GEOTECHNICAL REPORT

Heathers Ridge South
NE 100th Street and 134th Avenue NE
Redmond, Washington

Project No. T-7177

Terra Associates, Inc.

Prepared for:

Quadrant Homes
Bellevue, Washington

February 17, 2015

TERRA ASSOCIATES, Inc.

Consultants in Geotechnical Engineering, Geology
and
Environmental Earth Sciences

February 17, 2015
Project No. T-7177

Mr. Matt Perkins
Quadrant Homes
14725 SE 36th Street, Suite 200
Bellevue, Washington 98006

Subject: Geotechnical Report
Heathers Ridge South
NE 100th Street and 134th Avenue NE
Redmond, Washington

Dear Mr. Perkins:

As requested, we conducted a geotechnical engineering study for the subject project. The attached report presents our findings and recommendations for the geotechnical aspects of project design and construction.

Our study indicates the site soils generally consist of about 4 to 12 inches of duff and topsoil overlying glacial till deposits comprised predominantly of silty fine to medium sand with varying amounts of gravel. Groundwater seepage was not observed in any of the test pits.

In our opinion, there are no geotechnical conditions that would preclude the planned residential development. Residences can be supported on conventional spread footings bearing on competent native soils underlying the organic surface soils or on structural fill placed on competent native soils. Floor slabs and pavements can be similarly supported.

Detailed recommendations addressing these issues and other geotechnical design considerations are presented in the attached report. We trust the information presented is sufficient for your current needs. If you have any questions or require additional information, please call.

Sincerely yours,
TERRA ASSOCIATES, INC.

John C. Sadler, L.E.G., L.H.G.
Project Manager

2-17-15

Theodore J. Schepper, P.E.
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1.0 PROJECT DESCRIPTION

The proposed project consists of a residential development. A Preliminary Short Plat Plan by LDC dated January 20, 2015 indicates that the property will be developed with 8 single-family residential lots. Proposed site grading will consist primarily of fills ranging in thicknesses from about two feet to a maximum of about eight feet along the eastern margin of the planned development area.

A 2- to 8-foot high rockery is shown supporting the vertical grade change along the eastern margin of the fill. Proposed grades above the rockery are flat except for an approximately 75-foot long section adjacent to Lot 6 and the southern portion of Lot 5, which slopes up about 4 to 6 feet at an inclination of 3:1 (Horizontal:Vertical). The southern approximately 90 feet of the rockery will support the downgradient margin of a utility maintenance access roadway.

Site stormwater will be detained in a buried vault located in the southern portion of the planned development area. Preliminary information provided by LDC indicates that the vault will be approximately 48 feet long, 37 feet wide, and approximately 15 feet deep.

Building plans are not available; however, we expect that the residences would be two-story, wood-frame structures, with their main floors constructed at grade or framed over a crawl space. Foundation loads should be relatively light, in the range of 2 to 3 kips per foot for bearing walls and 25 to 50 kips for isolated columns.

The recommendations contained in the following sections of this report are preliminary and based on our understanding of the above design features. We should review design drawings as they become available to verify that our recommendations have been properly interpreted and incorporated into project design and to amend or supplement our recommendations, if required.

2.0 SCOPE OF WORK

Geotech Consultants, Inc. (GCI) explored subsurface conditions at the site by excavating 12 test pits in October 2001 and presented the test pit logs in a report titled *Geotechnical Engineering Study, Proposed Ellsworth Estates*, dated October 18, 2011. Using the subsurface information provided on the GCI Test Pit Logs and the results GCI grain size determinations, analyses were undertaken to develop geotechnical recommendations for project design and construction. Specifically, this report addresses the following:

- Soil and groundwater conditions
- Geologic hazards per the Redmond Zoning Code
- Seismic design parameters per the current International Building Code (IBC)
- Site preparation and grading
- Excavations
- Foundations

- Slab-on-grade floors
- Fill retention and rockeries
- Infiltration feasibility
- Stormwater detention
- Drainage
- Utilities
- Pavements

It should be noted that recommendations outlined in this report regarding drainage are associated with soil strength, design earth pressures, erosion, and stability. Design and performance issues with respect to moisture as it relates to the structure environment (i.e., humidity, mildew, mold) is beyond Terra Associates' purview. A building envelope specialist or contactor should be consulted to address these issues, as needed.

3.0 SITE CONDITIONS

3.1 Surface

The site is an undeveloped 1.53-acre parcel located south of and adjacent to the intersection of NE 100th Street and 134th Avenue NE in Redmond, Washington. The approximate location of the site is shown on Figure 1.

Existing surface gradients are relatively flat with a gentle slope down to the east-southeast. Topography shown on the Preliminary Short Plat Plan indicates that surface gradients in the vast majority of the planned development area are about four to five percent. The exception to this is along the eastern margin of the site, which slopes gently to moderately down to a south-flowing stream channel. The slope adjacent to the stream channel varies in height and inclination from about 6 feet and 18 percent in the northeastern portion of the site to about 20 feet and 48 percent in the southeastern portion of the site. We did not observe any indications of instability, significant erosion, or groundwater seepage on the slope areas adjacent to the stream channel. We did not observe any indications of significant recent channel erosion or instability related to channel erosion in the drainage. The materials observed in the bottom of the channel consist primarily of sand and gravel.

Site vegetation consists primarily of mature coniferous and deciduous trees and thin brush undergrowth. The central portion of the site is generally clear of mature trees and is vegetated with grasses. A construction access road that is surfaced with hog fuel and quarry spalls has been graded into the site from NE 100th Street. We observed several localized areas of standing surface water along the access road. Based on our observations, the surface water appears to be from recent precipitation.

3.2 Soils

Our review of the GCI Test Pit Logs indicate that the site soils generally consist of about 4 to 12 inches of duff and topsoil overlying glacial till deposits comprised predominantly of silty fine to medium sand with varying amounts of gravel. The upper approximately three to five feet of soil has weathered to a loose to medium dense condition, and was typically moist and mottled. The unweathered till soils observed below these depths are generally dense to very dense, and moist.

The *Geologic map of the Kirkland quadrangle, Washington*, by James P. Minard (1983) shows site geology mapped as Vashon till (Qvt). The dense to very dense silty sand with gravel observed below depths of about three to five feet in all of the test pits is generally consistent with this geologic map unit.

Detailed descriptions of the subsurface conditions observed in the test pits are presented on the Test Pit Logs in Appendix A. The approximate locations of the test pits are shown on Figure 2.

3.3 Groundwater

Groundwater seepage was not documented on any of the Test Pit Logs. However, mottling was observed in the upper weathered till soils in 9 of the 12 test pits. Mottling is an indication that the soils are impacted by fluctuating perched groundwater at times.

The development of perched groundwater is typical for sites underlain by till and till-like soils. Perched groundwater levels and flow rates will fluctuate seasonally, and typically reach their highest levels during and shortly following the wet winter months (October through May).

3.4 Geologic Hazards

We evaluated site conditions for the presence of geologic hazards. Section 21.64.060 (Geologically Hazardous Areas) of the City of Redmond Zoning Code (RZC) defines geologically hazardous areas as erosion hazard areas, landslide hazard areas, and seismic hazard areas.

3.4.1 Erosion Hazard Areas

Section 21.64.060A.1.a of the RZC defines erosion hazard areas as "...lands or areas underlain by soils identified by the U.S. Department of Agriculture Soil Conservation Service (SCS) as having "severe" or "very severe" rill and inter-rill erosion hazards. This includes, but is not limited to, the following group of soils when they occur on slopes of 15 percent or greater: Alderwood-Kitsap (AkF), Alderwood gravelly sandy loam (AgD), Kitsap silt loam (KpD), Everett (EvD), and Indianola (InD)."

The Soil Conservation Service (SCS) has classified the soils underlying the site as *Alderwood gravelly sandy loam, 0 to 8 percent slopes* (AgB) and *Alderwood gravelly sandy loam, 6 to 15 percent slopes* (AgC). Alderwood soils are described as formed over till, which is generally consistent with the soils observed in the test pits. The SCS describes the erosion hazard of AgB soils and AgC soils as slight and moderate, respectively, which does not meet the above criteria defining an erosion hazard area. However, the soils underlying the slope areas that are steeper than 15 percent in the eastern portion of the site would be better classified as *Alderwood gravelly sandy loam, 15 to 30 percent slopes* (AgD) soils. As described above, areas underlain by AgD soils are considered erosion hazard areas.

We did not observe any indications of significant erosion at the site; however, the site soils will be susceptible to erosion when exposed during construction. In our opinion, proper implementation and maintenance of Best Management Practices (BMPs) for erosion prevention and sedimentation control will adequately mitigate the erosion potential in the planned development area. Erosion protection measures as required by the City of Redmond will need to be in place prior to and during grading activity on the site.

3.4.2 Landslide Hazard Areas

Section 21.64.060A.1.b of the RZC defines landslide hazard areas as "…areas potentially subject to significant or severe risk of landslides based on a combination of geologic, topographic, and hydrogeologic factors.

They include areas susceptible because of any combination of bedrock, soil, slope, slope aspect, structure, hydrology, or other factors. They are areas of the landscape that are at a high risk of failure or that presently exhibit downslope movement of soil and/or rocks and that are separated from the underlying stationary part of the slope by a definite plane of separation. The plane of separation may be thick or thin and may be composed of multiple failure zones depending on local conditions, including soil type, slope gradient, and groundwater regime." Landslide hazard areas include the following:

i. Areas of historic failures, such as:

   a. Areas designated as quaternary slumps or landslides on maps published by the United States Geologic Survey (USGS).

   b. Those areas designated by the United States Department of Agriculture (USDA) Soil Conservation Service (SCS) as having a "severe" limitation for building site development.

ii. Areas containing a combination of slopes steeper than 15 percent, springs or groundwater seepage, and hillsides intersecting geologic contacts with a relatively permeable sediment overlying a relatively impermeable sediment or bedrock.

iii. Areas that have shown movement during the Holocene epoch (from 10,000 years ago to the present) or which are underlain or covered by mass wastage debris of that epoch.

iv. Slopes that are parallel or subparallel to planes of weakness in subsurface materials.

v. Slopes having gradients steeper than 80 percent subject to rockfall during seismic shaking.

vi. Areas potentially unstable as a result of rapid stream incision, stream bank erosion, and undercutting by wave action.

vii. Any area with a slope 40 percent or steeper with a vertical relief of 10 feet or more.

A localized slope area adjacent to the stream channel in the southeastern portion of the site is steeper than 40 percent with a slope height of about 18 feet. The geometry of this slope area meets the criteria for a landslide hazard area given in above Item vii. Section 21.64.060B.2 of the RZC states that a minimum buffer width of 50 feet shall be applied to the top, toe, and sides of a landslide hazard area. However, per RZC Section 21.64.060B.3, the buffer may be reduced to a minimum width of 15 feet provided a qualified professional demonstrates through technical studies that the reduction will adequately protect the proposed and surrounding development from the critical landslide hazard.

Given our observations of existing slope conditions and the documented subsurface conditions near the top of the slope, it is our opinion that a 15-foot buffer, in conjunction with proper site drainage, will adequately mitigate any potential hazard associated with the landslide hazard area. The use of the reduced buffer width is supported by the results of our site stability analysis discussed below.

Stability Analysis

We performed stability analyses of the steep slope using the computer program WINSTABL. The soil parameters used for our analyses are based on field and laboratory data and our past experience with similar soils. These parameters are shown on the analysis output text and profiles in Appendix B.

Analyses were performed on a section line identified on Figure 2 as Section A-A' for both static and pseudostatic (seismic) conditions for the existing slope conditions and for post development condition. The pseudostatic analysis used a horizontal earthquake coefficient value of 0.15g to model ground motions expected from a severe earthquake. The seismic acceleration of 0.15g was based on current USGS seismic hazard maps for a seismic event having a 10 percent probability of exceedance in a 50-year period. The USGS map indicates the subject site is located within an area where the peak horizontal ground acceleration for this return period is expected to range between 0.25g and 0.3g. Our analysis considered a horizontal acceleration equal to one-half the maximum value of this range. The lowest safety factors determined by our analyses are presented in the following table:

Section Analyzed	**Minimum Safety Factors**	
	Static	Pseudostatic
A-A' Existing Slope Condition	2.72	1.95
A-A' Post-Development Condition	2.57	1.80

The results of the stability analyses indicate that the slope is stable with respect to deep-seated failure under static and pseudostatic conditions. The safety factors listed above are higher than the minimum safety factors considered acceptable for stable slopes by local geotechnical engineering practice.

Potential impacts to the landslide hazard area due to site grading and construction include increasing the potential for erosion on and/or adjacent to the slope by exposing soils during grading and allowing surface runoff to flow onto the steep slope. In our opinion, potential erosion and sedimentation impacts to the landslide hazard area resulting from site development activities would be eliminated or significantly reduced with proper implementation and maintenance of BMPs for erosion prevention sedimentation containment. It is also our opinion that slope stability would be improved as a result of enhanced site drainage measures associated with the planned development.

3.4.3 Seismic Hazard Areas

Section 21.64.060A.1.c of the RZC defines seismic hazard areas as "…lands subject to severe risk of damage as a result of earthquake-induced ground shaking, slope failure, settlement, soil liquefaction, or surface faulting."

Based on the soil and groundwater conditions we observed at the site, it is our opinion that the risk for damage resulting from earthquake induced slope failure, ground settlement, surface faulting, or soil liquefaction is negligible. Therefore, in our opinion, unusual seismic hazard areas do not exist at the site, and design in accordance with local building codes for determining seismic forces would adequately mitigate impacts associated with ground shaking.

3.5 Seismic Design Parameters

Based on the site soil conditions and our knowledge of the area geology, per the 2012 International Building Code (IBC), site class "C" should be used in structural design. Based on this site class, in accordance with the 2012 IBC, the following parameters should be used in computing seismic forces:

Seismic Design Parameters (IBC 2012)

Spectral response acceleration (Short Period), S_{Ms}	1.254 g
Spectral response acceleration (1 – Second Period), S_{M1}	0.635 g
Five percent damped .2 second period, S_{Ds}	0.836 g
Five percent damped 1.0 second period, S_{D1}	0.424 g

Values determined using the United States Geological Survey (USGS) Ground Motion Parameter Calculator accessed on February 12, 2015 at the web site http://earthquake.usgs.gov/designmaps/us/application.php.

4.0 DISCUSSION AND RECOMMENDATIONS

4.1 General

Based on our study, there are no geotechnical conditions that would preclude the planned development. Residences can be supported on conventional spread footings bearing on competent native soils underlying organic topsoil or on structural fill placed on the competent native soils. Floor slabs and pavements can be similarly supported.

The site soils contain a sufficient amount of fines (silt- and clay-sized particles) such that they will be difficult to compact as structural fill when too wet or too dry. If grading activities will take place during the winter season, the owner should be prepared to import free-draining granular material for use as structural fill and backfill.

Detailed recommendations regarding these issues and other geotechnical design considerations are provided in the following sections of this report. These recommendations should be incorporated into the final design drawings and construction specifications.

4.2 Site Preparation and Grading

To prepare the site for construction, all vegetation, organic surface soils, and other deleterious materials should be stripped and removed from the site. We expect surface stripping depths of about six to ten inches will be required to remove the organic surficial soils. Stripped vegetation debris should be removed from the site. Organic soils will not be suitable for use as structural fill, but may be used for limited depths in nonstructural areas or for landscaping purposes. Once clearing and grubbing operations are complete, cut and fill operations to establish desired building grades can be initiated.

A representative of Terra Associates, Inc. should examine all bearing surfaces to verify that conditions encountered are as anticipated and are suitable for placement of structural fill or direct support of building and pavement elements. Our representative may request proofrolling exposed surfaces with a heavy rubber tired vehicle to determine if any isolated soft and yielding areas are present. If unstable yielding areas are observed, they should be cut to firm bearing soil and filled to grade with structural fill. If the depth of excavation to remove unstable soils is excessive, use of geotextile fabric such as Mirafi 500X or equivalent in conjunction with structural fill can be considered in order to limit the depth of removal. In general, our experience has shown that a minimum of 18 inches of clean, granular structural fill over the geotextile fabric should establish a stable bearing surface.

The native soils observed at the site contain a sufficient amount of fines (silt and clay size particles) that will make them difficult to compact as structural fill if they are too wet or too dry. Accordingly, the ability to use these soils from site excavations as structural fill will depend on their moisture content and the prevailing weather conditions when site grading activities take place. Soils that are too wet to properly compact could be dried by aeration during dry weather conditions, or mixed with an additive such as cement or lime to stabilize the soil and facilitate compaction. If an additive is used, additional Best Management Practices (BMPs) for its use will need to be incorporated into the Temporary Erosion and Sedimentation Control (TESC) plan for the project. Soils that are dry of optimum should be moisture conditioned by controlled addition of water and blending prior to material placement.

If grading activities are planned during the wet winter months, or if they are initiated during the summer and extend into fall and winter, the owner should be prepared to import wet weather structural fill. For this purpose, we recommend importing a granular soil that meets the following grading requirements:

U.S. Sieve Size	Percent Passing
6 inches	100
No. 4	75 maximum
No. 200	5 maximum*

*Based on the 3/4-inch fraction.

Prior to use, Terra Associates, Inc. should examine and test all materials imported to the site for use as structural fill.

Structural fill should be placed in uniform loose layers not exceeding 12 inches and compacted to a minimum of 95 percent of the soil's maximum dry density, as determined by American Society for Testing and Materials (ASTM) Test Designation D-698 (Standard Proctor). The moisture content of the soil at the time of compaction should be within two percent of its optimum, as determined by this ASTM standard. In nonstructural areas, the degree of compaction can be reduced to 90 percent.

4.3 Excavations

All excavations at the site associated with confined spaces, such as lower building level retaining walls, must be completed in accordance with local, state, and federal requirements. Based on the Washington State Safety and Health Administration (WSHA) regulations, the upper weathered medium dense to dense native soils would typically be classified as Type C soils. Unweathered, dense to very dense till and till-like soils would typically be classified as Type A soils.

Accordingly, for temporary excavations of more than 4 feet and less than 20 feet in depth, the side slopes in Type C soils should be laid back at a slope inclination of 1.5:1 (Horizontal:Vertical) or flatter. Temporary excavations in Type A soils can be laid back at inclinations of 0.75:1 or flatter. For temporary excavation slopes less than 8 feet in height in Type A soils, the lower 3.5 feet can be cut to a vertical condition with a 0.75:1 slope graded above. For temporary excavation slopes greater than 8 feet in height up to a maximum height of 12 feet, the slope above the 3.5-foot high vertical portion should be laid back to an inclination of 1:1 or flatter. No vertical cut with a backslope immediately above is allowed for excavation depths that exceed 12 feet. In this case, a 4-foot high vertical cut with an equivalent horizontal bench to the cut slope toe is required. If there is insufficient room to complete the excavations in the manners discussed above, or if excavations greater than 20 feet deep are planned, you may need to use temporary shoring to support the excavations.

Seepage of perched groundwater should be anticipated within excavations extending to the dense to very dense till soils. In our opinion, the volume of water and rate of flow into the excavation should be relatively minor and would not be expected to impact the stability of the excavations when completed as described above. Conventional sump pumping procedures along with a system of collection trenches, if necessary, should be capable of maintaining a relatively dry excavation for construction purposes.

The above information is provided solely for the benefit of the owner and other design consultants, and should not be construed to imply that Terra Associates, Inc. assumes responsibility for job site safety. It is understood that job site safety is the sole responsibility of the project contractor.

4.4 Foundations

Residential structures may be supported on conventional spread footing foundations bearing on competent native soils or on structural fill placed above the native soils. Foundation subgrades should be prepared, as recommended in Section 4.2 of this report.

Perimeter foundations exposed to the weather should bear at a minimum depth of 1.5 feet below final exterior grades for frost protection. Interior foundations can be constructed at any convenient depth below the floor slab. We recommend designing foundations for a net allowable bearing capacity of 2,500 pounds per square foot (psf). For short-term loads, such as wind and seismic, a one-third increase in this allowable capacity can be used in design. With the anticipated loads and this bearing stress applied, building settlements should be less than one-half inch total and one-fourth inch differential.

For designing foundations to resist lateral loads, a base friction coefficient of 0.35 can be used. Passive earth pressure acting on the sides of the footings may also be considered. We recommend calculating this lateral resistance using an equivalent fluid weight of 300 pounds per cubic foot (pcf). We recommend not including the upper 12 inches of soil in this computation because they can be affected by weather or disturbed by future grading activity. This value assumes the foundations will be constructed neat against competent native soil or the excavations are backfilled with structural fill, as described in Section 4.2 of this report. The recommended passive and friction values include a safety factor of 1.5.

4.5 Slab-on-Grade Floors

Slab-on-grade floors may be supported on a subgrade prepared as recommended in Section 4.2 of this report. Immediately below the floor slab, we recommend placing a four-inch thick capillary break layer composed of clean, coarse sand or fine gravel that has less than three percent passing the No. 200 sieve. This material will reduce the potential for upward capillary movement of water through the underlying soil and subsequent wetting of the floor slab.

The capillary break layer will not prevent moisture intrusion through the slab caused by water vapor transmission. Where moisture by vapor transmission is undesirable, such as covered floor areas, a common practice is to place a durable plastic membrane on the capillary break layer and then cover the membrane with a layer of clean sand or fine gravel to protect it from damage during construction, and aid in uniform curing of the concrete slab. It should be noted that if the sand or gravel layer overlying the membrane is saturated prior to pouring the slab, it will be ineffective in assisting uniform curing of the slab and can actually serve as a water supply for moisture seeping through the slab and affecting floor coverings. Therefore, in our opinion, covering the membrane with a layer of sand or gravel should be avoided if floor slab construction occurs during the wet winter months and the layer cannot be effectively drained.

4.6 Fill Retention and Rockeries

As discussed, a rockery will be used to support a near-vertical grade transition in fill on the eastern side of the planned development area. It should be noted that rockeries are not engineered structures that are designed to retain earth in a manner similar to a cast-in-place concrete or gravity block wall systems. Rocks used to construct the wall will by virtue of their mass enhance stability; however, the soil against which the rockery is constructed must be inherently stable and able to stand unsupported in a near-vertical condition.

In our opinion, a rockery can be used to face unreinforced structural fill to a maximum height of four feet provided the fill is placed and compacted in accordance with recommendations outlined in Section 4.2 of this report, and the structural fill face is overbuilt, and then cut back, prior to rock placement.

For fill heights greater than four feet or where the rockery will be surcharged by structures or driveway/parking areas, the structural fill immediately behind the rockery facing should be reinforced with geosynthetic reinforcement. A recommended reinforced fill/rockery construction detail for a maximum wall height of eight feet is attached as Figure 4.

4.7 Infiltration Feasibility

Based on the subsurface conditions documented in the Test Pit Logs, it is our opinion that on-site infiltration is not a viable option for management of site stormwater. Based on the presence of mottling in the vast majority of soils observed at the site, it is also our opinion that the site conditions would generally not be suitable for applying other natural drainage practices (NDPs).

4.8 Stormwater Detention

As discussed, on-site detention of stormwater runoff will be provided by a buried vault located in the southern portion of the planned development area. Because the subsurface investigation performed by GCI occurred prior to development of the project plans, no location-specific exploration occurred in the proposed detention vault area. However, subsurface conditions appear quite uniform across the site. Therefore, we anticipate that dense to very dense glacial deposits exist at the planned bottom of vault elevation; however, this should be verified prior to construction.

Vault foundations supported by dense to very dense native soils at a depth greater than 8 feet may be designed for an allowable bearing capacity of 5,000 psf. For short-term loads, such as seismic, a one-third increase in this allowable capacity can be used. Friction at the base of foundations and passive earth pressure will provide resistance to these lateral loads. For designing foundations to resist lateral loads, a base friction coefficient of 0.35 can be used. Passive earth pressure acting on the sides of the vault footings may also be considered. We recommend calculating this lateral resistance using an equivalent fluid weight of 300 pounds per cubic foot (pcf).

The magnitude of earth pressures developing on the vault walls will depend in part on the quality and compaction of the wall backfill. We recommend placing and compacting wall backfill as structural fill as recommended in Section 4.2.

To prevent development of hydrostatic pressure and uplift on the vault, wall drainage must be installed. A typical recommended wall drainage detail is shown on Figure 3. If it is not possible to discharge collected water at the footing invert elevation, we recommend setting the invert elevation of the wall drainpipe equivalent to the outfall invert and connecting the drain to the outfall pipe for discharge.

With the recommended wall backfill and drainage, we recommend designing the vault walls for an earth pressure imposed by an equivalent fluid weighing 50 pcf. For any portion of the wall that falls below the invert elevation of the wall drain, an earth pressure equivalent to a fluid weighing 85 pcf should be used. For evaluating walls under seismic loading, an additional uniform earth pressure equivalent to 8H psf, where H is the height of the below-grade wall in feet, can be used. These values assume a horizontal backfill condition. If necessary, a uniform horizontal traffic surcharge value of 75 psf should be included in design of vault walls.

The vault will be subject to uplift pressures if drainage is not provided the full depth of the structure. The weight of the structure and the weight of the backfill soil above its foundation will provide resistance to uplift. A soil unit weight of 125 pcf can be used for the vault backfill provided the backfill is placed and compacted as structural fill as recommended in Section 4.2.

4.9 Drainage

Surface

Final exterior grades should promote free and positive drainage away from the building areas. We recommend providing a positive drainage gradient away from the building perimeter. If a positive gradient cannot be provided, provisions for collection and disposal of surface water adjacent to the structure should be provided.

Surface water from developed areas must not be allowed to flow in an uncontrolled and concentrated manner over the crests of site slopes and embankments. Surface water should be directed away from the slope crests to a point of collection and controlled discharge. If site grades do not allow for directing surface water away from the slopes, then the water should be collected and tightlined to an approved point of controlled discharge.

Subsurface

We recommend installing a continuous drain along the outside lower edge of the perimeter building foundations. The drains can be laid to grade at an invert elevation equivalent to the bottom of footing grade. The drains can consist of four-inch diameter perforated PVC pipe that is enveloped in washed ½- to ¾-inch gravel-sized drainage aggregate. The aggregate should extend six inches above and to the sides of the pipe. The foundation drains and roof downspouts should be tightlined separately to an approved point of controlled discharge. All drains should be provided with cleanouts at easily accessible locations. These cleanouts should be serviced at least once each year.

4.10 Utilities

Utility pipes should be bedded and backfilled in accordance with American Public Works Association (APWA) or local jurisdictional requirements. At minimum, trench backfill should be placed and compacted as structural fill as described in Section 4.2 of this report. As noted, soils excavated on-site should generally be suitable for use as backfill material. However, the vast majority of the site soils are fine grained and moisture sensitive; therefore, moisture conditioning may be necessary to facilitate proper compaction. If utility construction takes place during the winter, it may be necessary to import suitable wet weather fill for utility trench backfilling.

4.11 Pavements

Pavement subgrades should be prepared as described in the Section 4.2 of this report. Regardless of the degree of relative compaction achieved, the subgrade must be firm and relatively unyielding before paving. The subgrade should be proofrolled with heavy rubber-tire construction equipment such as a loaded 10-yard dump truck to verify this condition.

The pavement design section is dependent upon the supporting capability of the subgrade soils and the traffic conditions to which it will be subjected. For residential access, with traffic consisting mainly of light passenger vehicles with only occasional heavy traffic, and with a stable subgrade prepared as recommended, we recommend the following pavement sections:

- Two inches of hot mix asphalt (HMA) over six inches of crushed rock base (CRB)
- Five inches full depth HMA over prepared subgrade

The paving materials used should conform to the Washington State Department of Transportation (WSDOT) specifications for ½-inch class HMA and CRB.

Long-term pavement performance will depend on surface drainage. A poorly-drained pavement section will be subject to premature failure as a result of surface water infiltrating into the subgrade soils and reducing their supporting capability. For optimum pavement performance, we recommend surface drainage gradients of at least two percent. Some degree of longitudinal and transverse cracking of the pavement surface should be expected over time. Regular maintenance should be planned to seal cracks when they occur.

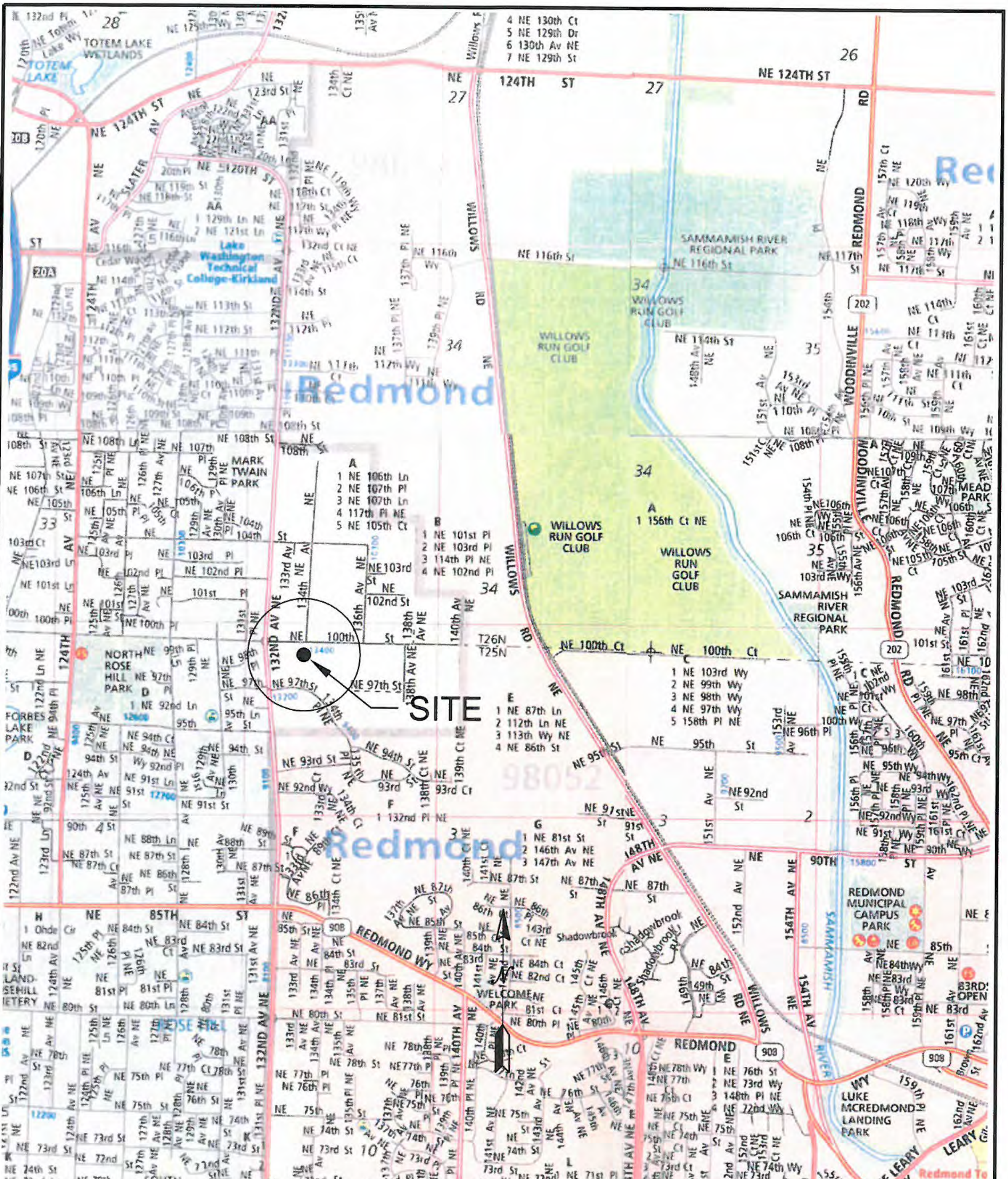
5.0 ADDITIONAL SERVICES

Terra Associates, Inc. should review the final designs and specifications in order to verify that earthwork and foundation recommendations have been properly interpreted and implemented in project design. We should also provide geotechnical services during construction in order to observe compliance with our design concepts, specifications, and recommendations. This will allow for design changes if subsurface conditions differ from those anticipated prior to the start of construction.

6.0 LIMITATIONS

We prepared this report in accordance with generally accepted geotechnical engineering practices. This report is the copyrighted property of Terra Associates, Inc. and is intended for specific application to the Heathers Ridge South project. This report is for the exclusive use of Quadrant Homes and their authorized representatives. No other warranty, expressed or implied, is made.

The analyses and recommendations presented in this report are based on data obtained from our on-site test pits. Variations in soil conditions can occur, the nature and extent of which may not become evident until construction. If variations appear evident, Terra Associates, Inc. should be requested to reevaluate the recommendations in this report prior to proceeding with construction.



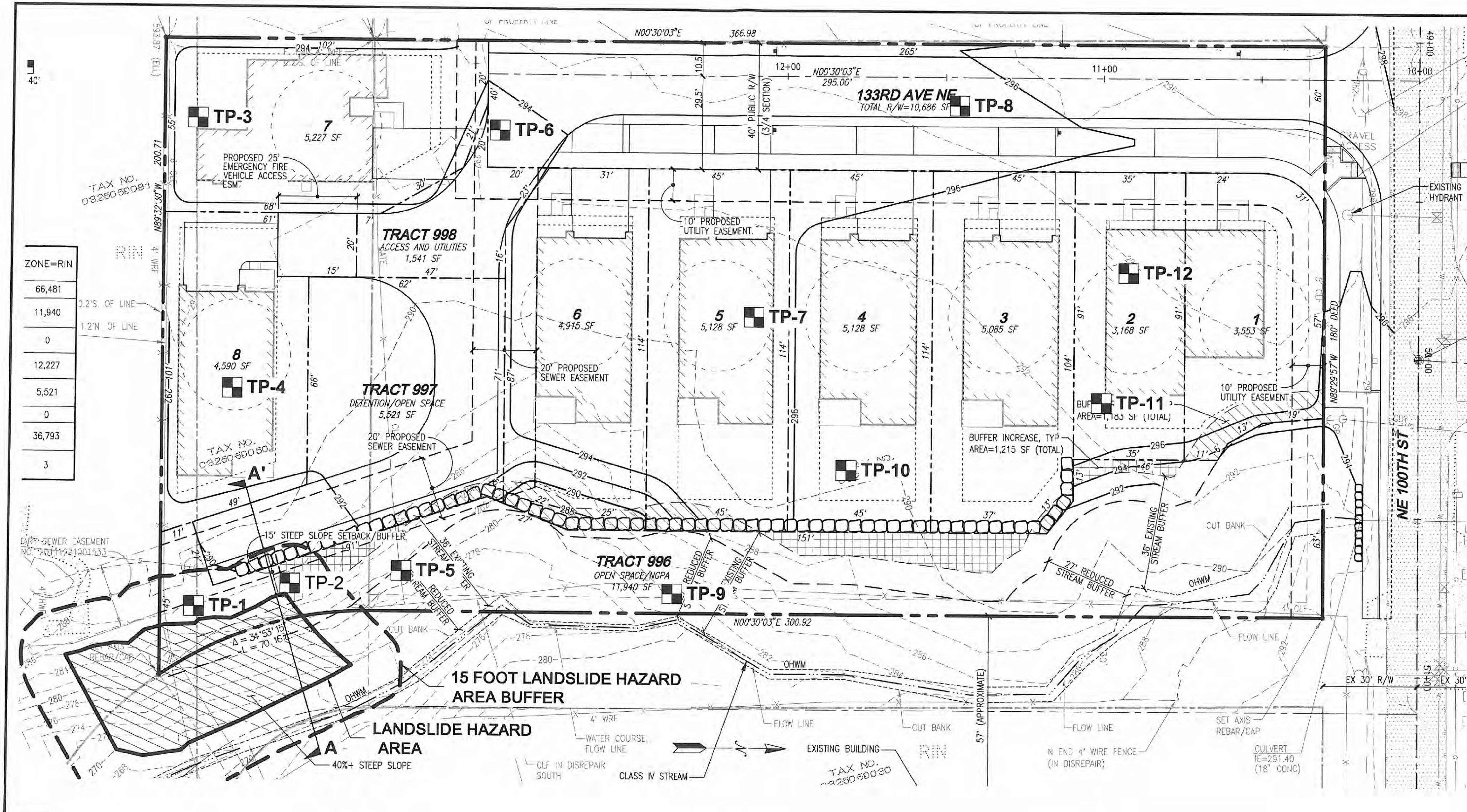
REFERENCE: THOMAS GUIDE (2008)

NOT TO SCALE

Terra Associates, Inc.
Consultants in Geotechnical Engineering
Geology and
Environmental Earth Sciences

VICINITY MAP
HEATHERS RIDGE SOUTH
REDMOND, WASHINGTON

Proj. No.T-7177	Date FEB 2015	Figure 1



NOTE:

THIS SITE PLAN IS SCHEMATIC. ALL LOCATIONS AND DIMENSIONS ARE APPROXIMATE. IT IS INTENDED FOR REFERENCE ONLY AND SHOULD NOT BE USED FOR DESIGN OR CONSTRUCTION PURPOSES.

REFERENCE:

SITE PLAN BY LDC

LEGEND:

APPROXIMATE TEST PIT LOCATION (GEOTECH CONSULTANTS, INC. 2011)

A A' SECTION A - A'

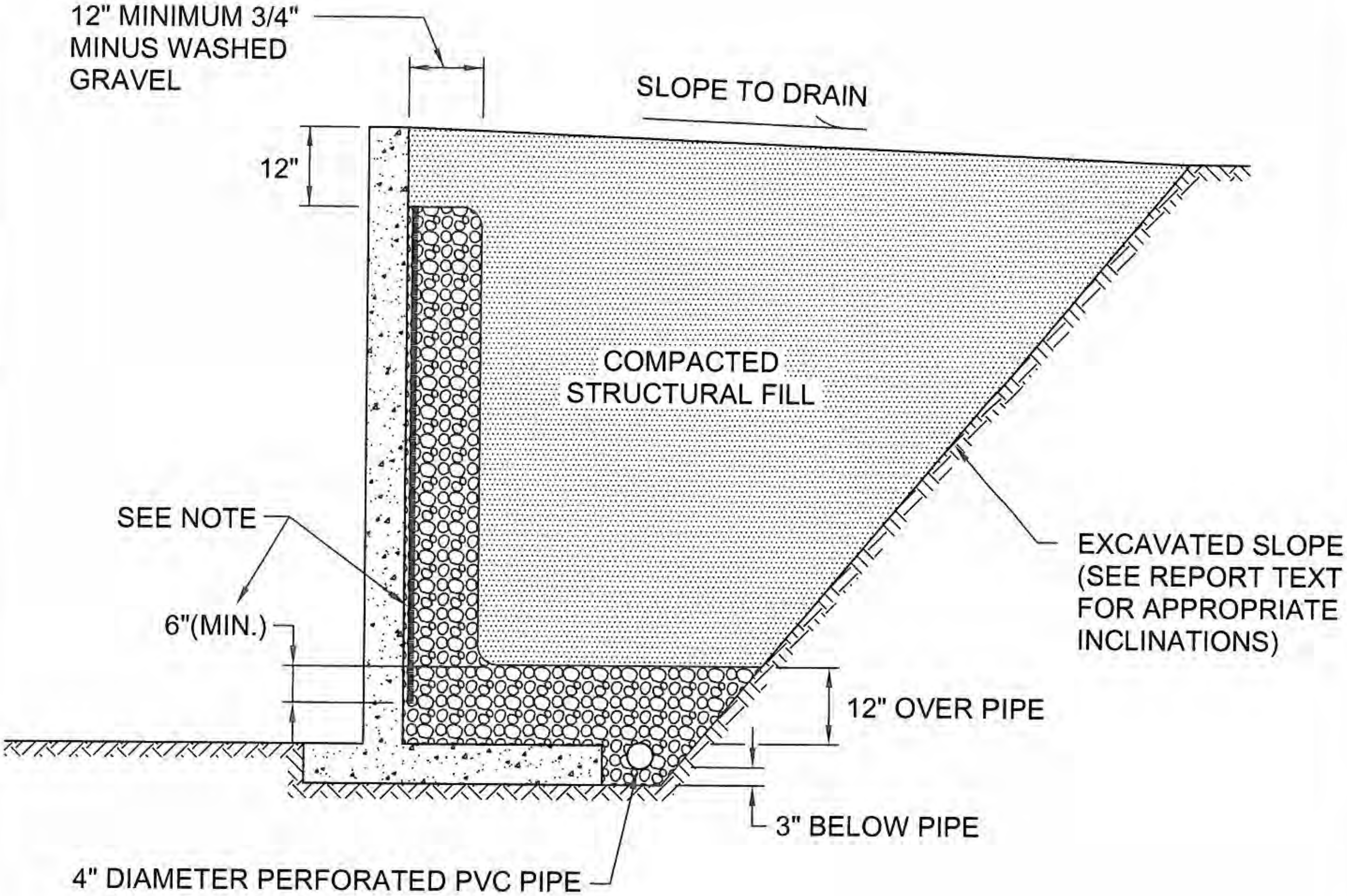
0 30 60

APPROXIMATE SCALE IN FEET

Terra Associates, Inc.
Consultants in Geotechnical Engineering
Geology and
Environmental Earth Sciences

EXPLORATION LOCATION PLAN
HEATHERS RIDGE SOUTH
REDMOND, WASHINGTON

Proj. No.T-7177	Date FEB 2015	Figure 2



NOT TO SCALE

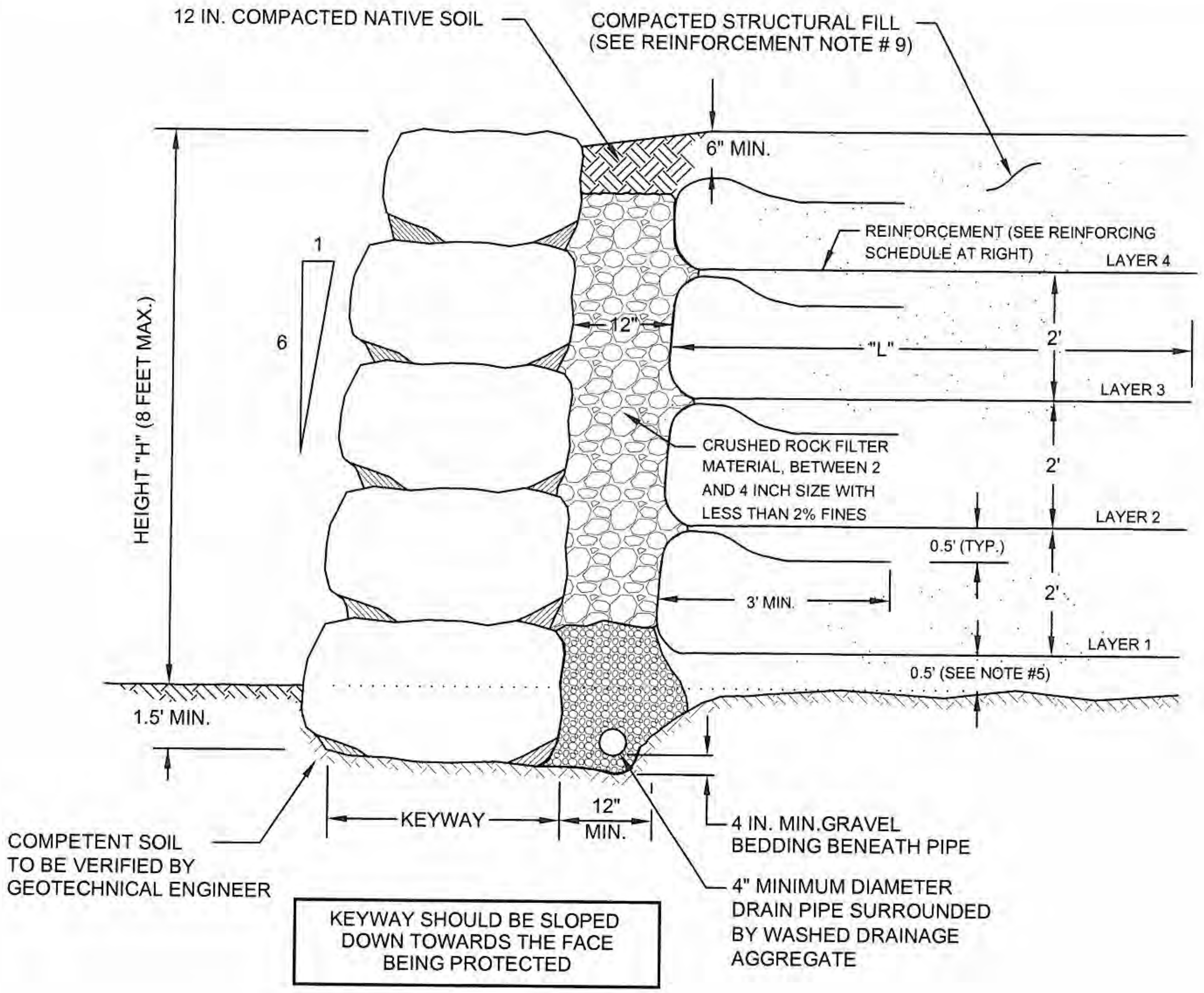
NOTE:

MIRADRAIN G100N PREFABRICATED DRAINAGE PANELS OR SIMILAR PRODUCT CAN BE SUBSTITUTED FOR THE 12-INCH WIDE GRAVEL DRAIN BEHIND WALL. DRAINAGE PANELS SHOULD EXTEND A MINIMUM OF SIX INCHES INTO 12-INCH THICK DRAINAGE GRAVEL LAYER OVER PERFORATED DRAIN PIPE.

Terra Associates, Inc.
Consultants in Geotechnical Engineering
Geology and
Environmental Earth Sciences

TYPICAL WALL DRAINAGE DETAIL
HEATHERS RIDGE SOUTH
REDMOND, WASHINGTON

Proj. No.T-7177	Date FEB 2015	Figure 3



NOT TO SCALE

ROCKERY NOTES

1. ROCKERY CONSTRUCTION SHALL BE COMPLETED IN ACCORDANCE WITH THE ASSOCIATION OF ROCKERY (ARC) CONTRACTORS GUIDELINES.
2. ROCK USED MUST MEET THE REQUIREMENTS FOR ROCK QUALITY SPECIFIED IN SECTIONS 9-13.7(1) OF THE WSDOT STANDARDS SPECIFICATIONS (2014).
3. ALL CAP ROCKS MUST BE SECURE AND NOT ABLE TO BE DISLODGED BY HAND.

REINFORCING SCHEDULE

ROCKERY HEIGHT 'H' (FEET)	REINFORCEMENT LAYER NO.	REINFORCEMENT (MIRAFI OR EQUIVALENT)	REINFORCEMENT LENGTH "L" (FEET)	REINFORCEMENT HEIGHT (FEET) (SEE NOTE NO. 4)
<4	NO REINFORCING REQUIRED			
4 (HORIZ. AND 3:1 BS)	1 2	MIRAFI HP570 MIRAFI HP570	3.5 3.5	0.5 2.5
4 (TRAFFIC SURCHARGE)	1 2	MIRAFI HP570 MIRAFI HP570	4.0 4.0	0.5 2.5
6 (SEE NOTE 10)	1 2 3	MIRAFI HP570 MIRAFI HP570 MIRAFI HP570	5.0 5.0 5.0	0.5 2.5 4.5
8 (SEE NOTE 10)	1 2 3 4	MIRAFI HP570 MIRAFI HP570 MIRAFI HP570 MIRAFI HP570	6.5 6.5 6.5 6.5	0.5 2.5 4.5 6.5

GENERAL REINFORCED FILL NOTES

1. REFER TO CIVIL DRAWINGS FOR ROCKERY ALIGNMENT, LOCATIONS AND ELEVATIONS.
2. REFER TO REINFORCING SCHEDULE FOR GEOTEXTILE REINFORCEMENT LENGTHS AND ELEVATIONS.
3. GEOTEXTILE SHALL BE INSTALLED BEHIND WALL WITH MACHINE DIRECTION (STRONGEST AXIS) PERPENDICULAR TO WALL.
4. GEOTEXTILE SHALL BE INSTALLED ON HORIZONTAL SURFACE OF COMPACTED STRUCTURAL FILL AT ELEVATIONS SHOWN ON SCHEDULE.
5. GEOTEXTILE LAYER HEIGHT MEASURED RELATIVE TO GRADE IN FRONT OF ROCKERY.
6. GEOTEXTILE SHALL BE PULLED TIGHT BEHIND WALL. STAKE END OF GEOGRID AS REQUIRED TO MAINTAIN TENSION BEFORE COVERING WITH STRUCTURAL FILL.
7. PROTECT GEOTEXTILE FROM CONSTRUCTION DAMAGE PER MANUFACTURERS CONSTRUCTION EQUIPMENT SHALL NOT TRAVEL DIRECTLY ON GEOTEXTILE. ANY GEOTEXTILE THAT IS DAMAGED SHALL BE REPLACED WITH NEW GEOTEXTILE AT CONTRACTORS EXPENSE.
8. GEOTEXTILE SHALL BE AS SHOWN ON SCHEDULE. ALL GEOTEXTILE SHALL BE CLEARLY IDENTIFIED AND LABELED IN THE FIELD. ANY UNMARKED ROLLS, OR PORTIONS THEREOF, OF GEOTEXTILE THAT CANNOT BE IDENTIFIED SHALL NOT BE USED IN WALL CONSTRUCTION.
9. ALL STRUCTURAL FILL TO BE PLACED AND COMPACTED PER GEOTECHNICAL REPORT PREPARED BY TERRA ASSOCIATES, INC., PROJECT NO. T-7177 DATED FEB 2015. STRUCTURAL FILL IN REINFORCED ZONE SHALL BE GRANULAR MATERIAL WITH A MAXIMUM AGGREGATE SIZE OF 3 INCHES AND A MAXIMUM OF 30 PERCENT PASSING THE NO. 200 SIEVE (FINES CONTENT) BASED ON THE 3/4" GRAVEL FRACTION.
10. REINFORCEMENT LENGTHS FOR 6' AND 8' ROCKERIES APPLICABLE FOR SURCHARGE CONDITION IMPOSED BY 3:1 (H:V) BACKSLOPE AND TRAFFIC.

Terra Associates, Inc. Consultants in Geotechnical Engineering Geology and Environmental Earth Sciences	REINFORCED FILL/ROCKERY DETAIL HEATHERS RIDGE SOUTH REDMOND, WASHINGTON		
	Proj. No.T-7177	Date FEB 2015	Figure 4

APPENDIX A

TEST PIT LOGS AND LABORATORY TESTING RESULTS BY OTHERS

TEST PIT 1

Depth (ft.)	Moisture Content (%)	Water Table	USCS	Description
			SM	Wood and deadfall over 1 foot of topsoil and roots, then Light-brown, slightly silty SAND with gravel,fine- to medium-grained, damp loose -becomes light-gray, medium-dense -becomes gray and dense to very dense (Glacial Till)

* Test Pit terminated at 5.5 feet on October 14, 2011.
* No groundwater seepage was observed during excavation.
* No caving observed during excavation.

TEST PIT 2

Depth (ft.)	Moisture Content (%)	Water Table	USCS	Description
			SM	Wood and deadfall over 1 foot of topsoil, then Light-gray to brown, slightly silty SAND with gravel, fine- to medium-grained, damp, loose -becomes gray, more silt, dense (Glacial Till)

* Test Pit terminated at 6.0 feet on October 14, 2011.
* No groundwater seepage was observed during excavation.
* No caving observed during excavation.

GEOTECH CONSULTANTS, INC.

TEST PIT LOG

N.E. 100th Street at 134th Ave. N.E.
Redmond Washington

Job	Date:	Logged by:	Plate:
11335	October 2011		3

TEST PIT 3

Depth (ft.)	Moisture Content (%)	Water Table	USCS	Description
0–8			SM	Wood and deadfall over 4- to 6-inches of topsoil, then Reddish-brown, silty SAND with gravel, fine- to medium-grained, damp, loose -becomes light brown, medium-dense - becomes mottled with light gray, dense (Glacial Till) -becomes gray, very dense

* **Test Pit terminated at 8 feet on October 14, 2011**
* **No groundwater seepage was observed during excavation.**
* **No caving observed during excavation.**

TEST PIT 4

Depth (ft.)	Moisture Content (%)	Water Table	USCS	Description
0–5.5			SM	Wood and deadfall over 4- to 6-inches of topsoil, then Reddish-brown, silty SAND with gravel, fine- to medium-grained, damp, loose -becomes light gray-brown mottled with orange, medium-dense -becomes dense (Glacial Till) -becomes light graydense to very dense

* **Test Pit terminated at 5.5 feet on October 14, 2011**
* **No groundwater seepage was observed during excavation.**
* **No caving observed during excavation.**

GEOTECH CONSULTANTS, INC.

TEST PIT LOG
N.E. 100th Street at 134th Ave. N.E.
Redmond Washington

Job	Date:	Logged by:	Plate:
11335	October 2011		4

TEST PIT 5

Depth (ft.)	Moisture Content (%)	Water Table	USCS	Description
			SM	One to 2 feet of organic silt topsoil and roots over
				Light-tan to brown, silty SAND with gravel, fine- to medium-grained, damp, medium-dense
				-becomes light-gray, dense (Glacial Till)
5				-becomes all gray, dense to very dense

* **Test Pit terminated at 6.5 feet on October 14, 2011**
* **No groundwater seepage was observed during excavation.**
* **No caving observed during excavation.**

10

TEST PIT 6

Depth (ft.)	Moisture Content (%)	Water Table	USCS	Description
			SM	Wood and deadfall over 6-inches of topsoil, then
				Light-brown to reddish-brown mottled with light-gray, silty SAND with gravel, fine- to medium-grained, damp, loose
				-becomes light gray, medium-dense
				-becomes gray, dense (Glacial Till)

5

* **Test Pit terminated at 4.5 feet on October 14, 2011**
* **No groundwater seepage was observed during excavation.**
* **No caving observed during excavation.**

10

GEOTECH
CONSULTANTS, INC.

TEST PIT LOG

N.E. 100th Street at 134th Ave. N.E.
Redmond Washington

Job	Date:	Logged by:	Plate:
11335	October 2011		5

TEST PIT 7

Depth (ft.) | Moisture Content (%) | Water Table | USCS | Description

SM

Six to 8 inches of Topsoil over
Light-brown to orange-brown, silty SAND with gravel, fine- to medium-grained, damp to moist, loose
-becomes light-gray mottled with orange, medium-dense

-becomes gray to light-gray, dense (Glacial Till)

5

10

*** Test Pit terminated at 7 feet on October 14, 2011**
*** No groundwater seepage was observed during excavation.**
*** No caving observed during excavation.**

TEST PIT 8

Depth (ft.) | Moisture Content (%) | Water Table | USCS | Description

SM

Four to 6 inches of Topsoil over
Light-brown to orange-brown, silty SAND with gravel, fine- to medium-grained, damp to moist, loose
-becomes light-gray mottled with some orange, medium-dense

-becomes gray and dense (Glacial Till)

-becomes very dense

5

10

*** Test Pit terminated at 6 feet on October 14, 2011**
*** No groundwater seepage was observed during excavation.**
*** No caving observed during excavation.**

GEOTECH
CONSULTANTS, INC.

TEST PIT LOG

N.E. 100th Street at 134th Ave. N.E.
Redmond Washington

Job	Date:	Logged by:	Plate:
11335	October 2011		6

TEST PIT 9

Depth (ft.)	Moisture Content (%)	Water Table	USCS	Description
			SM	Wood and deadfall over 4 to 6 inches of topsoil, then Light-brown to orange-brown, silty SAND with gravel, fine- to medium-grained, damp to moist, loose -becomes light-gray mottled with some orange, medium-dense -becomes gray, dense (Glacial Till)
5				
10				

* Test Pit terminated at 5 feet on October 14, 2011
* No groundwater seepage was observed during excavation.
* No caving observed during excavation.

TEST PIT 10

Depth (ft.)	Moisture Content (%)	Water Table	USCS	Description
			SM	Six to 8 inches of Topsoil over Light-brown to orange-brown, silty SAND with gravel, fine- to medium-grained, damp to moist, loose -becomes light-gray mottled with some orange, medium-dense -becomes gray and dense (Glacial Till)
5				
10				

* Test Pit terminated at 6 feet on October 14, 2011
* No groundwater seepage was observed during excavation.
* No caving observed during excavation.

GEOTECH
CONSULTANTS, INC.

TEST PIT LOG

N.E. 100th Street at 134th Ave. N.E.
Redmond Washington

Job	Date:	Logged by:	Plate:
11335	October 2011		7

TEST PIT 11

Depth (ft.)	Moisture Content (%)	Water Table	USCS	Description
			SM	Six to 8 inches of Topsoil over Brown to orange-brown, silty SAND with gravel, fine- to medium-grained, moist, loose -becomes light-gray mottled with some orange, medium-dense -becomes gray and dense (Glacial Till)
5				
10				

* Test Pit terminated at 6 feet on October 14, 2011
* No groundwater seepage was observed during excavation.
* No caving observed during excavation.

TEST PIT 12

Depth (ft.)	Moisture Content (%)	Water Table	USCS	Description
			SM	Six to 8 inches of Topsoil over Brown to orange-brown, silty SAND with gravel, fine- to medium-grained, moist, loose
	14.9%			-becomes light-gray mottled with orange, medium-dense
	13.9%			-becomes gray and dense (Glacial Till)
5				
10				

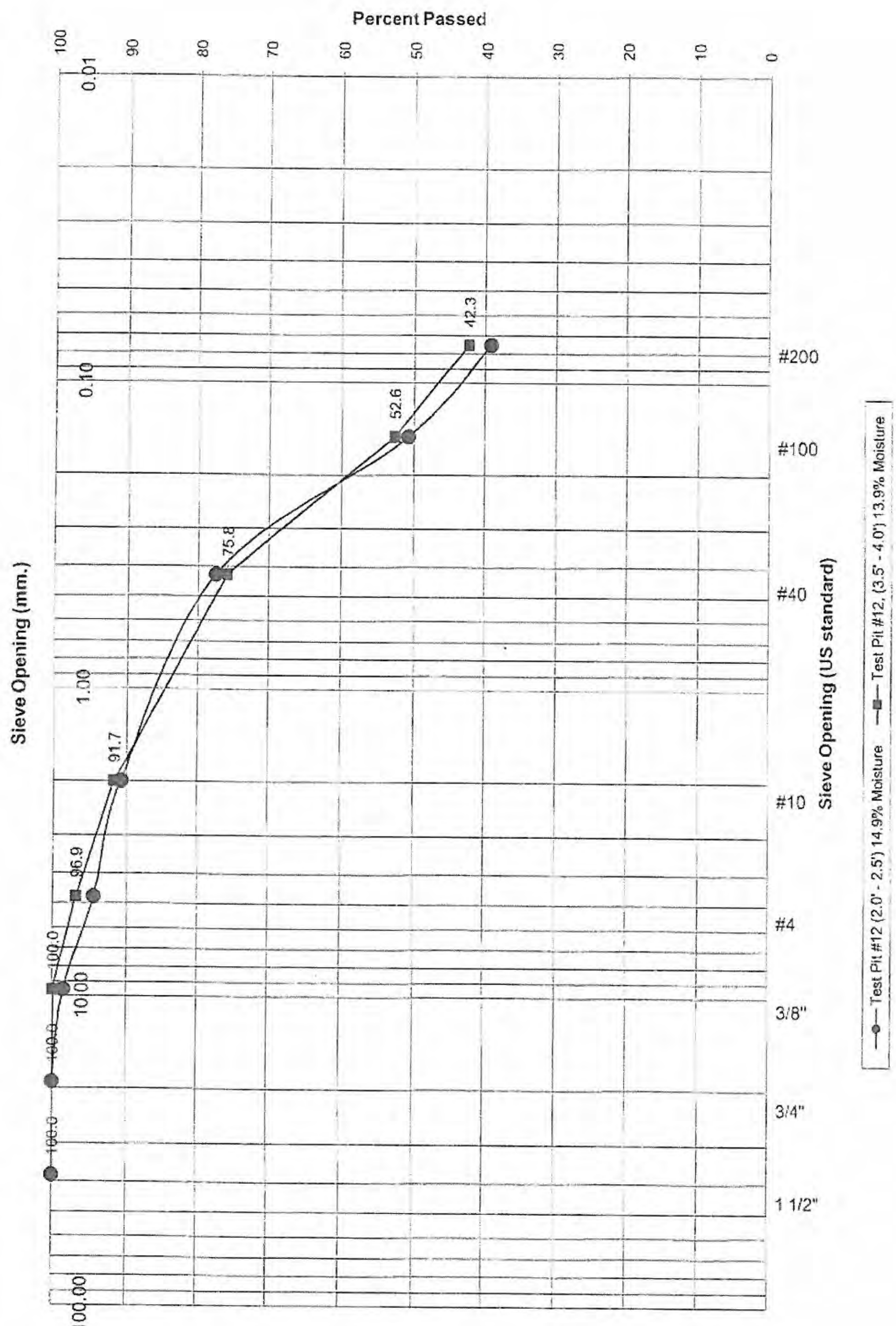
* Test Pit terminated at 7 feet on October 14, 2011
* No groundwater seepage was observed during excavation.
* No caving observed during excavation.

GEOTECH
CONSULTANTS, INC.

TEST PIT LOG

N.E. 100th Street at 134th Ave. N.E.
Redmond Washington

Job	Date:	Logged by:	Plate:
11335	October 2011		8



GEOTECH
CONSULTANTS, INC.

GRAIN SIZE ANALYSES

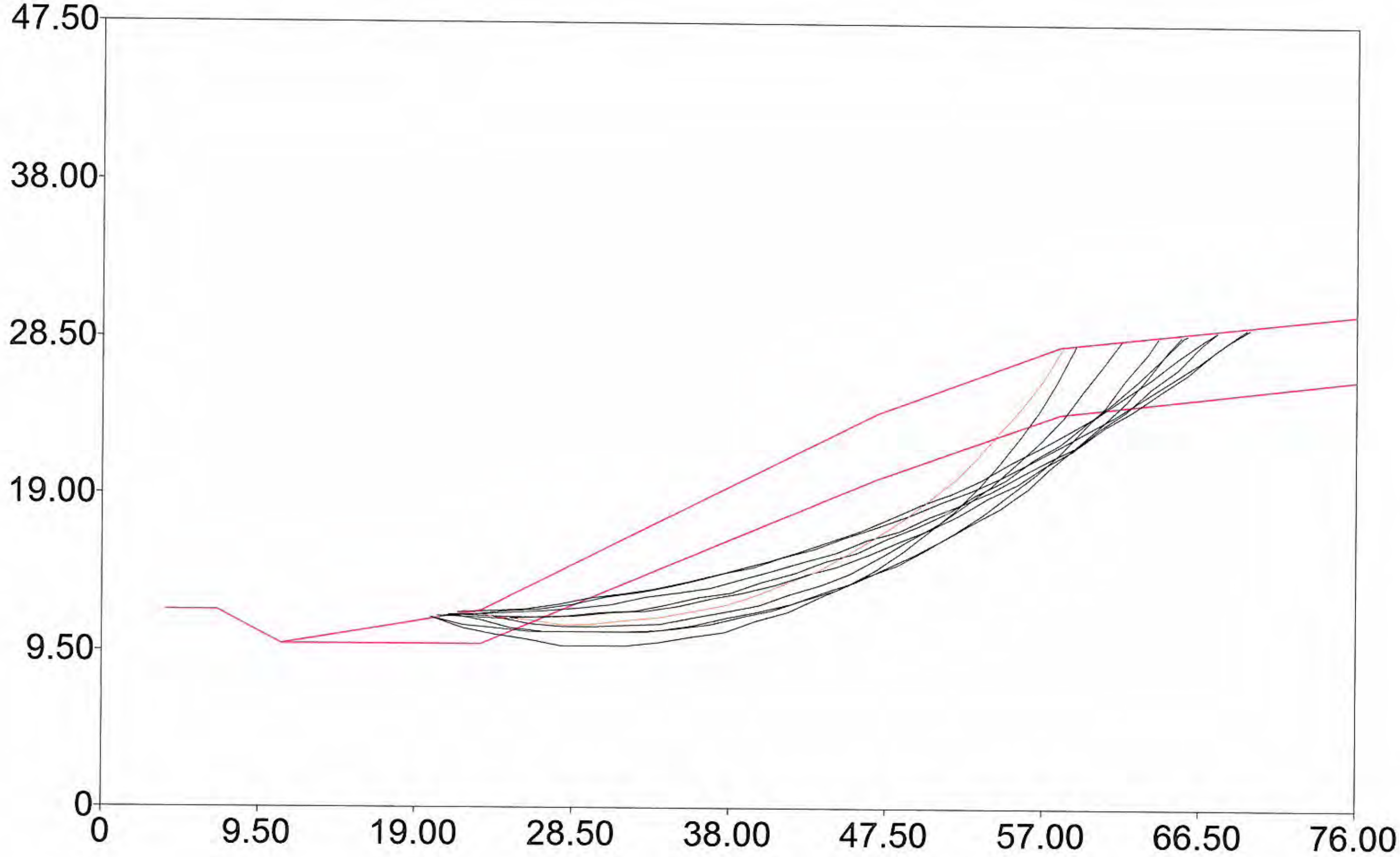
N.E. 100th Street at 134th Ave. N.E.
Redmond, Washington

Job No:	Date:		Plate:
11335	Oct. 2011		9

APPENDIX B

WINSTABL OUTPUT

A-A' Existing Static



Safety Factors

2.72
2.75
2.77
2.78
2.80
2.81
2.81
2.83
2.84
2.84

** PCSTABL6 **

by
Purdue University

modified by
Peter J. Bosscher
University of Wisconsin-Madison

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer`s Method of Slices

PROBLEM DESCRIPTION A-A' Existing Static

BOUNDARY COORDINATES

6 Top Boundaries
10 Total Boundaries

Type Bnd	Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Below
	1	4.00	12.00	7.00	12.00	2
	2	7.00	12.00	11.00	10.00	2
	3	11.00	10.00	23.00	12.00	1
	4	23.00	12.00	47.00	24.00	1
	5	47.00	24.00	58.00	28.00	1
	6	58.00	28.00	76.00	30.00	1
	7	11.00	10.00	23.00	10.00	2
	8	23.00	10.00	47.00	20.00	2
	9	47.00	20.00	58.00	24.00	2
	10	58.00	24.00	76.00	26.00	2

ISOTROPIC SOIL PARAMETERS

2 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	125.0	125.0	0.0	32.0	0.00	0.0	0
2	130.0	130.0	200.0	38.0	0.00	0.0	0

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.

100 Trial Surfaces Have Been Generated.

10 Surfaces Initiate From Each Of 10 Points Equally Spaced Along The Ground Surface Between X = 20.00 ft.
and X = 23.00 ft.

Each Surface Terminates Between X = 58.00 ft.
and X = 76.00 ft.

Unless Further Limitations Were Imposed, The Minimum Elevation At Which A Surface Extends Is Y = 2.00 ft.

2.00 ft. Line Segments Define Each Trial Failure Surface.

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Examined. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Bishop Method * *

Failure Surface Specified By 23 Coordinate Points

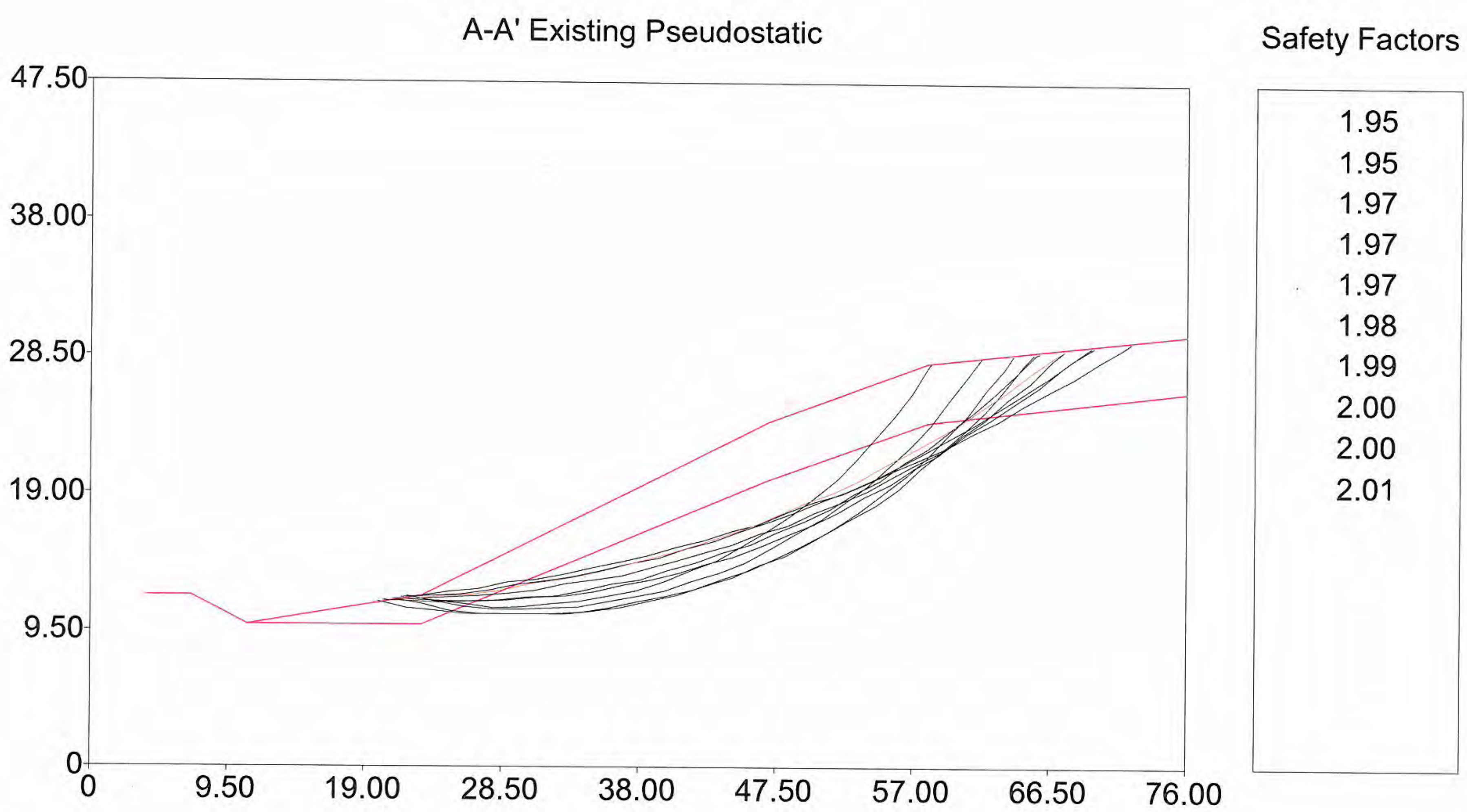
Point No.	X-Surf (ft)	Y-Surf (ft)
1	22.00	11.83
2	23.97	11.49
3	25.96	11.27
4	27.96	11.16
5	29.96	11.17
6	31.95	11.30
7	33.94	11.54
8	35.90	11.90
9	37.85	12.37
10	39.76	12.96
11	41.63	13.66
12	43.46	14.47
13	45.24	15.38
14	46.96	16.40
15	48.62	17.51
16	50.22	18.72
17	51.74	20.03
18	53.17	21.41
19	54.53	22.88
20	55.80	24.43
21	56.97	26.05
22	58.05	27.74
23	58.21	28.02

Circle Center At X = 28.8 ; Y = 45.2 and Radius, 34.1

*** 2.720 ***

Failure Surface Specified By 27 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	20.33	11.56
2	22.33	11.46
3	24.33	11.43
4	26.33	11.47
5	28.33	11.57
6	30.32	11.75
7	32.31	11.99
8	34.28	12.30
9	36.24	12.68
10	38.19	13.13
11	40.13	13.64
12	42.04	14.22


A-A' Existing Pseudostatic
47.50
38.00
28.50
19.00
9.50
0
0
9.50
19.00
28.50
38.00
47.50
57.00
66.50
76.00
Safety Factors
1.95
1.95
1.97
1.97
1.97
1.98
1.99
2.00
2.00
2.01

** PCSTABL6 **

by
Purdue University

modified by
Peter J. Bosscher
University of Wisconsin-Madison

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer`s Method of Slices

PROBLEM DESCRIPTION A-A' Existing Pseudostatic

BOUNDARY COORDINATES

6 Top Boundaries
10 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	4.00	12.00	7.00	12.00	2
2	7.00	12.00	11.00	10.00	2
3	11.00	10.00	23.00	12.00	1
4	23.00	12.00	47.00	24.00	1
5	47.00	24.00	58.00	28.00	1
6	58.00	28.00	76.00	30.00	1
7	11.00	10.00	23.00	10.00	2
8	23.00	10.00	47.00	20.00	2
9	47.00	20.00	58.00	24.00	2
10	58.00	24.00	76.00	26.00	2

ISOTROPIC SOIL PARAMETERS

2 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	125.0	125.0	0.0	32.0	0.00	0.0	0
2	130.0	130.0	200.0	38.0	0.00	0.0	0

A Horizontal Earthquake Loading Coefficient
Of0.150 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of0.000 Has Been Assigned

Cavitation Pressure =   0.0 psf

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.

100 Trial Surfaces Have Been Generated.

10 Surfaces Initiate From Each Of 10 Points Equally Spaced
Along The Ground Surface Between X = 20.00 ft.
and X = 23.00 ft.

Each Surface Terminates Between X = 58.00 ft.
and X = 76.00 ft.

Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 2.00 ft.

2.00 ft. Line Segments Define Each Trial Failure Surface.

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Examined. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Bishop Method * *

Failure Surface Specified By 27 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	20.67	11.61
2	22.66	11.73
3	24.66	11.90
4	26.65	12.11
5	28.63	12.38
6	30.60	12.69
7	32.57	13.05
8	34.53	13.45
9	36.48	13.91
10	38.41	14.41
11	40.34	14.96
12	42.24	15.55
13	44.14	16.20
14	46.02	16.88
15	47.88	17.62
16	49.72	18.39
17	51.54	19.22
18	53.35	20.08
19	55.13	20.99
20	56.89	21.95
21	58.62	22.94
22	60.33	23.98
23	62.01	25.06
24	63.67	26.18
25	65.30	27.34
26	66.90	28.53
27	67.58	29.06

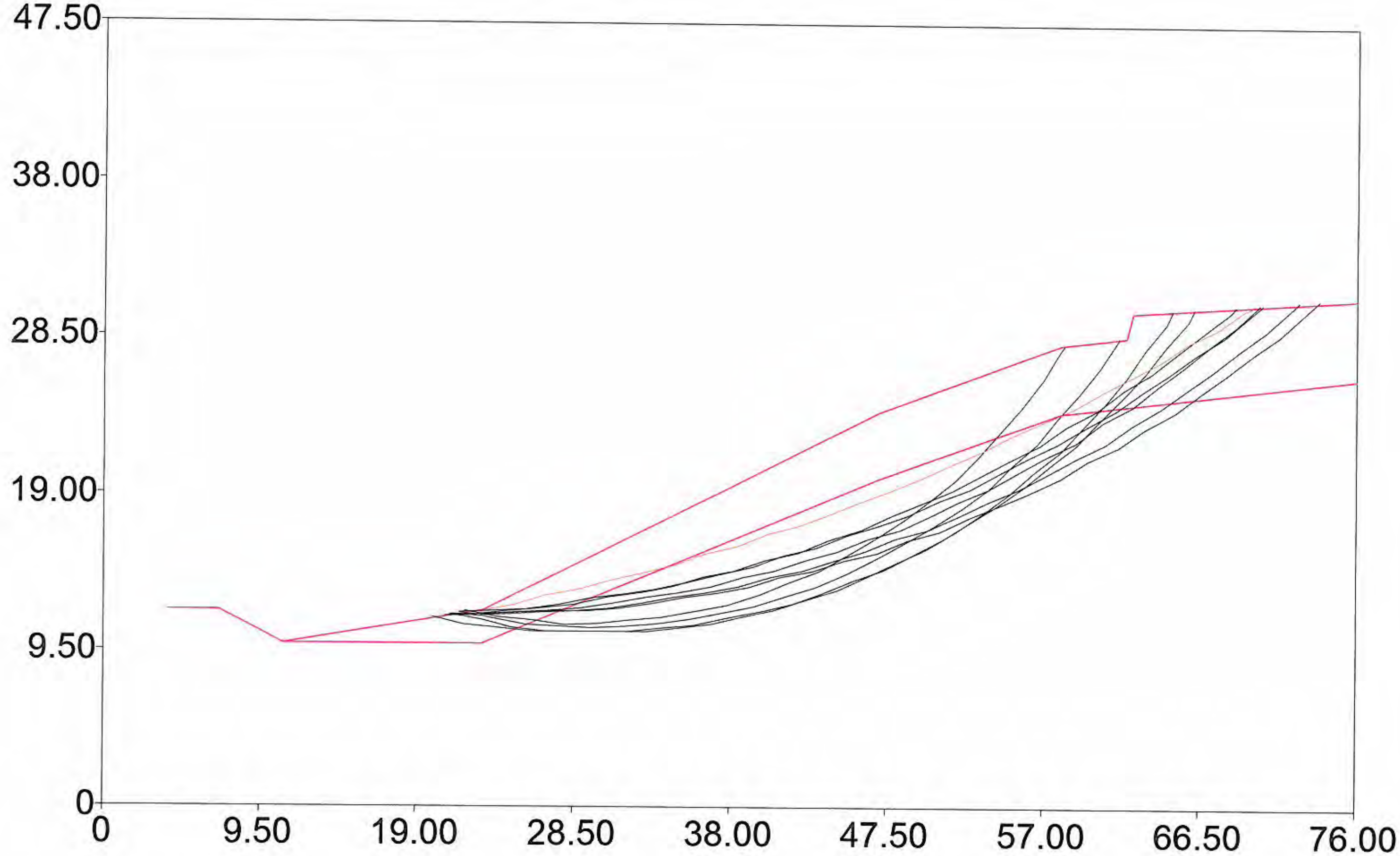
Circle Center At X = 16.8 ; Y = 93.8 and Radius, 82.3

*** 1.946 ***

Failure Surface Specified By 27 Coordinate Points

Point X-Surf Y-Surf

A-A' Proposed Static



Safety Factors

2.57
2.64
2.68
2.68
2.72
2.72
2.74
2.76
2.77
2.79

** PCSTABL6 **

by
Purdue University

modified by
Peter J. Bosscher
University of Wisconsin-Madison

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer`s Method of Slices

PROBLEM DESCRIPTION A-A' Proposed Static

BOUNDARY COORDINATES

8 Top Boundaries
12 Total Boundaries

Type Bnd	Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Below
	1	4.00	12.00	7.00	12.00	2
	2	7.00	12.00	11.00	10.00	2
	3	11.00	10.00	23.00	12.00	1
	4	23.00	12.00	47.00	24.00	1
	5	47.00	24.00	58.00	28.00	1
	6	58.00	28.00	62.00	28.50	1
	7	62.00	28.50	62.50	30.00	1
	8	62.50	30.00	76.00	31.00	1
	9	11.00	10.00	23.00	10.00	2
	10	23.00	10.00	47.00	20.00	2
	11	47.00	20.00	58.00	24.00	2
	12	58.00	24.00	76.00	26.00	2

ISOTROPIC SOIL PARAMETERS

2 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	125.0	125.0	0.0	32.0	0.00	0.0	0
2	130.0	130.0	200.0	38.0	0.00	0.0	0

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.

100 Trial Surfaces Have Been Generated.

10 Surfaces Initiate From Each Of 10 Points Equally Spaced
Along The Ground Surface Between X = 20.00 ft.
and X = 23.00 ft.

Each Surface Terminates Between X = 58.00 ft.
and X = 76.00 ft.

Unless Further Limitations Were Imposed, The Minimum Elevation At Which A Surface Extends Is Y = 2.00 ft.

2.00 ft. Line Segments Define Each Trial Failure Surface.

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Examined. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Bishop Method * *

Failure Surface Specified By 27 Coordinate Points

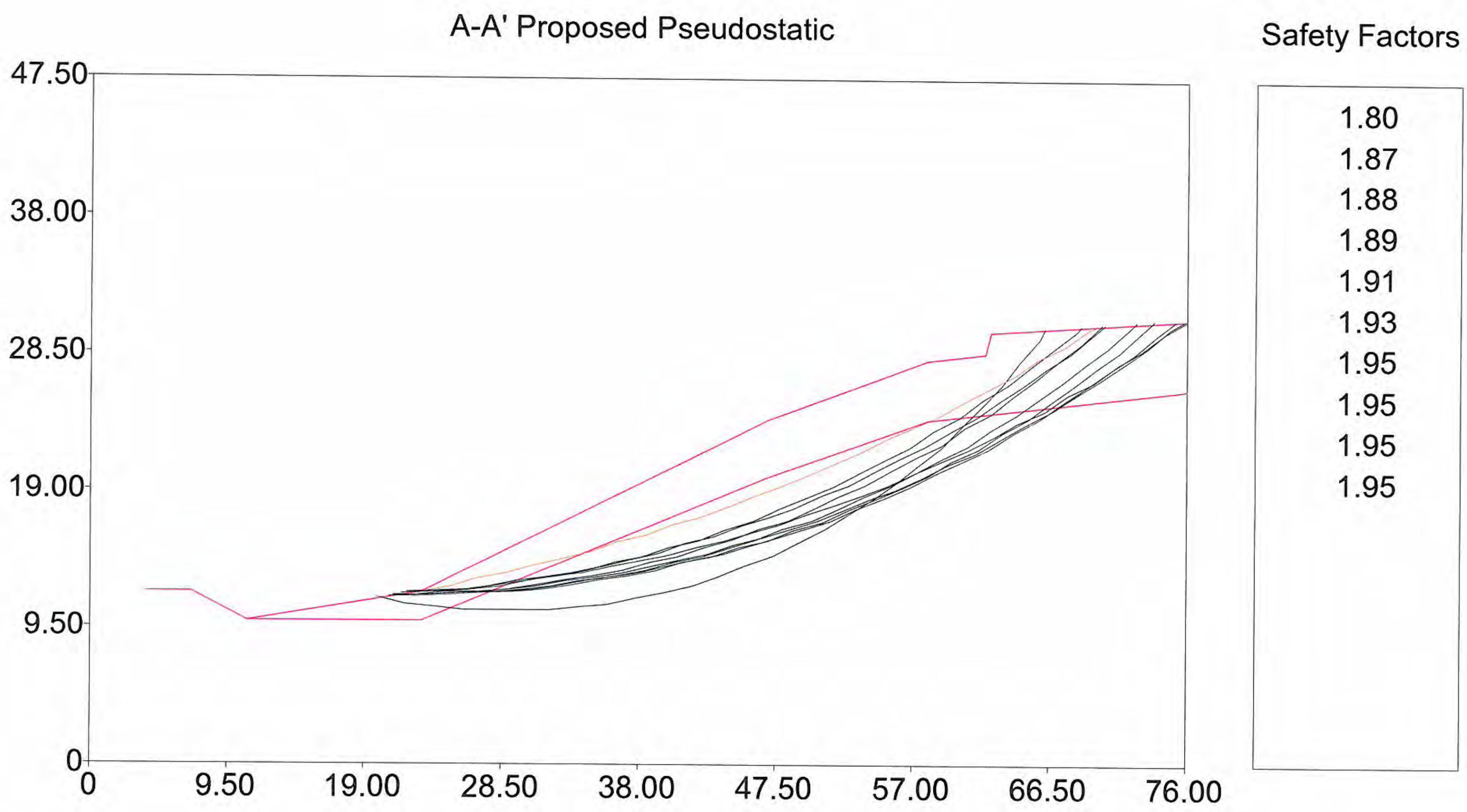
Point No.	X-Surf (ft)	Y-Surf (ft)
1	23.00	12.00
2	24.96	12.40
3	26.92	12.82
4	28.86	13.27
5	30.80	13.75
6	32.74	14.26
7	34.66	14.80
8	36.58	15.37
9	38.49	15.96
10	40.39	16.59
11	42.28	17.24
12	44.17	17.92
13	46.04	18.62
14	47.90	19.36
15	49.75	20.12
16	51.58	20.91
17	53.41	21.72
18	55.22	22.57
19	57.03	23.43
20	58.81	24.33
21	60.59	25.25
22	62.35	26.20
23	64.10	27.17
24	65.83	28.17
25	67.55	29.20
26	69.25	30.25
27	69.70	30.53

Circle Center At X = -2.8 ; Y = 145.2 and Radius, 135.7

*** 2.568 ***

Failure Surface Specified By 28 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	20.67	11.61
2	22.66	11.73
3	24.66	11.90
4	26.64	12.12
5	28.63	12.38
6	30.60	12.70


A-A' Proposed Pseudostatic
47.50
38.00
28.50
19.00
9.50
0
0
9.50
19.00
28.50
38.00
47.50
57.00
66.50
76.00
Safety Factors
1.80
1.87
1.88
1.89
1.91
1.93
1.95
1.95
1.95
1.95

** PCSTABL6 **

by
Purdue University

modified by
Peter J. Bosscher
University of Wisconsin-Madison

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer`s Method of Slices

PROBLEM DESCRIPTION   A-A' Proposed Pseudostatic

BOUNDARY COORDINATES

8 Top Boundaries
12 Total Boundaries

Type Bnd	Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Below
	1	4.00	12.00	7.00	12.00	2
	2	7.00	12.00	11.00	10.00	2
	3	11.00	10.00	23.00	12.00	1
	4	23.00	12.00	47.00	24.00	1
	5	47.00	24.00	58.00	28.00	1
	6	58.00	28.00	62.00	28.50	1
	7	62.00	28.50	62.50	30.00	1
	8	62.50	30.00	76.00	31.00	1
	9	11.00	10.00	23.00	10.00	2
	10	23.00	10.00	47.00	20.00	2
	11	47.00	20.00	58.00	24.00	2
	12	58.00	24.00	76.00	26.00	2

ISOTROPIC SOIL PARAMETERS

2 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	125.0	125.0	0.0	32.0	0.00	0.0	0
2	130.0	130.0	200.0	38.0	0.00	0.0	0

A Horizontal Earthquake Loading Coefficient
Of0.150 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of0.000 Has Been Assigned

Cavitation Pressure = 0.0 psf

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.

100 Trial Surfaces Have Been Generated.

10 Surfaces Initiate From Each Of 10 Points Equally Spaced
Along The Ground Surface Between X = 20.00 ft.
and X = 23.00 ft.

Each Surface Terminates Between X = 58.00 ft.
and X = 76.00 ft.

Unless Further Limitations Were Imposed, The Minimum Elevation At Which A Surface Extends Is Y = 2.00 ft.

2.00 ft. Line Segments Define Each Trial Failure Surface.

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Examined. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Bishop Method * *

Failure Surface Specified By 27 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	23.00	12.00
2	24.96	12.40
3	26.92	12.82
4	28.86	13.27
5	30.80	13.75
6	32.74	14.26
7	34.66	14.80
8	36.58	15.37
9	38.49	15.96
10	40.39	16.59
11	42.28	17.24
12	44.17	17.92
13	46.04	18.62
14	47.90	19.36
15	49.75	20.12
16	51.58	20.91
17	53.41	21.72
18	55.22	22.57
19	57.03	23.43
20	58.81	24.33
21	60.59	25.25
22	62.35	26.20
23	64.10	27.17
24	65.83	28.17
25	67.55	29.20
26	69.25	30.25
27	69.70	30.53

Circle Center At X = -2.8 ; Y = 145.2 and Radius, 135.7

*** 1.800 ***

Failure Surface Specified By 28 Coordinate Points

APPENDIX C

REINFORCED FILL/ROCKERY DESIGN CALCULATIONS

AASHTO DESIGN METHOD
Heathers Ridge South

PROJECT IDENTIFICATION

Title:	Heathers Ridge South
Project Number:	T-7177
Client:	Quadrant Homes
Designer:	JCS
Station Number:	N/A

Description:

4-foot high geotextile wrap-face reinforced fill rockery.

Company's information:

Name: Terra Associates, Inc.
Street: 12525 Willows Rd.
Ste. 101
Kirkland, WA 98034
Telephone #: 425-821-7777
Fax #: 425-821-4334
E-Mail: jsadler@terra-associates.com

Original file path and name: G:\USERS\JSADLER\7000\7177 Ellsworth Property\Reinforce.....
.....ft Reinf Rockery.BEN
Original date and time of creating this file: February 13, 2015

PROGRAM MODE: ANALYSIS
of a SIMPLE STRUCTURE
using GEOTEXTILE as reinforcing material.

SOIL DATA

REINFORCED SOIL

Unit weight, γ	125.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	34.0 °

RETAINED SOIL

Unit weight, γ	125.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	32.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{equiv.}$	125.0 lb/ft ^{3}
Equivalent internal angle of friction, $\phi_{equiv.}$	32.0 °
Equivalent cohesion, c $_{equiv.}$	100.0 lb/ft ^{2}

Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Inclination of internal slip plane, ψ= 62.00° (see Fig. 28 in DEMO 82).
Ka (external stability) = 0.3073 (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 35.49 N γ= 30.21

SEISMICITY

Maximum ground acceleration coefficient, A = 0.290
Design acceleration coefficient in Internal Stability: Kh = Am = 0.336
Design acceleration coefficient in External Stability: Kh = 0.134 (Am = 0.134)
(Kh in External Stability is based on allowable displacement, d = 100 mm. using FHWA-NHI-00-043 equation)

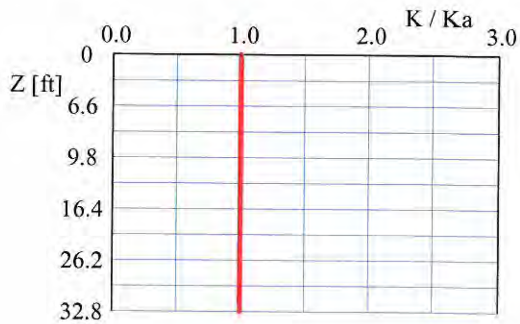
Kae (Kh > 0) = 0.3310 Kae (Kh = 0) = 0.2474 Δ Kae = 0.0836 (see eq. 37 in DEMO 82)
Seismic soil-geotextile friction coefficient, F* is 80.0% of its specified static value.

INPUT DATA: Geotextiles (Analysis)

D A T A		Geotextile type #1	Geotextile type #2	Geotextile type #3	Geotextile type #4	Geotextile type #5
Tult [lb/ft]		4800.0				
Durability reduction factor, RFd		1.10				
Installation-damage reduction factor, RFid		1.20				
Creep reduction factor, RFc		1.63	N/A	N/A	N/A	N/A
Fs-overall for strength		N/A				
Coverage ratio, Rc		1.000				
Friction angle along geotextile-soil interface,	ρ	28.35				
Pullout resistance factor, F*		$0.80 \cdot \tan\phi$	N/A	N/A	N/A	N/A
Scale-effect correction factor,	α	0.8				

Variation of Lateral Earth Pressure Coefficient With Depth

Z	K / Ka
0 ft	1.00
3.3 ft	1.00
6.6 ft	1.00
9.8 ft	1.00
13.1 ft	1.00
16.4 ft	1.00
19.7 ft	1.00

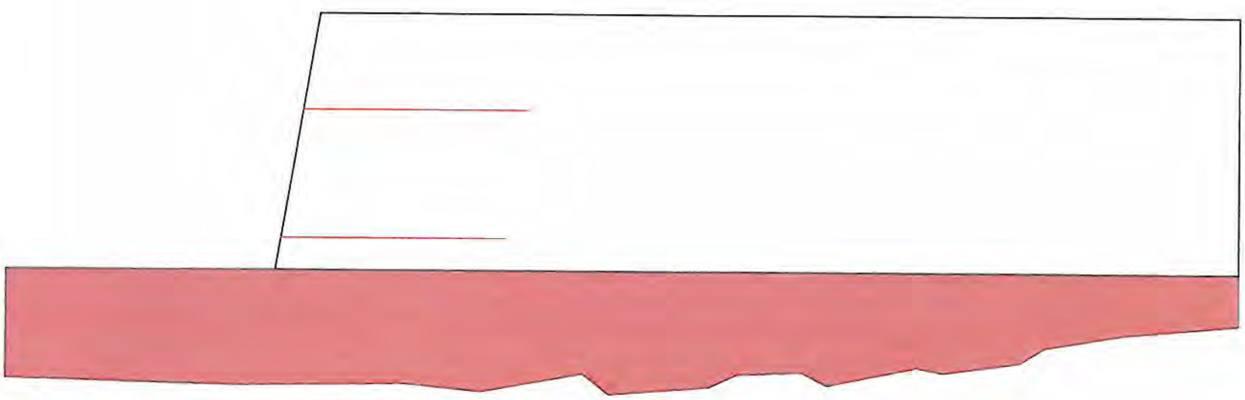


INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

Design height, Hd	4.00	[ft]	{ Embedded depth is E = 0.00 ft, and height above top of finished bottom grade is H = 4.00 ft }
Batter, ω	9.5	[deg]	
Backslope, β	0.0	[deg]	
Backslope rise	0.0	[ft]	Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82)

UNIFORM SURCHARGE
Uniformly distributed dead load is 0.0 [lb/ft ^{2}]

ANALYZED REINFORCEMENT LAYOUT:



SCALE:

0 2 4 6 [ft]

ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, Fs = 20.45, Meyerhof stress = 479 lb/ft².

Foundation Interface: Direct sliding, Fs = 3.474, Eccentricity, e/L = 0.0278, Fs-overturning = 7.38

GEOTEXTILE				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [pullout resistance]	Fs-overall [connection break]	Fs-overall [geotextile strength]	Geotextile strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.50	3.50	1	N/A	N/A	N/A	12.949	6.969	3.219	0.0153	Mirafi HP570
2	2.50	3.50	1	N/A	N/A	N/A	20.201	3.795	7.902	-0.0080	Mirafi HP570

ANALYSIS: CALCULATED FACTORS (Seismic conditions)

Bearing capacity, Fs = 16.67, Meyerhof stress = 543 lb/ft².

Foundation Interface: Direct sliding, Fs = 2.315, Eccentricity, e/L = 0.0837, Fs-overturning = 4.21

GEOTEXTILE				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [pullout resistance]	Fs-overall [connection break]	Fs-overall [geotextile strength]	Geotextile strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.50	3.50	1	N/A	N/A	N/A	10.400	3.983	2.145	0.0575	Mirafi HP570
2	2.50	3.50	1	N/A	N/A	N/A	15.558	2.042	5.266	-0.0006	Mirafi HP570

AASHTO DESIGN METHOD
Heathers Ridge South

PROJECT IDENTIFICATION

Title:	Heathers Ridge South
Project Number:	T-7177
Client:	Quadrant Homes
Designer:	JCS
Station Number:	N/A

Description:

4-foot high geotextile wrap-face reinforced fill rockery (3-1 Backslope)

Company's information:

Name: Terra Associates, Inc.
Street: 12525 Willows Rd.
Ste. 101
Kirkland, WA 98034
Telephone #: 425-821-7777
Fax #: 425-821-4334
E-Mail: jsadler@terra-associates.com

Original file path and name: G:\USERS\JSADLER\7000\7177 Ellsworth Property\Reinforce..... ..... (3-1 Backslope).BEN

Original date and time of creating this file: February 13, 2015

PROGRAM MODE:

ANALYSIS
of a SIMPLE STRUCTURE
using GEOTEXTILE as reinforcing material.

SOIL DATA

REINFORCED SOIL

Unit weight, γ	125.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	34.0 °

RETAINED SOIL

Unit weight, γ	125.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	32.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{equiv.}$	125.0 lb/ft ^{3}
Equivalent internal angle of friction, $\phi_{equiv.}$	32.0 °
Equivalent cohesion, $c_{equiv.}$	100.0 lb/ft ^{2}

Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Inclination of internal slip plane, ψ = 62.00° (see Fig. 28 in DEMO 82).
Ka (external stability) = 0.3602 (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 35.49 N γ= 30.21

SEISMICITY

Maximum ground acceleration coefficient, A = 0.290
Design acceleration coefficient in Internal Stability: Kh = Am = 0.336
Design acceleration coefficient in External Stability: Kh = 0.134 (Am = 0.134)
(Kh in External Stability is based on allowable displacement, d = 100 mm. using FHWA-NHI-00-043 equation)

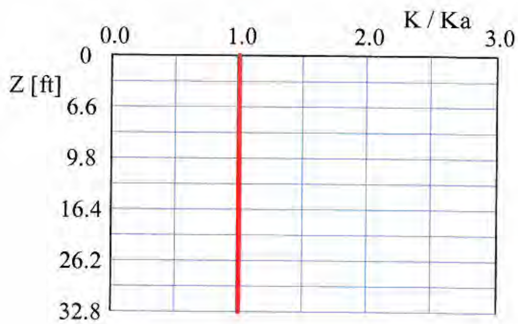
Kae (Kh > 0) = 0.4349 Kae (Kh = 0) = 0.2749 Δ Kae = 0.1600 (see eq. 37 in DEMO 82)
Seismic soil-geotextile friction coefficient, F* is 80.0% of its specified static value.

INPUT DATA: Geotextiles (Analysis)

D A T A		Geotextile type #1	Geotextile type #2	Geotextile type #3	Geotextile type #4	Geotextile type #5
Tult [lb/ft]		4800.0				
Durability reduction factor, RFd		1.10				
Installation-damage reduction factor, RFid		1.20				
Creep reduction factor, RFc		1.63	N/A	N/A	N/A	N/A
Fs-overall for strength		N/A				
Coverage ratio, Rc		1.000				
Friction angle along geotextile-soil interface,	ρ	28.35				
Pullout resistance factor, F*		$0.80 \cdot \tan\phi$	N/A	N/A	N/A	N/A
Scale-effect correction factor, α		0.8				

Variation of Lateral Earth Pressure Coefficient With Depth

Z	K / Ka
0 ft	1.00
3.3 ft	1.00
6.6 ft	1.00
9.8 ft	1.00
13.1 ft	1.00
16.4 ft	1.00
19.7 ft	1.00

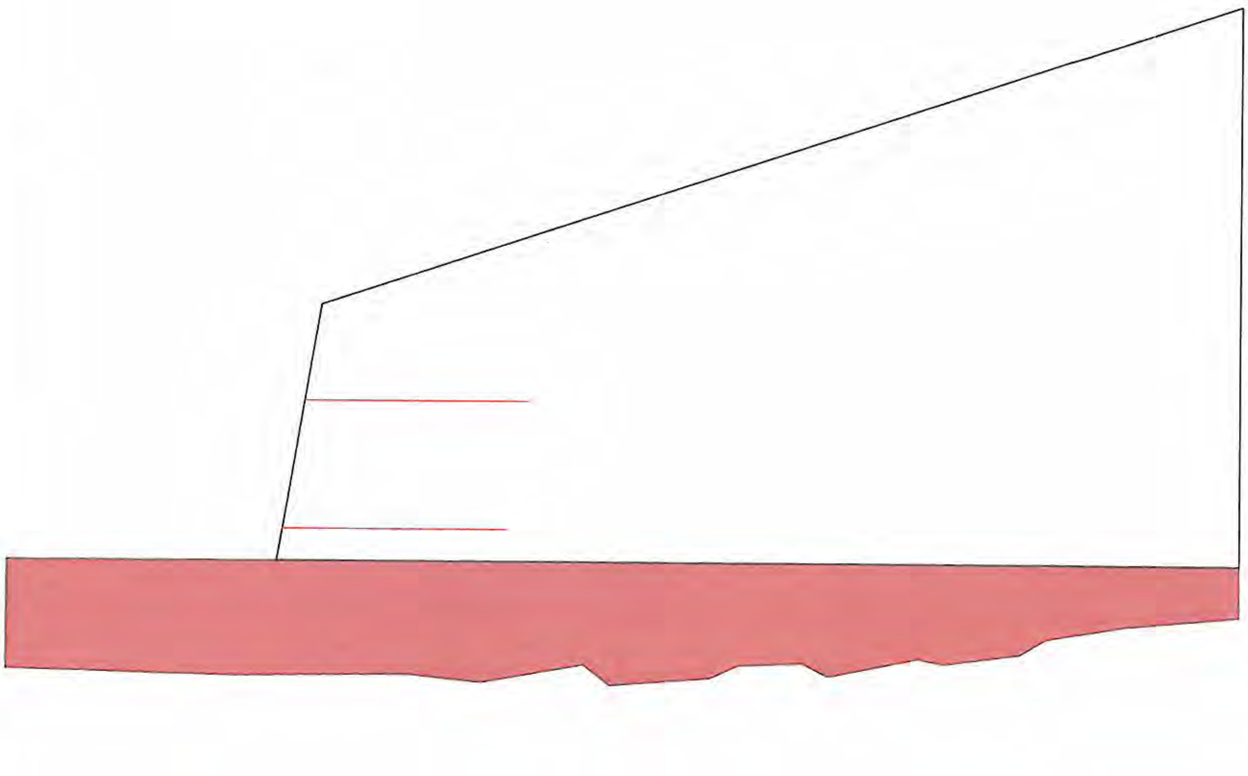


INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

Design height, Hd	4.00	[ft]	{ Embedded depth is E = 0.00 ft, and height above top of finished bottom grade is H = 4.00 ft }
Batter, ω	9.5	[deg]	
Backslope, β	18.3	[deg]	
Backslope rise	6.0	[ft]	Broken back equivalent angle, I = 18.26° (see Fig. 25 in DEMO 82)

UNIFORM SURCHARGE
Uniformly distributed dead load is 0.0 [lb/ft ^{2}]

ANALYZED REINFORCEMENT LAYOUT:



SCALE:

0 2 4 6 [ft]

ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, Fs = 17.03, Meyerhof stress = 577 lb/ft².
Foundation Interface: Direct sliding, Fs = 2.488, Eccentricity, e/L = 0.0248, Fs-overturning = 4.73

GEOTEXTILE				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [pullout resistance]	Fs-overall [connection break]	Fs-overall [geotextile strength]	Geotextile strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.50	3.50	1	N/A	N/A	N/A	11.338	6.870	2.179	0.0086	Mirafi HP570
2	2.50	3.50	1	N/A	N/A	N/A	14.750	3.962	3.423	-0.0461	Mirafi HP570

ANALYSIS: CALCULATED FACTORS (Seismic conditions)

Bearing capacity, Fs = 12.51, Meyerhof stress = 703 lb/ft².
Foundation Interface: Direct sliding, Fs = 1.745, Eccentricity, e/L = 0.1007, Fs-overturning = 2.96

GEOTEXTILE				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [pullout resistance]	Fs-overall [connection break]	Fs-overall [geotextile strength]	Geotextile strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.50	3.50	1	N/A	N/A	N/A	9.100	3.923	1.556	0.0666	Mirafi HP570
2	2.50	3.50	1	N/A	N/A	N/A	11.803	2.253	2.791	-0.0356	Mirafi HP570

AASHTO DESIGN METHOD
Heathers Ridge South

PROJECT IDENTIFICATION

Title:	Heathers Ridge South
Project Number:	T-7177
Client:	Quadrant Homes
Designer:	JCS
Station Number:	N/A

Description:

4-foot high geotextile wrap-face reinforced fill rockery (Traffic Surcharge)

Company's information:

Name: Terra Associates, Inc.
Street: 12525 Willows Rd.
Ste. 101
Kirkland, WA 98034
Telephone #: 425-821-7777
Fax #: 425-821-4334
E-Mail: jsadler@terra-associates.com

Original file path and name: G:\USERS\JSADLER\7000\7177 Ellsworth Property\Reinforce.....
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Original date and time of creating this file: February 13, 2015

PROGRAM MODE:

ANALYSIS
of a SIMPLE STRUCTURE
using GEOTEXTILE as reinforcing material.

SOIL DATA

REINFORCED SOIL

Unit weight, γ	125.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	34.0 °

RETAINED SOIL

Unit weight, γ	125.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	32.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{equiv.}$	125.0 lb/ft ^{3}
Equivalent internal angle of friction, $\phi_{equiv.}$	32.0 °
Equivalent cohesion, $c_{equiv.}$	100.0 lb/ft ^{2}

Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Inclination of internal slip plane, ψ= 62.00° (see Fig. 28 in DEMO 82).
Ka (external stability) = 0.3073 (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 35.49 N_γ= 30.21

SEISMICITY

Maximum ground acceleration coefficient, A = 0.290
Design acceleration coefficient in Internal Stability: Kh = Am = 0.336
Design acceleration coefficient in External Stability: Kh = 0.134 (Am = 0.134)
(Kh in External Stability is based on allowable displacement, d = 100 mm. using FHWA-NHI-00-043 equation)

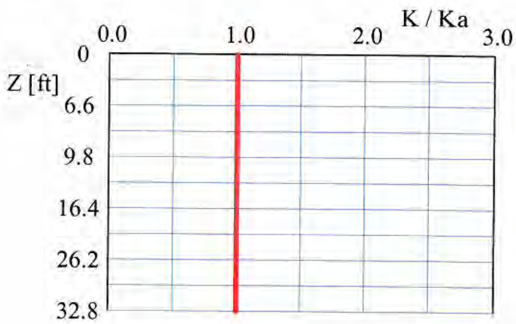
Kae (Kh > 0) = 0.3310 Kae (Kh = 0) = 0.2474 Δ Kae = 0.0836 (see eq. 37 in DEMO 82)
Seismic soil-geotextile friction coefficient, F* is 80.0% of its specified static value.

INPUT DATA: Geotextiles (Analysis)

D A T A	Geotextile type #1	Geotextile type #2	Geotextile type #3	Geotextile type #4	Geotextile type #5
Tult [lb/ft]	4800.0				
Durability reduction factor, RFd	1.10				
Installation-damage reduction factor, RFid	1.20				
Creep reduction factor, RFc	1.63	N/A	N/A	N/A	N/A
Fs-overall for strength	N/A				
Coverage ratio, Rc	1.000				
Friction angle along geotextile-soil interface, ρ	28.35				
Pullout resistance factor, F*	$0.80 \cdot \tan\phi$	N/A	N/A	N/A	N/A
Scale-effect correction factor, α	0.8				

Variation of Lateral Earth Pressure Coefficient With Depth

Z	K / Ka
0 ft	1.00
3.3 ft	1.00
6.6 ft	1.00
9.8 ft	1.00
13.1 ft	1.00
16.4 ft	1.00
19.7 ft	1.00



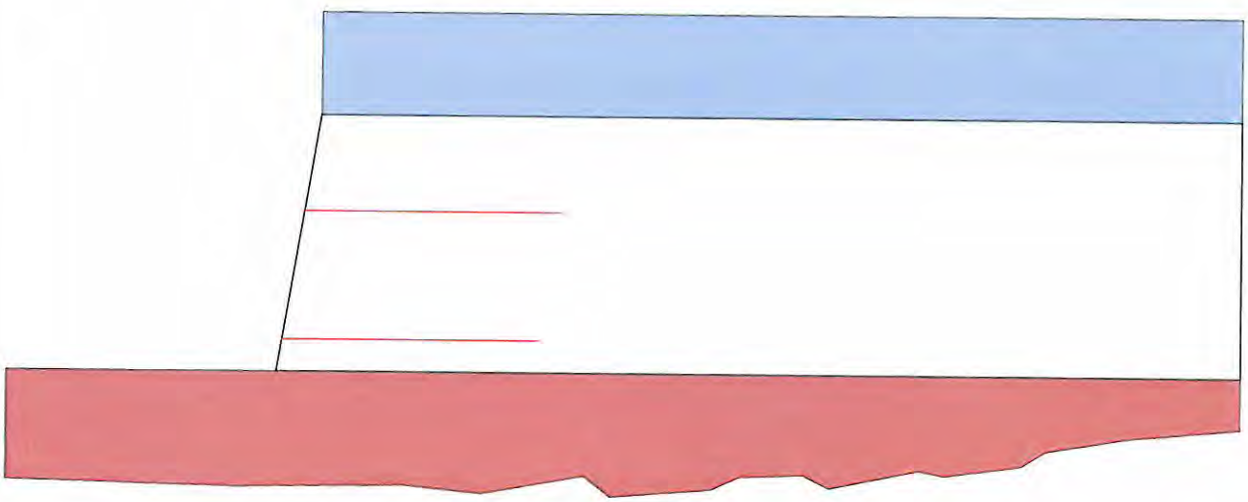
INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

Design height, Hd	4.00	[ft]	{ Embedded depth is E = 0.00 ft, and height above top of finished bottom grade is H = 4.00 ft }
Batter, ω	9.5	[deg]	
Backslope, β	0.0	[deg]	
Backslope rise	0.0	[ft]	Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82)

UNIFORM SURCHARGE

Uniformly distributed dead load is 0.0 [lb/ft ^{2}], and live load is 240.0 [lb/ft ^{2}]

ANALYZED REINFORCEMENT LAYOUT:



SCALE:

0 2 4 6 [ft]

ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, Fs = 14.61, Meyerhof stress = 717 lb/ft².

Foundation Interface: Direct sliding, Fs = 2.053, Eccentricity, e/L = 0.0958, Fs-overturning = 3.96

GEOTEXTILE				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [pullout resistance]	Fs-overall [connection break]	Fs-overall [geotextile strength]	Geotextile strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.50	4.00	1	N/A	N/A	N/A	8.140	5.070	1.774	0.0763	Mirafi HP570
2	2.50	4.00	1	N/A	N/A	N/A	7.966	1.785	2.548	0.0205	Mirafi HP570

ANALYSIS: CALCULATED FACTORS (Seismic conditions)

Bearing capacity, Fs = 13.10, Meyerhof stress = 766 lb/ft².

Foundation Interface: Direct sliding, Fs = 1.635, Eccentricity, e/L = 0.1380, Fs-overturning = 3.03

GEOTEXTILE				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [pullout resistance]	Fs-overall [connection break]	Fs-overall [geotextile strength]	Geotextile strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.50	4.00	1	N/A	N/A	N/A	7.068	3.252	1.432	0.1082	Mirafi HP570
2	2.50	4.00	1	N/A	N/A	N/A	7.112	1.194	2.234	0.0261	Mirafi HP570

AASHTO DESIGN METHOD
Heathers Ridge South

PROJECT IDENTIFICATION

Title:	Heathers Ridge South
Project Number:	T-7177
Client:	Quadrant Homes
Designer:	JCS
Station Number:	N/A

Description:

6-foot high geotextile wrap-face reinforced fill rockery.

Company's information:

Name: Terra Associates, Inc.
Street: 12525 Willows Rd.
Ste. 101
Kirkland, WA 98034
Telephone #: 425-821-7777
Fax #: 425-821-4334
E-Mail: jsadler@terra-associates.com

Original file path and name: G:\USERS\JSADLER\7000\7177 Ellsworth Property\Reinforce..... .....affic Surcharge).BEN

Original date and time of creating this file: February 15, 2015

PROGRAM MODE: ANALYSIS
of a SIMPLE STRUCTURE
using GEOTEXTILE as reinforcing material.

SOIL DATA

REINFORCED SOIL

Unit weight, γ	125.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	34.0 °

RETAINED SOIL

Unit weight, γ	125.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	32.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{equiv.}$	125.0 lb/ft ^{3}
Equivalent internal angle of friction, $\phi_{equiv.}$	32.0 °
Equivalent cohesion, $c_{equiv.}$	100.0 lb/ft ^{2}

Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Inclination of internal slip plane, ψ = 62.00° (see Fig. 28 in DEMO 82).
Ka (external stability) = 0.3073 (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 35.49 N γ= 30.21

SEISMICITY

Maximum ground acceleration coefficient, A = 0.290
Design acceleration coefficient in Internal Stability: Kh = Am = 0.336
Design acceleration coefficient in External Stability: Kh = 0.134 (Am = 0.134)
(Kh in External Stability is based on allowable displacement, d = 100 mm. using FHWA-NHI-00-043 equation)

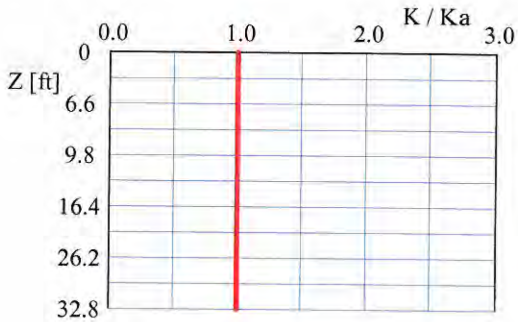
Kae (Kh > 0) = 0.3310 Kae (Kh = 0) = 0.2474 Δ Kae = 0.0836 (see eq. 37 in DEMO 82)
Seismic soil-geotextile friction coefficient, F* is 80.0% of its specified static value.

INPUT DATA: Geotextiles (Analysis)

D A T A	Geotextile type #1	Geotextile type #2	Geotextile type #3	Geotextile type #4	Geotextile type #5
Tult [lb/ft]	4800.0				
Durability reduction factor, RFd	1.10				
Installation-damage reduction factor, RFid	1.20				
Creep reduction factor, RFc	1.63	N/A	N/A	N/A	N/A
Fs-overall for strength	N/A				
Coverage ratio, Rc	1.000				
Friction angle along geotextile-soil interface, ρ	28.35				
Pullout resistance factor, F*	$0.80 \cdot \tan\phi$	N/A	N/A	N/A	N/A
Scale-effect correction factor, α	0.8				

Variation of Lateral Earth Pressure Coefficient With Depth

Z	K / Ka
0 ft	1.00
3.3 ft	1.00
6.6 ft	1.00
9.8 ft	1.00
13.1 ft	1.00
16.4 ft	1.00
19.7 ft	1.00

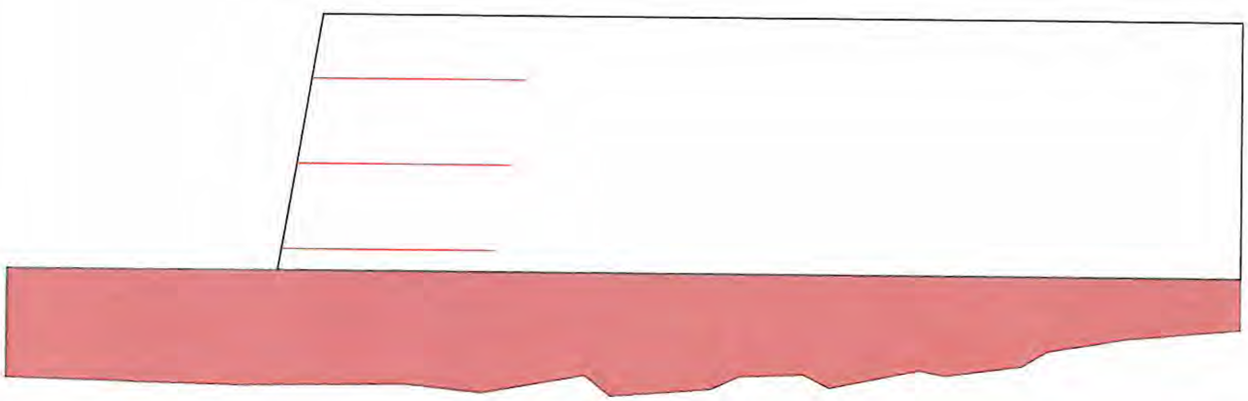


INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

Design height, Hd	6.00	[ft]	{ Embedded depth is E = 0.00 ft, and height above top of finished bottom grade is H = 6.00 ft }
Batter, ω	9.5	[deg]	
Backslope, β	0.0	[deg]	
Backslope rise	0.0	[ft]	Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82)

UNIFORM SURCHARGE
Uniformly distributed dead load is 0.0 [lb/ft ^{2}]

ANALYZED REINFORCEMENT LAYOUT:



SCALE:

0 2 4 6 [ft]

ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, Fs = 17.08, Meyerhof stress = 723 lb/ft².
Foundation Interface: Direct sliding, Fs = 3.291, Eccentricity, e/L = 0.0336, Fs-overturning = 6.69

GEOTEXTILE				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [pullout resistance]	Fs-overall [connection break]	Fs-overall [geotextile strength]	Geotextile strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.50	5.00	1	N/A	N/A	N/A	8.016	9.643	2.899	0.0238	Mirafi HP570
2	2.50	5.00	1	N/A	N/A	N/A	9.018	6.241	4.724	-0.0020	Mirafi HP570
3	4.50	5.00	1	N/A	N/A	N/A	20.201	4.925	11.414	-0.0077	Mirafi HP570

ANALYSIS: CALCULATED FACTORS (Seismic conditions)

Bearing capacity, Fs = 13.41, Meyerhof stress = 834 lb/ft².
Foundation Interface: Direct sliding, Fs = 2.194, Eccentricity, e/L = 0.0955, Fs-overturning = 3.81

GEOTEXTILE				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [pullout resistance]	Fs-overall [connection break]	Fs-overall [geotextile strength]	Geotextile strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.50	5.00	1	N/A	N/A	N/A	6.471	5.553	1.932	0.0753	Mirafi HP570
2	2.50	5.00	1	N/A	N/A	N/A	7.344	3.640	3.148	0.0181	Mirafi HP570
3	4.50	5.00	1	N/A	N/A	N/A	14.229	2.340	7.607	-0.0041	Mirafi HP570

AASHTO DESIGN METHOD
Heathers Ridge South

PROJECT IDENTIFICATION

Title:	Heathers Ridge South
Project Number:	T-7177
Client:	Quadrant Homes
Designer:	JCS
Station Number:	N/A

Description:

6-foot high geotextile wrap-face reinforced fill rockery (3:1 Backslope)

Company's information:

Name: Terra Associates, Inc.
Street: 12525 Willows Rd.
Ste. 101
Kirkland, WA 98034
Telephone #: 425-821-7777
Fax #: 425-821-4334
E-Mail: jsadler@terra-associates.com

Original file path and name: G:\USERS\JSADLER\7000\7177 Ellsworth Property\Reinforce..... ..... (3-1 Backslope).BEN

Original date and time of creating this file: February 15, 2015

PROGRAM MODE: ANALYSIS
of a SIMPLE STRUCTURE
using GEOTEXTILE as reinforcing material.

SOIL DATA

REINFORCED SOIL

Unit weight, γ	125.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	34.0 °

RETAINED SOIL

Unit weight, γ	125.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	32.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{equiv.}$	125.0 lb/ft ^{3}
Equivalent internal angle of friction, $\phi_{equiv.}$	32.0 °
Equivalent cohesion, $c_{equiv.}$	100.0 lb/ft ^{2}

Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Inclination of internal slip plane, ψ = 62.00° (see Fig. 28 in DEMO 82).
Ka (external stability) = 0.3602 (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 35.49 N γ= 30.21

SEISMICITY

Maximum ground acceleration coefficient, A = 0.290
Design acceleration coefficient in Internal Stability: Kh = Am = 0.336
Design acceleration coefficient in External Stability: Kh = 0.134 (Am = 0.134)
(Kh in External Stability is based on allowable displacement, d = 100 mm. using FHWA-NHI-00-043 equation)

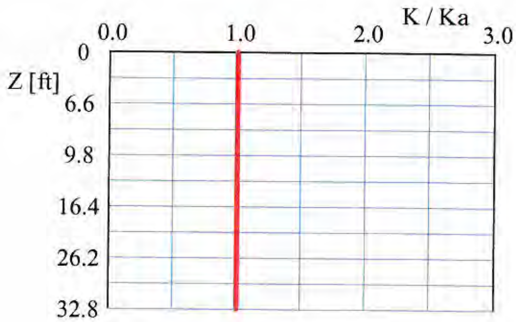
Kae (Kh > 0) = 0.4349 Kae (Kh = 0) = 0.2749 Δ Kae = 0.1600 (see eq. 37 in DEMO 82)
Seismic soil-geotextile friction coefficient, F* is 80.0% of its specified static value.

INPUT DATA: Geotextiles (Analysis)

D A T A	Geotextile type #1	Geotextile type #2	Geotextile type #3	Geotextile type #4	Geotextile type #5
Tult [lb/ft]	4800.0				
Durability reduction factor, RFd	1.10				
Installation-damage reduction factor, RFid	1.20				
Creep reduction factor, RFc	1.63	N/A	N/A	N/A	N/A
Fs-overall for strength	N/A				
Coverage ratio, Rc	1.000				
Friction angle along geotextile-soil interface, ρ	28.35				
Pullout resistance factor, F*	$0.80 \cdot \tan\phi$	N/A	N/A	N/A	N/A
Scale-effect correction factor, α	0.8				

Variation of Lateral Earth Pressure Coefficient With Depth

Z	K / Ka
0 ft	1.00
3.3 ft	1.00
6.6 ft	1.00
9.8 ft	1.00
13.1 ft	1.00
16.4 ft	1.00
19.7 ft	1.00

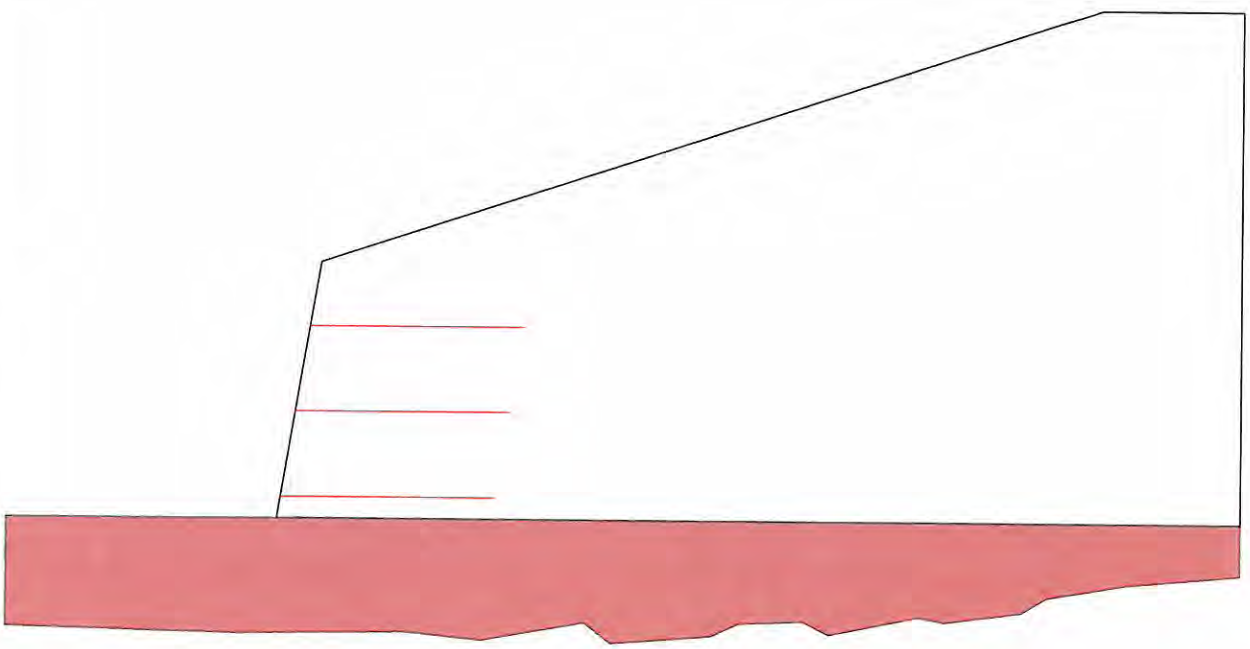


INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

Design height, Hd	6.00	[ft]	{ Embedded depth is E = 0.00 ft, and height above top of finished bottom grade is H = 6.00 ft }
Batter, ω	9.5	[deg]	
Backslope, β	18.3	[deg]	
Backslope rise	6.0	[ft]	Broken back equivalent angle, I = 18.26° (see Fig. 25 in DEMO 82)

UNIFORM SURCHARGE
Uniformly distributed dead load is 0.0 [lb/ft ²]

ANALYZED REINFORCEMENT LAYOUT:



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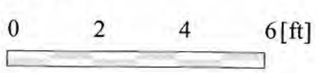

ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, Fs = 14.22, Meyerhof stress = 872 lb/ft².

Foundation Interface: Direct sliding, Fs = 2.404, Eccentricity, e/L = 0.0318, Fs-overturning = 4.42

GEOTEXTILE				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [pullout resistance]	Fs-overall [connection break]	Fs-overall [geotextile strength]	Geotextile strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.50	5.00	1	N/A	N/A	N/A	7.082	9.459	2.044	0.0198	Mirafi HP570
2	2.50	5.00	1	N/A	N/A	N/A	7.528	6.373	2.707	-0.0208	Mirafi HP570
3	4.50	5.00	1	N/A	N/A	N/A	12.997	5.308	3.879	-0.0614	Mirafi HP570

ANALYSIS: CALCULATED FACTORS (Seismic conditions)

Bearing capacity, Fs = 9.90, Meyerhof stress = 1087 lb/ft².

Foundation Interface: Direct sliding, Fs = 1.676, Eccentricity, e/L = 0.1156, Fs-overturning = 2.73

GEOTEXTILE				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [pullout resistance]	Fs-overall [connection break]	Fs-overall [geotextile strength]	Geotextile strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.50	5.00	1	N/A	N/A	N/A	5.703	5.428	1.441	0.0900	Mirafi HP570
2	2.50	5.00	1	N/A	N/A	N/A	6.180	3.762	2.037	0.0074	Mirafi HP570
3	4.50	5.00	1	N/A	N/A	N/A	9.926	2.823	3.344	-0.0563	Mirafi HP570

AASHTO DESIGN METHOD
Heathers Ridge South

PROJECT IDENTIFICATION

Title:	Heathers Ridge South
Project Number:	T-7177
Client:	Quadrant Homes
Designer:	JCS
Station Number:	N/A

Description:

6-foot high geotextile wrap-face reinforced fill rockery (Traffic surcharge)

Company's information:

Name: Terra Associates, Inc.
Street: 12525 Willows Rd.
Ste. 101
Kirkland, WA 98034
Telephone #: 425-821-7777
Fax #: 425-821-4334
E-Mail: jsadler@terra-associates.com

Original file path and name: G:\USERS\JSADLER\7000\7177 Ellsworth Property\Reinforce.....
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Original date and time of creating this file: February 15, 2015

PROGRAM MODE: ANALYSIS
of a SIMPLE STRUCTURE
using GEOTEXTILE as reinforcing material.

SOIL DATA

REINFORCED SOIL

Unit weight, γ	125.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	34.0 °

RETAINED SOIL

Unit weight, γ	125.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	32.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{equiv.}$	125.0 lb/ft ^{3}
Equivalent internal angle of friction, $\phi_{equiv.}$	32.0 °
Equivalent cohesion, $c_{equiv.}$	100.0 lb/ft ^{2}

Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Inclination of internal slip plane, ψ = 62.00° (see Fig. 28 in DEMO 82).
Ka (external stability) = 0.3073 (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 35.49 N γ= 30.21

SEISMICITY

Maximum ground acceleration coefficient, A = 0.290
Design acceleration coefficient in Internal Stability: Kh = Am = 0.336
Design acceleration coefficient in External Stability: Kh = 0.134 (Am = 0.134)
(Kh in External Stability is based on allowable displacement, d = 100 mm. using FHWA-NHI-00-043 equation)

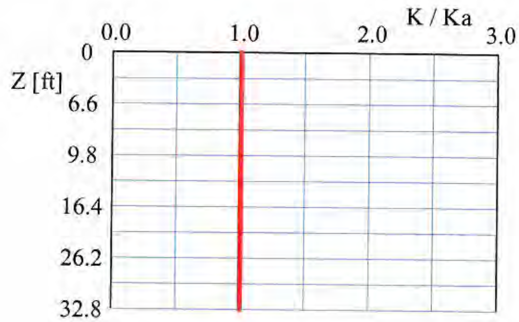
Kae (Kh > 0) = 0.3310 Kae (Kh = 0) = 0.2474 Δ Kae = 0.0836 (see eq. 37 in DEMO 82)
Seismic soil-geotextile friction coefficient, F* is 80.0% of its specified static value.

INPUT DATA: Geotextiles (Analysis)

D A T A	Geotextile type #1	Geotextile type #2	Geotextile type #3	Geotextile type #4	Geotextile type #5
Tult [lb/ft]	4800.0				
Durability reduction factor, RFd	1.10				
Installation-damage reduction factor, RFid	1.20				
Creep reduction factor, RFc	1.63	N/A	N/A	N/A	N/A
Fs-overall for strength	N/A				
Coverage ratio, Rc	1.000				
Friction angle along geotextile-soil interface, ρ	28.35				
Pullout resistance factor, F*	$0.80 \cdot \tan\phi$	N/A	N/A	N/A	N/A
Scale-effect correction factor, α	0.8				

Variation of Lateral Earth Pressure Coefficient With Depth

Z	K / Ka
0 ft	1.00
3.3 ft	1.00
6.6 ft	1.00
9.8 ft	1.00
13.1 ft	1.00
16.4 ft	1.00
19.7 ft	1.00



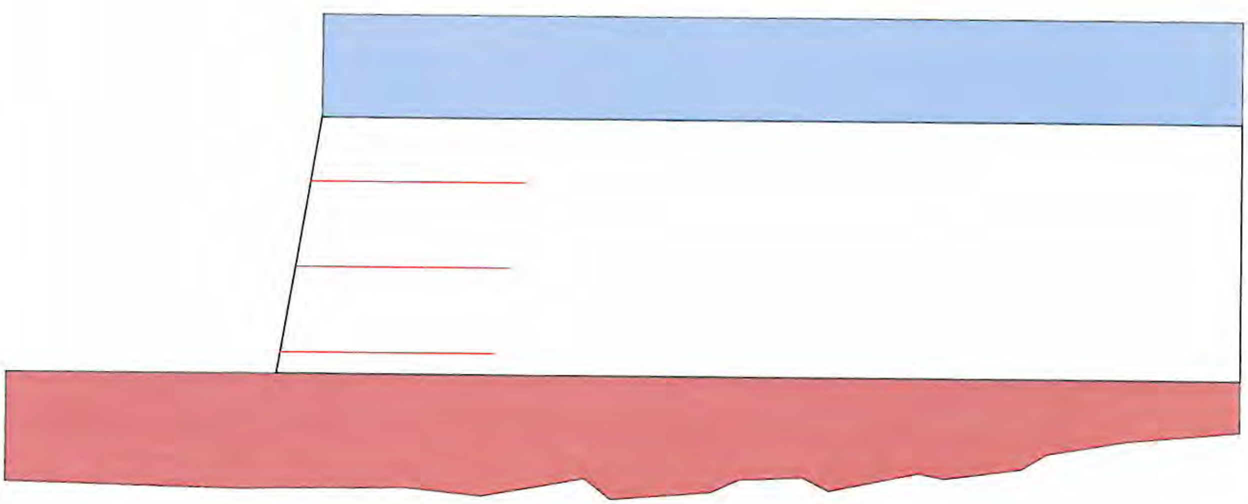
INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

Design height, Hd	6.00	[ft]	{ Embedded depth is E = 0.00 ft, and height above top of finished bottom grade is H = 6.00 ft }
Batter, ω	9.5	[deg]	
Backslope, β	0.0	[deg]	
Backslope rise	0.0	[ft]	Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82)

UNIFORM SURCHARGE

Uniformly distributed dead load is 0.0 [lb/ft ²], and live load is 240.0 [lb/ft ²]

ANALYZED REINFORCEMENT LAYOUT:



SCALE:

0 2 4 6[ft]

ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, Fs = 11.80, Meyerhof stress = 997 lb/ft².
Foundation Interface: Direct sliding, Fs = 2.007, Eccentricity, e/L = 0.1123, Fs-overturning = 3.41

GEOTEXTILE				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [pullout resistance]	Fs-overall [connection break]	Fs-overall [geotextile strength]	Geotextile strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.50	5.00	1	N/A	N/A	N/A	5.870	7.061	1.707	0.0953	Mirafi HP570
2	2.50	5.00	1	N/A	N/A	N/A	5.824	4.030	2.253	0.0418	Mirafi HP570
3	4.50	5.00	1	N/A	N/A	N/A	7.966	1.942	3.206	0.0104	Mirafi HP570

ANALYSIS: CALCULATED FACTORS (Seismic conditions)

Bearing capacity, Fs = 9.68, Meyerhof stress = 1121 lb/ft².
Foundation Interface: Direct sliding, Fs = 1.538, Eccentricity, e/L = 0.1742, Fs-overturning = 2.46

GEOTEXTILE				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [pullout resistance]	Fs-overall [connection break]	Fs-overall [geotextile strength]	Geotextile strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.50	5.00	1	N/A	N/A	N/A	4.996	4.396	1.319	0.1468	Mirafi HP570
2	2.50	5.00	1	N/A	N/A	N/A	5.076	2.600	1.819	0.0620	Mirafi HP570
3	4.50	5.00	1	N/A	N/A	N/A	6.835	1.224	2.811	0.0140	Mirafi HP570

AASHTO DESIGN METHOD
Heathers Ridge South

PROJECT IDENTIFICATION

Title:	Heathers Ridge South
Project Number:	T-7177
Client:	Quadrant Homes
Designer:	JCS
Station Number:	N/A

Description:

8-foot high geotextile wrap-face reinforced fill rockery.
3:1 backslope

Company's information:

Name: Terra Associates, Inc.
Street: 12525 Willows Rd.
Ste. 101
Kirkland, WA 98034
Telephone #: 425-821-7777
Fax #: 425-821-4334
E-Mail: jsadler@terra-associates.com

Original file path and name: G:\USERS\JSADLER\7000\7177 Ellsworth Property\Reinforce.....
..... (3-1 Backslope).BEN

Original date and time of creating this file: February 13, 2015

PROGRAM MODE: ANALYSIS
of a SIMPLE STRUCTURE
using GEOTEXTILE as reinforcing material.

SOIL DATA

REINFORCED SOIL

Unit weight, γ	125.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	34.0 °

RETAINED SOIL

Unit weight, γ	125.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	32.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{\text{equiv.}}$	125.0 lb/ft ^{3}
Equivalent internal angle of friction, $\phi_{\text{equiv.}}$	32.0 °
Equivalent cohesion, $c_{\text{equiv.}}$	100.0 lb/ft ^{2}

Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Inclination of internal slip plane, ψ = 62.00° (see Fig. 28 in DEMO 82).
Ka (external stability) = 0.3602 (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 35.49 N γ= 30.21

SEISMICITY

Maximum ground acceleration coefficient, A = 0.290
Design acceleration coefficient in Internal Stability: Kh = Am = 0.336
Design acceleration coefficient in External Stability: Kh = 0.134 (Am = 0.134)
(Kh in External Stability is based on allowable displacement, d = 100 mm. using FHWA-NHI-00-043 equation)

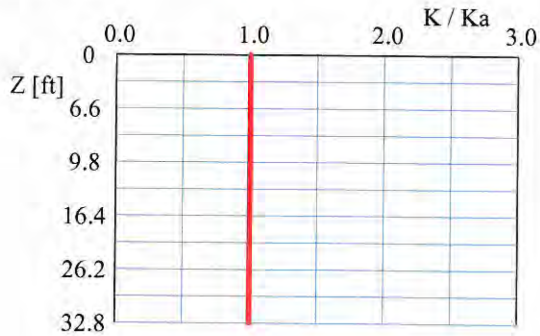
Kae (Kh > 0) = 0.4349 Kae (Kh = 0) = 0.2749 Δ Kae = 0.1600 (see eq. 37 in DEMO 82)
Seismic soil-geotextile friction coefficient, F* is 80.0% of its specified static value.

INPUT DATA: Geotextiles
(Analysis)

DATA		Geotextile type #1	Geotextile type #2	Geotextile type #3	Geotextile type #4	Geotextile type #5
Tult [lb/ft]		4800.0				
Durability reduction factor, RFd		1.10				
Installation-damage reduction factor, RFid		1.20				
Creep reduction factor, RFc		1.63	N/A	N/A	N/A	N/A
Fs-overall for strength		N/A				
Coverage ratio, Rc		1.000				
Friction angle along geotextile-soil interface,	ρ	28.35				
Pullout resistance factor, F*		0.80·tanϕ	N/A	N/A	N/A	N/A
Scale-effect correction factor,	α	0.8				

Variation of Lateral Earth Pressure Coefficient With Depth

Z	K / Ka
0 ft	1.00
3.3 ft	1.00
6.6 ft	1.00
9.8 ft	1.00
13.1 ft	1.00
16.4 ft	1.00
19.7 ft	1.00

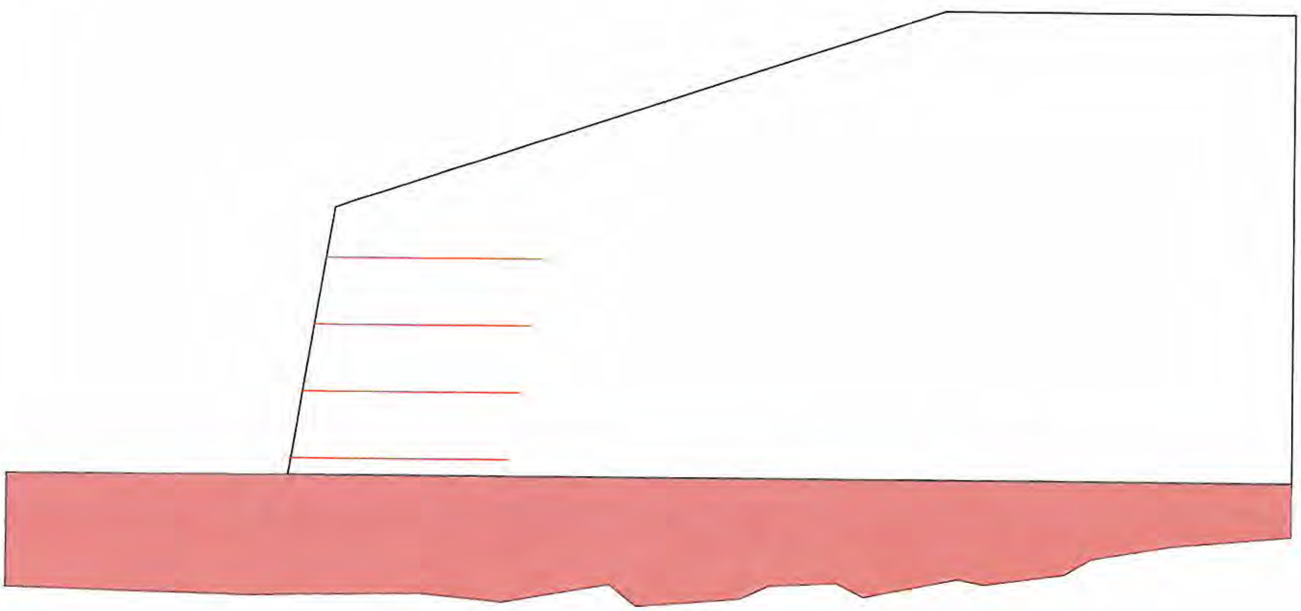


INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

Design height, Hd	8.00	[ft]	{ Embedded depth is E = 0.00 ft, and height above top of finished bottom grade is H = 8.00 ft }
Batter, ω	9.5	[deg]	
Backslope, β	18.3	[deg]	
Backslope rise	6.0	[ft]	Broken back equivalent angle, I = 18.26° (see Fig. 25 in DEMO 82)

UNIFORM SURCHARGE
Uniformly distributed dead load is 0.0 [lb/ft ^{2}]

ANALYZED REINFORCEMENT LAYOUT:



SCALE:

0 2 4 6 [ft]

ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, Fs = 12.81, Meyerhof stress = 1167 lb/ft².
Foundation Interface: Direct sliding, Fs = 2.361, Eccentricity, e/L = 0.0357, Fs-overturning = 4.27

GEOTEXTILE				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [pullout resistance]	Fs-overall [connection break]	Fs-overall [geotextile strength]	Geotextile strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.50	6.50	1	N/A	N/A	N/A	5.149	12.076	1.977	0.0261	Mirafi HP570
2	2.50	6.50	1	N/A	N/A	N/A	4.914	8.525	2.424	-0.0071	Mirafi HP570
3	4.50	6.50	1	N/A	N/A	N/A	7.135	7.799	3.097	-0.0355	Mirafi HP570
4	6.50	6.50	1	N/A	N/A	N/A	11.616	6.684	4.158	-0.0724	Mirafi HP570

ANALYSIS: CALCULATED FACTORS (Seismic conditions)

Bearing capacity, Fs = 8.62, Meyerhof stress = 1476 lb/ft².
Foundation Interface: Direct sliding, Fs = 1.641, Eccentricity, e/L = 0.1238, Fs-overturning = 2.62

GEOTEXTILE				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [pullout resistance]	Fs-overall [connection break]	Fs-overall [geotextile strength]	Geotextile strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.50	6.50	1	N/A	N/A	N/A	4.151	6.940	1.385	0.1035	Mirafi HP570
2	2.50	6.50	1	N/A	N/A	N/A	4.084	5.124	1.772	0.0343	Mirafi HP570
3	4.50	6.50	1	N/A	N/A	N/A	5.679	4.401	2.432	-0.0188	Mirafi HP570
4	6.50	6.50	1	N/A	N/A	N/A	8.574	3.388	3.716	-0.0693	Mirafi HP570

AASHTO DESIGN METHOD
Heathers Ridge South

PROJECT IDENTIFICATION

Title:	Heathers Ridge South
Project Number:	T-7177
Client:	Quadrant Homes
Designer:	JCS
Station Number:	N/A

Description:

8-foot high geotextile wrap-face reinforced fill rockery. Traffic Surcharge

Company's information:

Name: Terra Associates, Inc.
Street: 12525 Willows Rd.
Ste. 101
Kirkland, WA 98034
Telephone #: 425-821-7777
Fax #: 425-821-4334
E-Mail: jsadler@terra-associates.com

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Original date and time of creating this file: February 13, 2015

PROGRAM MODE: ANALYSIS of a SIMPLE STRUCTURE using GEOTEXTILE as reinforcing material.

SOIL DATA

REINFORCED SOIL

Unit weight, γ	125.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	34.0 °

RETAINED SOIL

Unit weight, γ	125.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	32.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{equiv.}$	125.0 lb/ft ^{3}
Equivalent internal angle of friction, $\phi_{equiv.}$	32.0 °
Equivalent cohesion, $c_{equiv.}$	100.0 lb/ft ^{2}

Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.2827 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Inclination of internal slip plane, ψ = 62.00° (see Fig. 28 in DEMO 82).
Ka (external stability) = 0.3073 (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 35.49 N γ= 30.21

SEISMICITY

Maximum ground acceleration coefficient, A = 0.290
Design acceleration coefficient in Internal Stability: Kh = Am = 0.336
Design acceleration coefficient in External Stability: Kh = 0.134 (Am = 0.134)
(Kh in External Stability is based on allowable displacement, d = 100 mm. using FHWA-NHI-00-043 equation)

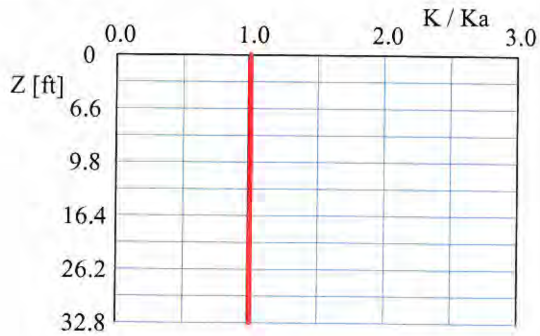
Kae (Kh > 0) = 0.3310 Kae (Kh = 0) = 0.2474 Δ Kae = 0.0836 (see eq. 37 in DEMO 82)
Seismic soil-geotextile friction coefficient, F* is 80.0% of its specified static value.

INPUT DATA: Geotextiles
(Analysis)

D A T A		Geotextile type #1	Geotextile type #2	Geotextile type #3	Geotextile type #4	Geotextile type #5
Tult [lb/ft]		4800.0				
Durability reduction factor, RFd		1.10				
Installation-damage reduction factor, RFid		1.20				
Creep reduction factor, RFc		1.63	N/A	N/A	N/A	N/A
Fs-overall for strength		N/A				
Coverage ratio, Rc		1.000				
Friction angle along geotextile-soil interface,	ρ	28.35				
Pullout resistance factor, F*		0.80·tanϕ	N/A	N/A	N/A	N/A
Scale-effect correction factor,	α	0.8				

Variation of Lateral Earth Pressure Coefficient With Depth

Z	K / Ka
0 ft	1.00
3.3 ft	1.00
6.6 ft	1.00
9.8 ft	1.00
13.1 ft	1.00
16.4 ft	1.00
19.7 ft	1.00



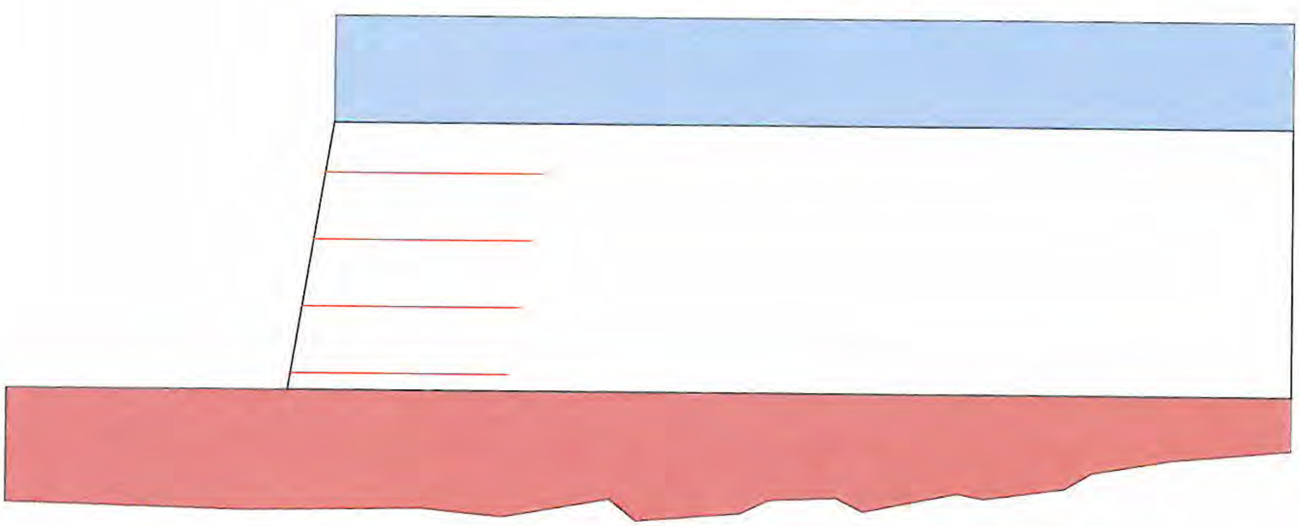
INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

Design height, Hd	8.00	[ft]	{ Embedded depth is E = 0.00 ft, and height above top of finished bottom grade is H = 8.00 ft }
Batter, ω	9.5	[deg]	
Backslope, β	0.0	[deg]	
Backslope rise	0.0	[ft]	Broken back equivalent angle, I = 0.00° (see Fig. 25 in DEMO 82)

UNIFORM SURCHARGE

Uniformly distributed dead load is 0.0 [lb/ft ^{2}], and live load is 240.0 [lb/ft ^{2}]

ANALYZED REINFORCEMENT LAYOUT:



SCALE:

0 2 4 6 [ft]

ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, Fs = 11.44, Meyerhof stress = 1247 lb/ft².

Foundation Interface: Direct sliding, Fs = 2.162, Eccentricity, e/L = 0.0992, Fs-overturning = 3.69

	GEOTEXTILE			CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [pullout resistance]	Fs-overall [connection break]	Fs-overall [geotextile strength]	Geotextile strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.50	6.50	1	N/A	N/A	N/A	4.589	9.757	1.819	0.0869	Mirafi HP570
2	2.50	6.50	1	N/A	N/A	N/A	4.254	6.324	2.271	0.0463	Mirafi HP570
3	4.50	6.50	1	N/A	N/A	N/A	5.824	4.792	2.970	0.0190	Mirafi HP570
4	6.50	6.50	1	N/A	N/A	N/A	7.966	2.387	4.193	0.0039	Mirafi HP570

ANALYSIS: CALCULATED FACTORS (Seismic conditions)

Bearing capacity, Fs = 9.10, Meyerhof stress = 1422 lb/ft².

Foundation Interface: Direct sliding, Fs = 1.616, Eccentricity, e/L = 0.1645, Fs-overturning = 2.57

	GEOTEXTILE			CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [pullout resistance]	Fs-overall [connection break]	Fs-overall [geotextile strength]	Geotextile strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.50	6.50	1	N/A	N/A	N/A	3.866	5.981	1.367	0.1438	Mirafi HP570
2	2.50	6.50	1	N/A	N/A	N/A	3.688	4.047	1.754	0.0761	Mirafi HP570
3	4.50	6.50	1	N/A	N/A	N/A	4.924	2.954	2.398	0.0307	Mirafi HP570
4	6.50	6.50	1	N/A	N/A	N/A	6.571	1.419	3.676	0.0060	Mirafi HP570