GEOTECH
CONSULTANTS, INC.

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October 17, 2018

JN 18485

Cihan Anisoglu
P.O. Box 10386
Bainbridge Island, Washington 98110
via email: cihan@anisoglu.com

Subject: **Critical Area Report and Update of Previous Geotechnical Engineering Study**
Proposed New Multi-Family Building
230 Madison Avenue South
Bainbridge Island, Washington

Reference: *Geotechnical Engineering Study, Proposed Windward Inn Hotel, 230 Madison Avenue South, Winslow, Washington*; December 4, 2002; Geotech Consultants, Inc.

Dear Mr. Anisoglu:

This Critical Area Report (CAR) is intended to satisfy the report requirements of section 16.20.180 of the Bainbridge Island Municipal Code (BIMC). The proposed site contains geologically hazardous areas, which must be addressed in a CAR. In order to prepare this report, we:

1. Reviewed the above-referenced report,
2. Revisited the subject property on October 8, 2018 to observe the current conditions on the subject lot and the adjoining properties,
3. Reviewed the development plans provided,
4. Discussed the planned development with you,
5. Researched the U.S. Geologic Survey's (USGS) website for the current seismic design parameters required by the International Building Code (IBC), and
6. Completed stability analyses for both static and seismic conditions on the steep, eastern slope.

CRITICAL AREA REPORT

Site and Project Plans:

A Vicinity Map for the site location is attached to the end of this report.

We were provided with architectural plans prepared by Anisoglu Architecture Art and Ideas dated February 8, 2018 and indicated to be for "Site Plan Review". Based on this information, and our discussions with you, we expect that a multi-family residential building will be constructed on the western two-thirds of the property. The new structure will extend no further east than the existing sanitary sewer, which runs along the south side of the lot from Madison Avenue South, before turning northward approximately two-thirds of the way into the site. This new building will have one below-grade level that will daylight to the east. It will have a finish floor elevation of approximately 28 feet. The majority of this below-grade level will contain parking, which will be accessed via a sloped driveway extending along the south side of the building from Madison Avenue South. Two residential units and covered outdoor space will occupy the eastern approximately one-fourth of this lower level. Two floors of residential units will overlie the basement level.

Temporary cuts of up to approximately 10 feet will be needed to reach the basement foundation level. The tallest cut will be located in the northwest corner of the building.

The structure is shown to be set back at least 25 feet from the crest of the steep slope that is located on the eastern portion of the property.

A waterfront trail is shown extending north to south through the eastern side of the development area, along the crest of the steep slope. This trail is indicated to continue off the site both to the east and to the south, along the top of the marine bluff. We expect that this trail will only be used for foot traffic.

A copy of the Site Plan contained within the provided set of drawings is attached to the end of this report as Plate 2. This Site Plan shows the property boundaries, the proposed footprints of the building and waterfront trail, the existing sanitary sewer, and the existing topography. We have also included on the Site Plan the approximate locations of the test pits conducted for our 2002 *Geotechnical Engineering Study* of the property. A copy of this previous study is attached as Appendix A.

The site lies within the Shoreline Management Act Jurisdiction. The conditions on the site are substantially unchanged since we completed our 2002 *Geotechnical Engineering Study*. There are no indications of grading since our 2002 work. The majority of the property slopes gently toward the east from Madison Avenue East. This portion of the property is covered with tall grass and weeds, with scattered trees. Along the east edge of site is a steep slope extending down to a small tidal inlet. This slope is approximately 15 feet in height, and is inclined at approximately 1:1 (Horizontal:Vertical). It is overgrown with blackberry vines and other underbrush, and there are a few medium-sized trees growing on the slope. There are no indications of recent movement on this steep slope. Its oversteepened condition within the boundaries of the site appears mostly the result of previous erosion from past uncontrolled discharge from a large storm drain outfall located in the northeastern portion of the lot. The base of the steep slope has been protected with rock armoring and does not appear to be subjected to wave attack. The *Coastal Zone Atlas of Washington* maps this waterfront area as stable.

Under BIMC 16.12.060, the steep slope on the eastern edge of the property meets the criteria for a landslide hazard area and an erosion hazard area. This is primarily due to the steep inclination of the waterfront slope, which extends down to a small tidal inlet. On the Critical Areas Plan, Plate 3, we have indicated the geologically hazardous areas (landslide hazard and erosion hazard areas as defined by BIMC 16.12.060). Based on our interpretation of the BIMC, the prescriptive buffer from a landslide hazard area is 50 feet or the height of the slope, whichever is greater. This 50-foot buffer zone is also indicated on the Critical Areas Plan.

Assessment of Geologic Characteristics:

As a part of our 2002 *Geotechnical Engineering Study* our firm completed five test pits spread over the site. Test Pits 3 and 4 found fill soils immediately below the existing ground surface. The native soil conditions found beneath the fill, and underneath the ground surface in the remaining test pits, consists of a layer of topsoil overlying gravelly, silty, fine-grained sand that is weathered, and loose to medium-dense to a depth of 2 to 3 feet below the original ground surface. Beneath this looser soil, the gravelly, silty sand is dense to very dense. It has been glacially-compressed, and is referred to as glacial till. The glacial till was difficult to excavate, and extended to the maximum 11-foot depth of the test pits.

Our research of the Washington Department of Natural Resources' *Geologic Information Portal* yielded logs of test holes conducted in 2002 at 305 Madison Avenue South, to the southwest of the subject site. These test holes also found several feet of loose, weathered soil overlying glacial till.

The glacial till soils have a high internal strength, due to their cemented, glacially-compressed condition. It is not uncommon for near-vertical banks of dense glacial till to stand stable for many years.

In addition to the fill soils exposed in the test pits, fill will likely be encountered in the areas previously disturbed by excavation and backfilling for the sanitary sewer that extends through the south and east sides of the site.

No groundwater seepage was encountered in the test pits, which were excavated during the summer months. Glacial till is essentially impervious to the downward percolation of water that infiltrates into the upper, looser soils. As a result, it is not uncommon to encounter at least localized zones of shallow groundwater perched on top of the glacial till soils following extended wet weather. Groundwater may also be trapped in the bottom of the trenches for the sanitary sewer. We noted that recent drainage improvements had been put in place on the outside of the north wall of the adjacent southern building. This may be the result of shallow seasonal perched water building up against the outside of that building, which is slightly lower in elevation than the subject site.

As discussed above, there have been no indications of recent instability on the eastern steep slope. The glacial till soils are not susceptible to deep-seated instability, or soil strength loss, even during a large seismic event. Even so, shallow instability in the form of skin slides can occur on steep slopes where the upper few feet of soils have weathered over time, due primarily to freeze/thaw cycles. This usually occurs only periodically, and typically affects only the uppermost one to 2 feet of soil.

Geologic Hazards Considerations:

Erosion Hazard Area: The majority of the site, the only portion of the property that will be disturbed for the planned development, slopes only gently. The steep, eastern portion of the lot is to remain undisturbed. The erosion potential of the on-site soils on the gently-sloped portion of the property is not severe. Implementation of appropriate temporary and permanent erosion control measures will be sufficient to protect the surrounding properties for adverse erosion impacts.

We expect that a Temporary Erosion and Sedimentation Control (TESC) plan will be required as a part of the permit process for this development. The extent of the temporary erosion control measures that will be appropriate will depend largely on the weather conditions during the clearing, excavation, and site grading operations. As a minimum, we recommend erosion control measures to include:

- Limiting vegetation clearing to areas that will be immediately worked, or that will be protected with straw, mulch, hog fuel, plastic sheeting, or some other measure.
- Installing wire-backed silt fences along the north, east and south boundaries of the work area. These fences should be bedded into mulch or compost.
- Constructing rock-covered access and staging areas to prevent vehicles that will enter and leave the site from driving onto bare soil.

- Covering soil stockpiles with plastic sheeting in wet and dry weather to control both erosion and dust.
- Mulching or covering all areas of bare soil in wet conditions. This is particularly important outside of the excavation where the ground surface slopes toward the neighboring properties or the steep slope.
- Preventing silty runoff from leaving the site and excavation. This may require that a temporary holding tank be kept at the site in wet weather until all bare areas are covered. Protecting the base of the excavation with a layer of clean rock is prudent to reduce the potential for generating silty water once the excavation is completed.

On most construction projects, it is necessary for the on-site contractor to periodically maintain or modify temporary erosion control measures to address specific site and weather conditions.

Landslide Hazard Area: The site is underlain at a relatively shallow depth by glacial till, a glacially-compressed mixture of gravel, silt and fine-grained sand. Experienced geotechnical engineers know that this soil has a high internal strength, and is not prone to deep-seated landslides. As a part of our work for this CAR, we completed a slope stability analysis for both static and seismic conditions. The results of these analyses are attached as Appendix B. As expected, the safety factors against slope movement extending into the glacial till soils exceeds 1.5 and 1.1 for static and seismic conditions.

The near-surface, looser soils (fill and native) are prone to future movement, most likely following extended wet weather. These shallow skin slides would not pose a risk to the planned building, which will be founded on competent glacial till soils. Considering the observed conditions, we expect that the crest of the slope will recede in periodic episodes involving the near-surface one to 2 feet of looser, weathered soil. This skin slides typically occur on approximate 20- to 30-year intervals, depending largely on weather patterns.

We recommend a building setback of at least 25 feet from the crest of the steep slope. This is the minimum setback allowed by the BIMC, consisting of a 15-foot setback from a minimum 10-foot buffer.

The proposed development plan includes the potential for a footpath to extend along the top of the steep slope and onto the adjacent properties. This would be within the minimum 10-foot buffer allowed by the BIMC from landslide hazard areas. It is our profession opinion that if this path is constructed using a lightweight surface, such as wood chips or gravel, and no more than 4 to 6 inches of this material is placed, the path should not adversely impact the stability of the steep slope. This assumes that the vegetation on the steep slope itself will be maintained. It is important to note, however, that future foreseeable shallow slope movement may damage or undermine this surface footpath.

Considering the above discussions, it is our professional opinion that:

- The proposed development will not create a net increase in geological instability, either on or off site,
- The proposed development will not increase the risk of life safety due to geological hazards above professionally acceptable levels,
- The proposed development will not increase the risk due to geological hazards above professionally acceptable levels for property loss to habitable structures or their necessary infrastructure on the site, or for property loss to off-site structures,

- The proposed building will be constructed using appropriate engineering methods that respond to the geologic characteristics specific to the site,
- The proposed development will not further degrade the geologic functions of the associated critical areas.

UPDATE TO GEOTECHNICAL ENGINEERING STUDY

The geotechnical findings and conclusions presented in our 2002 report are generally still applicable to the planned development. The new building can be supported on conventional foundations bearing on the dense glacial till soils. An allowable bearing capacity of 5,000 pounds per square foot (psf) can be assumed, with a one-third increase for short-term wind or seismic loads. It will be important that all foundation bearing surfaces be cleaned of loose or disturbed soils prior to pouring concrete. In wet conditions, it would be prudent to protect the bearing surfaces from disturbance by placing several inches of clean crushed rock.

The design considerations presented in the report for foundations, walls, slabs, subsurface drainage and temporary excavations are still appropriate. The following considerations supplement those presented in our 2002 report:

- The provided plans show that the south and east sides of the building will be close to the easement for a sanitary sewer that was installed previously. It will be critical for the new foundations to be located below a 1:1 (Horizontal:Vertical) zone sloping upward from the bottom of the trench that was excavated for the sanitary sewer. Depending on the depth of the sewer, which should be determined, this could require deepening of the nearby new building foundations.

- In accordance with the 2012/2015 International Building Code (IBC), the site class within 100 feet of the ground surface is best represented by Site Class Type C (Very Dense Soil and Soft Rock). As noted in the USGS website, the mapped spectral acceleration value for a 0.2 second (S_s) and 1.0 second period (S_1) equals 1.45g and 0.47g, respectively.

  The IBC requires that the potential for liquefaction (soil strength loss) during an earthquake be evaluated for the peak ground acceleration of the Maximum Considered Earthquake (MCE), which has a probability of occurring once in 2,475 years (2 percent probability of occurring in a 50-year period). The soils beneath the site that will support the foundations are not susceptible to seismic liquefaction under the ground motions of the MCE because of their dense nature.

- The on-site glacial till soils are essentially impervious to downward percolation of water. On-site infiltration of runoff from impervious areas is infeasible. Attempting to infiltrate or disperse storm water on the site will increase the shallow subsurface flow to the steep slope, and any downgradient properties.

- There is a potential for subsurface water to perch on the glacial till and bypass perimeter footing drains. In order to prevent a build up of water underneath the basement floor slab, it would be prudent to install at least a 6-inch layer of free-draining gravel combined with perforated drain pipes beneath the slab. This underslab drainage system would then be connected to the remainder of the foundation drainage system.

- The temporary cut slope recommendations in the report are still appropriate. If adequately-sloped temporary cuts cannot be made within the property lines, and a temporary easement cannot be obtained from the adjoining property owners, shoring could be designed. We can assist with this, if it is necessary.

If you have any questions regarding this report, or if we may be of further service, please do not hesitate to contact us.

Respectfully submitted,

GEOTECH CONSULTANTS, INC.

10/17/18

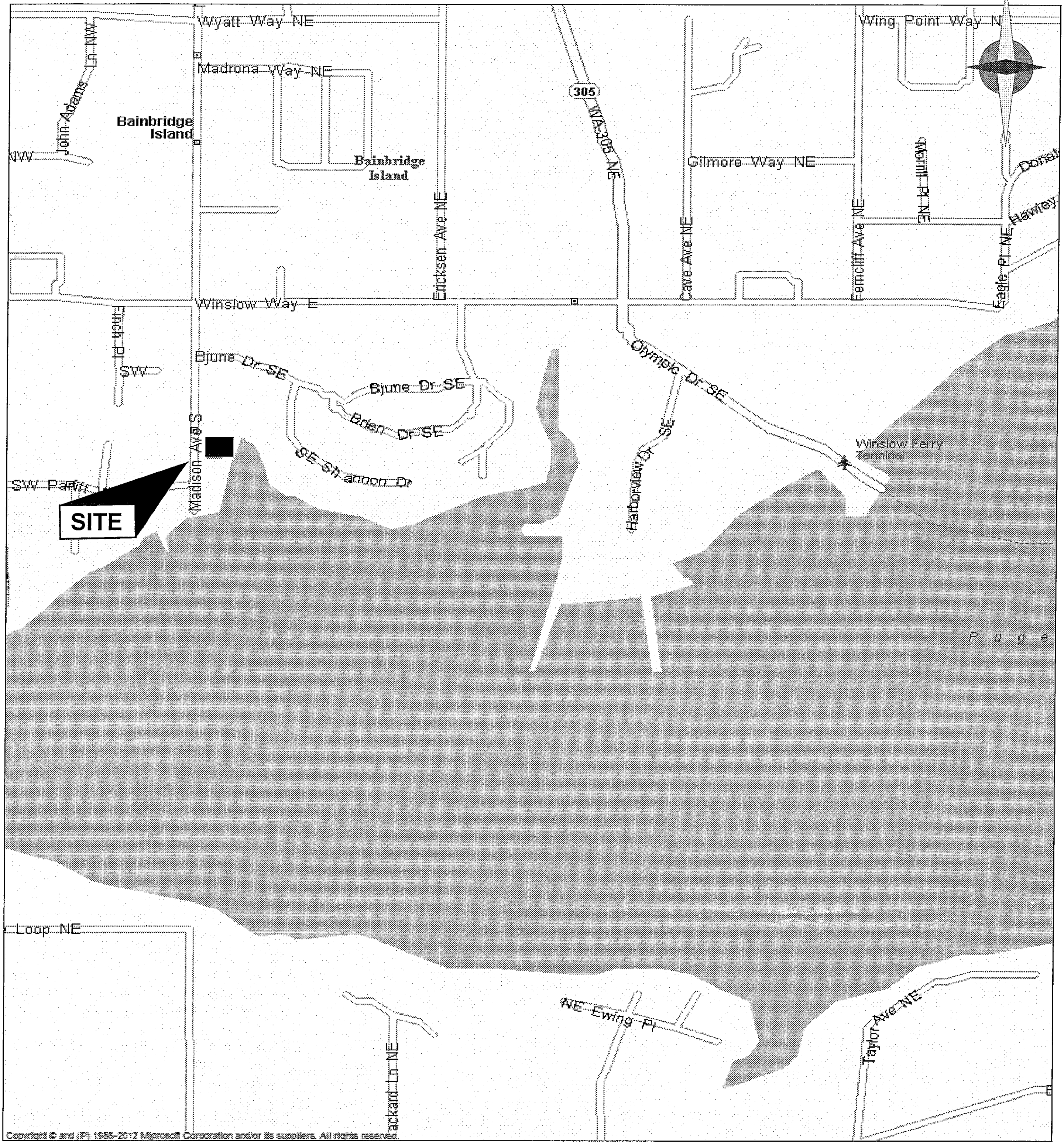
Marc R. McGinnis, P.E.
Principal

Attachments:

- Vicinity Map
- Site Plan
- Critical Areas Plan
- Appendix A – 2002 Geotechnical Engineering Study
- Appendix B – Slope Stability Analyses

MRM: kg

NORTH



(Source: Microsoft MapPoint, 2013)

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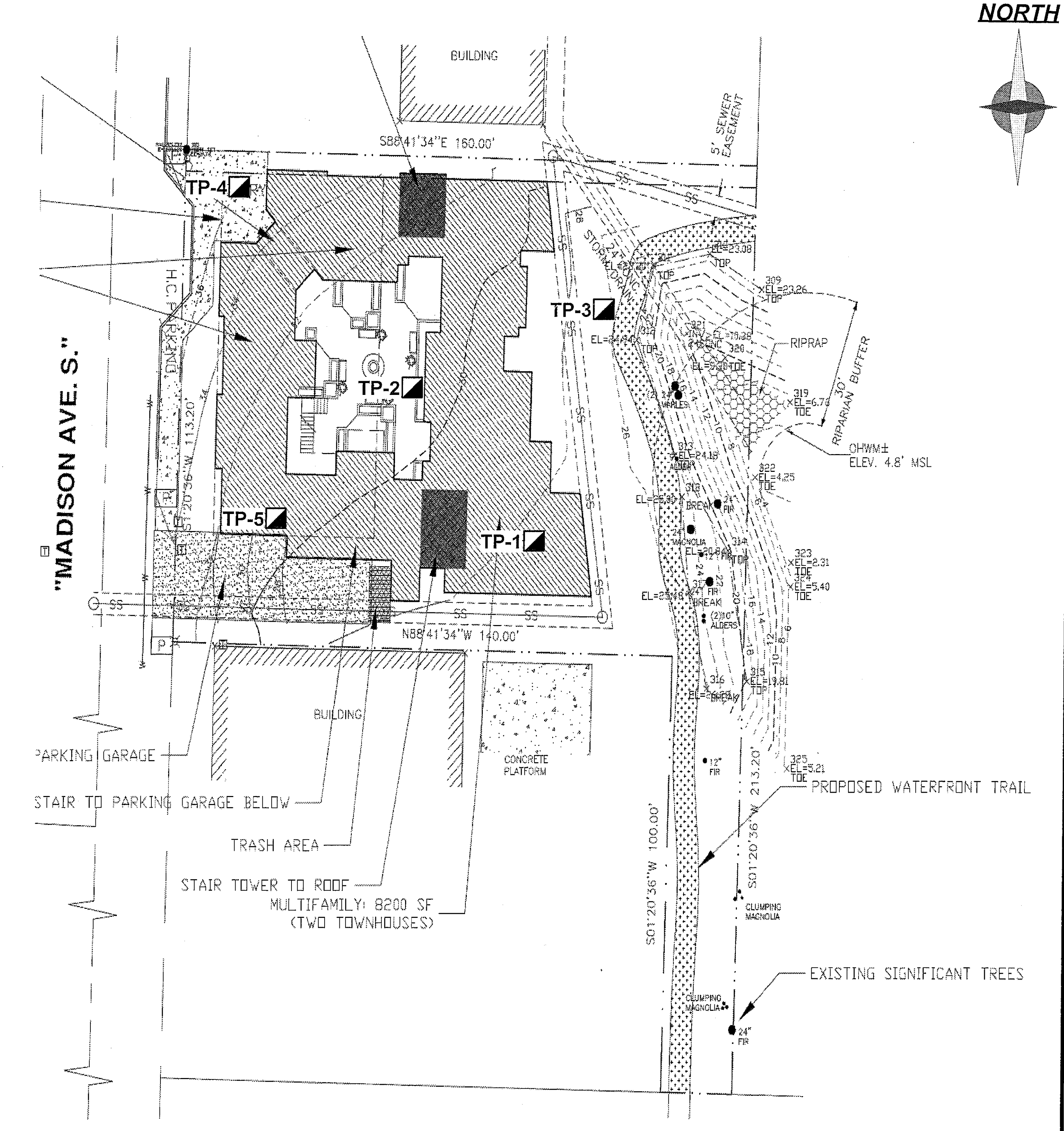
VICINITY MAP

230 Madison Avenue South
Bainbridge Island, Washington

Job No:	Date:		Plate:
18485	Oct. 2018		1



Legend:

Test Pit Location (2002 Geotech Consultants, Inc.)

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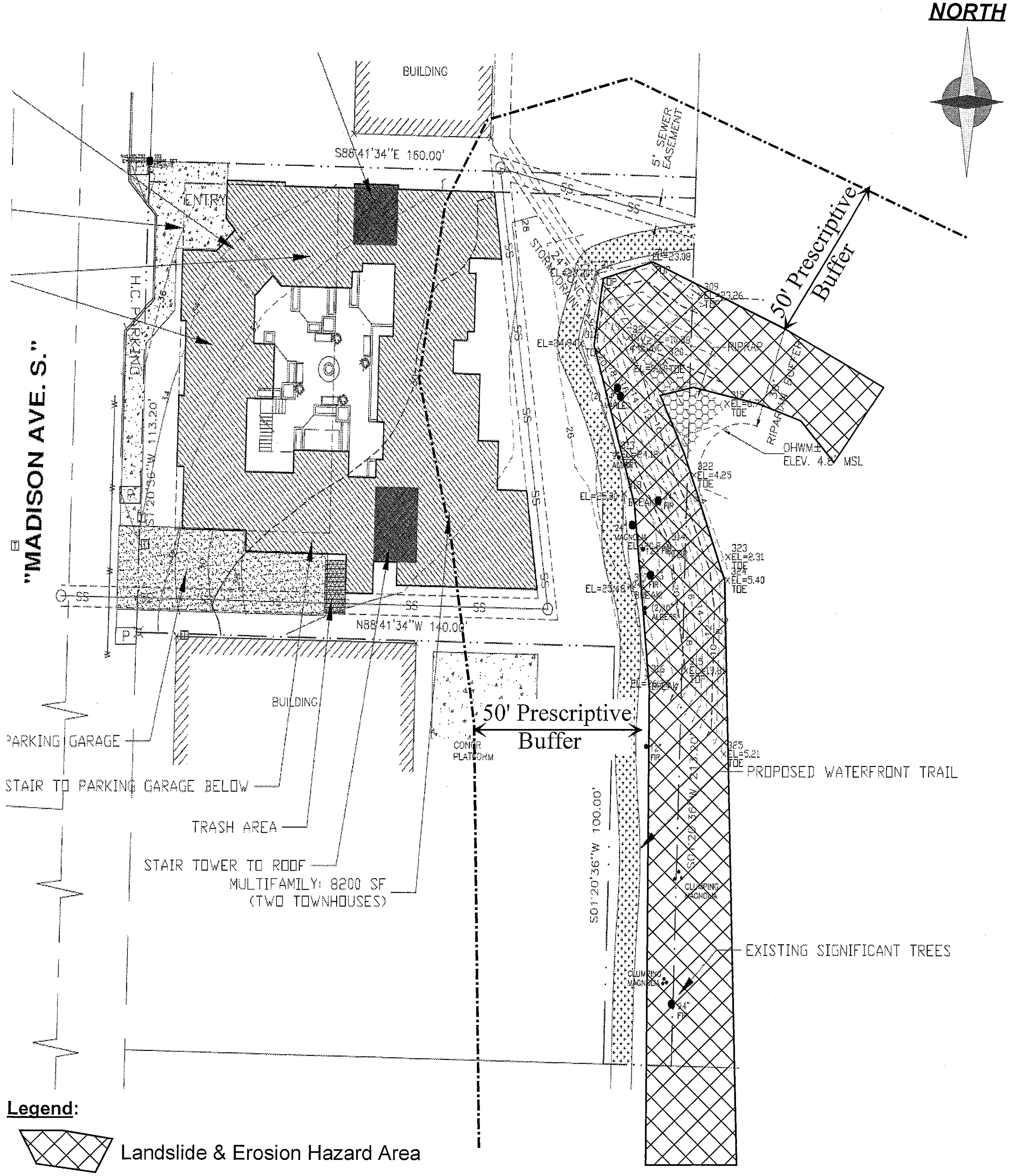
SITE PLAN

230 Madison Avenue South
Bainbridge Island, Washington

Job No:	Date:		Plate:
18485	Oct. 2018	No Scale	2



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CRITICAL AREAS PLAN

230 Madison Avenue South
Bainbridge Island, Washington

Job No:	Date:		Plate:
18485	Oct. 2018		3

Appendix A – 2002 Geotechnical Engineering Study

230 Madison Avenue South

Bainbridge Island, Washington

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13256 Northeast 20th Street, Suite 16
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(425) 747-5618 FAX (425) 747-8561

December 4, 2002

JN 02302

Larsen Architects
P.O. Box 10674
Bainbridge Island, Washington 98110

Attention: Garrett Larsen

Subject: **Geotechnical Engineering Study**
Proposed Windward Inn Hotel
230 Madison Avenue South
Winslow, Washington

Dear Mr. Larsen:

We are pleased to present this geotechnical engineering report for the proposed Windward Inn Hotel development to be constructed at 230 Madison Avenue South in Winslow, Washington. The scope of our services consisted of exploring site surface and subsurface conditions, and then developing this report to provide recommendations for general earthwork and design criteria for foundations, retaining walls, and pavements. This work was authorized by your acceptance of our proposal, P-02302, dated August 5, 2002.

PROJECT UNDERSTANDING

We were provided with a facsimile of a preliminary site plan for the Windward Inn Hotel. Larsen Architects prepared this sketch, which was undated. Based upon our review of the site plan, and discussions with you, it appears that the development will consist of an "L"-shaped building with 29 rooms in approximately two floors, above parking. We further understand that the building would be at grade along its western margin and a parking area beneath the building would daylight to the east. Access to the site would be off of Madison Avenue South, which bounds the proposed development on the west. Detailed information regarding final floor elevations and site grading was not available.

If the scope of the project changes from what we have described above, we should be provided with revised plans in order to determine if modifications to the recommendations and conclusions of this report are warranted.

SITE CONDITIONS

SURFACE

The Vicinity Map, Plate 1, illustrates the general location of the site. The subject property lies east of Madison Avenue South and west of an embayment of Eagle Harbor in the Winslow area of Bainbridge Island. The irregularly shaped property is comprised of a single tax parcel that totals approximately 0.46 acres. The property is currently undeveloped and is vegetated with scattered conifers and deciduous trees, and dense thickets of brambles. Beginning at the western property boundary, an approximate 20-foot-wide sanitary sewer easement parallels the southern property boundary. The easement turns toward the north-northwest approximately three-fourths the way into the site. It continues to the north-northwest across the property and exits the site approximately two-thirds the way from the western property line along the northern property line, then turns to the southeast. A manhole cover is visible near the southeastern corner of the adjoining northern building.

We were not provided with a topographic survey and do not have more information about ground surface elevations. The surface of the property gently descends from Madison Avenue South on the west, until reaching the eastern margin of the site, where the ground descends steeply to an embayment of Eagle Harbor.

We observed no indications of recent instability on the steep slope during our site visit. The slope was overgrown and its toe appears to be protected by rock armoring. Development adjacent to the site includes a real estate office on the north and a naval architects office on the south. Multi-family buildings are east of the embayment.

SUBSURFACE

The subsurface conditions were explored by excavating five test pits at the approximate locations shown on the Site Exploration Plan, Plate 2. Our exploration program was based on the proposed construction, anticipated subsurface conditions and those encountered during exploration, and the scope of services outlined in our proposal.

The test pits were excavated on August 8, 2002 with a trackhoe. A geologist from our staff observed the excavation process, logged the test pits, and obtained representative samples of the soil encountered. "Grab" samples of selected subsurface soil were collected from the trackhoe bucket. The Test Pit Logs are attached to this report as Plates 3 through 5.

Soil Conditions

With the exception of approximately 5.5 feet of fill in Test Pit 3, the test pits excavated across the site generally encountered similar conditions. We typically observed a very thin layer of topsoil and highly organic material, overlying medium-dense silty sand with gravel that became dense between 2 and 4 feet in depth and very dense within 5 feet. The fill encountered in Test Pit 3 was medium-dense with roots. A dark brown, one-inch-thick organic layer separated the fill from underlying native silty sand.

The dense to very dense silty sand with gravel encountered in our explorations has been glacially compressed and is commonly referred to as glacial till. The glacial till was encountered in all of the test pits to the maximum explored depth of 11 feet. The till soil contained occasional sandier zones.

We did encounter cobbles in one of the test pits. We anticipate that more cobbles and possibly occasional boulders may be encountered during site redevelopment activities.

Groundwater Conditions

No groundwater seepage was observed in any of the five test pits excavated. The test pits were excavated in the summer and were left open for only a short time period. Therefore, the absence of seepage levels on the logs does not preclude the presence of groundwater in future excavations. It should be noted that groundwater levels can vary seasonally with rainfall and other factors. We anticipate that perched groundwater could be found above and within the glacial till soils in the winter and spring months, especially in excavations near the wetlands.

The final logs represent our interpretations of the field logs and laboratory tests. The stratification lines on the logs represent the approximate boundaries between soil types at the exploration locations. The actual transition between soil types may be gradual, and subsurface conditions can vary between exploration locations. The logs provide specific subsurface information only at the locations tested. The relative densities and moisture descriptions indicated on the test pit logs are interpretive descriptions based on the conditions observed during excavation.

The compaction of backfill was not in the scope of our services. Loose soil will therefore be found in the area of the test pits. If this presents a problem, the backfill will need to be removed and replaced with structural fill during construction.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

THIS SECTION CONTAINS A SUMMARY OF OUR STUDY AND FINDINGS FOR THE PURPOSES OF A GENERAL OVERVIEW ONLY. MORE SPECIFIC RECOMMENDATIONS AND CONCLUSIONS ARE CONTAINED IN THE REMAINDER OF THIS REPORT. ANY PARTY RELYING ON THIS REPORT SHOULD READ THE ENTIRE DOCUMENT.

With the exception of encountering approximately 5.5 feet of fill in Test Pit 3, the test pits excavated for this study generally encountered thin topsoil and organic material overlying medium-dense silty sand with gravel or gravelly sand that became dense between 2 and 4 feet in depth. The silty sand with gravel encountered in our explorations has been glacially compressed and is commonly referred to as glacial till. It is our opinion that the proposed Windward Inn can be supported on conventional foundations bearing directly on the dense native soils.

The recommendations of this report are intended to protect the planned development from damage due to slope instability, and to prevent the planned work from reducing the stability of the steep slope. Based on the lack of evident recent slope instability, and the presence of dense, glacially compressed silty sands, the potential for deep instability appears negligible. It will be important that protection of the slope's toe be maintained to prevent undercutting by wave action. It is possible that future shallow instability affecting the fill and looser weathered soils on, or near, the steep slope could occur. We therefore recommend the following:

- Locate structures no closer than 25 feet to the crest of the steep slope.

- Maintain a minimum 15-foot separation between the on-grade parking and the steep slope.

- Place no fill or clearing debris, or disturb the existing vegetation on, or within 10 feet of, the steep slope.

- Allow no water from impervious surfaces to flow onto, or be discharged on, the steep slope. Collected water should either be piped to a storm sewer or the base of the steep slope.

If future slope movement occurs, it could be necessary to stabilize the affected area with a retaining wall or buttress, or simply revegetate the resulting bare area.

Due to the silty, moisture-sensitive nature of the majority of the site soils, earthwork will be easier and more economical if performed during the drier summer months. The fine-grained silty site soils are sensitive to moisture, which makes them difficult to impossible to adequately compact when they have moisture contents even 2 to 3 percent above their optimum moisture content. The on-site soils are not acceptable for reuse as fill beneath footings. The reuse of these soils as structural fill beneath slab or pavement areas will only be successful during hot, dry weather. When above optimum moisture content, aeration of each loose lift of soil will be required to dry it before the lift is compacted. Alternatively, the soil could be chemically dried by adding lime, kiln dust, or cement, provided this is allowed by the responsible building department. Regardless of the method of drying, the earthwork process will be slowed dramatically. The earthwork contractor must be prepared to rework areas that do not achieve proper compaction due to high moisture content. Utility trench backfill in structural areas, such as pavements, must also be dried before it can be adequately compacted. Improper compaction of backfill in utility trenches and around control structures is a common reason for pavement distress and failures. Imported granular fill will be needed wherever it is not possible to dry the on-site soils sufficiently before compaction.

The drainage and/or waterproofing recommendations presented in this report are intended only to prevent active seepage from flowing through concrete walls or slabs. Even in the absence of active seepage into and beneath structures, water vapor can migrate through walls, slabs, and floors from the surrounding soil, and can even be transmitted from slabs and foundation walls due to the concrete curing process. Excessive water vapor trapped within structures can result in a variety of undesirable conditions, including, but not limited to, moisture problems with flooring systems, excessively moist air within occupied areas, and the growth of molds, fungi, and other biological organisms that may be harmful to the health of the occupants. The architect must consider the potential vapor sources and likely occupant uses, and provide sufficient ventilation, either passive or mechanical, to prevent a build up of excessive water vapor within the planned structure.

The erosion control measures needed during the site development will depend heavily on the weather conditions that are encountered. We anticipate that a stout wire-backed silt fence will be needed around the downslope sides of any cleared areas. Rocked construction access roads should be extended into the site to reduce the amount of mud carried off the property by trucks and equipment. Wherever possible, these roads should follow the alignment of planned pavements. Cut slopes and soil stockpiles should be covered with plastic during wet weather. Following rough grading, it may be necessary to mulch or hydroseed bare areas that will not be immediately covered with landscaping or an impervious surface.

Geotech Consultants, Inc. should be allowed to review the final development plans to verify that the recommendations presented in this report are adequately addressed in the design. Such a plan review would be additional work beyond the current scope of work for this study, and it may include revisions to our recommendations to accommodate site, development, and geotechnical constraints that become more evident during the review process.

SEISMIC CONSIDERATIONS

The site is located within Seismic Zone 3, as illustrated on Figure No. 16-2 of the 1997 Uniform Building Code (UBC). In accordance with Table 16-J of the 1997 UBC, the site soil profile within 100 feet of the ground surface is best represented by Soil Profile Type S_C (Very Dense Soil). The site soils are not susceptible to seismic liquefaction because of their dense nature.

CONVENTIONAL FOUNDATIONS

The proposed structure can be supported on conventional continuous and spread footings bearing directly on the dense native soil. Structural fill should not be placed beneath the building's foundations. We recommend that continuous and individual spread footings have minimum widths of 16 and 24 inches, respectively. Footings should also be bottomed at least 18 inches below the lowest adjacent finish ground surface. The local building codes should be reviewed to determine if different footing widths or embedment depths are required.

Depending on the final site grades, overexcavation may be required below the footings to expose competent native soil. Unless lean concrete (minimum 1.5 sacks of cement per cubic yard) is used to fill an overexcavated hole, the foundation should be extended downward. If lean concrete backfill is used, the overexcavation need only extend 6 inches beyond the edges of the footing.

An allowable bearing pressure of 5,000 pounds per square foot (psf) is appropriate for footings supported on dense native soil. A one-third increase in this design bearing pressure may be used when considering short-term wind or seismic loads. For the above design criteria, it is anticipated that the total post-construction settlement of footings founded on competent native soil will be less than one inch, with differential settlements on the order of one-half inch in a distance of 50 feet along a continuous footing with a uniform load.

Lateral loads due to wind or seismic forces may be resisted by friction between the foundation and the bearing soil, or by passive earth pressure acting on the vertical, embedded portions of the foundation. For the latter condition, the foundation must be either poured directly against relatively level, undisturbed soil or be surrounded by level structural fill. We recommend using the following ultimate values for the foundation's resistance to lateral loading:

PARAMETER	ULTIMATE VALUE
Coefficient of Friction	0.50
Passive Earth Pressure	350 pcf

Where: (i) pcf is pounds per cubic foot, and (ii) passive earth pressure is computed using the equivalent fluid density.

If the ground in front of a foundation is loose or sloping, the passive earth pressure given above will not be appropriate. We recommend maintaining a safety factor of at least 1.5 for the foundation's resistance to lateral loading, when using the above ultimate values.

PERMANENT FOUNDATION AND RETAINING WALLS

Retaining walls backfilled on only one side should be designed to resist the lateral earth pressures imposed by the soil they retain. The following recommended parameters are for walls that restrain level backfill:

PARAMETER	VALUE
Active Earth Pressure *	35 pcf
Passive Earth Pressure	350 pcf
Coefficient of Friction	0.45
Soil Unit Weight	130 pcf

Where: (i) pcf is pounds per cubic foot, and (ii) active and passive earth pressures are computed using the equivalent fluid pressures.

*** For a restrained wall that cannot deflect at least 0.002 times its height, a uniform lateral pressure equal to 10 psf times the height of the wall should be added to the above active equivalent fluid pressure.**

The values given above are to be used to design permanent foundation and retaining walls only. The passive pressure given is appropriate for the depth of level structural fill placed in front of a retaining or foundation wall only. The values for friction and passive resistance are ultimate values and do not include a safety factor. We recommend a safety factor of at least 1.5 for overturning and sliding, when using the above values to design the walls. Restrained wall soil parameters should be utilized for a distance of 1.5 times the wall height from corners or bends in the walls. This is intended to reduce the amount of cracking that can occur where a wall is restrained by a corner.

The design values given above do not include the effects of any hydrostatic pressures behind the walls and assume that no surcharges, such as those caused by slopes, vehicles, or adjacent foundations will be exerted on the walls. If these conditions exist, those pressures should be added to the above lateral soil pressures. Where sloping backfill is desired behind the walls, we will need to be given the wall dimensions and the slope of the backfill in order to provide the appropriate design earth pressures. The surcharge due to traffic loads behind a wall can typically be accounted for by adding a uniform pressure equal to 2 feet multiplied by the above active fluid density.

Heavy construction equipment should not be operated behind retaining and foundation walls within a distance equal to the height of a wall, unless the walls are designed for the additional lateral pressures resulting from the equipment. The wall design criteria assume that the backfill will be well-compacted in lifts no thicker than 12 inches. The compaction of backfill near the walls should be accomplished with hand-operated equipment to prevent the walls from being overloaded by the higher soil forces that occur during compaction.

Retaining Wall Backfill and Waterproofing

Backfill placed behind retaining or foundation walls should be coarse, free-draining structural fill containing no organics. This backfill should contain no more than 5 percent silt or clay particles and have no gravel greater than 4 inches in diameter. The percentage of particles passing the No. 4 sieve should be between 25 and 70 percent. The native soils are not free draining. If the native glacial till soil is used as backfill, a minimum of 12 inches of free-draining gravel should be placed against the backfilled retaining walls. Free-draining backfill or gravel should be used for the entire width of the backfill where seepage is encountered. For increased protection, drainage composites should be placed along cut slope faces, and the walls should be backfilled entirely with free-draining soil.

The purpose of these backfill requirements is to ensure that the design criteria for a retaining wall are not exceeded because of a build-up of hydrostatic pressure behind the wall. The top 12 to 18 inches of the backfill should consist of a compacted, relatively impermeable soil or topsoil, or the surface should be paved. The ground surface must also slope away from backfilled walls to reduce the potential for surface water to percolate into the backfill. The section entitled ***General Earthwork and Structural Fill*** contains recommendations regarding the placement and compaction of structural fill behind retaining and foundation walls.

The above recommendations are not intended to waterproof below-grade walls. Over time, the performance of subsurface drainage systems can degrade, subsurface groundwater flow patterns can change, and utilities can break or develop leaks. Therefore, waterproofing should be provided where future seepage through the walls is not acceptable. This typically includes limiting cold-joints and wall penetrations, and using bentonite panels or membranes on the outside of the walls. Waterproofing systems should be installed by an experienced contractor familiar with the anticipated construction and subsurface conditions. Applying a thin coat of asphalt emulsion to the outside face of a wall is not considered waterproofing, and will only help to reduce moisture generated from water vapor or capillary action from seeping through the concrete. As with any project, adequate ventilation of basement and crawl space areas is important to prevent a build up of water vapor that is commonly transmitted through concrete walls from the surrounding soil, even when seepage is not present. This is appropriate even when waterproofing is applied to the outside of foundation and retaining walls.

SLABS-ON-GRADE

The building floors may be constructed as slabs-on-grade atop non-organic, medium-dense native soils, or on structural fill placed above this competent soil. The subgrade soil must be in a firm non-yielding condition at the time of slab construction or underslab fill placement. Any soft areas encountered should be excavated and replaced with select imported structural fill.

All slabs-on-grade should be underlain by a capillary break or drainage layer consisting of a minimum 4-inch thickness of coarse, free-draining structural fill with a gradation similar to that discussed in ***Permanent Foundation and Retaining Walls***. As noted by the American Concrete Institute (ACI) in the *Guides for Concrete Floor and Slab Structures*, proper moisture protection is desirable immediately below any on-grade slab that will be covered by tile, wood, carpet, impermeable floor coverings, or any moisture-sensitive equipment or products. ACI also notes that vapor *retarders*, such as 6-mil plastic sheeting, are typically used. A vapor retarder is defined as a material with a permeance of less than 0.3 US perms per square foot (psf) per hour, as determined by ASTM E 96. It is possible that concrete admixtures may meet this specification, although the manufacturers of the admixtures should be consulted. Where plastic sheeting is used under slabs, joints should overlap by at least 6 inches and be sealed with adhesive tape. The sheeting should extend to the foundation walls for maximum vapor protection.

If no potential for vapor passage through the slab is desired, a vapor *barrier* should be used. A vapor barrier, as defined by ACI, is a product with a water transmission rate of 0.00 perms per square foot per hour when tested in accordance with ASTM E 96. Reinforced membranes having sealed overlaps can meet this requirement.

In the recent past, ACI (Section 4.1.5) recommended that a minimum of 4 inches of well-graded compactable granular material, such as a 5/8-inch-minus crushed rock pavement base, be placed over the vapor retarder or barrier to protect them during slab construction and to act as a "blotter" for more even curing of the concrete slab. However, more current literature indicates that long-term vapor problems could result where the protection/blotter material becomes wet before the slab placement occurs. This is especially an issue in areas with wet climates, such as the Puget Sound. Therefore, if there is a potential that the protection/blotter material will become wet before the slab is installed, ACI now recommends that no protection/blotter material be used. However, they then recommend that the joint spacing in the slab be reduced, a low shrinkage concrete mixture be used, and "other measures" (steel reinforcing, etc.) be utilized to reduce the potential for irregular slab curing and excessive shrinkage cracking due to uneven curing.

We recommend that the contractor, architect, structural engineer, and the owner discuss these issues and review recent ACI literature and ASTM E-1643 for installation guidelines and guidance on the use of the protection/blotter material.

EXCAVATIONS AND SLOPES

Excavation slopes should not exceed the limits specified in local, state, and national government safety regulations. Temporary cuts to a depth of about 4 feet may be attempted vertically in unsaturated soil, if there are no indications of slope instability. However, vertical cuts should not be made near property boundaries, or existing utilities and structures. Based upon Washington Administrative Code (WAC) 296, Part N, the topsoil and loose to medium-dense soils at the subject site would generally be classified as Type B, while the underlying dense glacial till would be classified as Type A. Temporary cut slopes in the Type B soils should be excavated at an inclination no steeper than 1:1 (Horizontal:Vertical) and the Type A soils no steeper than 0.75:1 (H:V).

The above-recommended temporary slope inclinations are based on what has been successful at other sites with similar soil conditions. Temporary cuts are those that will remain unsupported for a relatively short duration to allow for the construction of foundations, retaining walls, or utilities. Temporary cut slopes should be protected with plastic sheeting during wet weather. The cut slopes should also be backfilled or retained as soon as possible to reduce the potential for instability. Please note that sand or loose soil can cave suddenly and without warning. Excavation, foundation, and utility contractors should be made especially aware of this potential danger.

All permanent cuts into native soil should be inclined no steeper than 2:1 (H:V). Fill slopes should also not be constructed with an inclination greater than 2:1 (H:V). As discussed in the ***General*** section, fill should not be placed on, or near, the steep east slope. To reduce the potential for shallow sloughing, fill must be compacted to the face of these slopes. This can be accomplished by overbuilding the compacted fill and then trimming it back to its final inclination. Adequate compaction of the slope face is important for long-term stability and is necessary to prevent excessive settlement of patios, slabs, foundations, or other improvements that may be placed near the edge of the slope.

Water should not be allowed to flow uncontrolled over the top of any temporary or permanent slope. All permanently exposed slopes should be seeded with an appropriate species of vegetation to reduce erosion and improve the stability of the surficial layer of soil.

DRAINAGE CONSIDERATIONS

Foundation drains should be used where (1) crawl spaces or basements will be below a structure, (2) a slab is below the outside grade, or (3) the outside grade does not slope downward from a building. Drains should also be placed at the base of all earth-retaining walls. These drains should be surrounded by at least 6 inches of 1-inch-minus, washed rock and then wrapped in non-woven, geotextile filter fabric (Mirafi 140N, Supac 4NP, or similar material). At its highest point, a perforated pipe invert should be at least 6 inches below the bottom of a slab floor or the level of a crawl space, and it should be sloped for drainage. All roof and surface water drains must be kept separate from the foundation drain system. A typical drain detail is attached to this report as Plate 6. For the best long-term performance, perforated PVC pipe is recommended for all subsurface drains.

Drainage inside the building's footprint should also be provided where a crawl space will slope or be lower than the surrounding ground surface, or an excavation encounters significant seepage. Considering the potential for perched groundwater, it would be prudent to provide at least a 4- to 6-inch gravel layer and several perforated drainpipes under the slab. We can provide additional recommendations for interior drains, should they become necessary, during excavation and foundation construction.

As a minimum, a vapor retarder, as defined in the ***Slabs-On-Grade*** section, should be provided in any crawl space area to limit the transmission of water vapor from the underlying soils. Also, an outlet drain is recommended for all crawl spaces to prevent a build up of any water that may bypass the footing drains.

No groundwater was observed during our field work. However, if seepage is encountered in an excavation, it should be drained from the site by directing it through drainage ditches, perforated pipe, or French drains, or by pumping it from sumps interconnected by shallow connector trenches at the bottom of the excavation.

The excavation and site should be graded so that surface water is directed off the site and away from the tops of slopes. Water should not be allowed to stand in any area where foundations, slabs, or pavements are to be constructed. Final site grading in areas adjacent to a buildings should slope away at least 2 percent, except where the area is paved. Surface drains should be provided where necessary to prevent ponding of water behind foundation or retaining walls.

GENERAL EARTHWORK AND STRUCTURAL FILL

All building and pavement areas should be stripped of surface vegetation, topsoil, organic soil, and other deleterious material. The stripped or removed materials should not be mixed with any materials to be used as structural fill, but they could be used in non-structural areas, such as landscape beds.

Structural fill is defined as any fill, including utility backfill, placed under, or close to, a building, behind permanent retaining or foundation walls, or in other areas where the underlying soil needs to support loads. All structural fill should be placed in horizontal lifts with a moisture content at, or near, the optimum moisture content. The optimum moisture content is that moisture content that results in the greatest compacted dry density. The moisture content of fill is very important and must be closely controlled during the filling and compaction process.

The allowable thickness of the fill lift will depend on the material type selected, the compaction equipment used, and the number of passes made to compact the lift. The loose lift thickness should not exceed 12 inches. We recommend testing the fill as it is placed. If the fill is not sufficiently compacted, it can be recompacted before another lift is placed. This eliminates the need to remove the fill to achieve the required compaction. The following table presents recommended relative compactions for structural fill:

LOCATION OF FILL PLACEMENT	MINIMUM RELATIVE COMPACTION
Beneath slabs or walkways	95%
Filled slopes and behind retaining walls	90%
Beneath pavements	95% for upper 12 inches of subgrade; 90% below that level

Where: Minimum Relative Compaction is the ratio, expressed in percentages, of the compacted dry density to the maximum dry density, as determined in accordance with ASTM Test Designation D 1557-91 (Modified Proctor).

The ***General*** section should be reviewed for considerations related to the reuse of on-site soils. Structural fill that will be placed in wet weather should consist of a coarse, granular soil with a silt or clay content of no more than 5 percent. The percentage of particles passing the No. 200 sieve should be measured from that portion of soil passing the three-quarter-inch sieve.

LIMITATIONS

The conclusions and recommendations contained in this report are based on site conditions as they existed at the time of our exploration and assume that the soil and groundwater conditions encountered in the test pits are representative of subsurface conditions on the site. If the subsurface conditions encountered during construction are significantly different from those observed in our explorations, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. Unanticipated soil conditions are commonly encountered on construction sites and cannot be fully anticipated by merely taking soil samples in test pits. Subsurface conditions can also vary between exploration locations. Such unexpected conditions frequently require making additional expenditures to attain a properly constructed project. It is recommended that the owner consider providing a contingency fund to accommodate such potential extra costs and risks. This is a standard recommendation for all projects.

This report has been prepared for Larsen Architects and its representatives for specific application to this project and site. Our conclusions and recommendations are professional opinions derived in accordance with current standards of practice within the scope of our services and within budget and time constraints.

No warranty is expressed or implied. The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design.

ADDITIONAL SERVICES

Geotech Consultants, Inc. should be retained to provide geotechnical consultation, testing, and observation services during construction. This is to confirm that subsurface conditions are consistent with those indicated by our exploration, to evaluate whether earthwork and foundation construction activities comply with the general intent of the recommendations presented in this report, and to provide suggestions for design changes in the event subsurface conditions differ from those anticipated prior to the start of construction. However, our work would not include the supervision or direction of the actual work of the contractor and its employees or agents. Also, job and site safety, and dimensional measurements, will be the responsibility of the contractor.

The following plates are attached to complete this report:

Plate 1	Vicinity Map
Plate 2	Site Exploration Plan
Plates 3 - 5	Test Pit Logs
Plate 6	Typical Footing Drain

We appreciate the opportunity to be of service on this project. If you have any questions, or if we may be of further service, please do not hesitate to contact us.

Respectfully submitted,

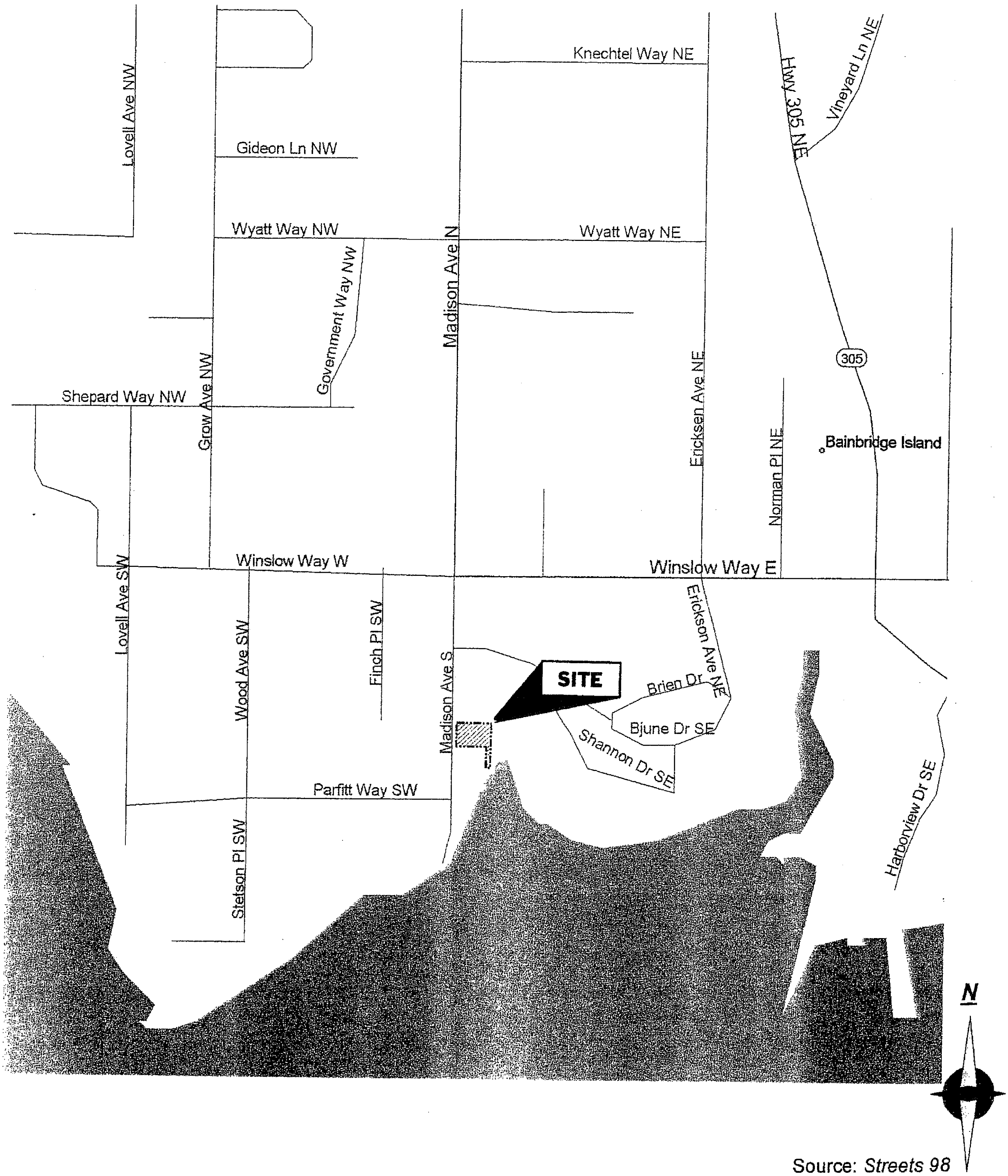
GEOTECH CONSULTANTS, INC.

Timothy A. Johnson
Geologist

MARC R. McGINNIS
STATE OF WASHINGTON
27845
REGISTERED PROFESSIONAL ENGINEER
12/4/02
EXPIRES 10/25/03

Marc R. McGinnis, P.E.
Principal

TAJ/MRM: esm



Source: *Streets 98*

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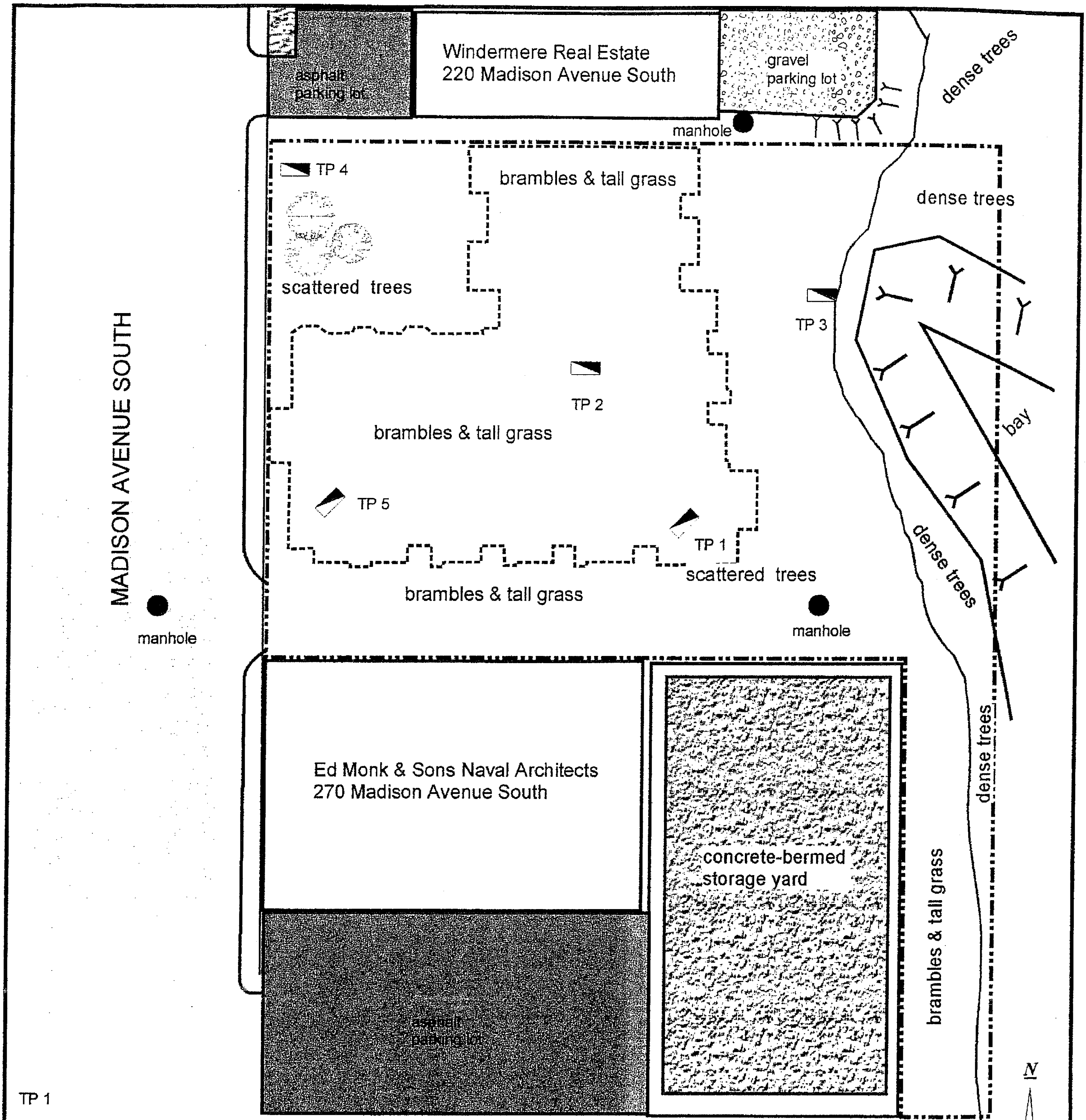
VICINITY MAP

Proposed Windward Inn Hotel
230 Madison Avenue South
Winslow, Washington

Job No:	Date:		Plate:
02302	November 2002		1



Approximate Location of Geotech Consultants Test Pit, August 2002

This Drawing is Based on a preliminary site plan "*The Windward Inn*"
Prepared by Larsen Architects, Not Dated

NOT TO SCALE

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SITE EXPLORATION PLAN

Proposed Windward Inn Hotel
230 Madison Avenue South
Winslow, Washington

Job No:	Date:		Plate:
02302	November 2002		2

TEST PIT 1

Depth (ft.)	Moisture Content (%)	Water Table	USCS	Description	Elevation:
				Topsoil with roots	
			SM	Tan, silty SAND, with gravel & cobbles, fine-grained, dry, medium dense - becomes dense - becomes gray, very dense	
5			SM	- becomes less gravely, moist - very dense, difficult to dig	
10					
15					

* **Test Pit was terminated at 8 feet on August 8, 2002.**
* **No groundwater seepage was observed during excavation.**
* **No caving was observed during excavation.**

TEST PIT 2

Depth (ft.)	Moisture Content (%)	Water Table	USCS	Description	Elevation:
				Topsoil with roots (3")	
			SM	Rusty brown, silty SAND, with gravel, fine-grained, dry, medium dense - becomes grayish tan, dense - becomes gray, very dense	
5					
10					
15					

* **Test Pit was terminated at 5 feet on August 8, 2002.**
* **No groundwater seepage was observed during excavation.**
* **No caving was observed during excavation.**

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TEST PIT LOGS
Proposed Windward Inn Hotel
230 Madison Avenue South
Winslow, Washington

Job No:	Date:	Logged By:	Plate:
02302	November 2002	T.A.J.	3

TEST PIT 3

Depth (ft.)	Moisture Content (%)	Water Table	USCS	Description	Elevation:
				Topsoil with roots (3")	
			FILL	Tan, silty SAND, with gravel, occasional cobbles, fine- to medium-grained, dry, medium dense ***(FILL)*** - roots	
5				Compressed vegetation	
			SM	Rusty brown, silty SAND, with gravel, fine- to medium-grained, moist, dense - some cobbles	
10				- becomes gray with orange mottling, moist, dense to very dense	
15					

* **Test Pit was terminated at 11 feet on August 8, 2002.**
* **No groundwater seepage was observed during excavation.**
* **No caving was observed during excavation.**

TEST PIT 4

Depth (ft.)	Moisture Content (%)	Water Table	USCS	Description	Elevation:
			FILL	Tan, silty SAND, with gravel, fine- to medium grained, dry, medium-dense ***(FILL)***	
				Vegetation layer	
			SM	Rusty brown, silty SAND, with gravel, fine- to medium-grained, dry, medium-dense - becomes grayish tan, moist, dense	
5				- becomes gray, very dense - very dense	
10					
15					

* **Test Pit was terminated at 8.5 feet on August 8, 2002.**
* **No groundwater seepage was observed during excavation.**
* **No caving was observed during excavation.**

GEOTECH CONSULTANTS, INC.

TEST PIT LOGS
Proposed Windward Inn Hotel
230 Madison Avenue South
Winslow, Washington

Job No:	*Date:*	*Logged By:*	*Plate:*
02302	November 2002	T.A.J.	4

TEST PIT 5

Depth (ft.)	Moisture Content (%)	Water Table	USCS	Description Elevation:
				Topsoil with roots (3")
			SM	Tan to rusty tan, silty SAND, fine- to medium-grained, dry, medium-dense - becomes dense
5			GP	Rusty gray, sandy GRAVEL, moist, very dense
			SM	Gray, silty SAND, with gravel, fine-grained, moist, very dense
10				
15				

*** Test Pit was terminated at 8.5 feet on August 8, 2002.**
*** No groundwater seepage was observed during excavation.**
*** No caving was observed during excavation.**

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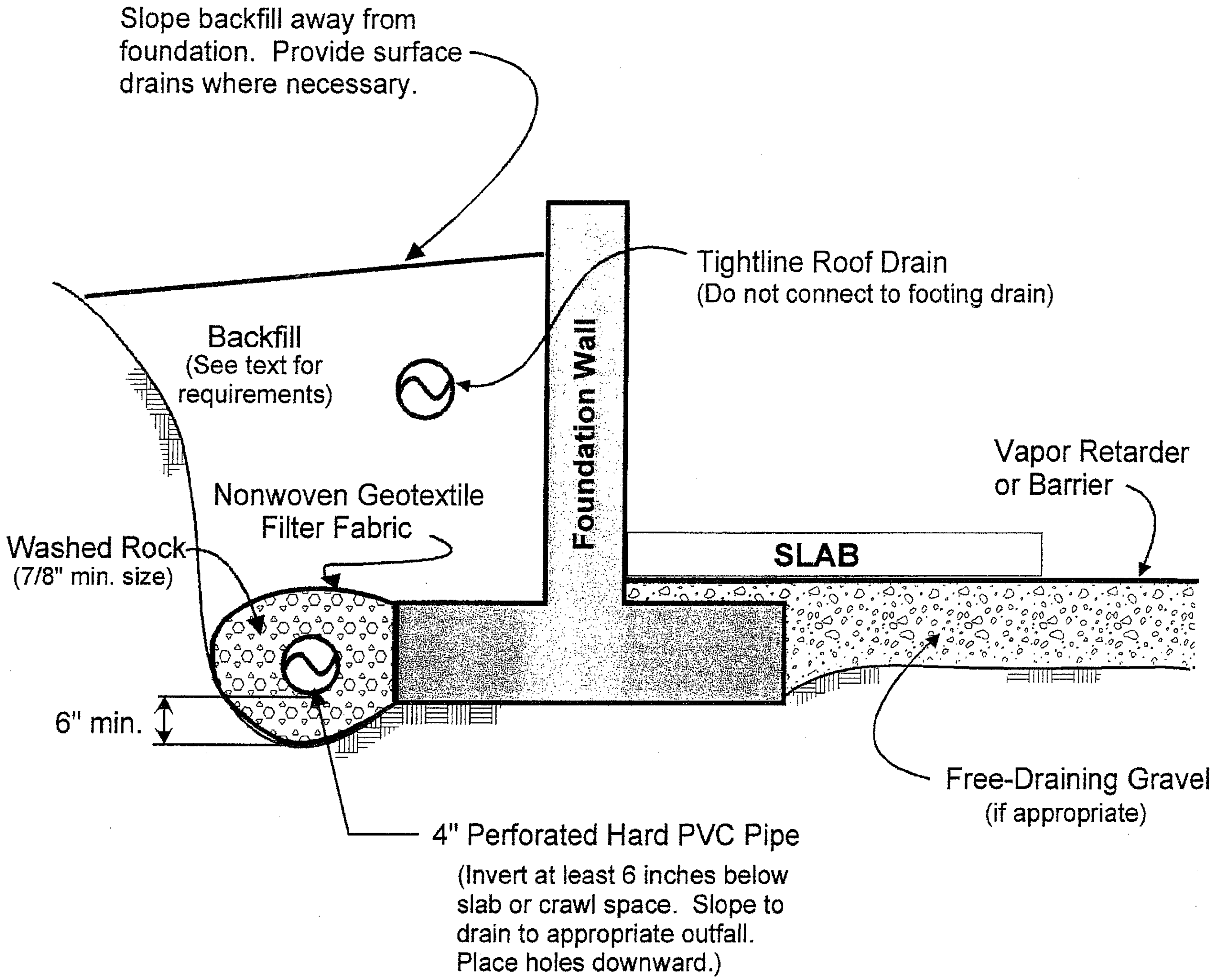
TEST PIT LOGS
Proposed Windward Inn Hotel
230 Madison Avenue South
Winslow, Washington

Job No:	Date:	Logged By:	Plate:
02302	November 2002	T.A.J.	5



NOTES:

(1) In crawl spaces, provide an outlet drain to prevent buildup of water that bypasses the perimeter footing drains.

(2) Refer to report text for additional drainage and waterproofing considerations.

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FOOTING DRAIN DETAIL

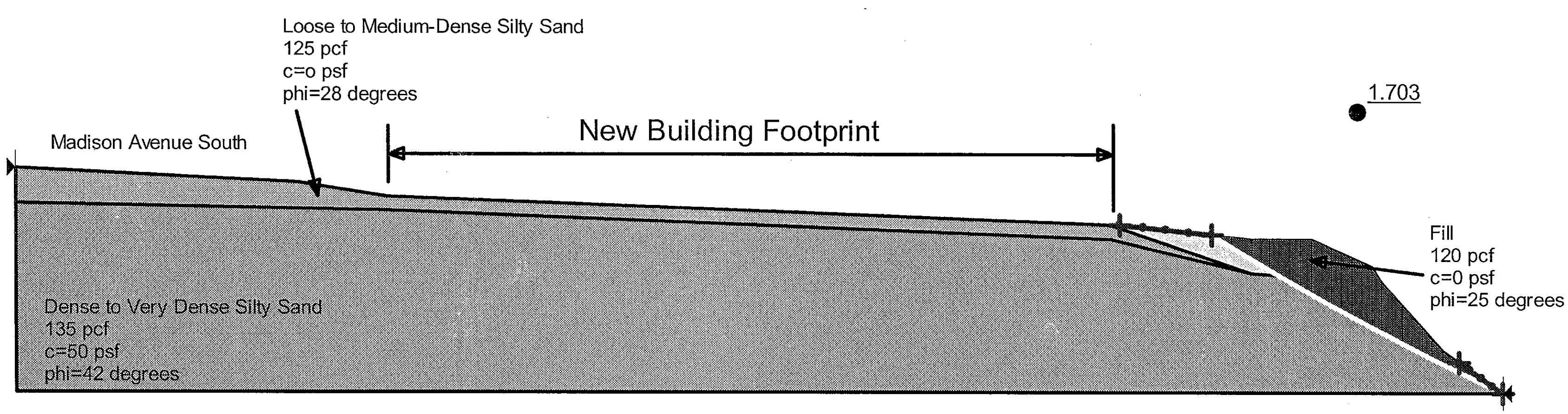
Proposed Windward Inn Hotel
230 Madison Avenue South
Winslow, Washington

Job No:	Date:		Plate:
02302	November 2002		6

Appendix B - Slope Stability Analyses
230 Madison Avenue South
Bainbridge Island, Washington

18485 - Cihan Anisoglu

Static



Static

File Information

File Version: 8.15
Title: 18485 Cihan Slope Stability
Created By: Matt McGinnis
Last Edited By: Matt McGinnis
Revision Number: 8
Date: 10/4/2018
Time: 12:33:45 PM
Tool Version: 8.15.4.11512
File Name: 18485 Slope stability - Existing with New Building Overlay.gsz
Directory: S:\2018 Jobs\18485 Anisoglu (MRM)\
Last Solved Date: 10/4/2018
Last Solved Time: 12:33:47 PM

Project Settings

Length(L) Units: Feet
Time(t) Units: Seconds
Force(F) Units: Pounds
Pressure(p) Units: psf
Strength Units: psf
Unit Weight of Water: 62.4 pcf
View: 2D
Element Thickness: 1

Analysis Settings

Static

- Kind: SLOPE/W
- Method: Morgenstern-Price
- Settings
  - Side Function
    - Interslice force function option: Half-Sine
  - PWP Conditions Source: (none)
- Slip Surface
  - Direction of movement: Left to Right
  - Use Passive Mode: No
  - Slip Surface Option: Entry and Exit
  - Critical slip surfaces saved: 1
  - Resisting Side Maximum Convex Angle: 1 °
  - Driving Side Maximum Convex Angle: 5 °
  - Optimize Critical Slip Surface Location: No
  - Tension Crack
    - Tension Crack Option: (none)
- F of S Distribution
  - F of S Calculation Option: Constant

Advanced

Number of Slices: 30
F of S Tolerance: 0.001
Minimum Slip Surface Depth: 0.1 ft
Search Method: Root Finder
Tolerable difference between starting and converged F of S: 3
Maximum iterations to calculate converged lambda: 20
Max Absolute Lambda: 2

Materials

Fill

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion': 0 psf
Phi': 25 °
Phi-B: 0 °

Loose to Medium-Dense Silty Sand

Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion': 0 psf
Phi': 28 °
Phi-B: 0 °

Dense to Very Dense Silty Sand

Model: Mohr-Coulomb
Unit Weight: 135 pcf
Cohesion': 50 psf
Phi': 42 °
Phi-B: 0 °

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (157, 27.8607) ft
Left-Zone Right Coordinate: (170, 26.56716) ft
Left-Zone Increment: 4
Right Projection: Range
Right-Zone Left Coordinate: (205.12103, 8.38068) ft
Right-Zone Right Coordinate: (211.4, 4) ft
Right-Zone Increment: 4
Radius Increments: 4

Slip Surface Limits

Left Coordinate: (0, 36) ft
Right Coordinate: (211.4, 4) ft

Seismic Coefficients

Horz Seismic Coef.: 0

Points

	X (ft)	Y (ft)
Point 1	0	36
Point 2	40	34
Point 3	52.8	32
Point 4	155.6	28
Point 5	175.7	26
Point 6	184.2	26
Point 7	192.8	22
Point 8	194.2	20
Point 9	202.8	10
Point 10	211.4	4
Point 11	0	4
Point 12	155.6	26
Point 13	155.6	20
Point 14	175.7	21
Point 15	175.6	15
Point 16	52.8	30
Point 17	52.8	23.5
Point 18	4	4
Point 19	0	31

Regions

	Material	Points	Area (ft^2)
Region 1	Dense to Very Dense Silty Sand	18,10,9,8,14,12,16,19,11	4,684
Region 2	Loose to Medium-Dense Silty Sand	1,2,3,4,14,12,16,19	437.7
Region 3	Fill	4,5,6,7,8,14	127.8

Current Slip Surface

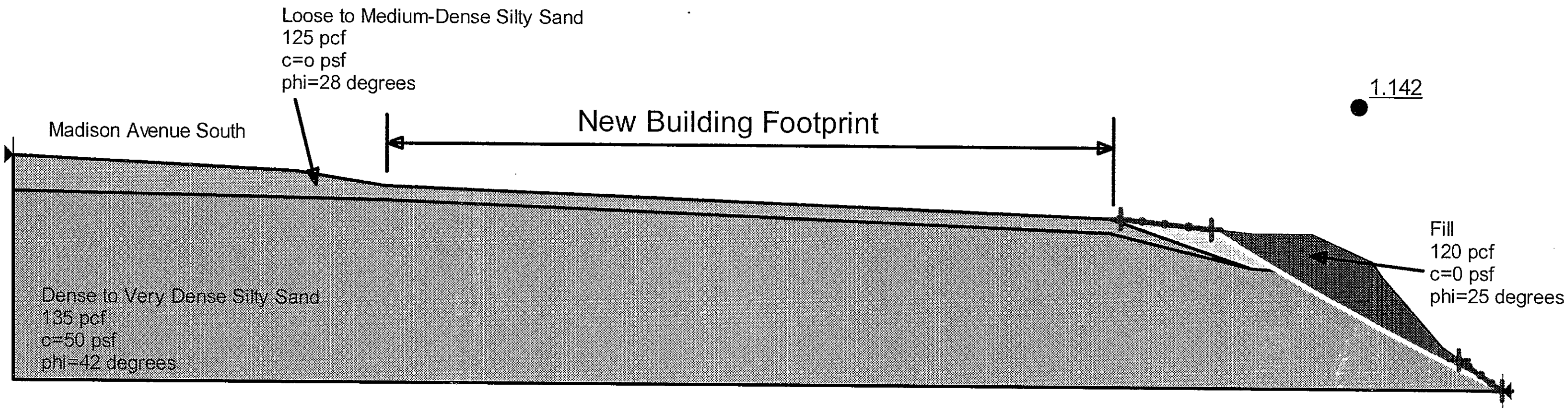
Slip Surface: 121
F of S: 1.703
Volume: 185.05693 ft^3
Weight: 23,752.794 lbs
Resisting Moment: 7,741,017.2 lbs-ft
Activating Moment: 4,544,669.2 lbs-ft
Resisting Force: 17,105.499 lbs
Activating Force: 10,042.829 lbs
F of S Rank (Analysis): 1 of 125 slip surfaces
F of S Rank (Query): 1 of 125 slip surfaces
Exit: (211.4, 4) ft
Entry: (170, 26.567164) ft
Radius: 397.36763 ft
Center: (380.55013, 363.56817) ft

Slip Slices

	X (ft)	Y (ft)	PWP	Base Normal Stress	Frictional Strength	Cohesive Strength

			(psf)	(psf)	(psf)	(psf)
Slice 1	170.7125	26.1241	0	37.334877	17.409539	0
Slice 2	172.1375	25.242128	0	108.67722	50.67702	0
Slice 3	173.5625	24.368436	0	175.3934	81.787287	0
Slice 4	174.9875	23.50296	0	238.30555	111.1237	0
Slice 5	176.33965	22.68907	0	301.2404	140.47071	0
Slice 6	177.61896	21.925925	0	364.90458	170.1578	0
Slice 7	178.89827	21.169262	0	426.9805	199.10428	0
Slice 8	180.31494	20.339246	0	491.47504	442.52611	50
Slice 9	181.86896	19.437346	0	587.11522	528.64092	50
Slice 10	183.42299	18.544802	0	682.80332	614.79887	50
Slice 11	184.91667	17.695498	0	744.72403	670.55253	50
Slice 12	186.35	16.888678	0	772.51625	695.57676	50
Slice 13	187.78333	16.089645	0	799.95558	720.28324	50
Slice 14	189.21667	15.298343	0	826.89265	744.53749	50
Slice 15	190.65	14.514721	0	853.15471	768.18395	50
Slice 16	192.08333	13.738727	0	878.54977	791.04977	50
Slice 17	193.5	12.979159	0	839.63534	756.01106	50
Slice 18	194.91667	12.227032	0	742.61413	668.65276	50
Slice 19	196.35	11.473448	0	650.35941	585.58624	50
Slice 20	197.78333	10.727297	0	555.53054	500.20194	50
Slice 21	199.21667	9.9885303	0	458.15788	412.52721	50
Slice 22	200.65	9.2571026	0	358.35369	322.66311	50
Slice 23	202.08333	8.5329684	0	256.30765	230.78045	50
Slice 24	203.51667	7.8160831	0	188.2236	169.4773	50
Slice 25	204.95	7.1064029	0	154.53567	139.14454	50
Slice 26	206.38333	6.403885	0	119.49406	107.59294	50
Slice 27	207.81667	5.708487	0	83.276793	74.982762	50
Slice 28	209.25	5.0201677	0	46.076468	41.487438	50
Slice 29	210.68333	4.3388863	0	8.0915334	7.2856494	50

18485 - Cihan Anisoglu
Seismic



Seismic

File Information

- File Version: 8.15
- Title: 18485 Cihan Slope Stability
- Created By: Matt McGinnis
- Last Edited By: Matt McGinnis
- Revision Number: 8
- Date: 10/4/2018
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- File Name: 18485 Slope stability - Existing with New Building Overlay.gsz
- Directory: S:\2018 Jobs\18485 Anisoglu (MRM)\
- Last Solved Date: 10/4/2018
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Project Settings

- Length(L) Units: Feet
- Time(t) Units: Seconds
- Force(F) Units: Pounds
- Pressure(p) Units: psf
- Strength Units: psf
- Unit Weight of Water: 62.4 pcf
- View: 2D
- Element Thickness: 1

Analysis Settings

Seismic

- Kind: SLOPE/W
- Method: Morgenstern-Price
- Settings
  - Side Function
    - Interslice force function option: Half-Sine
  - PWP Conditions Source: (none)
- Slip Surface
  - Direction of movement: Left to Right
  - Use Passive Mode: No
  - Slip Surface Option: Entry and Exit
  - Critical slip surfaces saved: 1
  - Resisting Side Maximum Convex Angle: 1 °
  - Driving Side Maximum Convex Angle: 5 °
  - Optimize Critical Slip Surface Location: No
  - Tension Crack
    - Tension Crack Option: (none)
- F of S Distribution
  - F of S Calculation Option: Constant

Advanced

Number of Slices: 30
F of S Tolerance: 0.001
Minimum Slip Surface Depth: 0.1 ft
Search Method: Root Finder
Tolerable difference between starting and converged F of S: 3
Maximum iterations to calculate converged lambda: 20
Max Absolute Lambda: 2

Materials

Fill

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion': 0 psf
Phi': 25 °
Phi-B: 0 °

Loose to Medium-Dense Silty Sand

Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion': 0 psf
Phi': 28 °
Phi-B: 0 °

Dense to Very Dense Silty Sand

Model: Mohr-Coulomb
Unit Weight: 135 pcf
Cohesion': 50 psf
Phi': 42 °
Phi-B: 0 °

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (157, 27.8607) ft
Left-Zone Right Coordinate: (170, 26.56716) ft
Left-Zone Increment: 4
Right Projection: Range
Right-Zone Left Coordinate: (205.12103, 8.38068) ft
Right-Zone Right Coordinate: (211.4, 4) ft
Right-Zone Increment: 4
Radius Increments: 4

Slip Surface Limits

Left Coordinate: (0, 36) ft
Right Coordinate: (211.4, 4) ft

Seismic Coefficients

Horz Seismic Coef.: 0.2

Points

	X (ft)	Y (ft)
Point 1	0	36
Point 2	40	34
Point 3	52.8	32
Point 4	155.6	28
Point 5	175.7	26
Point 6	184.2	26
Point 7	192.8	22
Point 8	194.2	20
Point 9	202.8	10
Point 10	211.4	4
Point 11	0	4
Point 12	155.6	26
Point 13	155.6	20
Point 14	175.7	21
Point 15	175.6	15
Point 16	52.8	30
Point 17	52.8	23.5
Point 18	4	4
Point 19	0	31

Regions

	Material	Points	Area (ft^2)
Region 1	Dense to Very Dense Silty Sand	18,10,9,8,14,12,16,19,11	4,684
Region 2	Loose to Medium-Dense Silty Sand	1,2,3,4,14,12,16,19	437.7
Region 3	Fill	4,5,6,7,8,14	127.8

Current Slip Surface

Slip Surface: 121
F of S: 1.142
Volume: 185.05693 ft^3
Weight: 23,752.794 lbs
Resisting Moment: 7,059,671.3 lbs-ft
Activating Moment: 6,183,401.9 lbs-ft
Resisting Force: 15,619.499 lbs
Activating Force: 13,676.514 lbs
F of S Rank (Analysis): 1 of 125 slip surfaces
F of S Rank (Query): 1 of 125 slip surfaces
Exit: (211.4, 4) ft
Entry: (170, 26.567164) ft
Radius: 397.36763 ft
Center: (380.55013, 363.56817) ft

Slip Slices

	X (ft)	Y (ft)	PWP	Base Normal Stress	Frictional Strength	Cohesive Strength

			(psf)	(psf)	(psf)	(psf)
Slice 1	170.7125	26.1241	0	33.513167	15.627446	0
Slice 2	172.1375	25.242128	0	92.600349	43.180252	0
Slice 3	173.5625	24.368436	0	141.08475	65.788897	0
Slice 4	174.9875	23.50296	0	181.00138	84.402331	0
Slice 5	176.33965	22.68907	0	217.46317	101.40474	0
Slice 6	177.61896	21.925925	0	252.05959	117.53732	0
Slice 7	178.89827	21.169262	0	283.20542	132.06086	0
Slice 8	180.31494	20.339246	0	392.23013	353.1656	50
Slice 9	181.86896	19.437346	0	477.19229	429.66587	50
Slice 10	183.42299	18.544802	0	563.51601	507.39209	50
Slice 11	184.91667	17.695498	0	623.1732	561.10767	50
Slice 12	186.35	16.888678	0	656.09133	590.74728	50
Slice 13	187.78333	16.089645	0	690.86542	622.05802	50
Slice 14	189.21667	15.298343	0	727.05259	654.6411	50
Slice 15	190.65	14.514721	0	763.98515	687.89532	50
Slice 16	192.08333	13.738727	0	800.79095	721.03541	50
Slice 17	193.5	12.979159	0	781.55315	703.71362	50
Slice 18	194.91667	12.227032	0	709.49899	638.83576	50
Slice 19	196.35	11.473448	0	637.79116	574.26974	50
Slice 20	197.78333	10.727297	0	559.26562	503.56502	50
Slice 21	199.21667	9.9885303	0	473.75615	426.57195	50
Slice 22	200.65	9.2571026	0	381.6419	343.63191	50
Slice 23	202.08333	8.5329684	0	283.78397	255.52024	50
Slice 24	203.51667	7.8160831	0	213.97379	192.66287	50
Slice 25	204.95	7.1064029	0	173.99659	156.66723	50
Slice 26	206.38333	6.403885	0	132.41969	119.23122	50
Slice 27	207.81667	5.708487	0	90.102252	81.128432	50
Slice 28	209.25	5.0201677	0	47.721625	42.968744	50
Slice 29	210.68333	4.3388863	0	5.7305448	5.1598057	50