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GEOTECHNICAL ENGINEERING REPORT
HAUPTMAN RESIDENCE FOUNDATION REPAIR
685 LUDLOW BAY ROAD
PORT LUDLOW, WASHINGTON
JEFFERSON COUNTY PARCEL # 969000006

PREPARED FOR:
MR. DWIGHT HAUPTMAN

BY:
OTTO ROSENAU & ASSOCIATES, INC.
ORA JOB NO. 06-124, REPORT NO. 1

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JEFFERSON COUNTY DCD

OTTO ROSENAU & ASSOCIATES, INC.
Geotechnical Engineering, Construction Inspection & Materials Testing

OTTO ROSENAU & ASSOCIATES, INC.

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May 19, 2006

Mr. Dwight Hauptman
685 Ludlow Bay Road
Port Ludlow, Washington 98363

Re: Geotechnical Engineering Report
Hauptman Residence Foundation Repair
685 Ludlow Bay Road
Port Ludlow, Washington
Jefferson County Parcel # 969000006
ORA Project Number: 06-124, Report 1

Dear Mr. Hauptman:

We are pleased to provide this report for the referenced project. Based on our subsurface explorations and our analyses, it is our opinion that the settlement at the residence is likely a result of consolidation of poorly-compacted fill under the weight of the existing structure. Based on the results of our slope stability analyses, it is our opinion that the settlement is not likely a result of slope instability. Furthermore, it is our opinion that the proposed foundation repair structure can be completed satisfactorily with minimal risk of adversely impacting the stability of site slopes, or adjacent properties provided that the work is completed in accordance with the recommendations of this report.

It is our opinion that the proposed residential structure can be satisfactorily underpinned with the hydraulically advanced, Grip-Tite pier and bracket system or with a conventional, driven 3-inch pin piles and bracket system.

Detailed underpinning, slope stability, and earthwork recommendations are presented in the attached report and plan sheet. If you have any questions, or if we may be of additional service, please contact us.

ANTHONY G. COYNE
STATE OF WASHINGTON
37856
REGISTERED
PROFESSIONAL ENGINEER
5/19/06
EXPIRES 10/14/2006

Sincerely,
Otto Rosenau & Associates, Inc.

Anthony Coyne

Anthony G. Coyne, P.E.
Geotechnical Engineer

Copies to: Addressee (5)

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GEOTECHNICAL ENGINEERING REPORT
RESIDENTIAL FOUNDATION REPAIR
685 LUDLOW BAY ROAD
PORT LUDLOW, WASHINGTON
JEFFERSON COUNTY PARCEL # 969000006

Prepared for

Mr. Dwight Hauptman

by Otto Rosenau & Associates, Inc.

May 19, 2006

1. INTRODUCTION

This report presents the results of our geotechnical engineering services for the residence at 685 Ludlow Bay Road in Port Ludlow, Washington. The location of the approximate site is shown on the Vicinity Map on page A-1 of the appendix.

2. PROJECT DESCRIPTION

We understand that the west side of the existing residence has experienced up to 4 inches of differential settlement. The existing residence consists of a two-story, wood-framed structure with a crawl space beneath the living areas and a slab-on-grade at the attached garage. The west side of the residence is located in close proximity to an existing rockery that is typically 15 feet in height at locations adjacent to the residence. The foundations along the west side of the residence appear to be supported on fill retained by the rockery.

Based on a review of a report prepared by Northwestern Territories, Inc. titled "Homesite Preparation-Port Ludlow Bay" dated April 23, 1985, we understand that 10- to 12-inch diameter, "drilled-in-place" reinforced concrete piles at an eight foot spacing were recommended to support the existing structure. It was recommended that the piles extend through the fill retained behind the rockery and several feet into the "firm in-place gravelly-sand soils".

3. SCOPE OF SERVICES

The scope of services included a reconnaissance of the site by the geologist, a review of geologic literature, and witnessing the drilling of two borings (B-1 and B-2) at the approximate location shown on the Site Plan on page A-2 of the appendix. Soil samples were taken of the subsurface soils at the depths shown on the boring logs presented on pages A-3 through A-4 of the appendix.

The engineering recommendations and advice presented in this report have been made in accordance with generally accepted geotechnical engineering practices in the area. The recommendations are based on our understanding of the geology of the area and on experience with similar projects.

The geotechnical engineering services were performed by Otto Rosenau and Associates, Inc. (ORA) to provide the following information:

- existing conditions of the foundation and rockery made during a visit to the site,
- available geologic information,
- suitability of use of Grip-Tite piers, or driven pin piles for the proposed foundation repair,
- recommended spacing of the Grip-Tite piers, or driven pin piles based on estimated building loads,
- evaluation of the rockery and recommendation for repair or reconstruction, if necessary.

4. SITE CHARACTERIZATION

We reviewed the "Geologic Map of Washington, Northwest Quadrant, Washington Division of Geology and Earth Resources, Geologic Map GM-50, 2002 compiled by Dragovich, Joe D., Logan, Robert L., Schasse, Henry W., et al. The soils at the project site are predominantly mapped as "Fraser Age glaciation advance and undifferentiated outwash and till deposits" (Qga, Qgo, and Qgt). Qga deposits typically consist of glaciofluvially deposited sand and gravel and lacustrine clay, silt, and sand deposited during the advance of the glaciers. Qgt deposits typically consist of unsorted, unstratified, highly compacted mixture of clay, silt, sand, gravel, and boulders deposited by glacial ice. Qgo deposits typically consist of recessional and proglacial stratified sand, gravel, and cobbles with minor silt and clay interbeds deposited in delta, ice contact beach and melt water stream environments. The Crescent Formation Basalts (Evc) are mapped to the west of the site. The Evc deposits are thoeleiitic basalts flows, basaltic flow breccia, filled tubes and volcaniclastic conglomerates from the lower Eocene to Middle Eocene Epochs (54.8 million years ago to 45 million years ago).

5. SURFACE CONDITIONS

The site is located along the south shore of Ludlow Bay. The site grades slope gently downwards to the north from approximately Elevation 60 feet along Ludlow Bay Road to about Elevation 40 feet to the north of the existing residence. The site grades slope steeply downwards towards Ludlow Bay to the north and towards a ravine to the west at the north end of the site. The site grades along the east side of the property continue evenly across onto the adjacent lot to the east. A rockery is present along a significant portion of the west property line. The rockery appears to have been constructed as part of the development of the site to provide a relatively level building pad area. The rockery is up to 15 feet in height at locations adjacent to the existing residence. Overall, the rockery appears to be in good condition. The rockery is inclined at an angle ranging from about 5 to 10 degrees from vertical. We did not observe indications of on-going instability at the rockery location such as bulging or loss of stones or partial collapses. We also did not observe the presence of drainage material between the retained

fill and the rockery. The stone used in the rockery appears to be generally sound and the quality of the construction and fitting of the stones was good overall. The west side of the house is located between 8 feet and 25 feet from the front of the rockery. The site grades in front of the rockery slopes downwards to a ravine located to the west of the residence.

6. SUBSURFACE CONDITIONS

We evaluated the subsurface soil and groundwater conditions by completing two boring (B-1 and B-2) at the site using a man-portable, hollow-stem auger drill rig on March 28, 2006. The borings were completed along the west side of the residence. Both borings were completed to a depth of 26.5 feet beneath the existing ground surface. Please refer to the Site Plan on page A-2 of the appendix for the approximate location of the boring. The details and explanations of our explorations are presented on pages A-3 through A-7 of the appendix.

Similar soil conditions were observed at each boring location with several feet of very loose to medium dense sandy fill placed over the native subgrade soils. The surface conditions at B-1 and B-2 consisted of bark, groundcover plants, and loose, light brown, fine to medium SAND with Silt (SP-SM) with organics. Very loose to loose, brown, fine to medium SAND with Silt (SP-SM) fill was encountered beneath the bark at B-1 from depths of about 0.3 to 14 feet below the existing ground surface. Medium dense to very dense, native, brown and gray sand with less than about 12 percent fines (SP and SP-SM) was encountered beneath the fill to the bottom of the exploration at a depth of about 26.5 feet below the existing ground surface.

Very loose to loose, brown, fine to medium SAND with Silt (SP-SM) and fine to coarse (SW-SM) fill was encountered beneath the bark at B-2 from depths of about 0.3 feet to 6.3 feet below the existing ground surface. Medium dense to dense, light brown and brown fine to medium SAND with Silt (SP-SM) and fine to coarse (SW-SM) was encountered beneath the upper fill layer from depths of about 6.3 to 18 feet below the ground surface. A wet, medium dense Silty SAND (SM) layer was encountered at depths of approximately 18 to 20.5 feet and was underlain by a medium dense fine SAND with Silt (SP-SM) to a depth of about 23 feet below the existing site grade. Very dense, gray fine to coarse SAND (SW-SM) with silt was encountered from a depth of about 23.0 feet to the bottom of the exploration at a depth of about 26.5 feet below the existing ground surface.

7. LABORATORY TESTING

We performed moisture content determinations on each sample collected from the borings. The results of moisture content determinations are presented on the boring logs.

8. DISCUSSION

The recommendations presented in this report are based on our understanding of the project as presented in the Project Description Section and on the assumption that the subsurface conditions are as assumed herein. Project conditions, regarding type and location of structures and foundation loads can change, and subsurface conditions are not always similar to those encountered during the subsurface exploration. Therefore, if discrepancies are noticed, the geotechnical engineer must be contacted for review and for possible revision of the recommendations.

9. CONCLUSIONS AND RECOMMENDATIONS

9.1 GENERAL

It is our opinion that the settlement at the residence is likely a result of consolidation of poorly-compacted fill under the weight of the existing structure. The consolidation of the poorly-compacted fill could have been accelerated by poor drainage caused by roof downspouts that have discharged close to the foundation walls. Based on the results of our slope stability analyses, it is our opinion that the settlement is not likely a result of slope instability.

We did not observe the presence of piles beneath the existing footings during our brief site visit. It is possible that the drilled-in-place piles, which were recommended in the report prepared by Northwestern Territories, Inc., were not installed, or were not installed in accordance with the recommendations of the original report.

It is our opinion that the current condition of the existing, perimeter foundation elements is suitable for underpinning. It is also our opinion that the installation of underpinning at the perimeter foundation locations will allow the structure to be stabilized to reduce the risk of future settlement at perimeter foundation element locations and re-leveled, if desired. The areas recommended for underpinning is based on our discussions with the owner and our observations while on site. The installation of underpinning along the perimeter will not reduce the risk of settlement of the interior floor slabs or interior column footings that support the floor beams under the residence. The stabilization of individual column footings in the crawl space is possible, but is likely unfeasible, due to the significant amount of disruption to the main floor flooring and subfloor that would be caused.

It is our opinion that the existing rockery is relatively stable based on its performance over the last 20 years. However, the wall was not built in accordance with current accepted practice, which typically includes drainage behind the rockery. We recommend that long-term monitoring of the rockery be performed at several locations to monitor for movement. In general, rockeries require periodic maintenance.

9.2 UNDERPINNING

9.2.1 General: It is our opinion that the perimeter foundation elements of the residence may be successfully underpinned using a proprietary hydraulically-pushed pier and bracket system known as

Grip-Tite piers, or with conventional, 3-inch diameter pin piles that are driven using a hydraulic, hammer. The 3-inch diameter pin piles should also be attached to the existing foundation with a bracket, which allows for post-installation adjustment, if necessary. Certain locations will need to be underpinned using conventional, 2- or 3-inch diameter, driven, pin piles due to an insufficient amount of structural weight being available to install hydraulically-pushed piers, such as at the porch column footings and at the northwest corner of the residence and at the covered enclosure area at the southwest corner of the residence. Driven pin piles may also be required to underpin the column footing between the two garage doors.

For planning purposes, we anticipate that the Grip-Tite pier and pin pile installation depths should not exceed 25 feet, and may be significantly less depending on the thickness of fill present beneath the residence. Our supporting calculations for Grip-Tite piers and pin pile underpinning are presented on pages A-8 through A-14 of the appendix.

9.2.2 Grip-Tite Piers: The Grip-Tite piers consist of 3-inch diameter, high-strength steel tubing that is advanced to refusal using a hydraulic ram that bears against a bracket in contact with the bottom of the foundation element. Sections of steel tubing are installed using slip-joint connectors until practical refusal is achieved. The bracket is then locked off to the pier to prevent future movement. Post-installation adjustments can be made if additional settlement occurs after installation. ICC-ES Legacy Report 22-02 presents design recommendations for the use of the Grip-Tite piers and brackets. The Grip-Tite pier and bracket system are designed to support a maximum allowable downward load of 16,500 pounds per pier. A minimum of one Grip-Tite pier shall be load tested in general accordance with the ASTM D1143-81 test procedure. The installation of Grip-Tite piers should be monitored by an ORA representative.

9.2.3 Pin Piles: Pin pile underpinning consists of a 2- or 3-inch diameter steel pipe that is driven to refusal and is attached by a bracket to the foundation element. 2-inch diameter pin piles should consist of Schedule 80 Grade A53 A steel. 2-inch diameter pin piles should be installed by driving with a pneumatic jack hammer weighing no less than 90 pounds, or with a hydraulic hammer approved by the geotechnical engineer. 3-inch diameter pin piles should consist of Schedule 40 Grade A53 A steel. 3-inch diameter pin piles should be driven to refusal with a hydraulic hammer that weighs no less than 650 pounds. The geotechnical engineer should be contacted for specific refusal criteria for pin piles based on the type of hammer to be used. 2–inch diameter pin piles driven to refusal can provide an allowable downward capacity of 4,000 pounds per pile. 3–inch diameter pin piles driven to refusal can provide an allowable downward capacity of 12,000 pounds per pile. A minimum of one 3-inch diameter pin pile shall be load tested in general accordance with the ASTM D1143-81 test procedure. The installation of pin piles should be monitored by an ORA representative.

9.3 SEISMIC CONSIDERATIONS

2003 IRC SEISMIC ZONE D2 IS CURRENT

The seismic design of structures in the Jefferson County is governed by the requirements of the 1997 edition of the Uniform Building Code (UBC). According to Figure No. 16-2 of the UBC the project site is located in Seismic Zone 3. According to Table 16-J of the UBC, the site soil profile is best represented by a Soil Profile Type Sc. The soils encountered in our exploration at the site are generally not susceptible to liquefaction.

9.4 SLOPE STABILITY

We performed a visual reconnaissance of the slopes along the west side of the site to evaluate the current slope stability conditions at the site. We did not observe indications of on-going slope instability at the site. We did not observe tension cracks, sag ponds, slump blocks or other common indicators of slope instability at the site. We observed settlement of the house, and what appears to be distortion (sagging and bending) of a chain link fence running along the top of the rockery near the southwest corner of the residence. There are several possible reasons for the distortion of the chain link fence at the top of rockery, which may include one or more of the following:

- Slope instability;
- Lack of proper rockery drainage resulting in occasionally increased lateral earth pressures;
- Settlement of the residence and adjacent covered enclosure resulting in an increased lateral earth pressure pushing on the rockery wall;
- Activity above the wall disturbing the fence.

It is our opinion that the likelihood that slope instability has caused the distortion of the chain link fence is small. Instead, it is our opinion that the distortion of the chain link fence is more likely a result of a lack of rockery drainage, and settlement of the adjacent residence exerting and possibly surcharge loading from the nearby foundation elements.

ORA performed a computer-based slope stability analysis using XSTABL. We used the information from the two field-generated cross-sections A-A' and B-B' indicated on the Site Plan on page A-2 of the appendix and from the subsurface explorations as a basis for input into the slope stability analysis. Cross-section A-A' is located approximately 18 feet south of the northwest corner. Cross-section B-B' is located at the west side of the residence approximately 9 feet north of the southwest corner. We assumed that the existing residence is supported on conventional spread footings in our slope stability analyses. We evaluated the following two cases for this project at each cross-section location:

1. Static Loading Conditions – No earthquake forces applied, existing site conditions.
2. Seismic Loading Conditions – 30% g ~ earthquake with 500 year recurrence interval.

The existing slope was modeled assuming the presence of two soil layers – the upper loose SAND fill layer and the dense, native SAND layer. The following soil parameters were assumed:

Soil Unit Description	Assumed Unit Weight, γ, (pcf)	Angle of Internal Friction, ϕ, degrees	Cohesion (psf)
Upper Fill Unit (SP-SM)	100 pcf dry	32	50
	115 pcf saturated		
Lower SAND Unit (SP-SM)	115 pcf dry	36	50
	125 pcf saturated		

The results of our slope stability analysis are presented on pages A-15 through A-35 of the appendix. A summary of the results under the various loading conditions is presented in the following table:

Location	Loading Condition	Estimated Minimum Factor of Safety	Recommended Minimum Factor of Safety
Section A-A' (West wall NW corner of residence)	Case 1 - Static	1.4	1.3
Section A-A' (West wall NW corner of residence)	Case 2 – Seismic 0.3 g (500 year seismic)	1.1	1.1
Section B-B' (West wall SW corner of residence)	Case 1 - Static	2.3	1.1
Section B-B' (West wall SW corner of residence)	Case 2 – Seismic 0.3 g (500 year seismic)	1.7	1.1

A factor of safety of 1 indicates that the forces that cause instability are in equilibrium with the forces that are resisting instability. A factor of safety of less than 1 indicates that the forces that cause instability are greater than the forces resisting instability and that the slope will fail. Conversely, a factor of safety greater than 1 indicates that the forces resisting instability are greater than those causing instability and that the slope is stable.

The results from the slope stability analyses completed for this study indicate that the existing slopes at the site will likely be stable under static conditions and during a 500 year seismic event. The primary failure mechanism of slope failure predicted by the XSTABL analyses is a shallow failure that passes below the base of the rockery wall.

In order to reduce the risk of slope instability after construction, we recommend the following practices:

- All roof drains, footing drains, and other drains should be gathered and tightlined to a discharge location approved by Jefferson County.

- Any accumulations of yard waste and biodegradable construction waste (cut branches, lawn clippings, and lumber) that are present should be removed from the slope face and adjacent areas.
- Yard waste should not be placed on any of the existing slopes.
- Vegetative cover should be continuously maintained on overall steep slope areas to reduce erosion potential and to stabilize surficial soils.

9.5 EROSION AND SEDIMENTATION CONTROL

The migration of sediments from the site must be installed and controlled in accordance with Jefferson County requirements. We recommend that the following minimum erosion control measures be employed at the site:

- Provide silt fencing around the construction area to delineate the construction limits. No construction or soil disturbance should take place outside of the construction limits.
- Stockpiled soil at the site should be kept to a minimum. Any stockpiled soils should be covered with carefully secured plastic sheeting.

Additional erosion control measures may be required as construction progresses.

9.6 TEMPORARY CUT SLOPES

We anticipate that temporary cut slopes will be used at portions of the site. We recommend that the inclination of the temporary cut slopes be no greater than 1.5H:1V (horizontal to vertical) in the upper fill soil.

All temporary cut slopes and excavations must comply with the provisions of Washington Administrative Code (WAC) Chapter 296-155, Part N, "Excavation, Trenching and Shoring." The contractor performing the work has the primary responsibility for protection of workers and adjacent improvements. However, we recommend that the contractor submit a work plan and excavation support plan for our review prior to beginning work on the site.

9.7 DRAINAGE

9.7.1 Dewatering: Ground water seepage will likely not be encountered during construction. However, we anticipate that dewatering could be satisfactorily completed by routing water through ditches to a low spot or sump in the excavation. Water collected in the excavation should be removed as soon as possible and should be discharged to a location approved by the Jefferson County and in accordance with Jefferson County requirements.

9.7.2 Surface Drainage: Good surface drainage is an integral part of the performance of earth-supported structures such as foundations, retaining walls, and pavements. Therefore, construction grades and final site grades should be designed to prevent water from entering the foundations or gravel drains behind any retaining walls, or from ponding on or next to pavements. Where pavement

does not immediately abut structures, the ground surface should be sloped with an outfall of at least three (3) percent for a minimum distance of five (5) feet from exterior footings. These slopes should be capped with relatively impervious soils to prevent infiltration of water into the foundation soils.

Runoff water should be collected from all impervious surfaces on the project and should be routed away from steep slope area on the west side of the site to a discharge location approved by the Jefferson County. We strongly recommend that no on-site infiltration of runoff water be performed to minimize the flow of additional groundwater and to help minimize the risk of future slope instability.

9.8 CONSTRUCTION OBSERVATION AND TESTING

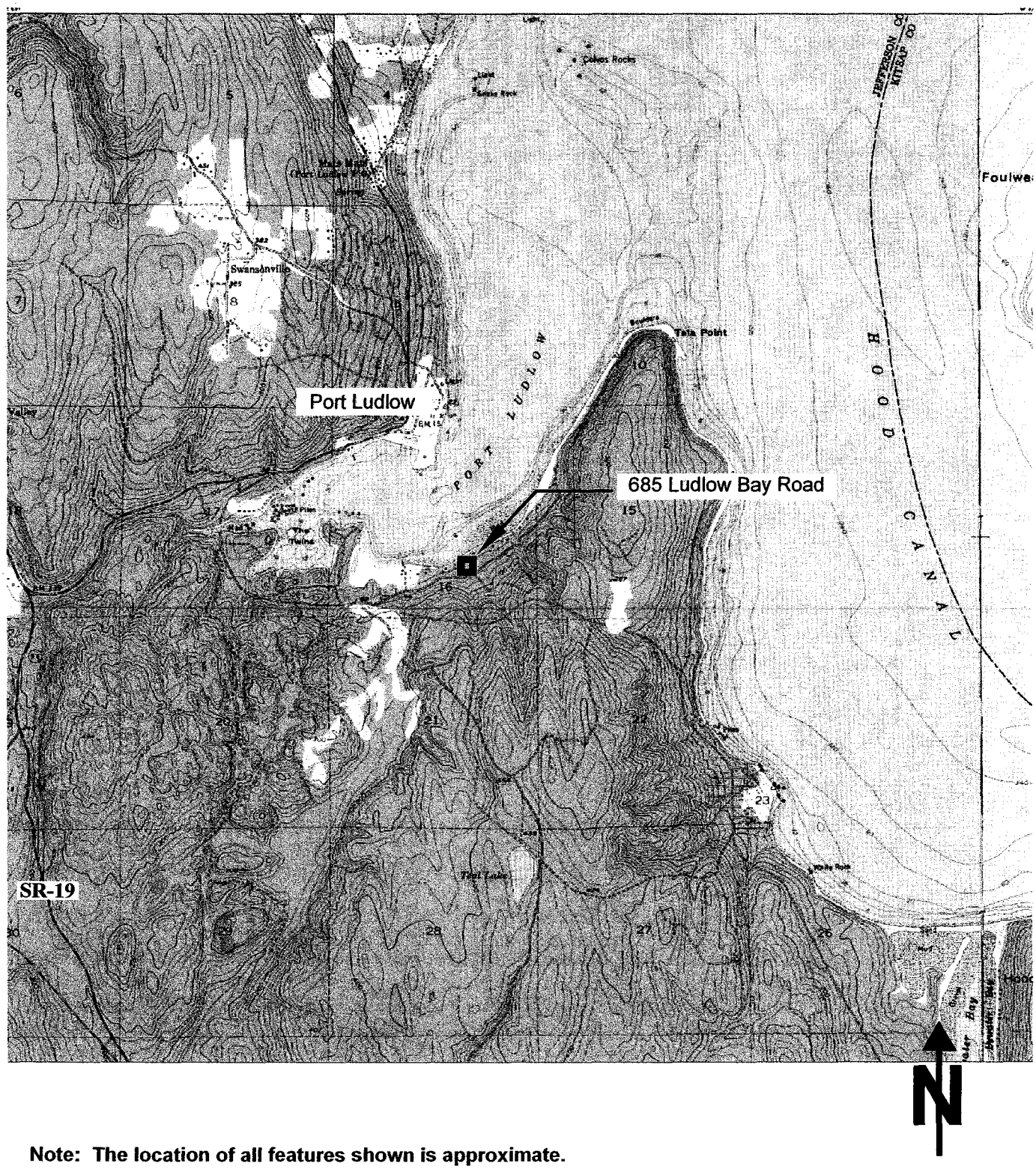
The recommendations presented in this report rely on adequate observation and testing of construction materials and procedures by the geotechnical engineer or his qualified representative. At a minimum, the testing program should include:

- Observation and review of pile driving and pile load testing to evaluate whether actual conditions are consistent with those encountered during exploration and used for design.

10. REPORT LIMITATIONS

The recommendations presented in this report are for the exclusive use of Mr. Dwight Hauptman and other members of the design team for the foundation repair project at 685 Ludlow Bay Road in Port Ludlow, Washington (Jefferson County Parcel # 969000006). The recommendations are based on the readily-available geologic literature and two borings completed on March 28, 2006. The recommendations of this report are not transferable to any other site. If there are any revisions to the plans or if deviations from the subsurface conditions noted in this report are encountered during construction, Otto Rosenau & Associates, Inc. (ORA) should be notified immediately to determine whether changes to the design recommendations are required.

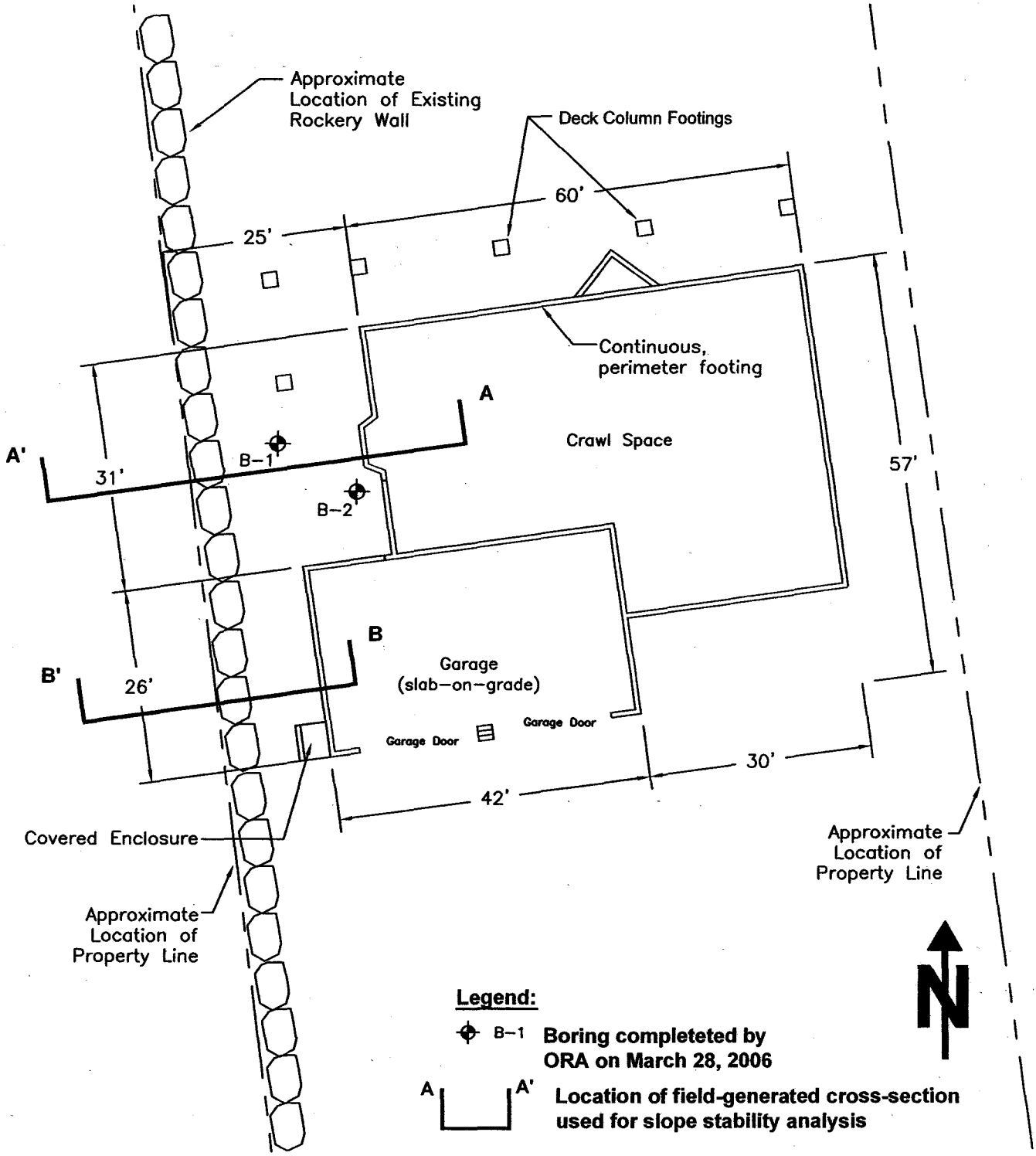
oOo



Note: The location of all features shown is approximate.
Reference: Port Ludlow, Washington USGS Quadrangle and portions of adjacent quadrangles, ALL TOPO MAPS: Washington.

VICINITY MAP

Project Name: Hauptman Residence Foundation Repair	OTTO ROSENAU & ASSOCIATES, INC.	**For:** Mr. Dwight Hauptman
Location: 685 Ludlow Bay Road, Port Ludlow, Washington		**ORA Project Number:** 06-124
Date: May 16, 2006		



Note: The location of all features shown is approximate. **Scale: 1" = 20'**
Reference: Field measurements made with measuring tape and original plat map of Ludlow Beach Tracts No. 2 recorded on November 4, 1948.

SITE PLAN

Project Name: Hauptman Residence Foundation Repair	OTTO ROSENAU & ASSOCIATES, INC.	**For:** Mr. Dwight Hauptman
Location: 685 Ludlow Bay Road, Port Ludlow, Washington		**ORA Project Number:** 06-124
Date: May 16, 2006		

OTTO ROSENAU & ASSOCIATES, INC.
6747 M.L. King Way South
Seattle, WA 98118
Telephone: (206) 725-4600
Fax: (206) 723-2221

BORING NUMBER B-1

CLIENT Dwight Hauptman
PROJECT NAME Hauptman Residence
PROJECT NUMBER 06-124
PROJECT LOCATION 685 Ludlow Bay Road, Port Ludlow, WA
DATE STARTED 3/28/06 **COMPLETED** 3/28/06
GROUND ELEVATION **HOLE SIZE** 7-inches
DRILLING CONTRACTOR CN Drilling
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING Not observed
LOGGED BY Craig Bechtold L.G. **CHECKED BY** Anthony Coyne P.E.
AT END OF DRILLING Not observed
NOTES
AFTER DRILLING Not observed

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	TESTS	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0							0.3 Brown bark
	SS 1	67	3-3-3 (6)	MC=27%	SP-SM		Loose, light brown, fine to medium SAND with Silt with organics and interbedded, oxidized, Sandy SILT (fill)
	SS 2	67	2-2-2 (4)	MC=15%	SP-SM		Loose, brown, fine to medium SAND with Silt (fill) 4.0
5							
	SS 3	67	1-1-1 (2)	MC=18%	SP		Very loose, brown, fine to medium SAND (fill)
	SS 4	33	3-3-3 (6)	MC=13%	SP		Loose, brown, fine to medium SAND (fill) 9.5
10							
	SS 5	33	3-3-2 (5)	MC=11%	SP-SM		Loose, brown SAND with Silt (fill) 14.0
15							
	SS 6	67	4-8-9 (17)	MC=19%	SP-SM		Medium dense, brown fine to medium SAND with Silt and interbedded Sandy SILT (native) 19.0
20							Gravelly drilling at approximately 19 feet
	SS 7	67	14-21-25 (46)	MC=4%	SP		Dense, gray, fine SAND
25							
	SS 8	67	18-32-41 (73)	MC=8%	SP		Very dense, gray, fine SAND 26.5
							Bottom of hole at 26.5 feet.

GENERAL BH / TP / WELL HAUPTMAN RESIDENCE.GPJ GINT US.GDT 5/19/06

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6747 M.L. King Way South
Seattle, WA 98118
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BORING NUMBER B-2

CLIENT Dwight Hauptman
PROJECT NAME Hauptman Residence
PROJECT NUMBER 06-124
PROJECT LOCATION 685 Ludlow Bay Road, Port Ludlow, WA
DATE STARTED 3/28/06 **COMPLETED** 3/28/06
GROUND ELEVATION ____ **HOLE SIZE** 7-inches
DRILLING CONTRACTOR CN Drilling
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING Not observed
LOGGED BY Craig Bechtold L.G. **CHECKED BY** Anthony Coyne P.E.
AT END OF DRILLING Not observed
NOTES ____
AFTER DRILLING Not observed

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	TESTS	U.S.C.S.	MATERIAL DESCRIPTION
0	SS 1	67	1-2-2 (4)	MC=15%	SP-SM	Very loose, dark brown, fine to medium SAND with Silt (fill)
	SS 2	67	1-2-2 (4)	MC=15%	SP-SM	Very loose, dark brown, fine to medium SAND with Silt and occasional gravel (fill)
4.5						
5	SS 3	100	4-4-18 (22)	MC=15%	SW-SM	Medium dense, brown, fine to coarse SAND with Silt (fill)
6.3						Medium dense, brown, fine to coarse SAND with Silt and occasional gravel and interbedded Silty SAND (native)
8.0	SS 4	67	24-25-16 (41)	MC=8%	SW-SM	
10	SS 5	78	10-15-20 (35)	MC=15%	SP-SM	Dense, light brown, moist fine SAND with Silt and traces of oxidation
15	SS 6	67	6-16-22 (38)	MC=9%	SP-SM	Dense, light brown, moist fine SAND with Silt and traces of oxidation
18.0					SM	
20	SS 7	100	9-13-14 (27)	MC=25%		Medium dense, gray, Silty fine SAND (significantly increased moisture content)
20.5					SP-SM	Medium dense, gray, fine SAND with Silt
23.0					SW-SM	
25	SS 8	50	27-50/6"	MC=5%		Very dense, gray, fine to coarse SAND with Silt and gravel
26.5						Bottom of hole at 26.5 feet.

GENERAL BH / TP / WELL HAUPTMAN RESIDENCE.GPJ GINT US.GDT 5/19/06

BORING LOG NOTES

These notes and boring logs are intended for use with this geotechnical report for the purposes and project described therein. The boring logs depict ORA's interpretation of subsurface conditions at the location of the boring on the date noted. Subsurface conditions may vary, and groundwater levels may change because of seasonal or numerous other factors. Accordingly, the boring logs should not be made a part of construction plans or be used to define construction conditions.

The approximate locations of the borings are shown on the Site Plan. The borings were located in the field by measuring from existing site features.

"Boring Size" refers the diameter and type of auger used. "HSA" denotes hollow-stem auger. "SSA" denotes solid-stem auger. "BA" denotes bucket auger.

"Sample Number and Type" refers to the sampling method and equipment used during exploration where:

- "AU" indicates a bulk sample taken from the ground surface or from the auger flights.
- "SS" indicates split-spoon sampler with 1-3/8" inside diameter and 2" outside diameter.
- "NR" indicates sample attempted with no recovery.

"N-Values" refer to the Standard Penetration Test which records number of blows from a 140-pound hammer falling 30 inches required to advance a standard sampler eighteen inches. The blow counts required to drive the sampler through each 6-inch interval is recorded. The number of blows to drive the sampler for the last 12 inches of driving are added together and is considered to be the N-Value. The N-Value is presented in parentheses on the boring logs. The actual blow count values for each 6-inch interval is also presented. If the sample is driven less than 6 inches for a given interval, the actual distance driven is recorded.

"Moisture Content (MC)" refers to the moisture content of the soil expressed in percent by weight of dry sample as determined in the laboratory.

"Qp" is an estimate of the unconfined compressive strength of the soil as determined with a handheld, calibrated, spring-loaded penetrometer.

"Description and USCS Classification" refer to the materials encountered in the boring. The descriptions and classifications are generally based on visual examination in the field and laboratory. Where noted, laboratory tests were performed to determine the soil classification. The terms and symbols used in the boring logs are in general accordance with the Unified Soil Classification System. Laboratory tests are performed in general

BORING LOG NOTES continued

accordance with applicable procedures described by the American Society for Testing and Materials.

"▼" Indicates location of groundwater at the time noted.

TERMS for RELATIVE DENSITY of NON-COHESIVE SOIL

Term	Standard Penetration Resistance "N"
Very Loose	4 or less
Loose	5 to 10
Medium Dense	11 to 30
Dense	31 to 50
Very Dense	Over 50 blows/foot

TERMS for RELATIVE CONSISTENCY of COHESIVE SOIL

Term	Unconfined Compressive Strength
Very Soft	0 to 0.25 tons/square-foot (tsf)
Soft	0.25 to 0.50 tsf
Medium Stiff	0.50 to 1.00 tsf
Stiff	1.00 to 2.00 tsf
Very Stiff	2.00 to 4.00 tsf
Hard	Over 4.00 tsf

DEFINITION of MATERIAL by DIAMETER of PARTICLE

Boulder	8-inches+
Cobble	3 to 8 inches
Gravel	3 inches to 5mm
Coarse Sand	5mm to 0.6mm
Medium Sand	0.6mm to 0.2mm
Fine Sand	0.2mm to 0.074mm
Silt	0.074mm to 0.005mm
Clay	less than 0.005mm

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (MORE THAN 12% FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (MORE THAN 12% FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: FINES ARE MATERIALS PASSING THE NO. 200 SIEVE.
COARSE GRAINED SOILS RECEIVE DUAL SYMBOLS IF THEY CONTAIN BETWEEN 5% AND 12% FINES.
FINE GRAINED SOILS RECEIVE DUAL SYMBOLS IF THEIR LIMITS PLOT LEFT OF THE "A" LINE WITH A PLASTICITY INDEX (PI) OF 4% TO 7%.

CALCULATIONS

HAUPTMAN RESIDENCE

FOUNDATION REPAIR

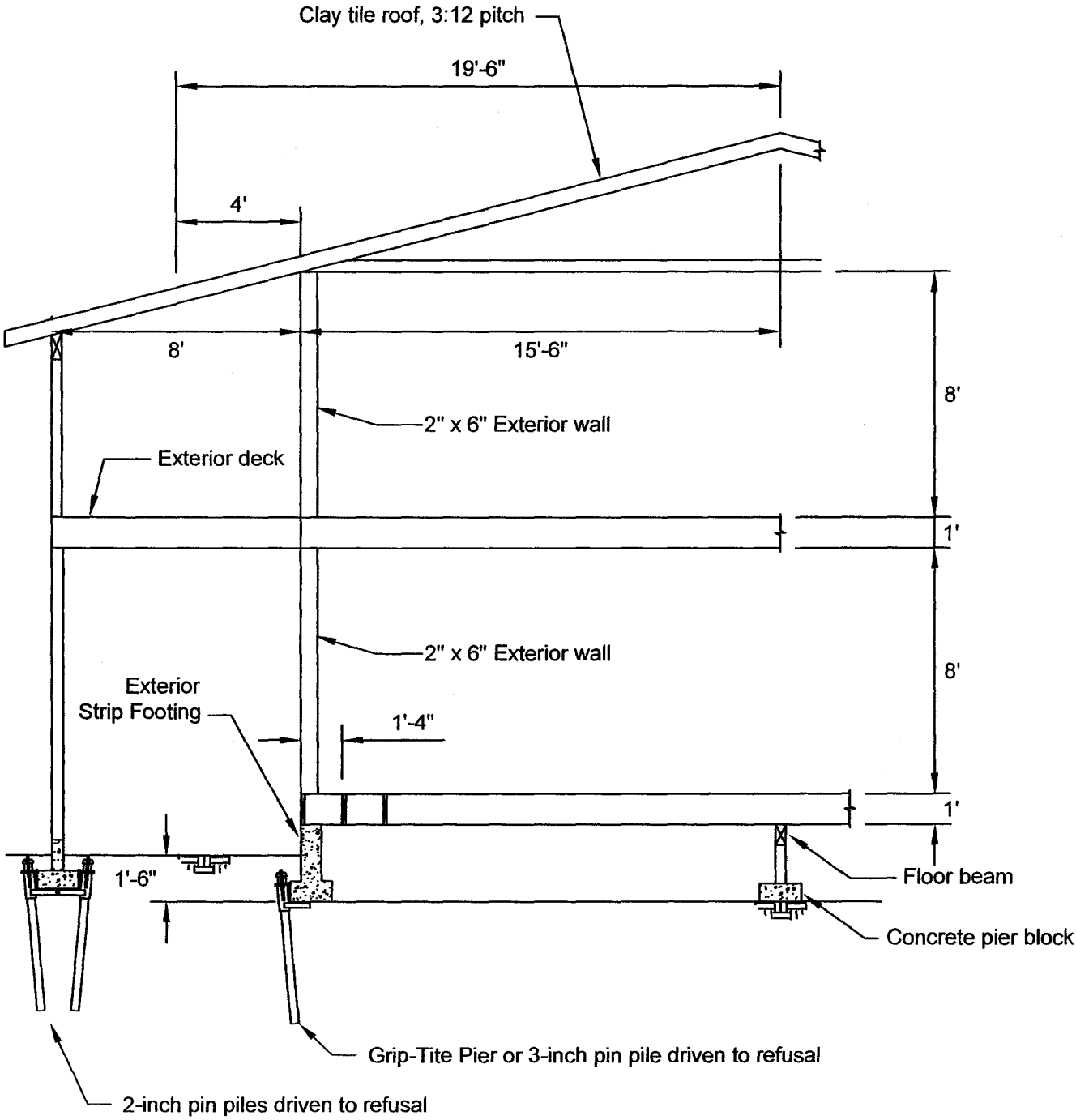
Hauptman Residence
Foundation Repair

Otto Rosenau & Associates
6747 M.L.King Way South
Seattle, WA 98118-3216
(206) 725-4600

Anthony Coyne, P.E
5/16/2006

1

TYPICAL BUILDING CROSS-SECTION AT NORTH WALL NEAR NORTHWEST CORNER



Otto Rosenau Associates, Inc.

Project: Hauptman Residence, North wall at northwest corner
Prepared By: Anthony Coyne, P.E.

Date: 5/17/2006

Dead Loads

Exterior Walls (including sheathing and siding)	Unit Weight (lb/sf)	Height (ft)	Weight (plf)	
1st floor 2 x 6	16	8	128	
2nd floor 2 x 6	16	8	128	
		Total	256	psf

Roof	Unit Weight (lb/sf)	Tributary Width (ft)	Tributary Length (ft)	Tributary Area (sf)	Weight (psf)	
Wood rafters and sheathing 2" x 6"	15	1	19.5	19.5	292.5	
Roofing (Cement Tile)	16	1	19.5	19.5	312	
				Total	604.5	psf

Floors	Unit Weight (lb/sf)	Tributary Width (ft)	Tributary Length (ft)	Tributary Area (sf)	Weight (psf)	
1st Floor	16	1	1.3	1.3	20.8	
2nd Floor	16	1	15.5	15.5	248	
Exterior deck	12	1	4	4	48	
				Total	316.8	psf

Foundation	Unit Weight (lb/sf)	Tributary Width (ft)	Tributary Height (ft)	Tributary Area (sf)	Weight (plf)	
Stem wall	150	0.67	0.83	0.56	84	
Footing	150	1.5	0.67	1.00	150	
				Total	234	psf
				Dead Load Total	1411	psf
				Total Load (D + L)	2731	psf

Live Loads

Snow Loads	Unit Weight (lb/sf)	Tributary Width (ft)	Tributary Length (ft)	Tributary Area (sf)	Weight (psf)	
Code minimum	25	1	19.5	19.5	488	
				Total	488	psf

Floor Live Loads	Unit Weight (lb/sf)	Tributary Width (ft)	Tributary Length (ft)	Tributary Area (sf)	Weight (plf)	
1st Floor	40	1	1.3	1.3	52	
2nd Floor	40	1	15.5	15.5	620	
Exterior deck	40	1	4	4	160	
				Total	832	psf
				Live Load Total	1320	psf

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5/17/2006

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Hauptman Residence
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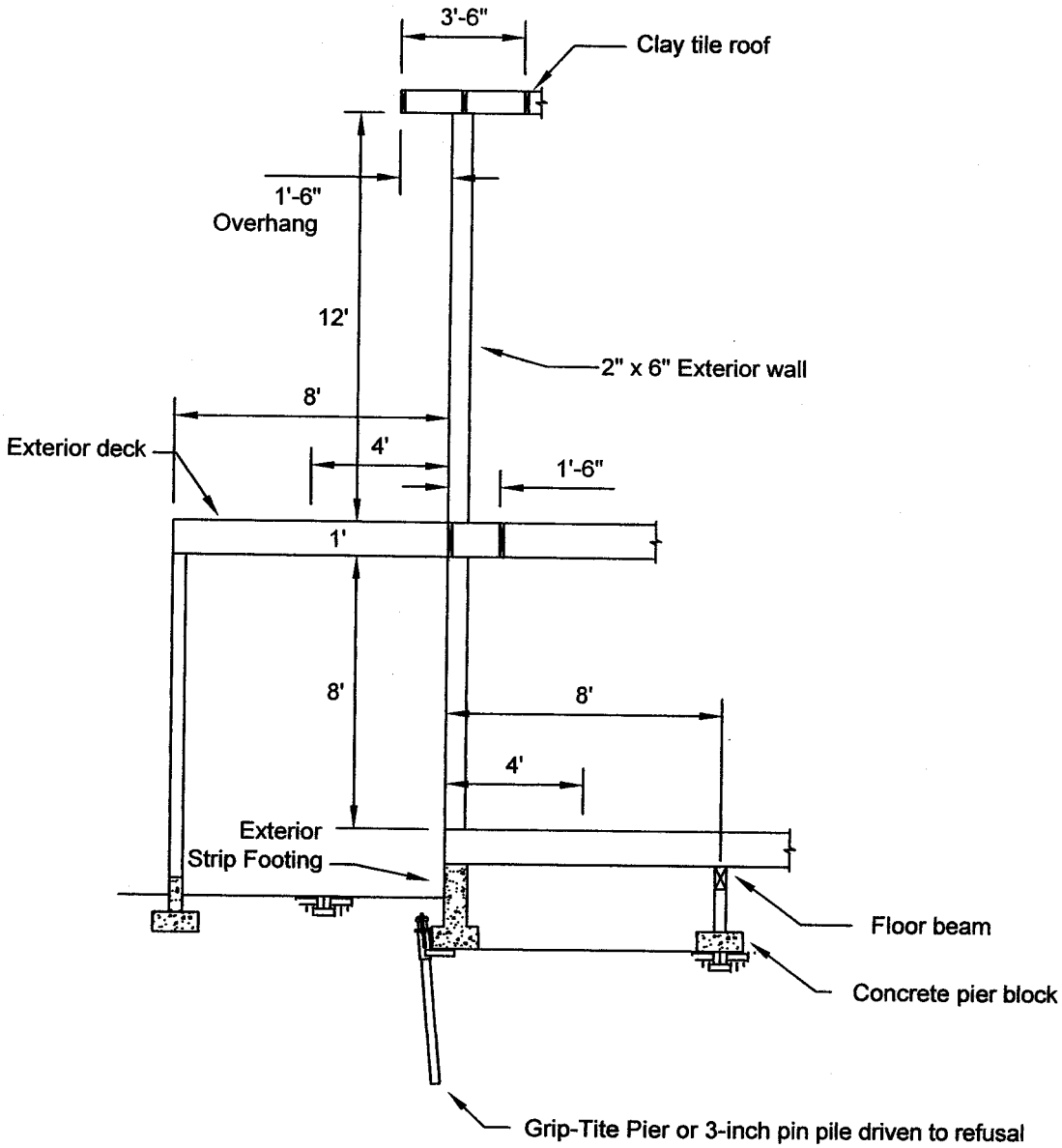
Anthony Coyne, P.E.
5/16/2006

3

TYPICAL BUILDING CROSS-SECTION AT WEST WALL NEAR NORTHWEST CORNER



Otto Rosenau Associates, Inc.

Project: Hauptman Residence, West wall at northwest corner
Prepared By: Anthony Coyne, P.E.

Date: 5/17/2006

Dead Loads

Exterior Walls (including sheathing and siding)	Unit Weight (lb/sf)	Height (ft)	Weight (plf)	
1st floor 2 x 6	16	8	128	
2nd floor 2 x 6	16	12	192	
		Total	320	psf

Roof	Unit Weight (lb/sf)	Tributary Width (ft)	Tributary Length (ft)	Tributary Area (sf)	Weight (psf)	
Wood rafters and sheathing 2" x 6"	15	1	3.5	3.5	52.5	
Roofing (Cement Tile)	16	1	3.5	3.5	56	
				Total	108.5	psf

Floors	Unit Weight (lb/sf)	Tributary Width (ft)	Tributary Length (ft)	Tributary Area (sf)	Weight (psf)	
1st Floor	16	1	4	4	64	
2nd Floor	16	1	1.3	1.3	20.8	
Exterior deck	12	1	4	4	48	
				Total	132.8	psf

Foundation	Unit Weight (lb/sf)	Tributary Width (ft)	Tributary Height (ft)	Tributary Area (sf)	Weight (plf)	
Stem wall	150	0.67	1.83	1.23	184	
Footing	150	1.5	0.67	1.01	151	
				Total	334.7	psf
				Dead Load Total	896	psf
				Total Load (D + L)	1343	psf

Live Loads

Snow Loads	Unit Weight (lb/sf)	Tributary Width (ft)	Tributary Length (ft)	Tributary Area (sf)	Weight (psf)	
Code minimum	25	1	3.5	3.5	87.5	
				Total	87.5	psf

Floor Live Loads	Unit Weight (lb/sf)	Tributary Width (ft)	Tributary Length (ft)	Tributary Area (sf)	Weight (plf)	
1st Floor	40	1	4	4	160	
2nd Floor	40	1	1	1	40	
Exterior deck	40	1	4	4	160	
				Total	360	psf
				Live Load Total	447.5	psf

Hauptman Residence
Foundation Repair

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5/17/2006

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Anthony Coyne, P.E.
5/17/2006

STAEDTLER® No. 937 811E Engineer's Computation Pad

GRIP-TITE PIER PROPERTIES

- 3-inch O.D. X 0.120 in wall thickness
  ⇒ Area of steel, $A_s = 7.07 in^2 - 5.98 in^2 = 1.09 in^2$
- Yield Stress of steel, $F_y = 70,000$ psi, Grade A513 steel

Per UBC 1997 1808.6.2, the maximum allowable stress in a pile is 35% to 50% of the yield stress

⇒ Allowable capacity $= 0.35 F_y A_s$

$= 0.35 (70,000 \frac{lb}{in^2})(1.09 in^2)$

$= 26,705$ lb ≈ 26.7 kips

Per ICC ES 22-02 Legacy Report, the maximum allowable capacity per pier is 16,500 lb = 16.5 kips

Max calculated load per foot of north wall = 2730 lb
Apply factor of safety of 1.5 to load = 4,095 lb

Max Grip Tite pier spacing $= \frac{16,500 \text{ lb}}{4,095 \text{ lb/ft}} = 4.02$ ft ≈ 4 ft.

Max calculated load per foot of west wall = 1343 lb
Apply factor of safety of 1.5 to load = 2014 lb

Max. Grip-Tite pier spacing $= \frac{16,500 \text{ lb}}{2014} = 8.19$ ft > 6 ft

6 ft max pier spacing per ICC ES 22-02

⇒ Max-allowable load per pier for 6-foot spacing @ west wall $= 2014 \frac{lb}{ft} * 6 ft = \underline{12,084 \text{ lb}}$

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5/17/2006

STAEDTLER® No. 937 811E Engineer's Computation Pad

3-INCH DIAMETER PIN PILES

Allowable downward pile capacity per pile = 12,000 pounds
Note: must be verified by pile load test
For 3-inch Sch. 40 pipe $A_s = 2.2285\ in^2$

F_y of Grade A53A steel = 30,000 psi

per UBC 1997 1808.6.2, the maximum allowable stress in a pile is 35% to 50% of the yield stress

$\Rightarrow$ Allowable capacity $= 0.35\ F_y\ A_s$

$$= 0.35\left(30{,}000\ \frac{lb}{in^2}\right)\left(2.2285\ in^2\right)$$

$$= 23{,}399\ lb \text{ per pile}$$

$$23{,}399\ lb < 12{,}000\ lb \Rightarrow OK$$

Max calculated load per foot of wall (N-S): 2730 lb
Apply factor of safety of 1.5 to load $\Rightarrow$ 4,095 lb

$\Rightarrow$ Max pin pile spacing at N-S walls $= \dfrac{12{,}000\ lb}{4{,}095\ lb/ft} = 2.93\ ft \approx 3.0'$

Max calculated load per foot of wall (E-W): = 1343 lb
Apply factor of safety of 1.5 to load $\Rightarrow$ 2014 lb

$\Rightarrow$ Max pin pile spacing at E-W walls $= \dfrac{12{,}000\ lb}{2{,}014\ lb/ft} = 5.95\ ft \approx 6.0'$

Hauptman Residence

XSTABL SLOPE STABILITY ANALYSIS RESULTS

Section A-A' (West wall NW corner of residence)

CASE 1 – Static Loading Conditions – No earthquake forces applied, existing site conditions

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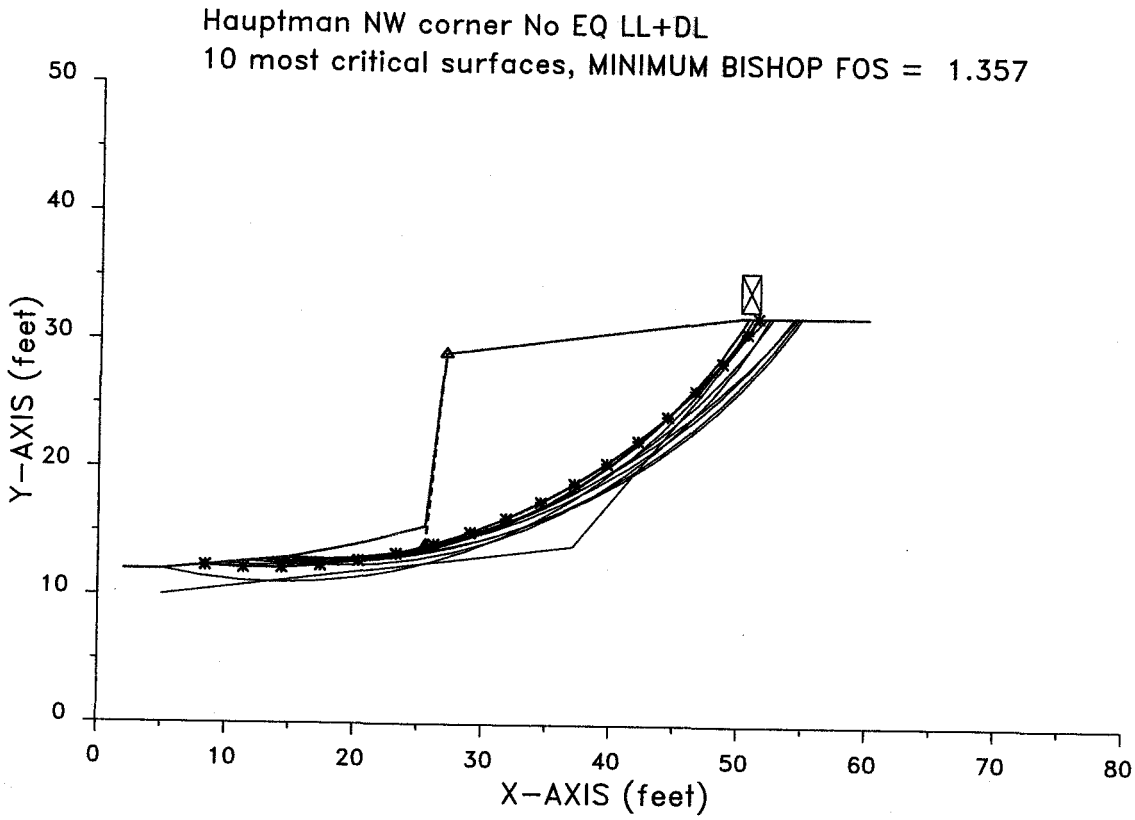

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*              X S T A B L              *
*                                       *
*        Slope Stability Analysis       *
*               using the               *
*            Method of Slices           *
*                                       *
*        Copyright (C) 1992 - 2002      *
*   Interactive Software Designs, Inc.  *
*        Moscow, ID 83843, U.S.A.       *
*                                       *
*          All Rights Reserved          *
*                                       *
*  Ver. 5.206                 96 - 1962 *
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Problem Description : Hauptman NW corner No EQ LL+DL

SEGMENT BOUNDARY COORDINATES
---

7 SURFACE boundary segments

Segment No.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)	Soil Unit Below Segment
1	2.0	12.0	5.0	12.0	1
2	5.0	12.0	15.0	13.0	1
3	15.0	13.0	20.0	14.0	1
4	20.0	14.0	25.5	15.5	1
5	25.5	15.5	27.0	29.0	1
6	27.0	29.0	50.0	32.0	1
7	50.0	32.0	60.0	32.0	1

4 SUBSURFACE boundary segments

Segment No.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)	Soil Unit Below Segment
1	5.0	10.0	37.0	14.0	2
2	37.0	14.0	47.0	26.0	2
3	47.0	26.0	50.0	30.0	2
4	50.0	30.0	52.0	32.0	2

ISOTROPIC Soil Parameters
---

2 Soil unit(s) specified

Soil Unit No.	Unit Weight Moist (pcf)	Unit Weight Sat. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Parameter Ru	Pore Pressure Constant (psf)	Water Surface No.
1	100.0	100.0	50.0	32.00	.000	.0	0
2	115.0	115.0	50.0	36.00	.000	.0	0

BOUNDARY LOADS
---

1 load(s) specified

Load No.	x-left (ft)	x-right (ft)	Intensity (psf)	Direction (deg)
1	50.0	51.5	1348.0	.0

NOTE - Intensity is specified as a uniformly distributed force acting on a HORIZONTALLY projected surface.

BOUNDARIES THAT LIMIT SURFACE GENERATION HAVE BEEN SPECIFIED

UPPER limiting boundary of 1 segments:

Segment No.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)
1	25.5	14.0	27.0	29.0

A critical failure surface searching method, using a random technique for generating CIRCULAR surfaces has been specified.

100 trial surfaces will be generated and analyzed.

10 Surfaces initiate from each of 10 points equally spaced along the ground surface between x = 5.0 ft
and x = 15.0 ft

Each surface terminates between x = 50.0 ft
and x = 60.0 ft

Unless further limitations were imposed, the minimum elevation at which a surface extends is y = 5.0 ft

* * * * * DEFAULT SEGMENT LENGTH SELECTED BY XSTABL * * * * *

3.0 ft line segments define each trial failure surface.

ANGULAR RESTRICTIONS

The first segment of each failure surface will be inclined within the angular range defined by :

Lower angular limit := -45.0 degrees
Upper angular limit := (slope angle - 5.0) degrees

Factors of safety have been calculated by the :

* * * * * SIMPLIFIED BISHOP METHOD * * * * *

The most critical circular failure surface is specified by 18 coordinate points

Point No.	x-surf (ft)	y-surf (ft)
1	8.33	12.33
2	11.33	12.16
3	14.33	12.17
4	17.32	12.37
5	20.30	12.76
6	23.24	13.34
7	26.14	14.10
8	28.99	15.04
9	31.78	16.15

10	34.49	17.44
11	37.11	18.89
12	39.64	20.51
13	42.06	22.28
14	44.36	24.20
15	46.54	26.27
16	48.59	28.46
17	50.49	30.78
18	51.37	32.00

**** Simplified BISHOP FOS = 1.357 ****

The following is a summary of the TEN most critical surfaces

Problem Description : Hauptman NW corner No EQ LL+DL

	FOS (BISHOP)	Circle Center x-coord (ft)	Circle Center y-coord (ft)	Radius (ft)	Initial x-coord (ft)	Terminal x-coord (ft)	Resisting Moment (ft-lb)
1.	1.357	12.61	59.96	47.82	8.33	51.37	8.679E+05
2.	1.381	15.67	53.44	40.97	11.67	50.56	7.143E+05
3.	1.384	15.95	52.93	40.49	11.67	50.58	7.151E+05
4.	1.410	16.04	57.13	44.29	13.89	52.51	8.612E+05
5.	1.451	18.43	51.34	38.97	12.78	52.23	8.441E+05
6.	1.456	14.11	53.90	42.88	5.00	50.96	9.173E+05
7.	1.485	12.57	66.56	54.20	9.44	54.32	1.128E+06
8.	1.486	15.52	61.03	48.33	12.78	54.14	1.024E+06
9.	1.521	16.99	58.56	45.97	12.78	54.51	1.040E+06
10.	1.539	18.17	57.46	44.58	15.00	54.74	1.014E+06

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Hauptman Residence

XSTABL SLOPE STABILITY ANALYSIS RESULTS

Section A-A' (West wall NW corner of residence)

CASE 2 – Seismic Loading Conditions 30 % g – existing site conditions

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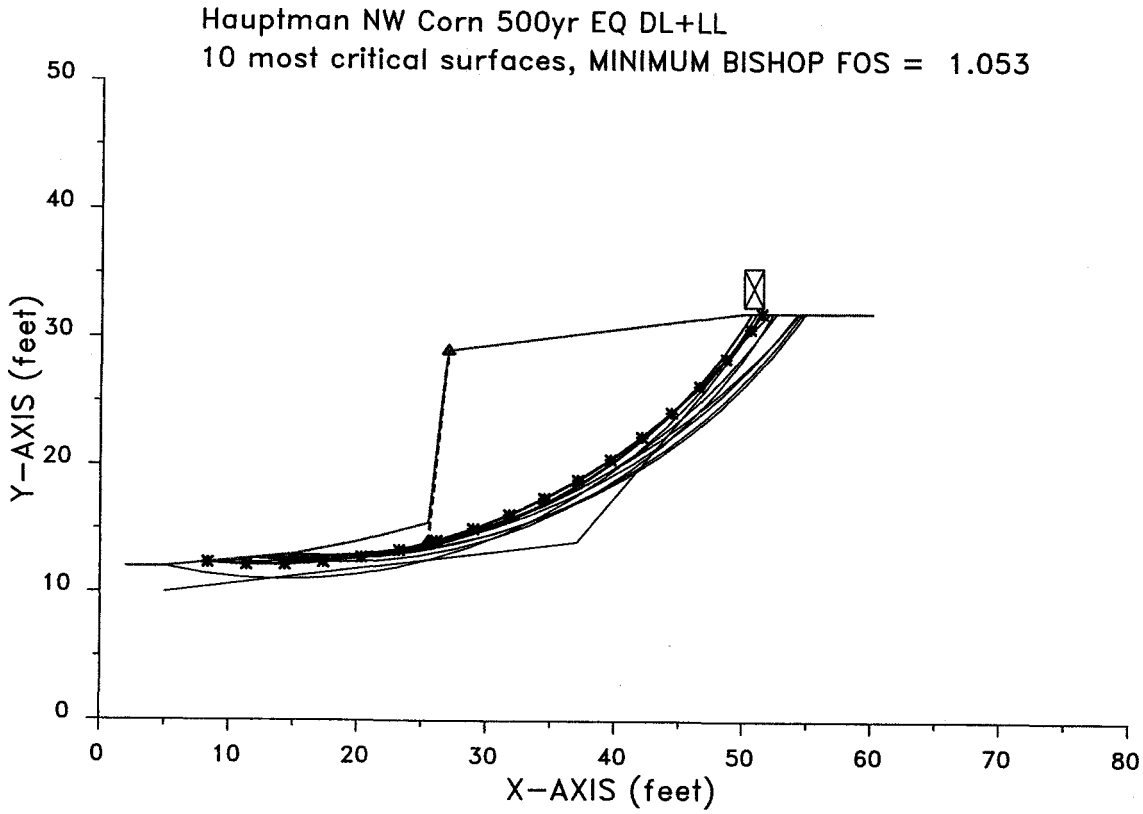

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*                                       *
*  Ver. 5.206                96 - 1962  *
*****************************************
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Problem Description : Hauptman NW Corn 500yr EQ DL+LL

SEGMENT BOUNDARY COORDINATES
---

7 SURFACE boundary segments

Segment No.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)	Soil Unit Below Segment
1	2.0	12.0	5.0	12.0	1
2	5.0	12.0	15.0	13.0	1
3	15.0	13.0	20.0	14.0	1
4	20.0	14.0	25.5	15.5	1
5	25.5	15.5	27.0	29.0	1
6	27.0	29.0	50.0	32.0	1
7	50.0	32.0	60.0	32.0	1

4 SUBSURFACE boundary segments

Segment No.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)	Soil Unit Below Segment
1	5.0	10.0	37.0	14.0	2
2	37.0	14.0	47.0	26.0	2
3	47.0	26.0	50.0	30.0	2
4	50.0	30.0	52.0	32.0	2

ISOTROPIC Soil Parameters

---

2 Soil unit(s) specified

Water Surface No.	Soil Unit No.	Unit Weight Moist (pcf)	Unit Weight Sat. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Parameter Ru	Pore Pressure Constant (psf)
0	1	100.0	100.0	50.0	32.00	.000	.0
0	2	115.0	115.0	50.0	36.00	.000	.0

A horizontal earthquake loading coefficient
of .148 has been assigned

A vertical earthquake loading coefficient
of .000 has been assigned

BOUNDARY LOADS
---

1 load(s) specified

Load No.	x-left (ft)	x-right (ft)	Intensity (psf)	Direction (deg)
1	50.0	51.5	1348.0	.0

NOTE - Intensity is specified as a uniformly distributed
force acting on a HORIZONTALLY projected surface.

BOUNDARIES THAT LIMIT SURFACE GENERATION HAVE BEEN SPECIFIED

UPPER limiting boundary of 1 segments:

Segment No.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)
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and x = 15.0 ft

Each surface terminates between x = 50.0 ft
and x = 60.0 ft

Unless further limitations were imposed, the minimum elevation
at which a surface extends is y = 5.0 ft

* * * * * DEFAULT SEGMENT LENGTH SELECTED BY XSTABL * * * * *

3.0 ft line segments define each trial failure surface.

ANGULAR RESTRICTIONS

The first segment of each failure surface will be inclined within the angular range defined by :

Lower angular limit := -45.0 degrees
Upper angular limit := (slope angle - 5.0) degrees

Factors of safety have been calculated by the :

* * * * * SIMPLIFIED BISHOP METHOD * * * * *

The most critical circular failure surface is specified by 18 coordinate points

Point No.	x-surf (ft)	y-surf (ft)
1	8.33	12.33
2	11.33	12.16

3	14.33	12.17
4	17.32	12.37
5	20.30	12.76
6	23.24	13.34
7	26.14	14.10
8	28.99	15.04
9	31.78	16.15
10	34.49	17.44
11	37.11	18.89
12	39.64	20.51
13	42.06	22.28
14	44.36	24.20
15	46.54	26.27
16	48.59	28.46
17	50.49	30.78
18	51.37	32.00

**** Simplified BISHOP FOS = 1.053 ****

The following is a summary of the TEN most critical surfaces

Problem Description : Hauptman NW Corn 500yr EQ DL+LL

	FOS (BISHOP)	Circle Center x-coord (ft)	Circle Center y-coord (ft)	Radius (ft)	Initial x-coord (ft)	Terminal x-coord (ft)	Resisting Moment (ft-lb)
1.	1.053	12.61	59.96	47.82	8.33	51.37	8.199E+05
2.	1.068	15.67	53.44	40.97	11.67	50.56	6.750E+05
3.	1.071	15.95	52.93	40.49	11.67	50.58	6.759E+05
4.	1.089	16.04	57.13	44.29	13.89	52.51	8.127E+05
5.	1.127	14.11	53.90	42.88	5.00	50.96	8.705E+05
6.	1.127	18.43	51.34	38.97	12.78	52.23	7.980E+05
7.	1.131	12.57	66.56	54.20	9.44	54.32	1.066E+06
8.	1.136	15.52	61.03	48.33	12.78	54.14	9.684E+05
9.	1.162	16.99	58.56	45.97	12.78	54.51	9.843E+05
10.	1.174	18.17	57.46	44.58	15.00	54.74	9.599E+05

* * * END OF FILE * * *

Hauptman Residence

XSTABL SLOPE STABILITY ANALYSIS RESULTS

Section B-B' (West wall SW corner of residence)

CASE 1 – Static Loading Conditions – No earthquake forces applied, existing site conditions

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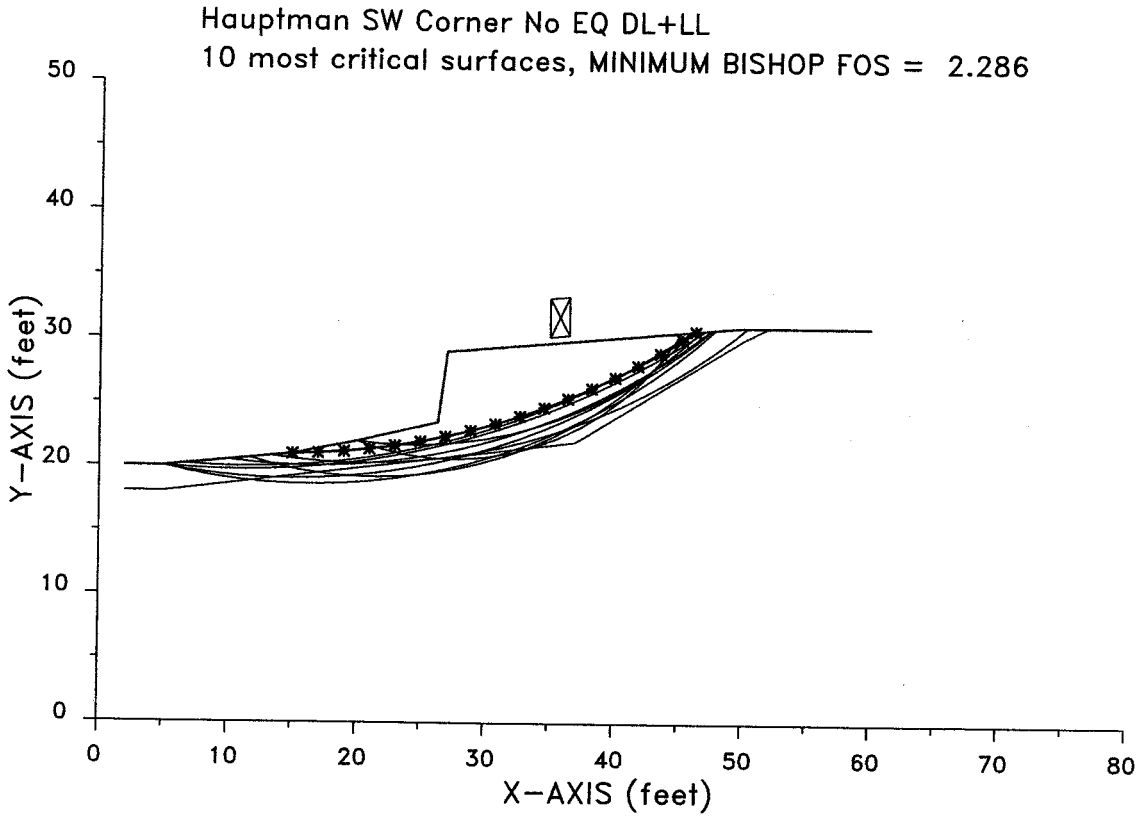

XSTABL File: 1HAUPT9 5-19-06 19:17

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*****************************************
*              X S T A B L              *
*                                       *
*        Slope Stability Analysis       *
*               using the               *
*            Method of Slices           *
*                                       *
*        Copyright (C) 1992 - 2002      *
*   Interactive Software Designs, Inc.  *
*        Moscow, ID 83843, U.S.A.       *
*                                       *
*          All Rights Reserved          *
*                                       *
*  Ver. 5.206                96 - 1962  *
*****************************************
```

Problem Description : Hauptman SW Corner No EQ DL+LL

SEGMENT BOUNDARY COORDINATES
---

8 SURFACE boundary segments

Segment No.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)	Soil Below Unit Segment
1	2.0	20.0	5.0	20.0	1
2	5.0	20.0	15.0	21.0	1
3	15.0	21.0	20.0	22.0	1
4	20.0	22.0	26.3	23.5	1
5	26.3	23.5	27.0	29.0	1
6	27.0	29.0	50.0	31.0	1
7	50.0	31.0	52.0	31.0	1
8	52.0	31.0	60.0	31.0	1

5 SUBSURFACE boundary segments

Segment No.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)	Soil Below Unit Segment
1	2.0	18.0	5.0	18.0	2
2	5.0	18.0	37.0	22.0	2
3	37.0	22.0	50.0	30.0	2
4	50.0	30.0	52.0	31.0	2
5	52.0	31.0	60.0	31.0	2

ISOTROPIC Soil Parameters
---

2 Soil unit(s) specified

Soil Unit No.	Unit Weight Moist (pcf)	Unit Weight Sat. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Parameter Ru	Pore Pressure Constant (psf)	Water Surface No.
1	100.0	100.0	50.0	32.00	.000	.0	0
2	115.0	115.0	50.0	36.00	.000	.0	0

BOUNDARY LOADS
---

1 load(s) specified

Load No.	x-left (ft)	x-right (ft)	Intensity (psf)	Direction (deg)
1	35.0	36.5	1348.0	.0

NOTE - Intensity is specified as a uniformly distributed force acting on a HORIZONTALLY projected surface.

A critical failure surface searching method, using a random technique for generating CIRCULAR surfaces has been specified.

100 trial surfaces will be generated and analyzed.

10 Surfaces initiate from each of 10 points equally spaced along the ground surface between x = 5.0 ft
and x = 20.0 ft

Each surface terminates between x = 45.0 ft
and x = 55.0 ft

Unless further limitations were imposed, the minimum

elevation
at which a surface extends is y = 5.0 ft

* * * * * DEFAULT SEGMENT LENGTH SELECTED BY XSTABL * * * *
*

2.0 ft line segments define each trial failure surface.

ANGULAR RESTRICTIONS

The first segment of each failure surface will be inclined within the angular range defined by :

Lower angular limit := -45.0 degrees
Upper angular limit := (slope angle - 5.0) degrees

Factors of safety have been calculated by the :

* * * * * SIMPLIFIED BISHOP METHOD * * * * *

The most critical circular failure surface is specified by 18 coordinate points

Point No.	x-surf (ft)	y-surf (ft)
1	15.00	21.00
2	17.00	21.06
3	18.99	21.19
4	20.98	21.39
5	22.97	21.66
6	24.94	21.99
7	26.90	22.40
8	28.84	22.87
9	30.77	23.41
10	32.67	24.01
11	34.56	24.68
12	36.42	25.41
13	38.25	26.21
14	40.06	27.07
15	41.84	27.99
16	43.58	28.97
17	45.29	30.01
18	46.30	30.68

**** Simplified BISHOP FOS = 2.286 ****

The following is a summary of the TEN most critical surfaces

Problem Description : Hauptman SW Corner No EQ DL+LL

	FOS (BISHOP)	Circle Center x-coord (ft)	Circle Center y-coord (ft)	Radius (ft)	Initial x-coord (ft)	Terminal x-coord (ft)	Resisting Moment (ft-lb)
1.	2.286	14.21	79.07	58.07	15.00	46.30	4.705E+05
2.	2.344	16.50	70.24	50.16	10.00	47.46	5.416E+05
3.	2.345	11.37	81.52	61.54	6.67	45.99	5.791E+05
4.	2.392	24.30	54.17	32.46	20.00	46.70	2.961E+05
5.	2.411	21.26	51.24	32.04	11.67	45.78	4.328E+05
6.	2.434	10.83	84.85	65.11	5.00	47.05	6.712E+05
7.	2.518	14.78	70.90	51.83	5.00	47.60	6.916E+05
8.	2.529	20.43	67.54	47.24	13.33	50.36	5.841E+05
9.	2.569	27.43	41.40	20.78	20.00	45.13	2.343E+05
10.	2.607	16.30	65.56	46.94	5.00	47.86	7.169E+05

* * * END OF FILE * * *

Hauptman Residence

XSTABL SLOPE STABILITY ANALYSIS RESULTS

Section B-B' (West wall SW corner of residence)

CASE 2 – Seismic Loading Conditions 30 % g – existing site conditions

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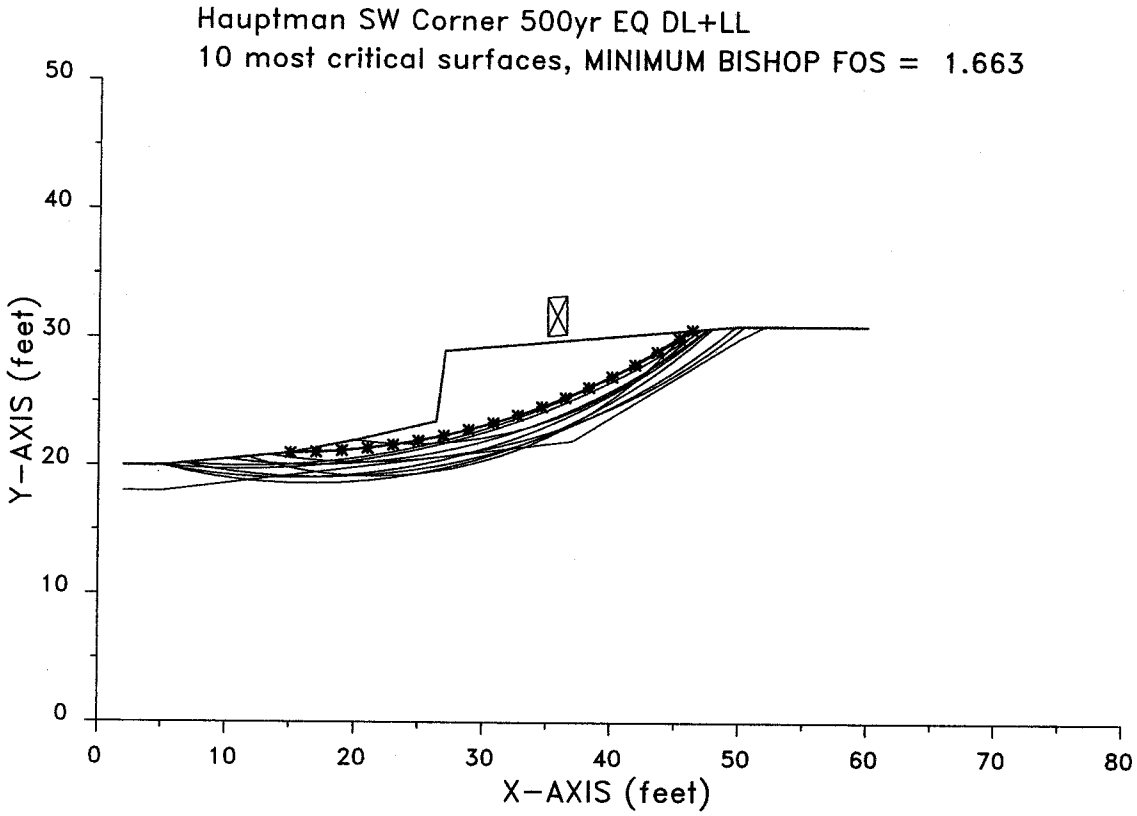

XSTABL File: 1HAUPT8 5-19-06 19:16

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*****************************************
*              X S T A B L              *
*                                       *
*        Slope Stability Analysis       *
*               using the               *
*            Method of Slices           *
*                                       *
*        Copyright (C) 1992 - 2002      *
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*        Moscow, ID 83843, U.S.A.       *
*                                       *
*          All Rights Reserved          *
*                                       *
*  Ver. 5.206                96 - 1962  *
*****************************************
```

Problem Description : Hauptman SW Corner 500yr EQ DL+LL

SEGMENT BOUNDARY COORDINATES
---

8 SURFACE boundary segments

Segment No.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)	Soil Below Unit Segment
1	2.0	20.0	5.0	20.0	1
2	5.0	20.0	15.0	21.0	1
3	15.0	21.0	20.0	22.0	1
4	20.0	22.0	26.3	23.5	1
5	26.3	23.5	27.0	29.0	1
6	27.0	29.0	50.0	31.0	1
7	50.0	31.0	52.0	31.0	1
8	52.0	31.0	60.0	31.0	1

5 SUBSURFACE boundary segments

Segment No.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)	Soil Below Unit Segment
1	2.0	18.0	5.0	18.0	2
2	5.0	18.0	37.0	22.0	2
3	37.0	22.0	50.0	30.0	2
4	50.0	30.0	52.0	31.0	2
5	52.0	31.0	60.0	31.0	2

ISOTROPIC Soil Parameters
---

2 Soil unit(s) specified

Soil Unit No.	Unit Weight Moist (pcf)	Unit Weight Sat. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Parameter Ru	Pore Pressure Constant (psf)	Water Surface No.
1	100.0	100.0	50.0	32.00	.000	.0	0
2	115.0	115.0	50.0	36.00	.000	.0	0

A horizontal earthquake loading coefficient
of .148 has been assigned

A vertical earthquake loading coefficient
of .000 has been assigned

BOUNDARY LOADS
---

1 load(s) specified

Load No.	x-left (ft)	x-right (ft)	Intensity (psf)	Direction (deg)
1	35.0	36.5	1343.0	.0

NOTE - Intensity is specified as a uniformly distributed force acting on a HORIZONTALLY projected surface.

A critical failure surface searching method, using a random technique for generating CIRCULAR surfaces has been specified.

100 trial surfaces will be generated and analyzed.

10 Surfaces initiate from each of 10 points equally spaced along the ground surface between x = 5.0 ft

and x = 20.0 ft

Each surface terminates between x = 45.0 ft
and x = 55.0 ft

Unless further limitations were imposed, the minimum elevation
at which a surface extends is y = 5.0 ft

* * * * * DEFAULT SEGMENT LENGTH SELECTED BY XSTABL * * * * *

2.0 ft line segments define each trial failure surface.

ANGULAR RESTRICTIONS

The first segment of each failure surface will be inclined within the angular range defined by :

Lower angular limit := -45.0 degrees
Upper angular limit := (slope angle - 5.0) degrees

Factors of safety have been calculated by the :

* * * * * SIMPLIFIED BISHOP METHOD * * * * *

The most critical circular failure surface is specified by 18 coordinate points

Point No.	x-surf (ft)	y-surf (ft)
1	15.00	21.00
2	17.00	21.06
3	18.99	21.19
4	20.98	21.39
5	22.97	21.66
6	24.94	21.99
7	26.90	22.40
8	28.84	22.87
9	30.77	23.41
10	32.67	24.01
11	34.56	24.68

12	36.42	25.41
13	38.25	26.21
14	40.06	27.07
15	41.84	27.99
16	43.58	28.97
17	45.29	30.01
18	46.30	30.68

**** Simplified BISHOP FOS = 1.663 ****

The following is a summary of the TEN most critical surfaces

Problem Description : Hauptman SW Corner 500yr EQ DL+LL

	FOS (BISHOP)	Circle Center x-coord (ft)	Circle Center y-coord (ft)	Radius (ft)	Initial x-coord (ft)	Terminal x-coord (ft)	Resisting Moment (ft-lb)
1.	1.663	14.21	79.07	58.07	15.00	46.30	4.557E+05
2.	1.670	16.50	70.24	50.16	10.00	47.46	5.247E+05
3.	1.696	11.37	81.52	61.54	6.67	45.99	5.615E+05
4.	1.715	24.30	54.17	32.46	20.00	46.70	2.873E+05
5.	1.733	10.83	84.85	65.11	5.00	47.05	6.513E+05
6.	1.747	21.26	51.24	32.04	11.67	45.78	4.198E+05
7.	1.749	20.43	67.54	47.24	13.33	50.36	5.667E+05
8.	1.769	14.78	70.90	51.83	5.00	47.60	6.711E+05
9.	1.821	16.30	65.56	46.94	5.00	47.86	6.959E+05
10.	1.836	16.95	70.20	51.08	6.67	49.66	7.382E+05

* * * END OF FILE * * *