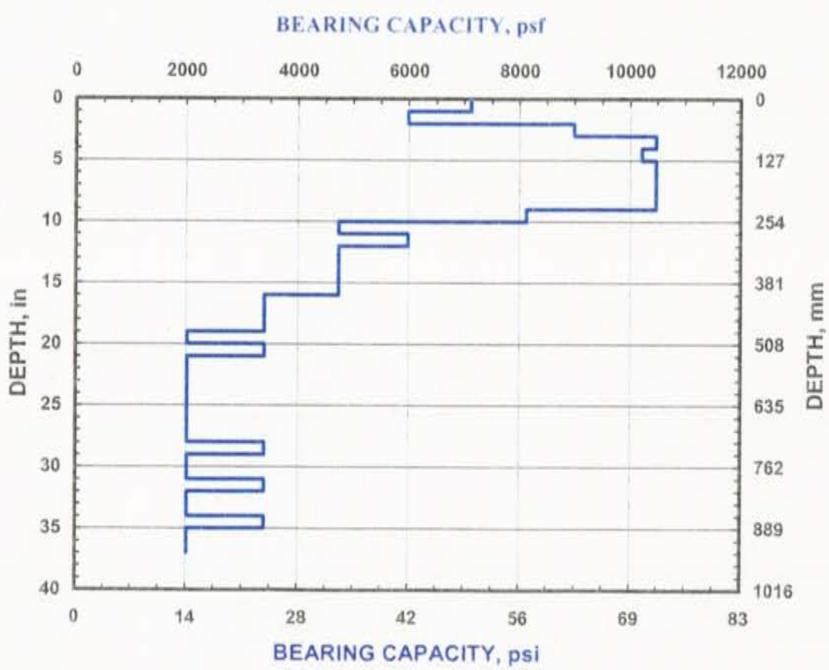
Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
5	25	1	1.0
4	51	1	2.0
7	76	1	3.0
11	102	1	4.0
9	127	1	5.0
10	152	1	6.0
14	178	1	7.0
13	203	1	8.0
19	229	1	9.0
6	254	1	10.0
3	279	1	11.0
4	305	1	12.0
3	330	1	13.0
3	356	1	14.0
3	381	1	15.0
3	406	1	16.0
2	432	1	17.0
2	457	1	18.0
2	483	1	19.0
1	508	1	20.0
2	533	1	21.0
1	559	1	22.0
1	584	1	23.0
1	610	1	24.0
1	635	1	25.0
1	660	1	26.0
1	686	1	27.0
1	711	1	28.0
2	737	1	29.0
1	762	1	30.0
1	787	1	31.0
2	813	1	32.0
1	838	1	33.0
1	864	1	34.0
2	889	1	35.0
1	914	1	36.0
1	940	1	37.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





DCP TEST DATA

DCP-30

Project: Saddleback Windfarm
Location: White Salmon, WA

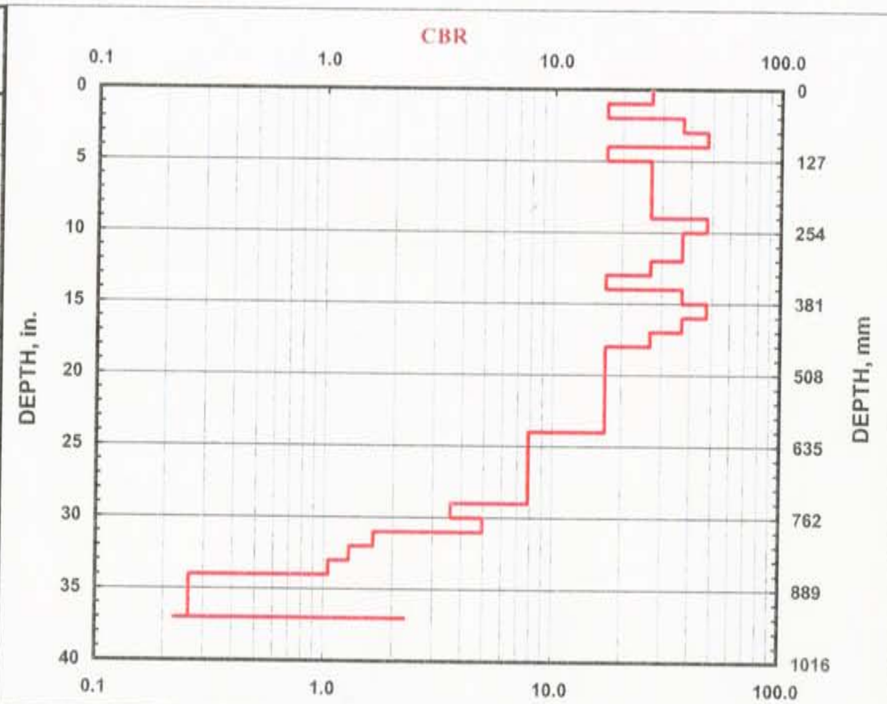
Date: 9/25/2007
Project No.: 33758687
Start: 0 feet bgs

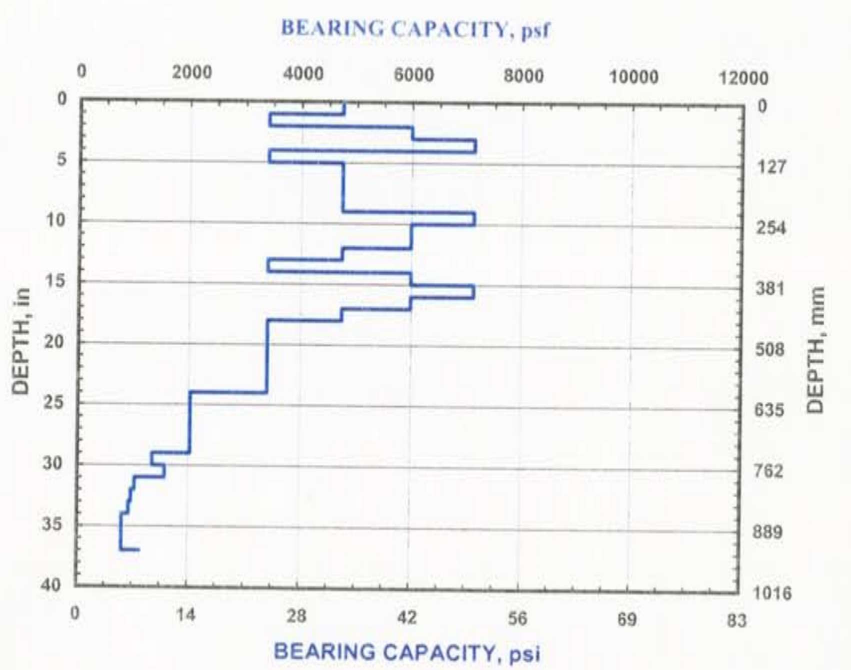
Hammer
- ○ 10.1 lbs.
- ◉ 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer	Scale Reading (in)
			0.0
3	25	1	1.0
2	51	1	2.0
4	76	1	3.0
5	102	1	4.0
2	127	1	5.0
3	152	1	6.0
3	178	1	7.0
3	203	1	8.0
3	229	1	9.0
5	254	1	10.0
4	279	1	11.0
4	305	1	12.0
3	330	1	13.0
2	356	1	14.0
4	381	1	15.0
5	406	1	16.0
4	432	1	17.0
3	457	1	18.0
2	483	1	19.0
2	508	1	20.0
2	533	1	21.0
2	559	1	22.0
2	584	1	23.0
2	610	1	24.0
1	635	1	25.0
1	660	1	26.0
1	686	1	27.0
1	711	1	28.0
1	737	2	29.0
1	762	3	30.0
2	787	4	31.0
1	813	5	32.0
1	838	6	33.0
1	864	7	34.0
1	940	8	37.0

Bearing Capacity based on correlations of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955)





APPENDIX D
LANDSLIDE HAZARDS

LANDSLIDE HAZARDS REPORT

Pursuant to Skamania County Code, Title 21A, Chapter 21A.06 - Landslide Hazard Areas, URS has conducted a preliminary landslide hazard evaluation of the proposed Saddleback Wind Energy (SWE) project wind turbine site. The project location is shown on Figure C-1.

Landslide Hazard Report Methodology

A URS Licensed Engineering Geologist conducted a site specific landslide hazard investigation. The investigation consisted of:

- Review of Sections of the County Code that address Geologically Hazardous Areas;
- Review of existing available topographic, geologic and soils literature and maps;
- Analysis of project-specific stereo aerial photographs;
- Review of project test pit logs and soil samples;
- A one day site reconnaissance.

According to the County Code, the primary criteria for landslide hazard designations are: presence of pre-existing, known mappable landslides; slope angle; and/or composition of the near-surface soils or rock.

URS has created a color-coded map of the study area using an existing USGS 10 meter digital terrain model (DTM) to segregate slopes into three categories: slopes less than 20%; slopes between 20% and 30%; and slopes greater than 30%. We then superimposed the United States Department of Agriculture, National Resources Conservation Services (NRCS) soil survey map onto the slope map to provide soil type information. The resulting Landslide Hazard Map is presented herein as Figure C-1.

Landslide Hazard Area Delineation

Skamania County recognizes three classes of landslide hazard areas (LHAs). Class I (Severe) LHAs are considered to present a severe landslide hazard and are distinguished as areas of known mappable landslide deposits which have been designated landslide hazard areas by the local legislative body. Class II (High) LHAs are areas with slopes between twenty and thirty percent that are underlain by soils that consist largely of silt, clay or bedrock, and all areas with slopes greater than thirty percent. Class III (Moderate) LHAs are areas with slopes between twenty percent and thirty percent not included in Class II.

URS reviewed available geologic and soils literature to develop a landslide hazard classification for the proposed SWE project. An existing published regional geologic map (partially recreated in Figure 3 of the main text of this report) indicates a large landslide in the northeast corner of the study area underlying Tower Line 'C'. Review of stereo photographs of the area where the landslide deposits are mapped, coupled with a site reconnaissance, indicate that there is little geomorphic evidence for landslide activity such as obvious scarps, hummocky or benched terrain, lobate toe areas, or redirected watercourses. No deep subsurface investigations have been carried out at the site to date, but future explorations in support of design for the turbine tower foundations will provide subsurface information that will provide information regarding the presence, or lack of, landslide deposits in the area. Based on our preliminary investigation, there does not appear to be any area of the site that meets Skamania County's criteria for a (Class I) LHA.

Class II LHAs are shown in red on Figure C-1. The Class II LHAs at the site are predominantly associated with the steep slopes to the west of proposed Tower Lines 'A' and 'B'. There are also steep slopes to the east of the 7 southernmost 'A' Line towers, and on both sides of Tower Line 'C'.

Although none of the proposed turbines are located within Class II LHAs, several of the towers along the western side of the project site (Tower Lines 'A' and 'B') are located along ridgelines with descending slopes that are locally greater than 35 degrees (70%). The heads of some of the drainages along these slopes are arcuate indicating possible mass-wasting activity such as landslides, debris flows, and / or earthflows.

Based on aerial photo and field observations, the primary mass wasting process below the ridgelines appears to be debris flows and soil creep. No evidence for deep-seated, block failure type landslides was observed. Local surficial creep of near-surface soils is indicated by the presence of pistol-butted trees on some of the slopes, primarily on the descending slope west of the northern portion of Tower Line 'A'. Other slopes have mature conifer stands that indicated little or no soil creep. Further subsurface investigation in support of final tower foundation design will help determine if there are weak rock or soil layers that could contribute to more deep-seated failure of the ridges and provide information on the quality of the rock mass underlying the ridgelines.

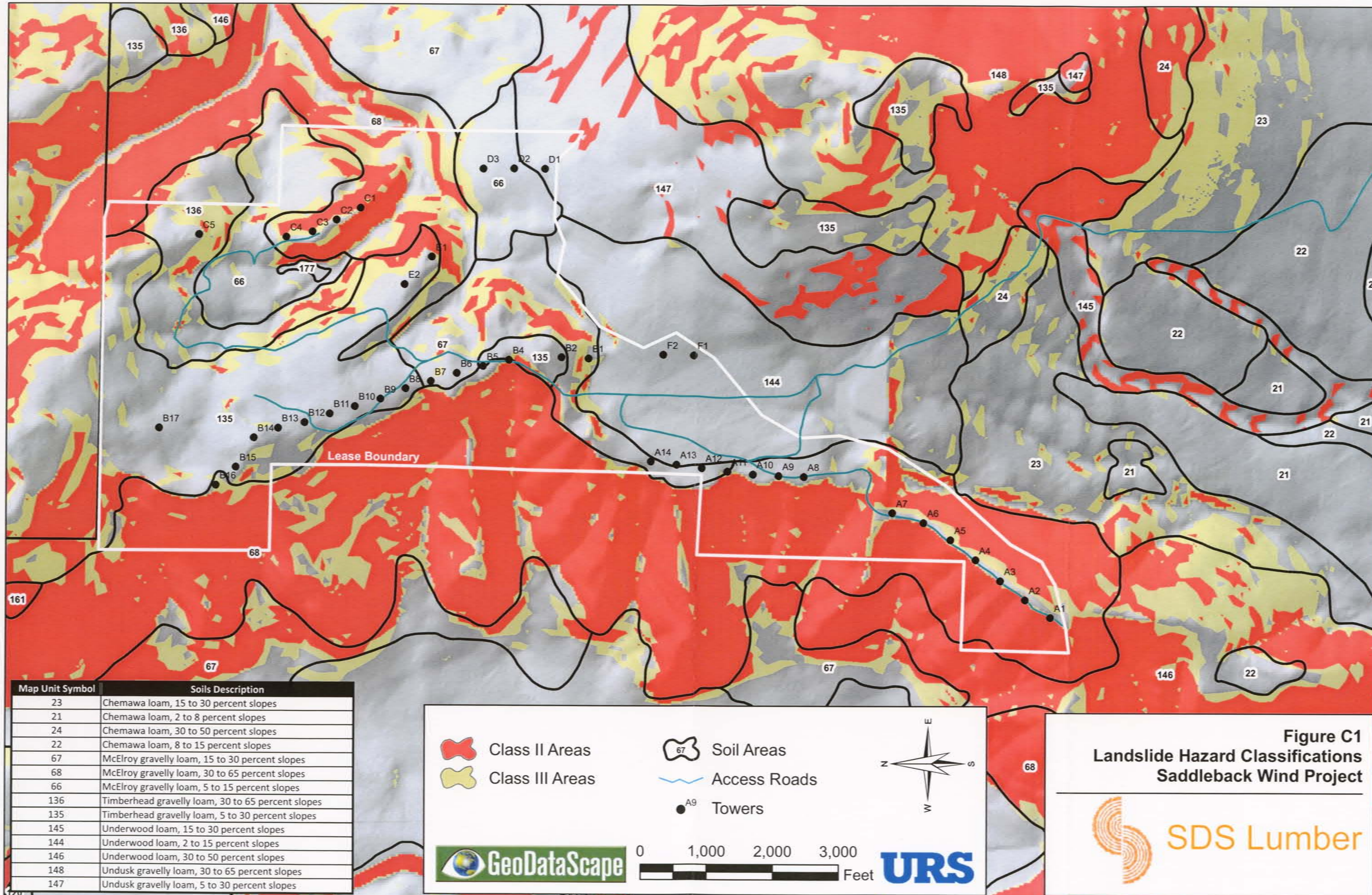
It appears that the primary concern for towers located adjacent to the Class II LHAs is the potential for headward erosion of the steep drainages by debris or earth flow processes. Erosion rates of these drainages are unknown, but no obvious recent mass wasting features were observed in the aerial photos or during the site reconnaissance.

Class III LHAs have been delineated adjacent to proposed wind turbines along the southern 'A' Line, and the 'C' Line. Class I LHAs are not anticipated to have any impact on the proposed facilities due to the robust nature of the proposed foundation designs.

Conclusions and Recommendations

URS has conducted a Landslide Hazard Evaluation for the proposed SWE wind farm project in Southeast Skamania County. The evaluation has identified several areas where the proposed wind turbine generators are located adjacent to slopes that meet Skamania County's criteria for Class II and Class III Landslide Hazard Areas. The primary hazard to the proposed towers appears to be the potential for exposure to headward erosion of steep drainages on the slopes below some of the tower locations. Exposure of the towers to headward erosion of the steep slope drainages can be minimized by providing maximum possible setbacks from the tops of the steep slopes and / or by siting the turbines along portions of the ridgelines that are above intervening spur ridges. The most critical area of exposure to Class II LHAs is the narrow ridge at the southern portion of the 'A' Line.

It is URS's opinion that the proposed SWE facilities can be constructed and operated without danger to human life or the surrounding environment due to landslide hazards.



Map Unit Symbol	Soils Description
23	Chemawa loam, 15 to 30 percent slopes
21	Chemawa loam, 2 to 8 percent slopes
24	Chemawa loam, 30 to 50 percent slopes
22	Chemawa loam, 8 to 15 percent slopes
67	McElroy gravelly loam, 15 to 30 percent slopes
68	McElroy gravelly loam, 30 to 65 percent slopes
66	McElroy gravelly loam, 5 to 15 percent slopes
136	Timberhead gravelly loam, 30 to 65 percent slopes
135	Timberhead gravelly loam, 5 to 30 percent slopes
145	Underwood loam, 15 to 30 percent slopes
144	Underwood loam, 2 to 15 percent slopes
146	Underwood loam, 30 to 50 percent slopes
148	Undusk gravelly loam, 30 to 65 percent slopes
147	Undusk gravelly loam, 5 to 30 percent slopes

Figure C1
Landslide Hazard Classifications
Saddleback Wind Project

BONNEVILLE POWER ADMINISTRATION

DOE/BP-4159 ▪ May 2010