Mr. Chad Wilson

Geotechnical Evaluation Report for Roads

Phase 1
Dave's View at Martins Bluff
Kalama, Washington

July 2003
V03-0133

GeoStandards Corporation

Engineers and geologists raising the quality of consulting in geosciences

Sam Adettiwar, MS, PE
Warren Krager, RG, CEG
Marcy Boyer, MS, PE
William Burns, MS, GIT
Sam Christie, EIT

Project No. V03-0133 Phase I Roads

July 9, 2003

Mr. Chad Wilson
5718 NE 45th Avenue
Vancouver, WA 98661

Re: Geotechnical Evaluation, Phase 1 Roads Report, Dave's View at Martins Bluff, Kalama, Cowlitz County, Washington

At your request, we have performed Geotechnical Evaluation for the referenced project. The accompanying report discusses results of our studies.

We appreciate the opportunity to be of service to you on this project. If you have any questions, please call us at 360-575-1101 (Longview/Kelso) or 503-646-9069 (Portland).

Sincerely,
GeoStandards

EXPIRES 9/04/03

Sam Adettiwar, MS, PE	Marcy Boyer, MS, PE
Principal Geotechnical Engineer	Senior Geotechnical Engineer

Attachments: Geotechnical Evaluation Report, Figures, and Appendix

Geotechnical Evaluation Report for Roads
Phase 1
Dave's View at Martins Bluff
Kalama, WA

At your request, we have performed Geotechnical Evaluation for the referenced project. The following report discusses results of our evaluation.

1.0 Purpose and Scope

The purpose and scope of this evaluation was to study surface and subsurface site conditions in order to provide geotechnical engineering recommendations for the Phase 1 road development. Our scope included geotechnical engineering analyses in order to provide site preparation, roadway and pavement design, and general construction recommendations. Specifically, our scope of services included following work items.

- Site and vicinity reconnaissance to evaluate surface conditions,
- Review of available literature on soils, geology, natural hazards, and USGS maps,
- Test pit and soil boring explorations
- Laboratory evaluation of collected soil samples,
- Development of geotechnical design and construction recommendations for roadways, and
- Preparation of this report.

A detailed description of specific tasks is given below. Any environmental or civil engineering evaluation of any kind was beyond our scope of services.

1.1 Site Reconnaissance and Literature Review

We performed detailed site and vicinity reconnaissance of the study area (Figure 1) and surrounding areas to evaluate surface conditions during October 2001 and the most recent one in July 2003. During our site visits, we performed detailed visual reconnaissance and subsurface explorations.

We reviewed following published reports and literature to identify any known hazards at the site or in the immediate site vicinity area.

- Soil Survey of Cowlitz County, USDA, SCS,
- USGS Topographic Quadrangle of Kalama and Deer Island, USGS, (Figure 1)
- Geology of the Southwest Quadrant of Washington, USGS,
- Geology of the Vancouver Quadrangle, Washington, WDNR
- Geologic Hazard Maps of Cowlitz County, WA, GIS of Cowlitz County,
- "Geologic Hazards Reconnaissance of the Wilson Property," Project No. V00-0268, GeoStandards Corporation.
- "Initial Geologic Hazards Evaluation & Initial Geotechnical Investigation, Wilson Property," Project No. V01-0278, GeoStandards Corporation.

1.2 Subsurface Exploration and Laboratory Evaluation

In September and October 2001, we explored the site by performing four soil borings (B-1, B-7 through B-9) and several test pits (TP-1 through TP-10, TP-13 to TP-16, TP 29, TP30, TP32, TP-50 through TP-54, TP-63, TP64, and TP-68) in the area now designated Phase 1. The soil borings and test pit locations are shown in Figure 2. General soil profile encountered in these explorations and noted in the geologic literature is discussed below. Detailed descriptions of subsurface soils encountered in our explorations are given in the exploration logs included in the attached appendix.

Selected samples of soils were transported to our laboratory for further evaluation, to aid in classification of the materials, and to help assess their strength and compressibility characteristics. Laboratory evaluation consisted of visual and textural examinations. We also performed moisture content testing on a sample from TP-3 and TP-15. We also conducted Atterberg Limits testing on a sample from TP-3. Laboratory evaluation testing details is included in the attached appendix.

1.3 Engineering Analyses

Using the results of our site reconnaissance, review of available subsurface exploration data, groundwater data, and laboratory test data, we performed slope stability analyses, embankment design, and pavement design for proposed roadways. In addition, we also developed geotechnical construction recommendations for general site preparation including fill placement and compaction, excavations, construction dewatering, and site drainage.

2.0 Proposed Development Plan

The project site (site) is located several miles south of downtown Kalama, WA, at the intersection of Cloverdale Road and Martins Bluff Road, as shown in an attached Site Location Map (Figure 1). The Roadway Grading plan, Figure 3, illustrates the proposed grading of the new roads. Phase 1 consists of constructing Dave's View Drive between Station 0+00 and Station 18+65, and all of Dalyn Court and Kalina Court. In addition to the streets, the proposed development will include civil amenities and paved areas. We understand that the roadways will be constructed in accordance with Cowlitz County's Private Road Ordinance.

3.0 Site Description

The entire site is a roughly triangular shaped, vacant, and partially cleared parcel of land. During our July 2, 2003 visit, we observed that a portion of the trees and all of the grass and brush has been removed from the Phase I area. The limit of Phase I is as shown in Figure 2.

Dave's view at Martins Bluff is surrounded by residential developments to the, south, north, and east. To the west is Interstate 5 and the Columbia River. The site is bordered to the south and west by South Cloverdale Road with Interstate 5 beyond to the west. Martins Bluff Road following Mill Creek bisects the site in a northeast-southwest direction.

3.1 General Conditions

At present, the site has been logged several times over the past at least 50 years, leaving access roads/paths with local cuts and fills at various locations throughout the site. The site topography generally consists of a western facing slopes leading down toward The Columbia River. Slopes in the central portions of the Phase 1 portion of the site are gentle to moderate (roughly 5H:1V to 2H:1V).

3.2 Surface Water Features

At the time our site visit, the site was very dry except in the main drainage along the northern property line. When we visited the site during last winter, we noticed very moist to wet surface conditions at many locations throughout the site. We also noticed soggy or wet soil conditions created by poor drainage condition as the result of old skid roads and other logging scars.

3.3 Vegetation

Phase 1 has been cleared of undergrowth and has bare soil at the ground surface. Some of the mature trees remain in-place. Along the steep slopes adjacent to the small tributary creek, the vegetation consists of second growth conifer and deciduous trees with a very dense under story of bushes and grasses.

4.0 Subsurface Conditions

In September and October 2001, we explored the site by performing four soil borings (B-1, B-7 through B-9) and several test pits (TP-1 through TP-10, TP-13 to TP-16, TP 29, TP30, TP32, TP-50 through TP-54, TP-63, TP64, and TP-68) in the area now designated Phase 1. The soil borings and test pit locations are shown in Figure 2. In addition, we reviewed following geologic literature.

- Soil Survey of Cowlitz County, USDA, SCS,
- Geology of the Southwest Quadrant of Washington, USGS, and
- Geology of the Vancouver Quadrangle, Washington, WDNR.

The general soil/rock profile noted in explorations and geologic literature review is discussed below. Detailed descriptions of subsurface soils encountered in our explorations are given in the soil borings and test pit logs included in Appendix A.

Topsoil – At present, there is minimal topsoil present at the site. The topsoil appears to have been removed during the recent site clearing and some grading activities.

Fill – There are local shallow fills (~ up to 6 feet) from old logging roads at isolated locations such as at TP-1 and TP-6. Generally, these local old road fills consist of soft to firm clayey silt with gravel mixtures with isolated organics.

Residual Soils – Where the surface has not been disrupted by man made cuts/fills and/or colluvium, the site is underlain by roughly 1 feet to 9 feet of residual soils consisting of very firm to stiff sandy silt with weathered rock fragments, derived from underlying Goble Volcanics Formation. Residual soil stratum is underlain by and grade into completely weathered bedrock or saprolite of Goble Volcanics, extending to varying depths.

Goble Volcanics – The Phase 1 area is underlain by residual soils and shallow bedrock derived from basaltic andesite flows, flow breccias, ash flow tuffs and lahar deposits of the Eocene-Oligocene age Goble Volcanics. Very steep canyon walls and slopes in the vicinity area generally consist of rock outcroppings of basaltic andesite or flow tuffs. This unit may be from a few to several tens of feet in thickness consisting of clayey silt and sand with weathered rock fragments including large rocks and boulders. The residual soils have a gradational contact with less weathered bedrock units at depth.

Cowlitz Formation/Grays River Formation – Upper Goble Volcanics Formation rocks are underlain by partially weathered to unweathered basaltic-andesite flows with interbeded pyroclastic flows of the Tertiary aged Goble Volcanics Formation. This formation extends for tens of feet where it is underlain by the Tertiary aged Cowlitz and/or the Grays River Formation consisting of interbeded sedimentary and volcanic deposits.

Groundwater – Groundwater seepage was generally not encountered during the test pit explorations. Small springs or seeps were observed within outcrops of the volcanic rocks of the Goble Volcanics. During our 2001 explorations, we did notice soggy or wet soil conditions created by poor drainage condition as the result of old skid roads and other logging scars. We did not observe ponded water on the site during our July 2003 visit. This is likely due to the dry weather.

We anticipate that the groundwater table will rise during months of peak precipitation. Surface infiltration and rainfall may generate higher perched water table conditions because of near surface rock conditions. Variations in groundwater levels should be expected seasonally, annually, and from location to location. Detailed descriptions on soil and groundwater conditions encountered in our explorations are given in attached test pit logs.

5.0 Laboratory Evaluation

Selected samples of soils were transported to our laboratory for further evaluation, to aid in classification of the materials, and to help assess their strength and compressibility characteristics. Laboratory evaluation consisted of visual and textural examinations. In addition, we performed moisture content testing on samples from test pits TP-3 and TP-15. We also conducted an Atterberg Limits test on a sample from test pit TP-3. Laboratory evaluation testing details are included in an appendix.

6.0 Slope Stability Analysis

Using the results of subsurface exploration, laboratory testing, and available survey data, we analyzed on-site slopes and proposed slopes by performing extensive slope stability analyses. We modeled proposed slopes at critical roadway locations using XSTABL computer software, and performed analyses using several analytical methods (Bishop, Janbu, Spencer, etc.) in order to obtain the lowest factor of safety against slope failures. We also analyzed the slopes using an effective stress approach. Results of our analyses are illustrated in Figure 4 and Figure 5. Analytical data is included in an appendix.

The XSTABL computer software calculates the most likely failure plane based on topography, subsurface conditions (including soil parameters), and groundwater conditions. The stability of this most likely failure plane is calculated as the factor of safety (FOS), which is a ratio of the resisting forces or shear strength to the driving forces or shear stress required for equilibrium of the slope. A FOS of 1.0 indicates the resistive forces and driving forces are equal. A FOS below 1.0 indicates the driving forces are greater and the landslide is active. A FOS above 1.0 indicates the resisting forces are greater and the slope is stable. Based on the engineering community and our experience, a factor of safety in the range of 1.5 is generally acceptable to assure slope stability. The following table summarizes results of our slope stability analyses performed at critical locations.

TABLE 1: Slope Stability Analyses Results

Critical Locations (Worst Case Scenarios)	Situation	Factor of Safety (Fig. 4&5)	Conclusion/Comments
Cross Section A-A' Station 7+90 Dave's View Drive	Proposed Fill Embankment over native soils.	2.948	Provide a minimum 8' long x 3' deep keyway to support the fill soils as shown in Figure 4 and Figure 6.
Cross Section B-B' Station 12+50 Dave's View Drive	Proposed Cut Embankment in the native slope	2.254	Cut Topsoil layer at a 2H:1V slope and the weathered Goble Volcanics at a 1.5H:1V slope as shown in Figure 5.

Based on the results of our slope stability analyses, the proposed construction does not lower the existing global slope stability at the site, provided our geotechnical recommendations are strictly followed. In our opinion, if our recommendations are strictly followed, the proposed construction will improve the slope stability conditions at the site due to proposed drainage improvements, regrading, construction of embankments and drainage improvements. Considering site slopes and adjacent and nearby topography, and the proposed improvements, in our opinion, the proposed construction will not adversely impact the slope stability of adjacent properties provided our geotechnical recommendations are strictly followed.

7.0 Geotechnical Construction Recommendations

Based on the results of our field exploration, laboratory evaluation, and engineering analyses, we believe the site is suitable for proposed development, provided following general construction recommendations are followed.

The following paragraphs provide general recommendations for site development. We recommend that the earthwork phases be completed during the dry summer months. Variations in soil conditions may be encountered during construction. In order to permit correlation between soil exploration data and actual soil conditions encountered during construction, we recommend that a geotechnical engineer be retained to perform inspections during construction and to provide specific recommendations for soils or foundation related phases of work.

7.1 Subgrade Preparation

In general, we recommend that any surface water within construction areas be drained away by cutting drainage ditches or by pumping from a sump hole, if necessary. Surface vegetation including any topsoil and/or surficial fill, any saturated/inundated and disturbed soil, and any non-soil or incompetent materials encountered at the time of construction should be removed. If any deep root systems or tree trunks are removed, then the excavated areas should be filled with densely compacted on-site sandy clean soils or imported crushed rock.

In case excessive moisture is encountered during construction, a 3-inch to 6-inch thick crushed rock layer should be placed immediately on any exposed subgrades after site grading and topsoil removal in order to protect the subgrade. For construction truck traffic areas, at least 12-inch thick granular working base is generally recommended with thicker sections and/or geotextile fabrics for heavily traveled areas or haul roads during construction.

7.2 Fill Placement and Embankment Construction

Fills up to 17 feet high are planned along the proposed roadway. In addition, existing fill is present along the existing logging roads. We recommend that this existing uncontrolled fill material be excavated and replaced as compacted structural fill in accordance with recommendations given below.

All fill must be placed only after the subgrade is properly prepared and then approved by a qualified geotechnical engineer. At the time of construction, a qualified geotechnical engineer may recommend that all exposed subgrades be proof-rolled with a loaded dump truck having a static weight of at least 45,000 pounds or may use other appropriate means of evaluating the subgrade based on moisture conditions. Generally, areas found to be soft or otherwise unsuitable for supporting anticipated structural loads are over-excavated and replaced with compacted structural fill.

Structural fill materials for the pavement and residential areas should be placed in layers that, when compacted, do not exceed about 6 to 8 inches for fine-grained soils (silts and clays) and about 10 to 12 inches for granular materials (sand and gravel). Fill materials should be moistened or dried to achieve near optimum moisture conditions and then compacted by mechanical means to a minimum of 95 percent of the maximum dry density determined from ASTM D1557 modified Proctor laboratory test. Landscape fill can be placed and compacted by mechanical means to a minimum of 90 percent of the maximum dry density determined from ASTM D1557 modified Proctor laboratory test. It should be noted that all slope fill should be compacted to 95% of the maximum dry density as determined by ASTM D1557.

The fill slopes and embankments should be constructed in accordance with instructions given in Figure 6. Specifically, fills should not be placed on or near steep slopes on the site prior to consulting with a qualified geotechnical engineer. In general, fill over naturally sloping ground steeper than 5H:1V should be constructed using a stepping and keying method. A 8-foot wide and 3-foot deep base key should be cut at the bottom of the slope and backfilled with granular fill. The next step or bench should be cut into the slopes as construction progresses from the bottom of the slope to the top of the slope and any modifications made by the field geotechnical engineer at the time of construction.

All fill embankment slopes must not be steeper than 2(H):1(V) for the on-site material and 1.5(H):1(V) if the embankment is constructed of crushed rock fill. The slopes may be constructed steeper only if specified by a qualified geotechnical engineer. For steeper fill slopes, soil reinforcement, retaining walls, etc. may become necessary. All fill slopes must be covered with adequate erosion protection such as erosion mats and/or vegetation.

We evaluated effects of fill surcharge on soils present at the site. Based on the results of our settlement analyses, we estimate roughly 1 inch to 1-1/2 inches of settlement due to a maximum of 15 feet of fill surcharge over native soils. These settlements are expected to occur mostly during construction and do not pose a long-term consolidation settlement hazard.

7.3 Cut Slopes, Drainage, and Construction Dewatering

Cuts up to 15 feet are planned along the proposed roadways. Figure 5 displays a typical cross-section of an area with cut slopes. In the proposed cut areas, we anticipate the presence of residual soil underlain by highly weathered Goble Volcanics. The topsoil and residual soil can be cut at with a slope of 2H:1V. The cut slopes in the highly weathered Goble Volcanics can be cut with a slope of 1.5H:1V. The cut slopes should be protected with erosion control as soon as possible after cutting. Possible erosion control could consist of erosion mats and/or vegetation.

In general, temporary earth slopes may be cut near vertical up to 5 feet deep. All temporary excavations should be performed in accordance with Department of Labor Occupational Safety and Health

Administration (OSHA) guidelines for Type A soils. Deeper excavations may be excavated at grades steeper than the recommended OSHA grades provided the excavations are monitored and certified by a qualified geotechnical engineer. Please note that site safety during construction is the sole responsibility of the project contractor and/or the owners.

The geotechnical evaluation for deep utility excavations that will penetrate or expose the bedrock formations underlying upper soil units may be required to evaluate cut stability and excavation feasibility. Based on our initial evaluation, in our opinion, on-site weathered rock formations in majority of the site can be excavated using a heavy trackhoe (20-30 ton) equipped with an appropriately toothed (duckbill or pointed) bucket.

Our subsurface exploration did not reveal a groundwater table within the proposed maximum excavation depth. However, some perched groundwater seepage in excavations should be anticipated during wet season of the year. Therefore, we do not anticipate the need for a subsurface drainage system or under drains for pavements. However, if groundwater seepage in excavations should be found during construction, specific dewatering system may be required if several springs or excessive subsurface seepage zones are encountered. GeoStandards can provide specific dewatering recommendations during construction if such conditions arise.

In general, we recommend that all concentrated seepage areas (springs) and other seepage areas should be evaluated to find the source and then adequately contained or intercepted to channelize collected water into suitable receptacle such as storm drainage systems. We recommend the installation of typical toe drains/ditch drain along the edge of the pavement and at the base of all cut slopes in order to collect surface and subsurface water run-off. In addition, we recommend the installation of culverts to provide adequate drainage for any springs, creeks, and drainage channels noted during excavation.

Following additional specific recommendations should be followed for any roadway cuts, or basement excavation or grading cuts for the lots:

- During rainy season, cuts should be completely covered with visquene to minimize saturation due to any rainfall until proposed retaining structures are constructed or until cuts are stabilized or covered with erosion protection.

- Surface water run-off (from uphill area) towards cuts should be diverted and appropriately discharged using interceptor drains on uphill side.

- Any springs or drainage features should be adequately contained and discharged away from excavated/cut areas.

- Soils exposed in excavated areas should be protected from rain, freezing, and excessive loading along edges. Surface water run-off should be intercepted and drained away from excavated areas.

- Common drainage systems (public utilities including recommended drains) should be checked and cleaned (if necessary) at least once in three years in September prior to the beginning of winter season in order to assure integrity of the systems.

Groundwater seepage in excavations should be anticipated during construction. For most of the excavations on this project, pumping from sumps outside the limits of the excavation should control groundwater seepage and surface water ponding. In general, we recommend that all exposed large/concentrated seepage areas (springs) and other seepage areas should be evaluated by a geotechnical engineer at the time of construction/excavation to find the source and then adequately contained or intercepted to channelize collected water into suitable receptacle such as storm drainage systems.

Soils exposed in excavated areas should be protected from rain, freezing, and excessive loading along edges. Surface water run-off should be intercepted and drained away from excavated areas. Ideally, in structural areas, concrete should be poured within 24 hours of the completion of excavation.

8.0 Pavement Design Recommendations

Based on the results of our field exploration, laboratory evaluation, and engineering analyses, we believe the site is suitable for proposed development at this time, provided following general design recommendations are followed.

The following paragraphs provide only general recommendations. Variations in soil conditions may be encountered during construction. In order to permit correlation between soil exploration data and actual soil conditions encountered during construction, we recommend that a geotechnical engineer be retained to perform inspections during construction and to provide specific recommendations for soils or foundation related phases of work.

Based on the results of field exploration, our experience, and engineering analyses, we assumed following parameters for pavement design. Please note that the traffic volume was estimated based on the presence of nearby schools, public facilities, commercial facilities, and our experience. If the estimated value is considerably different from the one obtained from actual traffic data, then we should be contacted to revise our design.

Design Life:	20 years
Design ESALs:	1,000,000
Initial Serviceability Index:	4.5
Terminal Serviceable Index:	2.0
Overall Standard Deviation:	0.45
California Bearing Ratio:	10.0
Reliability:	95%
Surface Drainage Quality:	Good, after improvements

Using these parameters, we designed the pavement in general accordance with AASHTO Pavement Design procedure. We recommend a preliminary pavement section consisting of following layers and subjected to given construction recommendations. See Figure 7.

Materials	Layer Thickness
Bituminous Concrete (AC) (Washington DOT Class B)	4 inches
Crushed Rock Base (Washington DOT Specification)	12 inches

In general, just before the placement of baserock, we recommend that any surface water within pavement areas be drained away by cutting drainage ditches or by pumping from a sump hole, if necessary. Any surface vegetation including any surficial topsoil/soft soil, any saturated/inundated and disturbed soil, and any non-soil or incompetent materials, shallow root systems or tree trunks should be removed.
The subgrade should be inspected and approved by a registered geotechnical engineer or his representative prior to the placement of base rock materials. For this purpose, the subgrade must be proof-rolled in the presence of a qualified geotechnical engineer with a loaded dump truck having a minimum static weight of 45,000 pounds to detect areas or pockets of unusually soft material. These failed areas should be excavated and replaced with suitable compacted fill.

If wet weather conditions exist, we should visit the site to evaluate soil moisture conditions and review proposed grading plans to provide specific recommendations such as soil-cement stabilization or cement-treated base construction. In wet season, to protect moisture sensitive soils during construction activities, a 3-inch to 6-inch thick crushed rock layer should be placed immediately on any silty subgrade throughout the site after site grading and topsoil removal. For construction truck traffic areas, at least 12-inch thick granular working base is generally recommended with thicker sections and/or geotextile fabrics for heavily traveled areas.

The base course and bituminous concrete materials should conform to the requirements given in the latest edition of the State of Washington DOT, Standard Specifications for Road, Bridge, and Municipal Construction handbook. Base course materials should consist of well-graded 1½-inch or ¾-inch minus crushed rock, having less than 5 percent material passing the No. 200 sieve. The base course material should be compacted to at least 95 percent of the maximum density determined by the ASTM D1557 laboratory test procedure.

The bituminous concrete material (AC) should be compacted to at least 90 percent of the theoretical maximum density determined by ASTM D2041 (Rice Specific Gravity) laboratory test procedure.
In general, all construction procedures should conform to the requirements given in the latest edition of the State of Washington DOT, Standard Specifications for Road, Bridge and Municipal Construction handbook.

9.0 Construction Monitoring

In order to assure that the project is constructed in accordance with geotechnical design and construction recommendations discussed in this report, we recommend following general construction monitoring criteria:

- A qualified geotechnical engineer should examine and identify all setbacks and excavated subgrades to verify building location and subgrade soil conditions. All structural subgrades should be proof-rolled in the presence of a qualified geotechnical engineer or approved by a qualified geotechnical engineer.

- Structural fill placement and compaction should be continuously and/or periodically observed and tested by a GeoStandards representative or a certified geotechnical testing laboratory depending upon the area of fill placement. All testing reports should be submitted to us for review.

10.0 General Conditions

Historically, with construction in hilly or sloped areas, there is an inherent risk associated with slope failures. Although no unstable areas have been noted, the owner is still responsible for taking any risks associated with any future potential for instability at the site or in the site vicinity.

We have completed this study in accordance with generally accepted geotechnical engineering principles and practices and conditions described in an ASFE document included in an appendix. GeoStandards is not responsible for the independent conclusions, opinions, or recommendations made by others based on the information presented in this report.