GEOTECHNICAL ENGINEERING STUDY
DETENTION POND WALL RECOMMENDATIONS

For

AL KUEHN PROPOERTY
28238, SR 410 HIGHWAY
BUCKLEY, PIERCE COUNTY, WASHINGTON

Prepared For

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feet tall, with a minimum embedment depth of 1.5 feet below the pond bottom. The site will be supported by two perimeter retaining walls consisted of ecology blocks. The perimeter ecology block retaining walls will be approximately 6 feet tall with exposed height of 4 feet and embedment depth of 2 feet below the toe grade. The location of the subject site, and the locations of the proposed detention pond and the walls are shown in the referenced plans, which are provided in Detention Pond Plans, Appendix A of this report.

The subsurface data used in generating the concrete wall design parameters, designing the ecology block walls, and developing the engineering recommendations for the construction of the walls are obtained from the information available from the subsurface explorations performed by PGE in the subject site. The findings of the explorations are presented in the following section of this report. The topographical information including the final grades, the proposed detention pond plan, and the wall geometries and the wall locations are obtained from the plans referenced above, which are provided in Detention Pond Plans, Appendix A of this report.

The following sections of this report address the details of the concrete wall design parameters, the actual design calculations and the details of the of the above grade perimeter ecology block walls, the details of the ecology block baffle wall, and the recommendations for the geotechnical aspects of the construction and inspection of these walls.

1.0 Soil and Groundwater Conditions

A brief description of the soil and groundwater conditions in the proposed detention pond area obtained from our field explorations are provided below.

Exploration Depth: One test pit was dug within the footprint area of the proposed detention pond. Our exploration depth extended upto 15 feet depth below the current grade.

Existing Fills: The soils in the proposed detention pond area are expected to be consisted of fills within the upper 4 feet depth, followed by native soils upto 15 feet depth below the current grade. The fills are consisted of brown, silty sandy gravel with some cobbles and boulders. According Mr. Al Kuehn, the upper 2 feet of fills have been in place for last 1.5 years and the lower 2 feet of fills have been in place for last 4.5 years. In general, the fills are found in moist conditions and in medium dense consistency.

Topsoils: The fills are underlain by original topsoil layer of approximately 12 inches thick, consisted of black-brown silty sandy soils containing some organics.

feet tall, with a minimum embedment depth of 1.5 feet below the pond bottom. The site will be supported by two perimeter retaining walls consisted of ecology blocks. The perimeter ecology block retaining walls will be approximately 6 feet tall with exposed height of 4 feet and embedment depth of 2 feet below the toe grade. The location of the subject site, and the locations of the proposed detention pond and the walls are shown in the referenced plans, which are provided in Detention Pond Plans, Appendix A of this report.

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Topsoils: The fills are underlain by original topsoil layer of approximately 12 inches thick, consisted of black-brown silty sandy soils containing some organics.

Native Soils: Below the original topsoil layer, the native soils are present, which consists of light brown to gray very gravelly soils consisted of some silty sand, cobbles, and boulders. Pockets of partly cemented silty sand lenses exist within the native soils. In general, the native soils are found in moist conditions upto 11 feet depth below the current grade, and in wet conditions below this depth. Signs of mottling were noticed below 5 feet depth of the test pit, which extended upto the bottom of the test pit depth. The mottling signs indicate the evidence of fluctuation of possible groundwater across these depths. The native soils are in medium to dense consistency.

Moderate caving was noticed within the native soils below 6 to 7 feet depths of the current grade. The digging through the native soils with the excavator was noticed somewhat difficult below 6 feet depth of the current grade. The test pit was left open for almost half an hour to notice if any seepage or ponding occurs, however, none of such was observed.

Seepage and Groundwater: As it is mentioned above, the native soils were found in very moist to wet condition below 11 feet depth of the current grade, and the signs mottling were noticed below 5 feet depth of the current grade to the bottom of the test pit. These observations made us believe that possibly the site is underlain by groundwater, and that the groundwater fluctuates with the season. Given the time of the year the field exploration was performed, which was almost at the end of the dry summer period (September), it is believed that the groundwater was below the test pit depth during our exploration therefore was not visible during our field exploration. However, the signs of the mottling shows the evidence that the groundwater rises in this site during the wet winter months. This is typical in the Puget Sound area for the groundwater level to fluctuate seasonally. The possibility of the presence of seepage and groundwater in this site within the excavation depths of the proposed detention pond is considered when developing the design parameters for the design of the concrete retaining wall for the pond. It is advised that the civil engineer of the project, the structural engineer for the concrete wall design, and the contractor must also consider the groundwater scenario when designing and building the proposed detention pond in this site.

The preceding discussion on the subsurface conditions of the site is intended as a general review to highlight the major subsurface stratification features and material characteristics. For more complete and specific information at individual test pit locations, please review the Test Pit Log (Page B-1) included in Appendix B. This log includes soil descriptions, stratification, and location of the samples and laboratory test data. It should be noted that the stratification lines shown on the individual logs represent the approximate boundaries between various soil strata; actual transitions may be more gradual or more severe. The subsurface conditions depicted in the log are for the test pit location indicated only, and it should not necessarily be expected that these conditions are representative at other locations of the site.

2.0 Detention Pond Concrete Retaining Wall

Based on the detention pond plan provided in the referenced plans, the proposed below grade detention pond will have a perimeter retaining wall consisted of a continuous concrete wall. The purposed of the wall will be to provide support to the cut to be made to build the pond. The concrete wall will have varying heights, ranging from 8 feet to 11.5 feet. The east and the south concrete walls will be approximately 11.5 feet tall and the west wall will be approximately 8 feet tall. These heights include a minimum embedment depth of 2 feet below the pond bottom elevation. The wall details are shown in Wall Details, Appendix C of this report.

Wall Design Parameters

The design parameters for the concrete retaining wall are developed based on the following assumptions.

(i) The pond's sidewall is to be designed as retaining wall to resist the lateral pressures induced by the retained soils, groundwater, traffic surcharge on the backhill grade of the wall, and the seismic surcharge on the wall. The appropriate combination of these pressures should be decided by the structural engineer depending on his understanding of the various loadings on the wall.

(ii) Because of the free-standing cantilever nature of the wall, the wall should be designed for active earth pressure condition.

(iii) Since it is expected that there may be groundwater within the excavation depths of the proposed detention pond, the below grade concrete perimeter wall may have water on its both sides, i.e., inside the pond and outside the wall. This scenario is expected to be taken in place during the wet winter months when the seasonal groundwater table may rise to its highest elevation and the detention pond will have its highest water table. The possibility of water on both sides of the wall eliminates the requirement of installing a drainage layer behind the wall. Also, due to the possibility of the presence of water on both sides of the wall, we believe that the wall may experience a buoyant effect on it. We recommend that the structural engineer should consider this scenario during his wall design.

(iv) Our design parameters has considered the hydrostatic pressure that is likely to be developed behind the wall.

(v) Our design parameters has considered that the void areas between the wall and the excavation face, and the excavation area at the toe of the wall must be backfilled with new structural fills compacted adequately to 95% of the fills' maximum dry density value.

(vi) The wall is considered with the level ground backhill slope above the wall.

The lateral earth pressures and the other design parameters developed based on the above assumptions are shown on Earth Pressure Diagrams, Figure 1, provided in Wall Details, Appendix C of this report.

Co-efficient of Earth Pressures: Our design parameters assume a saturated soil unit weight equal to approximately 125 pounds per cubic foot (pcf) and an angle of internal friction (effective) of approximately 26 degree for the backfill soils to be retained by the sidewalls. Based on the above values, the active earth pressure co-efficient (Ka) and the passive earth pressure co-efficient (Kp) are determined as 0.39 and 2.56.

Active Earth Pressure: Based on the above soil parameters, the active condition soil pressures equivalent to a fluid unit weight of 24H psf for level backhill slopes, where 'H' is the wall height, is to be used for designing the sidewalls. The above soil pressure is based on the effective soil unit weight of 62.5 pcf. The active lateral earth pressure of above amount will be acting as a triangular pressure distribution on the walls.

Hydrostatic Pressure: In addition to the active lateral earth pressure, hydrostatic pressure equal to 62.5H psf must be considered to be exerting on the sidewalls when the tank will be in empty condition. The hydrostatic pressure will have a triangular pressure distribution on the walls.

Seismic Pressure: The walls must be designed for the seismic loading conditions. Based on a design acceleration coefficient of 0.25 to 0.30 we recommend that a seismic loading equal to 4H psf for active condition and level backhill slope should be considered to be acting uniformly over the full height of the walls. The seismic pressure will have a rectangular pressure distribution on the walls.

Traffic Surcharge: Traffic surcharge loads are expected to be exerted on the retaining walls. The lateral impact of such traffic surcharge loads (q = 250 psf) on the walls should be considered and added to the other lateral pressures. To calculate the amount of the lateral pressure to be transferred from the traffic surcharge loads to the walls, a ratio of lateral stress to vertical stress of 0.39 is to be used on the traffic surcharge loads (q). Such lateral surcharge load (0.39q) will be acting uniformly over the entire height of the walls.

Passive Resistance: Where footings are incorporated as part of the walls, the footings can be resisted against sliding by a friction between the foundation base and the bearing soils, and by passive earth pressure acting on the face of the embedded portion of the footings. We recommend that a coefficient of friction between concrete and the bearing soils of 0.35 be used to calculate the resistance to sliding at the base of footings. The passive resistance developed by footings should be computed using an equivalent fluid pressure of 160 pounds per cubic foot (pcf) for static loading condition and a decreased passive pressure of 130 pcf for the seismic loading condition. The above soil parameters require to include a factor of safety value of 1.5. The above values are calculated based on the effective soil unit weight of 62.5 pcf and the effective angle of internal friction of 26 degree for the soil in the passive zone. The passive pressures should be applied for the design of the retaining walls against sliding and overturning failures. The passive resistance value assumes that the foundation must be poured "neat" against the firm and competent, undisturbed native soils or structural fill that placed and compacted as recommended later on in 'Fill Placement and Compaction' sub-section of Section 5.0 of this report, and that the footings are embedded for adequate depths below the lowest adjacent grades. We recommend to disregard the upper 12 inches of soil while computing the passive resistance value because this depth can be affected by weather or disturbed by future grading activity.

Wall Footing

The pond wall can be supported on conventional shallow strip footing, which must be placed on firm and unyielding native subgrades approved by PGE geotechnical engineer during the actual construction of the wall. The wall footing must be placed on a properly prepared subgrade as it is recommended later on in this report in 'Wall Footing Subgrade Preparation' sub-section of Section 5.0. Based on the soil conditions encountered in the exploratory test pit area of the pond and based on the footing elevations of the pond walls the footing subgrades are expected to be consisted of dense native silty sandy gravelly soils, which are considered to be competent to provide support to the wall. It should be noted that the actual subgrade conditions at the bottom of wall footing will be revealed during tho actual construction of the pond. Therefore, the competency of the subgrades to support the wall footing to be determined only after the pond excavations are completed upto the final bottom elevations of the wall footing. We recommend that a professional geotechnical engineer from PGE must be invited to examine the competency of the final footing subgrades to support the wall footing.

Allowable Bearing Capacity Value

If the final footing subgrades are found competent and the final footing subgardes are prepared as it is recommended later on in this report in 'Wall Footing Subgrade Preparation' sub-section of Section 5.0 then the final native subgrade would be able to provide a maximum net allowable bearing capacity of 2500 psf. This value includes a factor of safety of 2.5. For short-term loads, such as wind and seismic, a 1/3 increase in this allowable capacity can be used. The final footing widths should be determined based on the above recommended bearing capacity value, loading from the walls, and the lateral loads to be exerted on the walls from the retained soils. We recommend that all footings should be embedded a minimum of 24 inches below the bottom elevation of the pond into the native soils or adequately compacted structural fills to provide adequate confinement of the footings and to provide sufficient resistances to the lateral loadings on the footings. However, the actual embedment depths will be dependent on the heights of the wall, lateral loads on the wall, and the lateral resistances to be required for the stability of the wall.

Settlement

Based on our settlement potential evaluation, we anticipate that the properly constructed footing subgrades as recommended later on in this report in 'Wall Footing Subgrade Preparation' sub-section of Section 5.0 should experience total and differential settlements of less than 1 inch and 1/2 inch, respectively. Most of these settlements are expected to occur immediately following the vault is built. The predicted settlement values may be expected larger if soft, loose, organic soil is encountered, or if the foundation subgrade is disturbed and becomes soft during construction. The settlement evaluation was done without the aid of any laboratory consolidation test data, and on the basis of our experience with similar types of structures and subsoil conditions.

We recommend that the subsurface conditions considered above must be verified on-site during the actual construction of the wall. It should be noted that if the subsurface conditions considered above in the wall design are found significantly different during the actual construction then the wall design may need to be revised to reflect those variations in the soil conditions, or even the type of wall that is recommended in this report needs to be revised. The client needs to consider this situation and keep a contingency plan to accommodate the cost of the revisions of the wall design and the recommendations for the other type of wall.

The details of the concrete retaining walls of varying heights are shown on Concrete Retaining Wall Details, Figure 2 and 3, provided in Wall Details, Appendix C of this report.

3.0 Free-standing Ecology Block Baffle Wall in Pond

The detention pond will have an ecology block baffle wall, which will be approximately 6.5 feet tall, with 5 feet height above the pond bottom elevation and 1.5 feet below the pond bottom. This wall is designed as a free-standing wall, and with an allowable bearing capacity value 2500 psf to support the wall. The wall footing must be placed on a properly prepared subgrade as recommended later on in this report in 'Wall Footing Subgrade Preparation' sub-section of Section 5.0. In addition to this the wall must built with perfect vertical level to avoid the possibility of its tilting on the sides. Also, the footing subgrade must be leveled accurately to avoid any tilting of the wall on its sides. A reinforcement bar #5 must be installed at the center of each core of the blocks filled up with 2500 psi strength concrete at 28 days curing to provide the wall with additional vertical stability. A detail of the free-standing baffle wall with its various components is shown on Ecology Block Baffle Wall Details, Figure 4, provided in Wall Details, Appendix C of this report.

4.0 Perimeter Ecology Block Wall Design

Our wall design and the internal stability calculation of the block wall are performed based on the design considerations and the construction recommendations provided in the following sub-sections of this section, and in the other pertinent recommendations provided in the subsequent sections of this report. The details of the wall are shown in the Ecology Block Retaining Wall Details, Figure 5, provided in Wall Details, Appendix C of this report, and the wall design calculations are shown in Ecology Block Retaining Wall Design Calculation, provided in Appendix D of this report. The block wall must be installed as per the guidelines provided in the following section and its pertinent sections of this report and as per the details provided in the Ecology Block Retaining Wall Details, Figure 5.

Design Considerations and Construction Recommendations for Ecology Block Wall

Soil Parameters and Subsurface Conditions: The wall is designed based on the following soil parameters and the subsurface conditions:

Retained native soil in the wall area:	Dense gravelly sand or sandy gravel; USCS soil type: GP/ SP
Retained backfill behind the wall:	Imported structural fills.
Retained native soil unit weight:	125 pcf.

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Retained backfill soil unit weight:	125 pcf (after compacted to 95% of fills' dry density value based on Mod. Proctor Test (ATM 1557).
Retained native & backfill soil friction angle:	30 deg.
Retained native & backfill soil cohesion:	none.
Native, firm subgrade soil bearing capacity:	2500 psf (allowable).
Seepage from the excavation slope face:	If any, should be minor, not any significant amount.
Final excavation slope face:	Possibly consisted of dense soil, with firm and stable conditions, and w/ some sloughing.
Retained structural fills compactions:	Compacted to minimum 95% of the fills' maximum dry density value based on Mod. Proctor Test (ASTM 1557).
Native soils at the wall footing:	Firm and dense upto a min. depth equal to the width of the wall footing.
Groundwater table:	If there is any, must exist at least 2 feet below the bottom elevation of the wall footing.

We recommend that the subsurface conditions considered above must be verified on-site during the actual construction of the wall. It should be noted that if the subsurface conditions considered above in the wall design are found significantly different during the actual construction then the wall design may need to be revised to reflect those variations in the soil conditions, or even the type of wall that is recommended in this report needs to be revised. The client needs to consider this situation and keep a contingency plan to accommodate the cost of the revisions of the wall design and the recommendations for the other type of wall.

Earth Pressure: The earth pressure behind the wall is considered as active earth pressure.

Back Hill Slope: The wall is designed considering the existing grade of the backhill slope above the wall is in almost 2H:1V grade, which is considered to be remained unaltered throughout the life of the wall.

Surcharge Loading: The wall is designed with no temporary or permanent or traffic surcharge loadings on the backhill slope above the wall within the horizontal distance equal to the height of the wall.

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Seismic Loading: Due to the wall height not exceeding 6 feet, and the wall is of flexible nature, no seismic loading considered to be exerted on the wall.

Bearing Capacity: The wall is designed based on the assumption that the final native subgrades at the wall footing would be able to provide a maximum net allowable bearing capacity of approximately 2500 pounds per square foot (psf) to support the wall. Therefore, the wall must be installed on the native subgrades that are firm and unyielding and strong enough to support the wall and to be able to provide the above bearing capacity value.

Variations in the quality and strength of the potential bearing soils can occur with depth and distance within the proposed footing areas of the wall. Therefore, careful evaluation of these bearing materials is recommended at the time of construction to verify their suitability to support the above recommended bearing pressure. We recommend that a geotechnical engineer examine the bearing materials prior to placing base blocks.

Settlement: Considering that the above design bearing capacity value could be achieved and that the wall will be placed on properly prepared subgrades as recommended later on in this report in 'Wall Footing Subgrade Preparation' sub-section of Section 5.0 and placed over the recommended bearing materials should experience total and differential settlements of less than 1 inch and 1/2 inch, respectively. The majority of these settlements are expected to occur during construction.

Keyway or Embedment Depth: A keyway for the foundation bench or the embedment depth for the wall below the firm native soils consisting of a shallow trench of minimum 2 feet depth shall be constructed along the full length of the wall.

The grade of the keyway base for the block wall should be built slightly inclined (equal to the wall batter 1H:6V) towards the cut face to be protected by the wall. This is necessary for the block wall to be built with the recommended batter. The keyway shall be excavated for the full width equal to the wall width as well as the width of the drainage layer (12-inch thick). The keyway will provide lateral passive resistance for the walls against their sliding. The lateral passive resistances could be achieved from the passive earth pressures acting on the faces of the embedded portions of the wall footings.

It should be noted that the above embedment depth provided in the wall details in 'Ecology Block Wall Details' figure should be considered to be a minimum. The actual embedment depths along the length of the wall may vary depending on the factors such as the wall height and the depths of the firm and unyielding native subgrade conditions below the final grades. In our opinion, the appropriate wall embedment depth should be determined on-site during the actual wall construction by a geotechnical engineer.

Base Layer: After the final footing subgrades and the embedment depths are approved by the geotechnical engineer a minimum of 6 inches thick compacted crushed rock layer must be placed on the final footing subgrades prior to placing the base blocks. The gravel layer will provide additional frictional resistance for the wall against its sliding.

Filter Fabric: A nonwoven geosynthetic filter fabric (Mirafi 140N or equivalent) shall be placed between the drainage layer and the retained fill embankments to prevent migration of fines from the retained soils to the drainage layer. The fabric should be draped down across the width of the drainage layer. The toe edge of the fabric should be anchored in-place by wrapping it over the 12-inch thick washed drain rock layer surrounding the footing drainpipe. Once the wall is completed, and the drainage layer is within the 12 inch of the top of the wall, the filter fabric shall be folded over the drainage layer to seal the layer from receiving surface water runoff from backhill slopes and any fines carried by this water. The space above the fabric should be filled with topsoil of at least 12 inches thick. A swale is to be made within the top 6 inches of the topsoil layer to collect the surface runoff from the backhill slope above the top of the wall. The swale should be sloped along its longitudinal direction in such a way that the collected water would be directed towards the ends of the wall and finally to be discharged to approved points.

Footing Drainpipe: A minimum 4 to 6-inch perforated or slotted, flexible, Schedule 40 PVC drainpipe should be installed at the heel of the wall as shown in the 'Ecology Block Wall Details' figure. This will help collecting any water that may seep from the retained soils. The pipe shall be bedded on and surrounded by "Gravel Backfill for Drains" (WSDOT/APWA 9-03.12(4)) to a minimum thickness of 6 inches above and below the drainpipe. The drainpipe and the drain rock should be encapsulated in a geotextile filter fabric such as Mirafi 140N, or equivalent. The perforated pipe should be laid with a longitudinal slope towards its ends and finally be directed to approved discharge points. The drainpipe must not discharge at the toe of the walls.

Drainage Layer: To control seepage from behind the wall and to prevent any development of hydrostatic pressures behind the wall a backfill drainage layer of minimum 12 inches thick should be installed. The drainage layer should consist of 2 to 4 inch crushed rocks containing less than 2 percent material passing the No. 200 Sieve. This material shall be carefully placed so as not to damage the filter fabric.

Wall Batter: The block wall must be built with a proper batter as the wall erection progresses vertically upward. As mentioned earlier, this could be achieved by slightly inclining the grade of the keyway base equal to the wall batter (1H:6V) towards the back of the wall. The wall batter will prevent the walls to slide or tilt towards the front of the wall.

Toe Slope: The finished grades at the toe of the wall should be compacted adequately to firm and unyielding conditions. The final grades at the toe of the walls should be finished with 2 percent gradient away from the wall toe in order to divert surface runoff away from the toe of the wall.

Impermeable Cap and Swale: The space above the filter fabric should be filled with an impermeable soils such as a topsoil layer. The thickness of this layer should be a minimum of 12 inches. The purpose of this layer is to act as a cap to seal any transportation of the surface runoff from the slope to the swale and to the drainage layer behind the wall. A swale is to be made within the topsoil layer to collect the surface runoff from the backhill slope above the top of the wall. The swale should be sloped along its longitudinal direction in such a way that the collected water would be directed towards the ends of the wall and finally to be discharged to approved points. Water must not be discharged at the toe of the wall.

5.0 Other Construction Recommendations

Clearing and Grubbing

Initial site preparation for construction of walls and pond, and any other structures should include stripping of vegetation and topsoil from the site, if there is any. Stripped vegetation debris should be removed from the site. Stripped organic topsoils will not be suitable for use as structural fill but may be used for future landscaping purposes.

Wall Footing Subgrade Preparation

After the removal of the loose soils and any soft and wet soils near the surface, and after the excavations are completed upto the final footing subgrades that will provide direct support for the walls the exposed subgrades must be adequately proofrolled to evaluate their conditions. The footing subgrade proofrolling should be achieved using hand operated/walk-behind compaction equipment such as a small roller. We do not recommend jumping jack or vibratory plate equipment due to insufficient energy. Proofrolling should be done under the supervision of a geotechnical engineer from PGE, and/or must be probed with a T-probe by the geotechnical engineer to identify the presence of any isolated soft and yielding areas and to verify that stable subgrades are achieved to support the walls. If any subgrade area ruts and pumps excessively and cannot be stabilized in place by compaction, the affected soils should be over-excavated completely to firm and unyielding suitable bearing materials, and replaced with new structural fills (recommended fill is ¾ -inch crushed rock minus) to desired final footing subgrade levels. If the depth of overexcavation to remove unstable soils becomes excessive, a geotextile fabric, such as Mirafi 500X or equivalent in conjunction with the above structural fills may be considered. Such decision should be made on-site by a geotechnical engineer from PGE during the actual construction of the walls.

Reuse of On-Site Soils

The ability to use on-site soils obtained from the site excavations as structural fills depends on the gradation, moisture content of the soils, and the prevailing weather conditions exist during the grading activities. If the native soils are consisted of primarily fine-grained soils, and contains excessive fines (that portion passing the U.S. No. 200 sieve), then it becomes increasingly sensitive to small changes in moisture content, and hence provide challenge to its adequate compaction, and even sometimes make it almost impossible. With our experience, soils containing more than about 5 percent fines by weight cannot be consistently compacted to the recommend degree when the moisture content is more than about 2 percent above or below the optimum. Also, on the other hand, if the native soils are consisted of coarse-grained soils, which contains significant amounts of gravels, cobbles, and boulders with little or no fines to bind them, then such soils pose major problems to achieve their compactions, and hence should be avoided to use as structural fills. The presence of excessive amounts of gravels, cobbles, and boulders pose problems for the compaction of the finer particles present between the larger size particles.

The native soils in the subject site are primarily consisted of gravels, cobbles, and boulders, with less fines, which we consider as unsuitable for use as structural fills in its present state. If the native soils are to be used as the structural fills then the following recommendations must be followed. Care should be taken during the laying of the native soils to avoid clustering of excessive gravels, cobbles, and builders in any one location, which otherwise would pose problem during the compaction. Also, any particles larger than 3 to 4-inch diameters must be screened or handpicked from the native soils prior to their use as structural fills. An experienced geotechnical inspector should be monitoring the above processes on-site, and the fill placement and the compaction to avoid the above issues during the fill operations.

Structural Fills

If the native soils cannot be used structural fills then the structural fills must be imported. The structural fill should be non-organic soil, free of deleterious materials, and well-graded and free-draining granular material, with a maximum of 5 percent passing the No. 200 sieve by weight, and not exceeding 6 inches for any individual particle. A typical gradation for structural fill is presented in the following table.

Structural Fill	
U.S. Standard Sieve Size	Percent Passing by Dry Weight
3 inch	100
¾ inch	50 –100
No. 4	25 – 65
No. 10	10 – 50
No. 40	0 – 20
No. 200	5 Maximum*

* Based on the ¾ inch fraction.

In the event that whether the structural fill materials are to be imported to the site, or if on-site soils are to be reused as structural fill, we recommend that the potential structural fill materials be verified and approved by the project geotechnical engineer prior to their use.

Fill Placement and Compaction Requirements

Structural fills should be placed in uniform loose lifts not exceeding 6 inches in thickness for walk-behind low weight compaction equipment. Each lift should be compacted to a minimum of 95 percent of the soil's laboratory maximum dry density as determined by ASTM Test Designation D-1557 (Modified Proctor) method, or to the applicable minimum City or County standard, whichever is the more conservative. The fill should be moisture conditioned such that its final moisture content at the time of compaction should be at or near (typically within about 2 percent) of its optimum moisture content, as determined by the ASTM method. If the fill materials are on the wet side of optimum, they can be dried by periodic windrowing and aeration or by intermixing lime or cement powder to absorb excess moisture.

If field density tests indicate that the last lift of compacted fills has not been achieved the required percent of compaction or the surface is pumping and weaving under loading, then the fill should be scarified, moisture-conditioned to near optimum moisture content, re-compacted, and re-tested prior to placing additional lifts.

The loosely backfilled soils in the areas of exploratory test pit should be overexcavated completely upto the firm native soils and backfilled with adequately compacted new structural fills upto the final grades.

Construction Dewatering

If the regional groundwater levels rise above the planned excavation base during the winter and spring months, the contractor should be prepared to dewater the vault excavations.

If localized (perched) groundwater or minor seepage is encountered, we anticipate that internal collection ditches directing water inflow to sumpholes and then removal of water by conventional filtered sump pumps will be adequate to temporarily dewater the excavations and to maintain a relatively dry working area for construction purposes.

The dewatering must remain in operation to maintain a dry working condition throughout the construction period in the excavation areas. If severe water conditions encountered, more specialized dewatering techniques, such as vacuum wells, well points, etc., may be needed. However, these more extensive dewatering techniques can lead to settlement of the ground surface in the surrounding vicinity when the groundwater is drawn down. If such dewatering techniques are contemplated a geotechnical engineer should be consulted for specific design and construction recommendations for the excavation areas.

Temporary Excavation Slopes

As we understand from the project plan that the proposed detention pond construction would requires considerable amount of excavations. The owner and the contractor should be aware that in no case should the excavation slopes be greater than the limits specified in local, state, and federal safety regulations, particularly, the Occupational Safety and Health Administration (OSHA) regulations in the "Construction Standards for Excavations, 29 CFR, part 1926, Subpart P, dated October 31, 1989" of the Federal Register, Volume 54, the United States Department of Labor. As mentioned above, we also recommend that the owner and the contractor should follow the local and state regulations such as WSDOT section 2-09.3(3) B, Washington Industrial Safety and Health Act (WISHA), Chapter 49.17RCW, and Washington Administrative Code (WAC) Chapter 296-115, Part N. These documents are to better insure the safety of construction worker entering trenches or excavation. It is mandated by these regulations that excavations, whether they are for utility trenches or footings, be constructed in accordance with the guidelines provided in the above documents. We understand that these regulations are being strictly enforced and, if they are not closely followed, both the owner and the contractor could be liable for substantial penalties.

Stability of temporary excavations is a function of many factors including the presence of and abundance of groundwater and seepage, the type and density of the various soil strata, the depth of excavation, surcharge loadings adjacent to the excavation, and the length of time and weather conditions while the excavation remains open. It is exceedingly difficult under these unknown and variable circumstances to pre-establish a safe and maintenance-free temporary excavation slope angle at this time of the study. We therefore, strongly recommend that all temporary, as well as permanent, cuts and excavations in excess of 4 feet be examined by a geotechnical engineer during the actual construction to verify that the recommended slope inclinations in this section are appropriate for the actual soil and groundwater seepage conditions exposed in the cuts. If the conditions observed during the actual construction are different than anticipated during this study then, the proper inclination of the excavation and cut slopes or requirements of temporary shoring should be determined depending on the condition of the excavations and the slopes.

As a general rule, all temporary soil cuts greater than 4 feet in height associated with site regarding or excavations should be adequately sloped back or properly shored to prevent sloughing and collapse. As for the estimation purposes, in our opinion, for temporary excavations equal to the exploration depths, the side slopes should be laid back at a minimum slope inclination of 3:1 (Horizontal:Vertical) for the native loose to medium dense gravelly soil deposits.

The recommended inclinations assumes that the ground surface behind the cut slopes is level, that surface loads from equipment and materials are kept a sufficient distance away from the top of the slope. If these assumptions are not valid, we should be contacted for additional recommendations. Flatter slopes may be required if soils are loose or caving and/or water, are encountered along the slope faces. If such conditions occur and the excavation cannot stand by itself, or the excavation slope cannot be flattened because of the space limitations between the excavation line and the boundary of the property, temporary shoring may be considered. The shoring will assist in preventing slopes from failure and provide protection to field personnel during excavation. Because of the diversity available of shoring stems and construction techniques, the design of temporary shoring is most appropriately left up to the contractor engaged to complete the installation. We can assist in designing the shoring system by providing with detailed shoring design parameters including earth-retaining parameters, if required.

Where sloped embankments are used, the top of the slopes should be barricaded to prevent vehicles and storage loads within 10 feet of the top of the slopes. Greater setbacks may be necessary when considering heavy vehicles, such as concrete trucks and cranes. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the top of the slopes to prevent runoff water from entering the excavation and eroding the slope faces. All temporary slopes should be protected from surface water runoff.

The above information is provided solely for the benefit of the owner and other design consultants, and under no circumstances should not be construed to imply that PGE assumes responsibility for construction site safety or the contractor's activities; such responsibility is not being implied and should not be inferred. Therefore, the contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor's "responsible person", as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures.

We expect that the excavation can be completed using conventional equipments such as bulldozers or backhoes.

<u>Erosion Control Measurements</u>

Uncontrolled surface water with runoff over unprotected site surfaces during construction activities is considered the single most important factor that impacts the erosion potential of a site. The erosion process may be accelerated significantly when factors such as soils with high fines, sloped surface, and wet weather combines together. Taking into consideration of the very low fines content in the native soils, the project site will have minor impact due to erosion during the wet winter months.

The erosion hazard can be mitigated if the mass grading activities and the earthwork can be completed within the dry summer period. Also, measurements such as the control of surface water must be maintained during construction, and a temporary erosion and sedimentary control (TESC) plan, as a part of the Best Management Practices (BMP) must be developed and implemented as well. The TESC plan should include the use of geotextile barriers (silt fences) along any down-slope, straw bales to de-energize downward flow, controlled surface grading, limited work areas, equipment washing, storm drain inlet protection, and sediment traps. Also, vegetation clearing must be kept very limited in this site to reduce the exposed surface areas. A permanent erosion control plan is to be implemented following the completion of the construction. Permanent erosion control measurements such as establishment of landscaping, control of downspouts and surface drains, control of sheet flow over the wall, prevention of discharging water over the wall and at the toe of the wall are to be implemented following the completion of the construction

Dry Weather Construction

We recommend that the proposed detention pond should be built during the dry season of the year to minimize the water accumulation problem in the wall footing areas.

Wall Construction Considerations and Limitations

The stability of the walls can be detrimentally affected during or after the wall construction if the following situation occurs: (1) uncontrolled construction related activities such as heavy equipment traffic and unwise excavation be made adjacent to the top and the toe of the walls, (2) any additional loads such as stockpile of soils be placed on the adjacent ground to the top of the walls, and (3) alteration of the slope gradients above the walls than considered in this study takes place. Any of the above activities and the conditions may reduce the post-construction factor of safety values. If any of the above activities within the slope above the walls are required then the plans with new slope conditions be submitted to us for our review to develop corresponding change-specific recommendations. In our opinion, imposition of some restrictions to the proposed construction activities may help reducing the possibility of occurrence of local slope failures.

In order to protect the integrity of the final slope above the retaining walls, and to avoid any deterioration of them, we recommend that the final bcakhill slope be covered with grass after the construction is completed.

Care in the placement and compaction of fill behind walls must be taken in order to insure that undue lateral loads are not induced on the walls. Large equipment must not be allowed within the backside horizontal distance of the walls that equals to the wall height.

Compaction of the backfills behind the walls should be accomplished using hand operated or walk-behind equipment such as a small roller or a hand-operated plate compactor.

6.0 Geotechnical Special Inspections

Due the critically involved for maintaining the current stability of the existing sidewalk and the excavation faces during the construction, and also for achieving the stability of the wall for its future performances, the construction of the wall should be inspected by an experienced geotechnical inspector.

Items such as excavation, excavation face inclination, clearing of subgrades, footing subgrades preparation and proofrolling, crushed rocks base layer laying and compaction, block orientation and laying, drainage layer, geosynthetic filter fabric and footing drainpipe installation, encapsuling of footing drainpipe with rocks, footing embedment depth determination, wall batter, fill placement and compaction at the toe and at the back of the wall, and other components of the wall construction described in the report and in the relevant wall details figures are critical for the stability of the walls and for its long-term performances, therefore, these items should be verified on-site by an experienced geotechnical inspector. We recommend that a representative from PGE should be retained on-site by the owner or the contractor to observe the wall construction in order to verify that the recommendations provided in this report are properly understood and followed. It should be noted that the inspection of the wall construction by PGE could be performed in a separate contract and cost.

In conjunction with the above requirement, it is highly recommended that the wall construction and its inspection must be done by an experienced contractor and inspector with local experiences in similar type of wall construction.

7.0 Construction Monitoring

Problems associated with the construction of the walls can be avoided or corrected during the progress of the construction if proper inspection and testing services are provided. It is recommended that site preparation activities such as overexcavation of existing unsuitable soils, backfilling and compaction of new fills, compaction tests on new fills, footing subgrade option selection, final footing subgrade preparation and inspection, and determination of suitable structural fills be monitored by a representative from PGE to confirm that the recommendations provided in this report are understood, interpreted, and implemented properly by the contractor.

8.0 Report Limitations

The evaluation and recommendations presented in this report are based upon the information available from our subsurface explorations, and the project details furnished by Larson and Associates. The study was performed using a mutually agreed-upon scope of work, which is presented in this report.

It should be noted that PGE cannot take the responsibility regarding the accuracy of the information available from other consultant. If any of the information considered during this study is not correct or if there are any revisions to the plans for this project, PGE should be notified immediately of such information and the revisions so that necessary amendment of our geotechnical recommendations can be made. If such information and revisions are not notified to PGE, no responsibility should be implied on PGE for the impact of such information and the revisions on the project.

Variations in soil and groundwater conditions may exist between the locations of the explorations and the actual conditions underlying the site. The nature and the extent of variations in soil and groundwater conditions may not be evident until construction occurs. If any soil and groundwater conditions are encountered at the site that are different from those described in this report, we should be notified immediately to review the applicability of our recommendations if there are any changes in the project scope.

This report may be used only by the client and for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both off and on-site), or others factors including advances in our understanding of applied science, may change over time and could materially affect our findings. Therefore, this report should not be relied upon after 24 months from its issuance. PGE should be notified if the project is delayed by more than 24 months from the date of this report so that we may review to determine that the conclusions and recommendations of this report remain applicable to the changed conditions.

The scope of our work does not include services related to construction safety precautions. Our recommendations are not intended to direct the contractors' method, techniques, sequences or procedures, except as specifically described in our report for consideration in design. Additionally, the scope of our work specifically excludes the assessment of environmental characteristics, particularly those involving hazardous substances.

This report including its evaluation, conclusions, specifications, recommendations, or professional advice has been prepared for planning and design purposes for specific application to the proposed project in accordance with the generally accepted professional geotechnical engineering practices in the local areas at the time this report was written. No warranty, express or implied, is made.

This report is the property of our client, and has been prepared for the exclusive use of our client and its authorized representatives for the specific application to the proposed development at the subject site in Buckley, Washington.

It is the client's responsibility to see that all parties to this project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk. Any party other than the client who wishes to use this report shall notify PGE of such intended use and for permission to copy this report. Based on the intended use of the report, PGE may require that additional work be performed and that and updated report be reissued. Noncompliance with any of these requirements will release PGE from any liability resulting from the use of this report.

If there is a substantial lapse of time between the submission of this report and the start of the proposed construction work, or if the present conditions of the site changes during the lapsed time due to natural causes or construction activity at or adjacent to the site, it is recommended that this report be reviewed to determine that the conclusions and recommendations of this report remain applicable to the changed conditions.

9.0 Closure

We trust the information presented in this report is sufficient for your current needs. We appreciate the opportunity to provide the geotechnical services at this phase of the project and look forward to continued participation during the design and construction phase of this project. Should you have any questions or concerns, which have not been addressed, or if we may be of additional assistance, please do not hesitate to call us at 425-218-9316 or 425-643-2616 at your convenience.

Respectfully submitted,

Santanu Mowar

Santanu Mowar, MSCE, P.E.

PGE Pacific Geo Engineering

SANTANU MOWAR
STATE OF WASHINGTON
37403
REGISTERED
PROFESSIONAL ENGINEER
EXPIRES 01-01-2016

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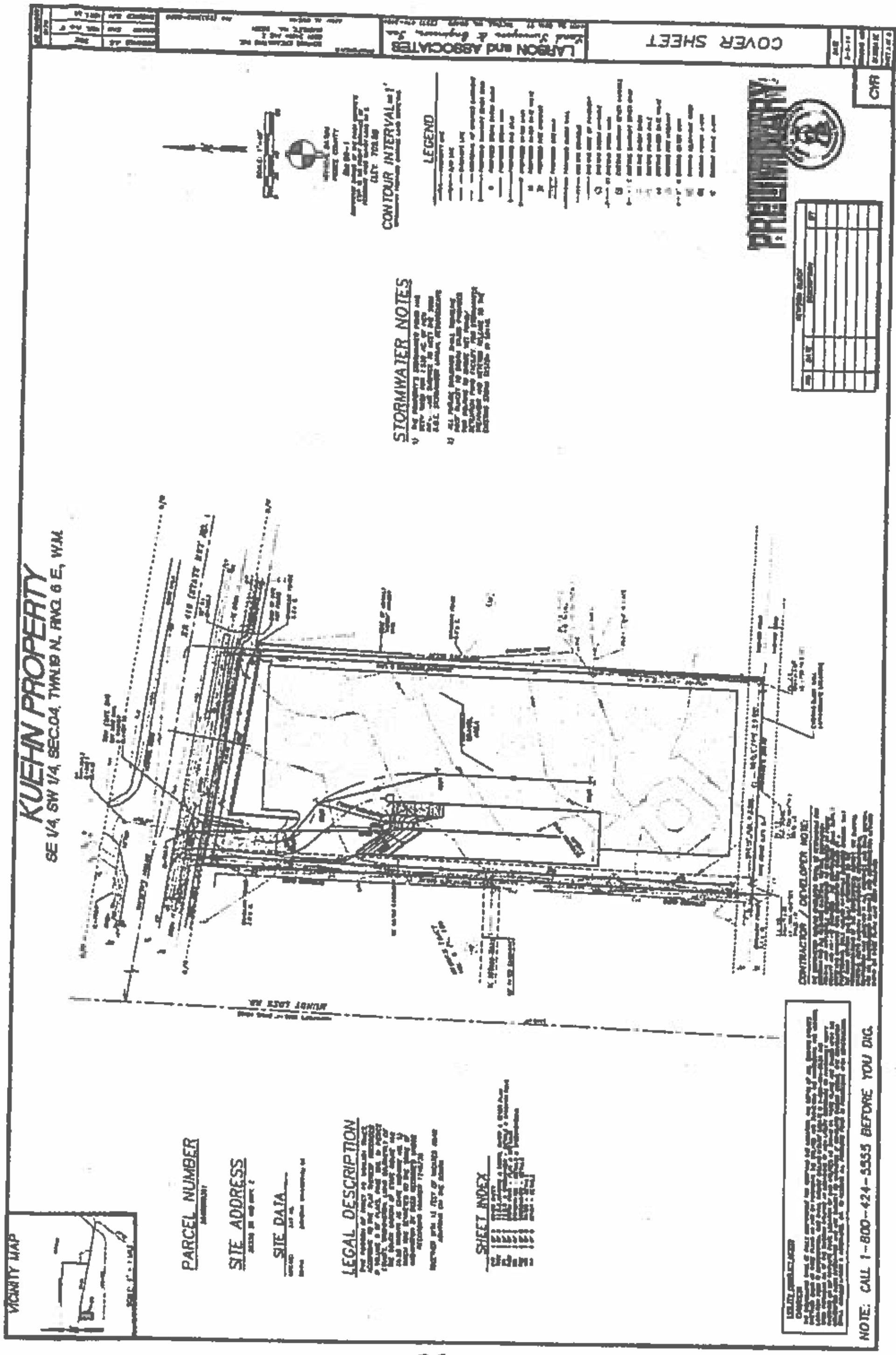
Attachments:	Appendix A:	Detention Pond Plans, prepared by Larson & Associates	
	Appendix B:	Soil Test Pit Log	
	Appendix C:	Wall Details -	
		Figure 1:	Earth Pressure Diagrams
		Figure 2:	Concrete Retaining Wall Details - E & S Walls
		Figure 3:	Concrete Retaining Wall Details - W Wall
		Figure 4:	Ecology Block Baffle Wall Details
		Figure 5:	Ecology Block Retaining Wall Details
	Appendix D:	Design Calculation for Ecology Block Retaining Wall	


KUEHN PROPERTY
SE 1/4, SW 1/4, SEC.04, TWN.19 N, RNG.6 E, W.M.
COVER SHEET
LARSON and ASSOCIATES
STORMWATER NOTES
LEGEND
CONTOUR INTERVAL=1'
PRELIMINARY
CVR
VICINITY MAP
PARCEL NUMBER
SITE ADDRESS
SITE DATA
LEGAL DESCRIPTION
SHEET INDEX
CONTRACTOR / DEVELOPER NOTE:
NOTE: CALL 1-800-424-5555 BEFORE YOU DIG.

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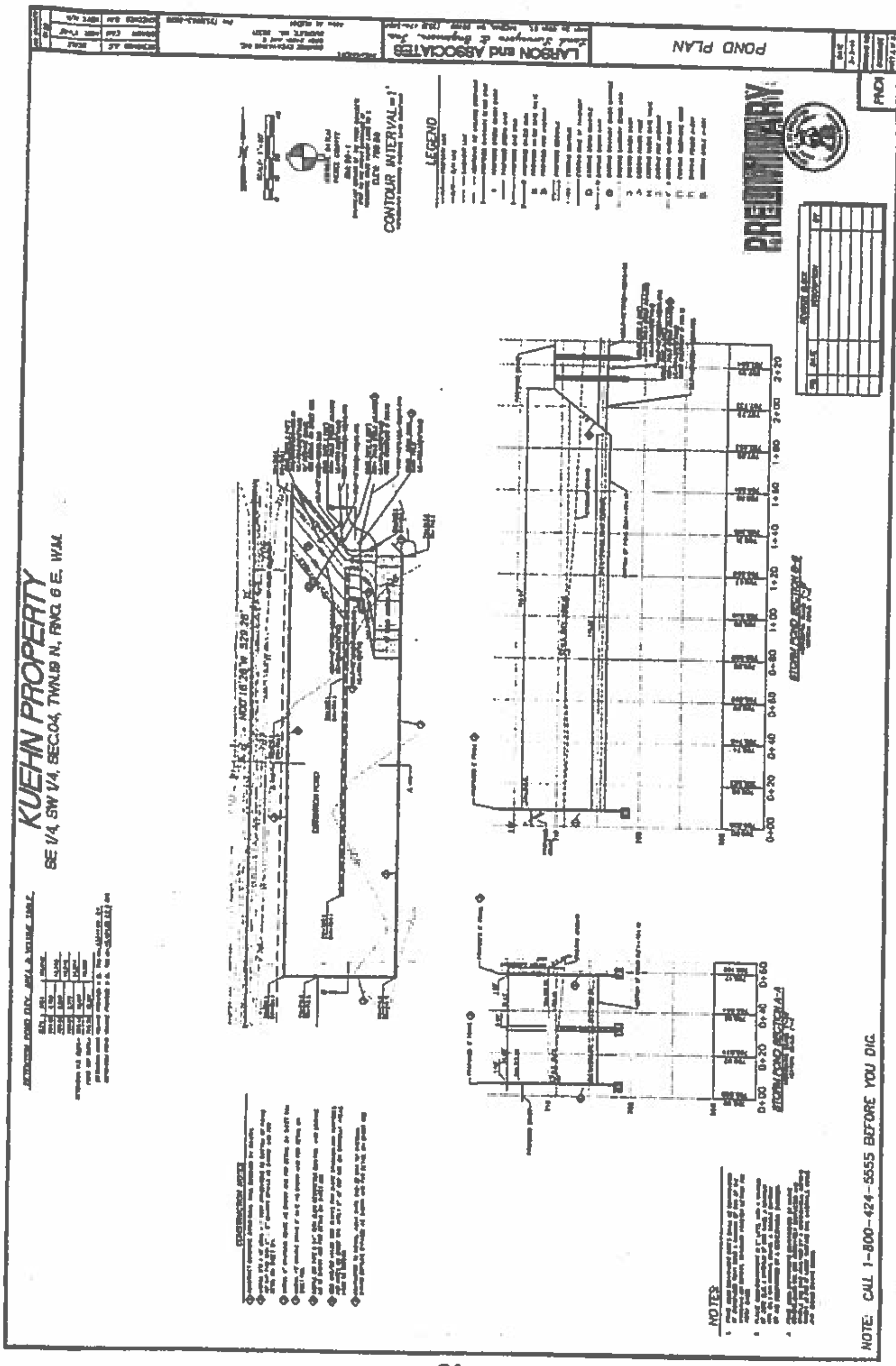
KUEHN PROPERTY
SE 1/4, SW 1/4, SEC.04, TWN.19 N, RNG. 6 E., W.M.
POND PLAN
LARSON and ASSOCIATES
PRELIMINARY
LEGEND
CONTOUR INTERVAL=1'
NOTES
NOTE: CALL 1-800-424-5555 BEFORE YOU DIG.

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Appendix B
Soil Test Pit Log

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General Sub-Surface Conditions in Test Pit		
Approximate Depths	**USCS Soil Class**	**Soil Descriptions**
0 – 2'	-	Fills - Brn. Silty Sandy Gravel w/ some Cobbles and Boulders, Moist, Medium Dense. This layer has been in place for last 1.5 years.
2' – 4'	-	Fills - Same as above. This layer has been in place for last 4.5 years.
4' – 5'	SM	Original Topsoil - Blk.-Brn. Silty Sand w/ organics.
5' - 11'	GP	Brn. Very Gravelly Silty Sand w/ abundant Cobbles and Boulders, pocket of partly cemented silty lenses, Mottled, Moist, Med. Dense to Dense.
11' - 15'	GP	Same as above, Mottled, Wet, Dense.

Note: Test pit was terminated at approximately 15 feet below the existing ground surfaces.
Signs of mottling were noticed between 5 to 15 feet depths below the existing ground surfaces.
No groundwater or seepage was encountered within the exploration depths.
Moderate caving was noticed below 6 to 7 feet depths of the existing ground surfaces.

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Appendix C
Wall Details

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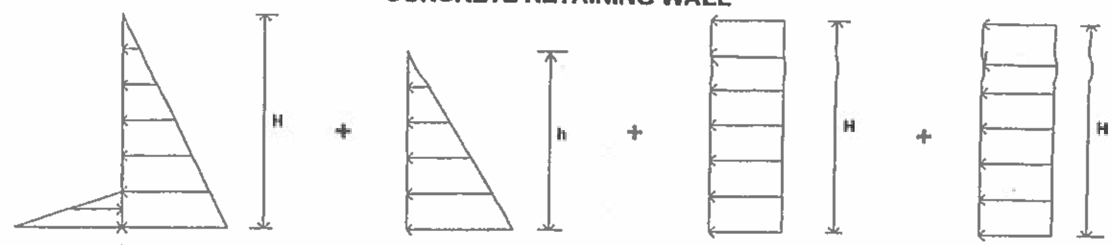
EARTH PRESSURE DIAGRAMS & DESIGN PARAMETERS FOR CONCRETE RETAINING WALL



Lateral passive earth pressure = 160 pcf for static condition and 130 pcf for seismic condition. A FS of 1.5 is to be used in the above values. Passive pressure is to be neglected for upper 1 ft of embedment depth

Active lateral earth pressure (effective) = 24H psf for level backhill slope above wall

Hydrostatic pressure = 62.5h psf

Seismic loading = 4H psf for active condition and level backhill slope

Lateral earth pressure for traffic surcharge = 0.39q psf

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Notes:-

1. The appropriate combination of the various components of the lateral pressures for pond wall design will be decided by the structural engineer based on the loadings on the wall, and wall's location, height, drainage condition, and water levels on its both sides.

2. The possibility of buoyant effect on the wall due to the presence of highest water levels inside the pond and outside the wall during the wet winter months should be considered during the wall design by the structural engineer (see Figure 2 & 3).

Design Considerations:-

1. Yielding (cantilever) wall is to be allowed to translate or rotate its free end a lateral distance equal to 0.001H to 0.002H to mobilize the active earth pressure on the wall.

2. The seismic pressures are based on a design acceleration coefficient of 0.25 to 0.30.

3. The static passive pressure is reduced by 20% for seismic condition.

4. The static bearing capacity is increased by ⅓ for the seismic condition.

Other Design Parameters:-

Retained saturated soil unit wt., γ_{sat} = 125 pcf
Unit wt. of water, γ_w = 62.5 pcf
Retained effective (buoyant) soil unit wt., γ_b = 62.5 pcf
Effective Angle of internal friction, ø' = 26°
Co-eff. of active earth pressure, K_a = 0.39
Co-eff. of passive earth pressure, K_p = 2.56
Base friction co-eff. μ = 0.35
Static bearing capacity = 2500 psf
Seismic bearing capacity = 3300 psf
Traffic surcharge, q = 250 psf

Figure 1 Not to Scale

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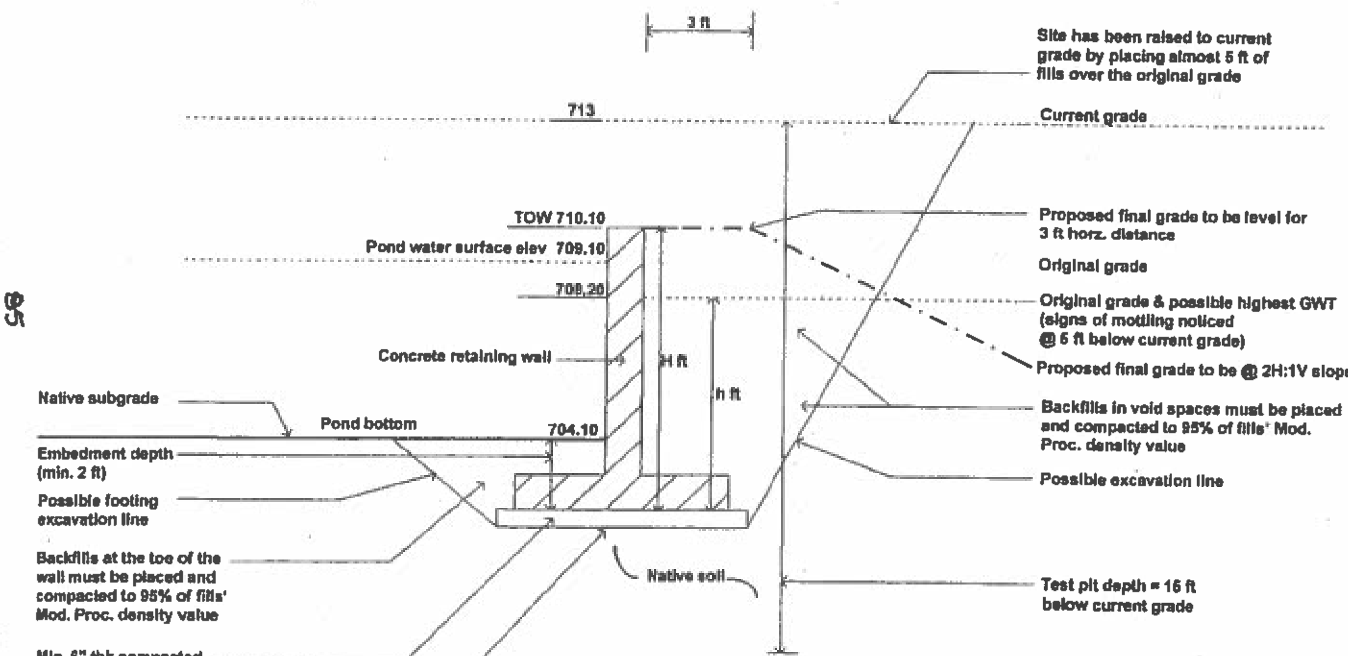
BELOW GRADE POND CONCRETE RETAINING WALL (WEST)



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Note:-

The recommendations provided in the relevant section of the text portion of the report must be followed to construct the wall.

Figure 3 — Not to Scale

Project – Al Kuehn Property
Project No. – 13-423

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BELOW GRADE POND CONCRETE RETAINING WALL (EAST & SOUTH)

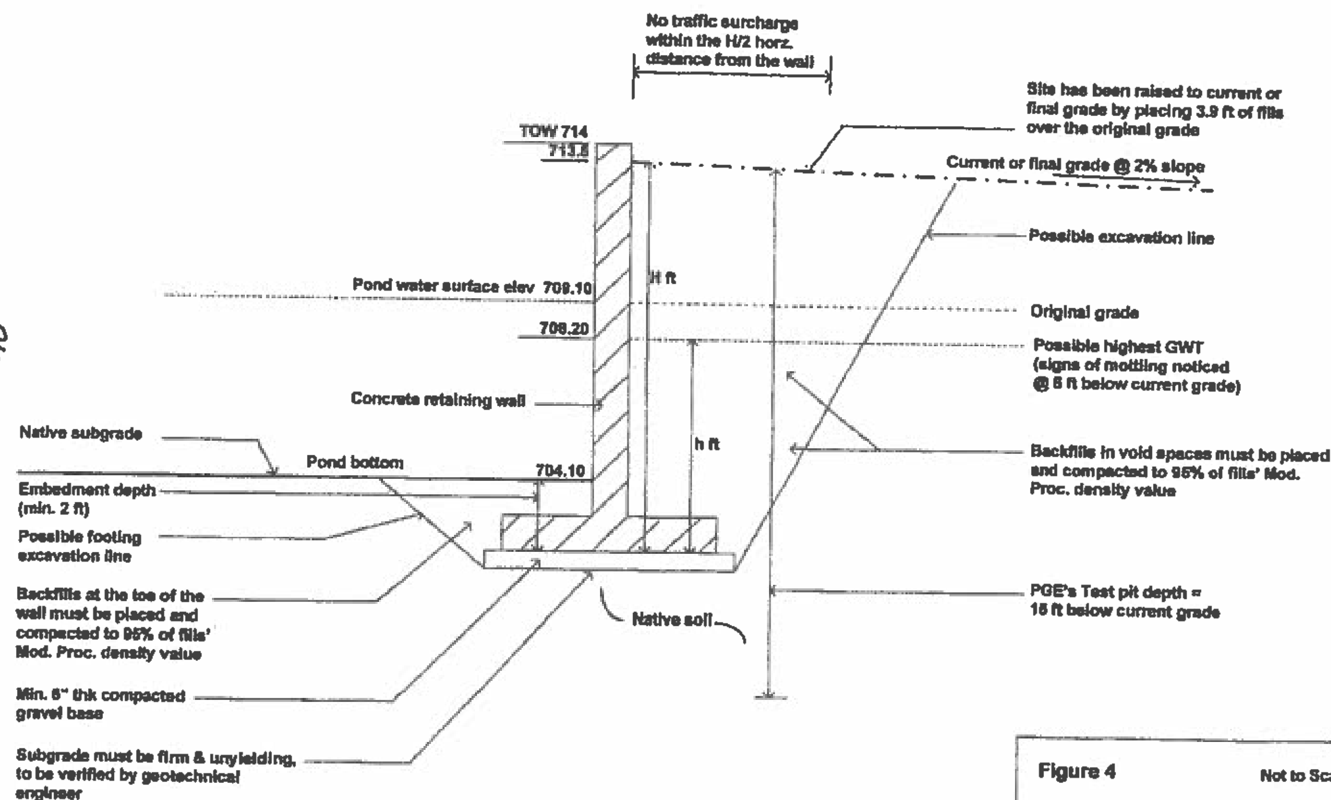

Figure 4 Not to Scale

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Project No. – 13-423

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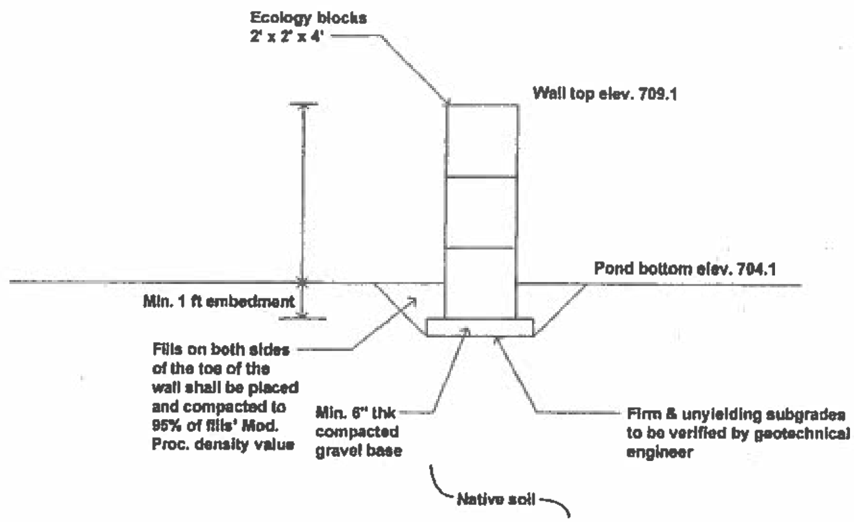
POND BAFFLE WALL DETAILS



Notes:-

1. The recommendations provided in the relevant section of the text portion of the report must be followed to construct the wall.

2. The wall must be placed on a horizontal grade to avoid any tilting of the wall. This is very critical for the vertical stability of the wall. The vertical level of the wall must be checked, with the horizontal level at the final footing grade must be checked by the contractor.

3. All cores in the blocks lined up together through the blocks shall be filled with 2,500 psi strength at 28 days concrete with (1) #5 bar located at the center of the cores.

Figure 4 Not to Scale

Project – Al Kuehn Property
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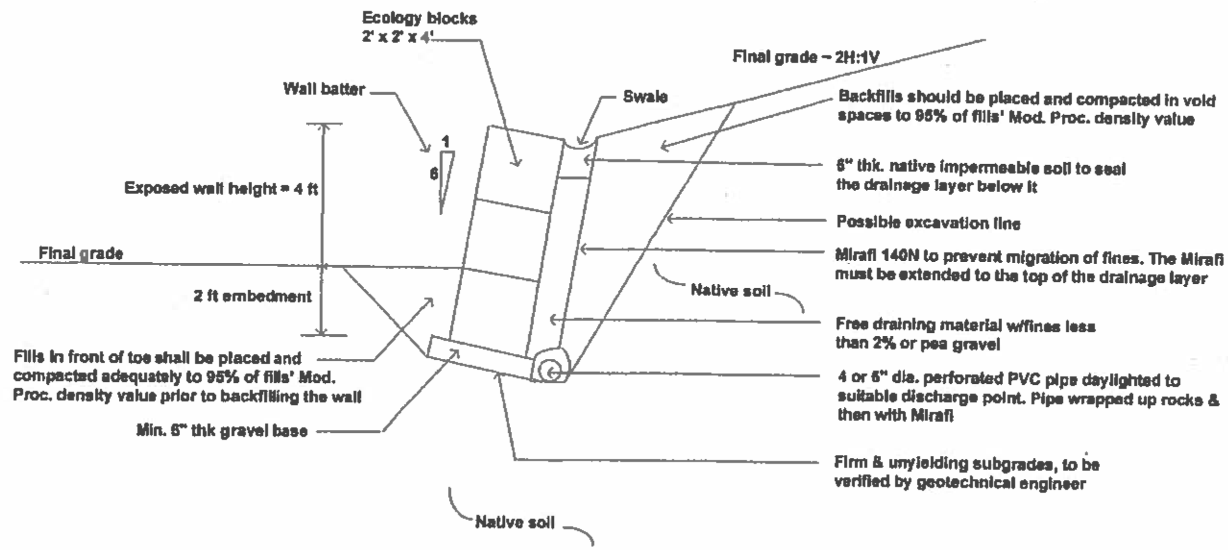
PERIMETER ECOLOGY BLOCK WALL DETAILS



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Notes:-

1. The recommendations provided in the relevant section of the text portion of the report must be followed to construct the wall.

2. All cores in the blocks lined up together through the blocks shall be filled with 2,500 psi strength at 28 days concrete with (1) #5 bar located at the center of the cores

Figure 5 Not to Scale

Project – Al Kuehn Property
Project No. – 13-423

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Appendix D
Ecology Block Retaining
Wall Design Calculations

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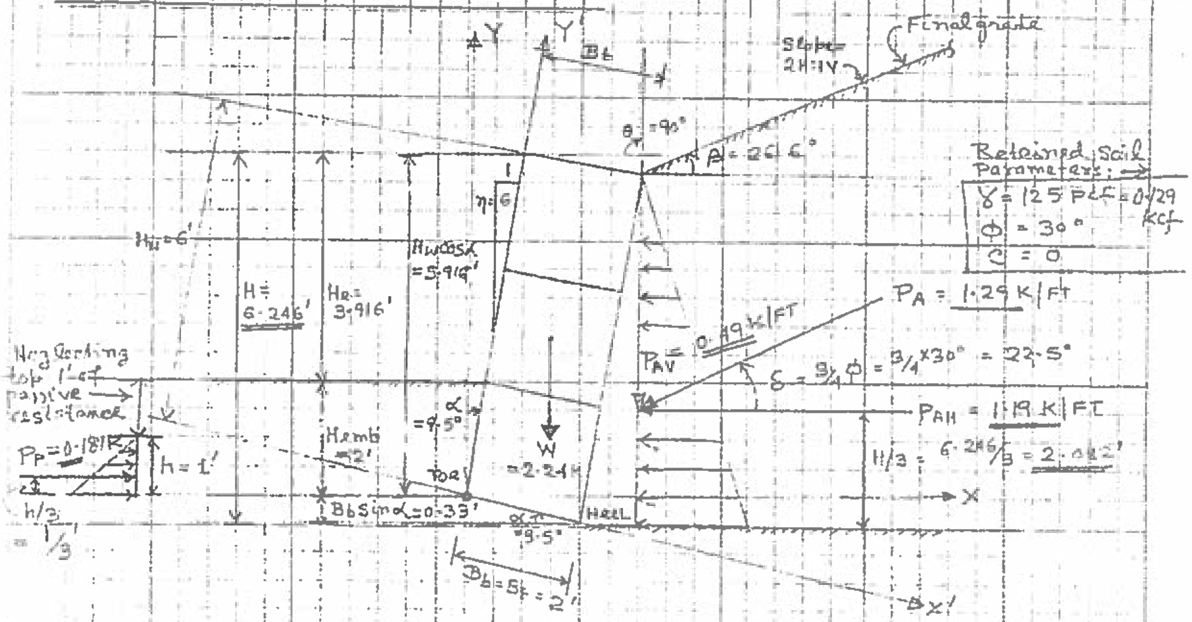
Geotechnical Engineering, Consulting & Inspection

PROJECT	AL KUEHN PROPERTY	PGE PROJECT NO.	13 - 423
PROJECT LOCATION	28238, SR 410 HWY. E, BUCKLEY, WA	CLIENT PROJ. NO.	
CLIENT	AL KUEHN	PERMIT/P.O. NO.	
OWNER	AL KUEHN	MEETING VENUE	
PGE REPRESENTATIVE		DATE	03-25-2014
PGE DESIGNER	SANTANU MOWAR	PAGE NO.	1 of 4
SUBJECT	PERIMETER ECOLOGY BLOCK WALL		
Project Notes	Meeting Notes	Telephone Conversation Notes	✓ Design Calculations

4' HIGH ECOLOGY BLOCK WALL DESIGN



$S_b = B_t = 2'$, $\alpha = 9.5°$ for batter = 1H:6V, $\sin\alpha = 0.165$, $\cos\alpha = 0.986$, $\tan\alpha = 0.167$

$\theta = 90°$, $\beta = 26.6°$ for backfill slope = 2H:1V

$H_w = 6'$, $H_w \cos\alpha = 6' \times 0.986 = 5.916'$, $H_{emb} = 2'$, $H_e = 5.916' - 2' = 3.916'$

~~$B_t \sin\alpha = 2' \sin 9.5° = 2' \times 0.165 = 0.33'$~~, ~~$H = 5.916 + 2' + 0.33'$~~ $= \underline{6.246}$

$$K_a = \frac{\sin^2(\theta+\phi)}{\sin^2\theta \, \sin(\theta-\delta)\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\,\sin(\phi-\beta)}{\sin(\theta-\delta)\,\sin(\theta+\beta)}}\right]^2}$$

~~$K_p = \tan^2(45° + \phi/2)$~~ $= \underline{3.0}$

$$= \frac{\sin^2(90+30)}{\sin^2 90 \, \sin(90-22.5)\left[1+\sqrt{\dfrac{\sin(30+22.5)\,\sin(30-26.6)}{\sin(90-22.5)\,\sin(90+26.6)}}\right]^2} = \frac{0.75}{1 \times 0.92 \times 1.53} = \underline{0.53}$$

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P.O. Box 1419, Issaquah, WA, 98027 (Tel) 425-643-7616 (Fax) 425-643-0436

PGE Pacific Geo Engineering LLC

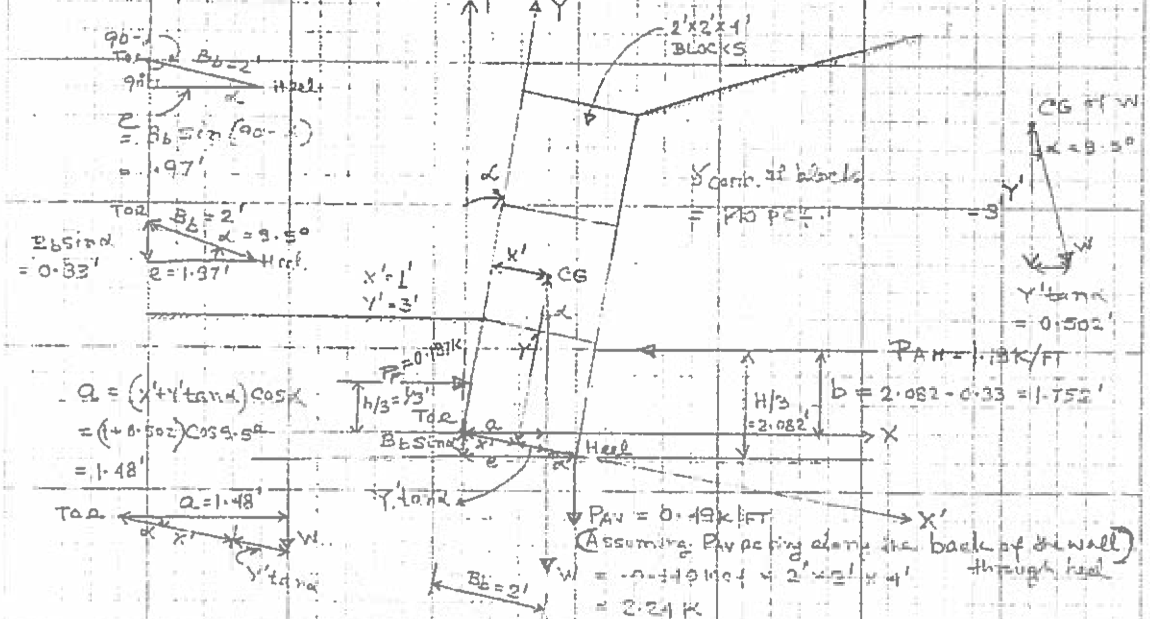
Geotechnical Engineering, Consulting & Inspection

PROJECT	AL KUEHN PROPERTY	**PGE PROJECT NO.**	13-423
PROJECT LOCATION	28838, SR410 HWY E, BUCKLEY, WA	**CLIENT PROJ. NO.**	
CLIENT	AL KUEHN	**PERMIT/P.O. NO.**	
OWNER	AL KUEHN	**MEETING VENUE**	
PGE REPRESENTATIVE		**DATE**	03-25-2014
PGE DESIGNER	SANTANU MONAR	**PAGE NO.**	2 of 4
SUBJECT	PERIMETER ECOLOGY BLOCK WALL		
Project Notes	Meeting Notes	Telephone Conversation Notes	Design Calculations

$P_A = \frac{1}{2} \times K_a \times \gamma \times H^2 = \frac{1}{2} \times 0.53 \times 0.125 \times 6.246^2 = 1.29\ K/Ft.$

$P_{AH} = P_A \cos(90° - \theta + \delta) = 1.29 \cos(90 - 90 + 22.5) = 1.19\ K/FT.$

$P_{AV} = P_A \sin(90 - \theta + \delta) = 1.29 \sin(90 - 90 + 22.5) = 0.49\ K/FT.$



$W = 2.24K,\quad a = 1.48',\quad b = 1.752',\quad e = B_b \sin(90 - \alpha) = 1.97',\quad h/3 = 1/3$

Taking moments about toe of all forces,

$M_R = Wa + P_{AV}\,e + P_P \times h/3$

$= 2.24 \times 1.48' + 0.49 \times 1.97' + 0.187 \times \frac{1}{3}'$

$= 4.33\ KFt.$

Passive resistance @ toe

$= P_P = \frac{1}{2} \times K_p \times \gamma \times h^2$

$= \frac{1}{2} \times 3.0 \times 0.125 \times 1^2$

$= 0.187\ K/FT.$

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Geotechnical Engineering, Consulting & Inspection

PROJECT	AL KUEHN PROPERTY	PGE PROJECT NO.	13-423
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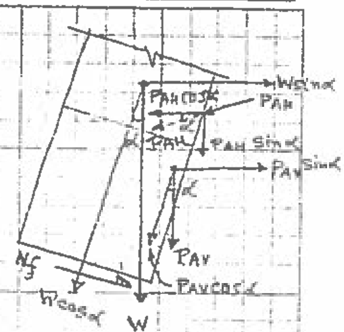
OWNER	AL KUEHN	MEETING VENUE	
PGE REPRESENTATIVE		DATE	03-25-2014
PGE DESIGNER	SANTANU MOWAR	PAGE NO.	3 of 4
SUBJECT	PERIMETER ECOLOGY BLOCK WALL		
Project Notes	Meeting Notes	Telephone Conversation Notes	Design Calculations

$M_D = P_{AH}\, b = 1.19 \times 1.752 = 2.08$ KFt.



overturning FOS $= \dfrac{M_R}{M_D} = \dfrac{4.32}{2.08} = 2.08 > 2$ ok.

$f = \tan\delta = \tan 22.5° = 0.41$

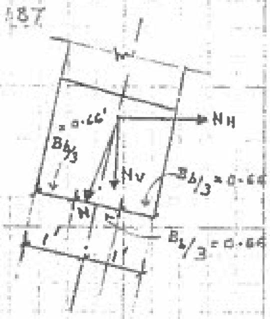
$N = W\cos\alpha + P_{AV}\cos\alpha + P_{AH}\sin\alpha$

$= 2.24 \times 0.986 + 0.49 \times 0.986 + 1.19 \times 0.165 = 2.876$ K/Ft.

$F_R = Nf + W\sin\alpha + P_{AV}\sin\alpha +$ (Passive resistance @ toe $= 0.187$ K/Ft)

$= 2.876 \times 0.41 + 2.24 \times 0.165 + 0.49 \times 0.165 + 0.187$

$= 1.807$ K/Ft.



$F_D = P_{AH}\cos\alpha = 1.19 \times 0.986 = 1.17$ K/Ft.

Sliding FOS $= \dfrac{F_R}{F_D} = \dfrac{1.807}{1.17} = 1.54 > 1.5$, So ok.

$N = 2.876$ K/Ft. $N_V = N\cos\alpha = 2.876 \times 0.986 = 2.835$ K/Ft

$N_H = N\sin\alpha = 2.876 \times 0.165 = 0.474$ K/Ft.

$\bar{x} = \dfrac{(M_R - M_D)}{N_V + N_H \tan\alpha} = \dfrac{4.33 - 2.08}{2.835 + 0.474 \times 0.167} = 0.773'$; $\dfrac{x}{\cos\alpha} = \dfrac{0.773'}{0.986} = 0.784'$

$e_x = \dfrac{B_b}{2} - \dfrac{x}{\cos\alpha} = \dfrac{2}{2} - \dfrac{0.773'}{0.986} = 0.216'$ from center of base measured along the plane parallel to the base.

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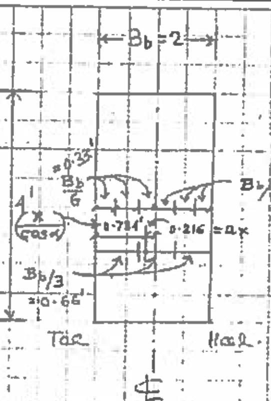
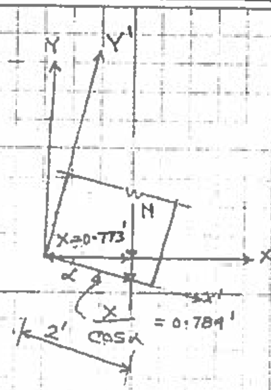


Eccentricity, $e_x = 0.216'$ is less than $B_b/6$ and falls within the middle third of the B_b, so ok.



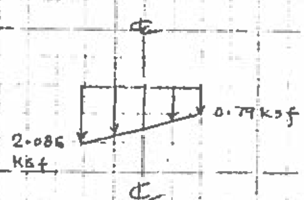
$$\frac{B_b}{3} \le \frac{X}{\cos\alpha} \qquad B_b - \frac{X}{\cos\alpha} \le \frac{2B_b}{3}$$

$$\Rightarrow \frac{2'}{3} \le \frac{0.773'}{0.786} \qquad \Rightarrow 2' - \frac{0.773'}{0.786} \le \frac{2\times 2'}{3}$$

$$\Rightarrow 0.66' \le 0.785', \text{ so ok.} \qquad \Rightarrow 1.216' \le 1.33', \text{ so ok.}$$

At toe, $q_{max} = \frac{N}{B_b} \times \frac{1+6e_x}{B_b} = \frac{2.876\,K}{2'} \times \frac{1+6\times(0.216')}{2'} = 2.086\ ksf$

At heel, $q_{min} = \frac{N}{B_b} \times \frac{1-6e_x}{B_b} = \frac{2.876\,K}{2'} \times \frac{1-6\times(0.216')}{2'} = 0.79\ ksf.$



The pressures are +ves, so no tension in base.

Also, the pressures are less than the allowable bearing pressure so ok.

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SECTION G: OTHER PERMITS

PERMITS REQUIRED

CITY-ISSUED CONSTRUCTION PERMIT

Required.

RIGHT-OF-WAY USE

A Right-of-Way use from the state is necessary to construct the driveway.

NPDES STORMWATER PERMIT

An NPDES permit from the State DOE is required because construction is over one acre.

PERMITS NOT REQUIRED

The threshold triggering a requirement for each of the following permits has not been met.

FOREST PRACTICES PERMIT, Department of Natural Resources

Project does not meet the threshold of 5,000 board feet of merchantable timber that triggers the requirement for a Forest Practices Permit.

SECTION 10, 401 and 404 PERMIT, Army Corps of Engineers

No discharge of dredge material or fill into waters of the U.S., including wetlands, is contemplated.

WATER QUALITY CERTIFICATION (401), Department of Ecology

The need for a 401 permit is automatically triggered by application for Federal license or permits, such as Section 404. It is certification by the State that the permitted activity complies with:

SECTION H: OPERATION AND MAINTENANCE MANUAL

Maintenance of the stormwater facilities will be the specific responsibility of Albert Kuehn, or his successor(s) in assigns. Upon transfer of parcel title, maintenance responsibility will transfer to the new owner or designee of the new owner.

MAINTENANCE REQUIREMENTS FOR FLOW CONTROL, CONVEYANCE, AND WQ FACILITIES

NO. 1 – DETENTION PONDS			
Maintenance Component	Defect or Problem	Conditions When Maintenance Is Needed	Results Expected When Maintenance Is Performed
General	Trash & Debris	Any trash and debris which exceed 1 cubic foot per 1,000 square feet (this is about equal to the amount of trash it would take to fill up one standard size office garbage can). In general, there should be no visual evidence of dumping.	Trash and debris cleared from site.
	Poisonous Vegetation or Noxious Weeds	Any poisonous or nuisance vegetation which may constitute a hazard to County personnel or the public.	No danger of poisonous vegetation where County personnel or the public might normally be. Coordination with Seattle-King County Health Department
	Contaminants and Pollution	Oil, gasoline, or other contaminants of one gallon or more, or any amount found that could: 1) cause damage to plant, animal, or marine life; 2) constitute a fire hazard; or 3) be flushed downstream during rain storms.	No contaminants present other than a surface film. (Coordination with Seattle/King County Health Department)
	Unmowed Grass/Ground Cover	If facility is located in private residential area, mowing is needed when grass exceeds 18 inches in height. In other areas, the general policy is to make the pond site match adjacent ground cover and terrain as long as there is no interference with the function of the facility.	When mowing is needed, grass/ground cover should be mowed to 2 inches in height. Mowing of selected higher use areas rather than the entire slope may be acceptable for some situations.
	Rodent Holes	Any evidence of rodent holes if facility is acting as a dam or berm, or any evidence of water piping through dam or berm via rodent holes or other causes.	Rodents destroyed and dam or berm repaired. (Coordination with Seattle/King County Health Department)
	Insects	When insects such as wasps and hornets interfere with maintenance activities. Mosquito complaints accompanied by presence of high mosquito larvae concentrations (aquatic phase).	Insects destroyed or removed from site. Mosquito control: Swallow nesting boxes or approved larvicide applied.
	Tree Growth	Tree growth threatens integrity of berms acting as dams, does not allow maintenance access, or interferes with maintenance activity (i.e., slope mowing, silt removal, vactoring, or equipment movements). If trees are a threat to berm integrity or not interfering with access, leave trees alone.	Trees do not hinder maintenance activities. Harvested trees should be recycled into mulch or other beneficial uses (e.g., alders for firewood).

NO. 4 – CONTROL STRUCTURE/FLOW RESTRICTOR

Maintenance Component	Defect or Problem	Condition When Maintenance is Needed	Results Expected When Maintenance is Performed
General	Trash and Debris (Includes Sediment)	Distance between debris build-up and bottom of orifice plate is less than 1.5 feet.	All trash and debris removed.
	Structural Damage	Structure is not securely attached to manhole wall and outlet pipe structure should support at least 1,000 lbs of up or down pressure.	Structure securely attached to wall and outlet pipe.
		Structure is not in upright position (allow up to 10% from plumb).	Structure in correct position.
		Connections to outlet pipe are not watertight and show signs of rust.	Connections to outlet pipe are water tight; structure repaired or replaced and works as designed.
		Any holes–other than designed holes—in the structure.	Structure has no holes other than designed holes.
Cleanout Gate	Damaged or Missing	Cleanout gate is not watertight or is missing.	Gate is watertight and works as designed.
		Gate cannot be moved up and down by one maintenance person.	Gate moves up and down easily and is watertight.
		Chain/rod leading to gate is missing or damaged.	Chain is in place and works as designed.
		Gate is rusted over 50% of its surface area.	Gate is repaired or replaced to meet design standards.
Orifice Plate	Damaged or Missing	Control device is not working properly due to missing, out of place, or bent orifice plate.	Plate is in place and works as designed.
	Obstructions	Any trash, debris, sediment, or vegetation blocking the plate.	Plate is free of all obstructions and works as designed.
Overflow Pipe	Obstructions	Any trash or debris blocking (or having the potential of blocking) the overflow pipe.	Pipe is free of all obstructions and works as designed.
Manhole	See "Detention Tanks and Vaults"	See "Detention Tanks and Vaults" Table No. 3	See "Detention Tanks and Vaults" Table No. 3

NO. 8 – FENCING

Maintenance Component	Defect or Problem	Conditions When Maintenance Is Needed	Results Expected When Maintenance Is Performed
General	Missing or Broken Parts	Any defect in the fence that permits easy entry to a facility.	Parts in place to provide adequate security.
	Erosion	Erosion more than 4 inches high and 12-18 inches wide permitting an opening under a fence.	No opening under the fence that exceeds 4 inches in height.
Wire Fences	Damaged Parts	Post out of plumb more than 6 inches.	Post plumb to within 1½ inches.
		Top rails bent more than 6 inches.	Top rail free of bends greater than 1 inch.
		Any part of fence (including post, top rails, and fabric) more than 1 foot out of design alignment.	Fence is aligned and meets design standards.
		Missing or loose tension wire.	Tension wire in place and holding fabric.
		Missing or loose barbed wire that is sagging more than 2½ inches between posts.	Barbed wire in place with less than ¾ inch sag between post.
		Extension arm missing, broken, or bent out of shape more than 1½ inches.	Extension arm in place with no bends larger than ¾ inch.
	Deteriorated Paint or Protective Coating	Part or parts that have a rusting or scaling condition that has affected structural adequacy.	Structurally adequate posts or parts with a uniform protective coating.
	Openings in Fabric	Openings in fabric are such that an 8-inch diameter ball could fit through.	No openings in fabric.

NO. 9 – GATES

Maintenance Component	Defect or Problem	Conditions When Maintenance Is Needed	Results Expected When Maintenance Is Performed
General	Damaged or Missing Members	Missing gate or locking devices.	Gates and Locking devices in place.
		Broken or missing hinges such that gate cannot be easily opened and closed by a maintenance person.	Hinges intact and lubed. Gate is working freely.
		Gate is out of plumb more than 6 inches and more than 1 foot out of design alignment.	Gate is aligned and vertical.
		Missing stretcher bar, stretcher bands, and ties.	Stretcher bar, bands, and ties in place.
	Openings in Fabric	See "Fencing" Table No. 8	See "Fencing" Table No. 8

SECTION I: BOND QUANTITIES WORKSHEET

The driveway is the only item that requires a bond. It is in State right-of-way. Bonding will fall under authority of the State.