Geo Engineers

May 3, 1999

Consulting Engineers
and Geoscientists
Offices in Washington,
Oregon, and Alaska

Thurston County Development Services
Building 1, Second Floor
2000 Lakeridge Drive SW
Olympia, Washington 98502

Attention: Don Krupp, Director

We are pleased to submit our "Report, Phase II Geotechnical Study, Carlyon Beach/Hunter Point Landslide, Thurston County, Washington."

Our services for this phase of study were requested on March 22, 1999 by Don Krupp, Director of Thurston County Development Services, following submittal of our Phase I report. The scope of our services is included as "Attachment B" to our contract with Thurston County dated March 15, 1999. The scope of services was developed during discussions with Thurston County.

We appreciate this opportunity to be of service to Thurston County on this project. Please call us if you have questions regarding this report, or if you require additional information.

Yours very truly,

GeoEngineers, Inc.

[signature] for

Jon W. Koloski
Principal

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THURSTON COUNTY, WASHINGTON
FOR
THURSTON COUNTY DEVELOPMENT SERVICES

EXECUTIVE SUMMARY

Above-normal precipitation during the past five years exceeds the 58-year average by between 3 and 65 percent and has caused the reactivation of an ancient landslide along the northern portion of the Steamboat Island Peninsula in northwest Thurston County, Washington. The landslide feature extends for 3,000 feet along the shoreline of Squaxin Passage and extends inland 900 feet. Forty-one homes have been damaged due to ground movement within the slide mass that began in early February 1999. Occupants of the homes have been requested to evacuate by Thurston County due to structural damage. Scarps within the landslide range up to 15 feet in height and many show horizontal offset. The headscarp of the landslide currently threatens the integrity of Hunter Point Road.

Thurston County Development Services authorized GeoEngineers in early March 1999 to perform a Phase I reconnaissance-level assessment of ground movement within the slide area and provide recommendations to the County. The results of GeoEngineers' Phase I study are presented in our report dated March 18, 1999.

Thurston County Development Services requested GeoEngineers undertake a Phase II geotechnical study of the landslide on March 22. The purpose of our Phase II Evaluation was to conduct subsurface explorations, monitor ground movement and ground water levels within and adjacent to the active landslide, and develop a series of options and associated costs to mitigate ground movement within the landslide.

GeoEngineers drilled 22 geotechnical soil borings to evaluate subsurface conditions within and adjacent to the landslide. We encountered glacially consolidated sand up to 60 feet thick in the area of Hunter Point Road. The sand overlies glacially consolidated silt and clay up to 200 feet thick. An unknown thickness of sand and gravel was encountered beneath the silt and clay unit.

Ground water occurs as a shallow, unconfined water table aquifer in the upper sand and as a confined aquifer in the lower sand and gravel. A water-bearing zone within the silt/clay unit was also observed in several borings at, or near, the location of the presumed failure plane. The shallow aquifer appears to be contributing a large quantity of ground water to the central portion of the landslide and may be a large contributory factor to movement. It does not appear that the confined sand and gravel aquifer is hydraulically connected to the landslide failure plane.

Twelve of the borings were completed as slope inclinometers or poor boys and ten of the borings were completed as ground water piezometers. Representative soil samples were collected from the borings at 5 to 10-foot intervals during drilling for classification of the geologic materials and for laboratory analysis. Movement within the landslide has resulted in crimping or shearing of seven of the inclinometer/poor boy casings since installation. Crimp depths range between 32 and 105 feet below ground surface and indicate multiple failure zones within the landslide.

Repeat measurements of slope inclinometer casings located outside the active portion of the landslide are continuing. Initial measurements may indicate some movement within the slope inclinometer casings, but are within the precision of the instrument and no conclusions may be drawn at this time.

SCA Engineering of Lacey, Washington has conducted repeat surveys of a network of reference points within the landslide. Results indicate that the zone of greatest movement (up to 1.4 feet over 3 weeks) occurs in the center of the landslide with less movement occurring at the eastern and western edges. Movement of reference points located outside the landslide near the headscarp is within the precision of the survey and is inconclusive at this time.

GeoEngineers conducted stability analyses of the landslide using the computer-modeling program XSTABL. The program identifies the most critical surface of failure within the landslide given the topography, soil properties, and ground water conditions. The stability is reported as a factor of safety, which is the ratio of driving forces over resisting forces. A factor of safety of 1.0 indicates marginal stability. XSTABL was also used to evaluate the factor of safety provided by several of the proposed mitigation options.

The landslide has continued to move since March 23, 1999 based on individual scarp measurements, survey data and poor boy measurements. It is our opinion that the landslide will continue to move until equilibrium conditions are restored, either by natural or engineered means. If no mitigation action is taken, we anticipate that regression of the headscarp toward the south will occur in the form of back-tilted blocks 50- to 100-feet wide. Southward regression may eventually involve currently unaffected homes on Lookout Drive and Hunter Point Road, and Hunter Point Road itself.

Mitigation scenarios include hydraulic controls to reduce ground water flow into or through the landslide and structural controls to resist movement. All scenario costs include estimated repairs to houses, infrastructure repairs and drainage improvements. We have estimated costs to control movement within the entire landslide. Portions of the landslide may be able to be stabilized using modified applications of these scenarios. We recommend that multiple and redundant measures be used to control ground movement. Remedial alternatives and associated costs are outlined in the following summary table.

SUMMARY OF REMEDIAL ALTERNATIVES

Remedial Alternative	Achievable Factor of Safety	Estimated Total Cost (millions)	Average Cost Per Affected Household
Hydraulic Controls			
Extraction Wells	1.3	$5.1 to $5.6	$140,000
Cutoff Wall	1.3	$4.0 to $4.6	$113,000

SUMMARY OF REMEDIAL ALTERNATIVES (CONTINUED)

Remedial Alternative	Achievable Factor of Safety	Estimated Total Cost (millions)	Average Cost Per Affected Household
Drains	1.1	$5.8	$153,000
Structural Controls			
Regrading	1.3+	$39	$1,000,000
Toe Buttress	1.4	$27.5	$723,000
Drilled-Pile Wall	Not Calculated	$10 to $15	$329,000

The costs presented in the above summary table, and within the report, are preliminary estimates and conceptual in nature. These preliminary cost estimates may differ from actual costs required to stabilize the landslide. We recommend that more detailed evaluations be conducted to refine the cost estimates and to incorporate data that has not been considered in our evaluations.

REPORT
PHASE II GEOTECHNICAL STUDY
CARLYON BEACH/HUNTER POINT LANDSLIDE
THURSTON COUNTY, WASHINGTON
FOR
THURSTON COUNTY DEVELOPMENT SERVICES

INTRODUCTION

This report summarizes our Phase II Geotechnical Study of the Carlyon Beach/Hunter Point landslide located in northwestern Thurston County, Washington. The landslide is located along the northern end of the Steamboat Island Peninsula and includes a portion of the private community of Carlyon Beach as well as a number of rural residential dwellings along Northwest Hunter Point Road. The area of the landslide slopes down to the shoreline and the marine waters of Squaxin Passage as shown in the Vicinity Map, Figure 1.

The purpose of our Phase II Evaluation was to conduct subsurface explorations and monitor ground movement and ground water levels within, and adjacent to, the active landslide. Our goals included developing an understanding of the geometry of the landslide, estimating the driving and resisting forces acting on the landslide, estimating the rate and direction of movement and developing a tiered set of actions based on the implementation of various levels of mitigative effort. Our services for this phase of work were conducted under contract to Thurston County Development Services and were authorized by Mr. Don Krupp, Director of Development Services.

Our services have been conducted over a relatively short time period and the results of our evaluations should be viewed in light of this constrained approach. The remedial options and associated cost estimates must be considered as preliminary and conceptual in nature. We recommend that our interpretation of subsurface conditions, conceptual remedial options and cost estimates be refined prior to implementing a remedial option and as additional data is available.

GeoEngineers previously completed a reconnaissance-level evaluation of the landslide for Thurston County Development Services. Our report, dated March 18, 1999, documents landslide features observed at that time, presents a discussion of previous studies and ground movement within the study area, events leading to the current ground movement and our recommendations for the Phase II study presented in this report.

Current ground movement in the Carlyon Beach/Hunter Point area appears to be the result of reactivation of an ancient landslide. Ground movement has been observed in areas at the east and west margins of the ancient landslide in recent years. Recent movement of the landslide resulted in cracks and settlement in streets and driveways that were observed beginning around February 6, 1999.

At the time of this report, the landslide encompasses portions of Mariner Drive, Crestridge Drive, Bowline Court and Jibstay Court within Carlyon Beach, as well as several properties located north of Northwest Hunter Point Road in unincorporated Thurston County. The current limit of the landslide, based on surface and subsurface information, is shown relative to surrounding physical features on the Site Plan, Figure 2.

Recent ground movement within the landslide has directly affected a total of 37 residences within Carlyon Beach Estates and four residences outside the private community. Ground movement in the form of subsidence and lateral offset has resulted in severe distress of building foundations and supports, rendering several homes unsuitable for occupation. Numerous homes have experienced differential settlement, resulting in racking of doorways and windows. Lateral as well as rotational offsets were observed in shoreline bulkheads in the Carlyon Beach and Hunter Point area. The ground movements have damaged streets in Carlyon Beach and currently threaten the stability of the eastern portion of Hunter Point Road.

SCOPE OF SERVICES

The purpose of our Phase II services was to evaluate the landslide geometry, obtain measurements of ground surface and subsurface soil movement, collect soil samples for laboratory testing and perform a computer analysis using collected data to assist in evaluating likely mitigation scenarios and associated costs. The specific tasks conducted for this phase include:

1. Drill 22 geotechnical soil borings to evaluate subsurface soil and ground water conditions within and adjacent to the landslide area. Collect relatively undisturbed soil samples during drilling to evaluate the geologic conditions and to provide samples for laboratory testing. The general locations of the borings are shown in Figure 2.
2. Install inclinometer casings in nine borings to conduct monitoring of slide movement and provide a potential for early warning should headward regression of the slide begin to occur. All inclinometer casings except SI-7 were slotted over the bottom 1 to 5 feet to allow for monitoring of ground water elevations.
3. Install piezometers in 10 borings to allow monitoring of ground water elevations and evaluate changes in ground water levels in response to precipitation events.
4. Install three "poor boy" inclinometers within the landslide. The poor boys consist of a 2-inch diameter PVC casing. A weighted line is installed to the base of the casing. The poor boy is used to evaluate the depth to the failure plane(s) within the landslide. Weighted lines were also installed in all of the inclinometer casings except SI-3.
5. Install survey reference points at various locations within the landslide to evaluate the direction and speed of ground movement. SCA Engineers of Lacey established the survey network and provided periodic monitoring of the reference points.
6. Conduct laboratory tests on selected soil samples to evaluate soil strength properties. Laboratory tests included direct shear, moisture density, Atterberg Limits, and grain size analysis.
7. Monitor the inclinometer casings for movement and the ground water levels in piezometers.
8. Evaluate the cause and characteristics of the landslide movement and the potential for expansion of the landslide using appropriate computer models.

BACKGROUND

HISTORICAL DEVELOPMENT

Carlyon Beach Country Club was first platted as a summer resort and camping area in 1959. By the late 1960s, permanent residences were being constructed on lots within the development. At that time, residences were served by individual, on-site septic systems. A wastewater treatment plant was constructed in 1972 to replace failing septic drainfields and reduce septic-related pollution into Squaxin Passage. Residents maintain enclosed septic holding tanks that are pumped on a regular basis. Effluent from the tanks is hauled by truck to the wastewater treatment plant for processing and discharge. Potable water is provided by several community wells and reservoirs. New construction within the development ceased in the early 1990s when a moratorium was placed on new residences due to the limited capacity of the wastewater treatment plant and water supply system.

Several houses located on Hunter Point Road are evident in the 1953 aerial photographs reviewed as a part of our Phase I study, indicating early development of this area.

PREVIOUS STUDIES

Bradley-Noble Geotechnical Services (Bradley-Noble) was retained in 1996 by Mr. John Arata, owner of the property located at 10127 Mariner Drive NW in Carlyon Beach, to evaluate ground movement in the area between 10103 and 10137 Mariner Drive. Bradley-Noble drilled three borings ranging between 19 and 24 feet below ground surface to evaluate subsurface conditions and found generally loose silt over firm silt. In their July 16, 1996 report, Bradley-Noble surmised that the soil failure appeared to be the result of erosion of soil from the beach area, resulting in reduced resisting forces along a sloping failure plane. Bradley-Noble suggested that the installation of a cutoff drain along Mariner Drive might be useful to divert subsurface water that could be contributing to the soil failure. It is our understanding that this drain was not installed.

During the winter of 1996-97, ground cracking was again observed within the asphalt of Mariner and Crestridge Drives as well as in several homes in Carlyon Beach. JW Morrissette and Associates, Inc. (Morrissette) of Olympia was retained by the Carlyon Beach Homeowners' Association to evaluate the cause of ground failure in the area. In their January 1997 report, Morrissette describes the failure as being attributable to loose, poorly compacted fill soil generated during site grading overlying relatively stiff, glacially consolidated silts in conjunction with a shallow ground water table. At that time, failure appeared to be in the area of an apparent buried drainage.

Morrissette was again retained by the Homeowners' Association in February 1999 to evaluate ground failure in the same area as observed in 1997, along with new failure features observed upslope in the area of Bowline Court NW. In their report dated February 16, 1999, Morrissette concluded that the landslide was a multi-block failure of a colluvial terrace driven by high soil-water pressures resulting from above average rainfall. Morrissette recommended the installation of several geotechnical borings and piezometers along Mariner and Crestridge Drive to evaluate subsurface conditions. It is our understanding that these investigations were not performed.

Bradley-Noble also conducted a geotechnical evaluation of the Dillon property located at 9710 Hunter Point Drive to assess ground cracking and movement in December 1997. Their report also refers to a previous investigation of soil movement in the western portion of the Dillon property. Bradley-Noble installed three piezometers ranging in depth between 19 and 29 feet below ground surface. The causes of ground movement at the Dillon property were attributed to increased precipitation and a resulting rise in the ground water table, removal of fine-grained soil from the subsurface due to ground water piping and lowering of the beach due to erosion from longshore currents.

Squier/HGI Associates conducted a foundation investigation for the Carlyon Beach Country Club Homeowners' Association in the fall of 1998. Their investigation focused on the suitability of soil in the area east of Lookout Drive to support a proposed 350,000-gallon water tank to serve the Carlyon Beach community. Soil conditions encountered in their boring and test pits were described as a thin veneer of sand and gravel overlying dense lacustrine silts.

CURRENT CONDITIONS

Evacuations

Thurston County Development Services began conducting inspections of dwellings affected by soil movement in Carlyon Beach and to the east toward Hunter Point shortly after movement began in February 1999. Inspections included evaluating structural damage and assessing the homes for occupant safety. In all, 41 homes were determined to be unsafe for occupancy and residents were instructed to evacuate the dwellings.

Phase I Results

GeoEngineers conducted a Phase I Reconnaissance-level study of the Carlyon Beach/Hunter Point landslide for Thurston County Development Services in March 1999. Our Phase I study revealed that the feature was an ancient landslide that had been reactivated and experienced soil movement within the past few years. We determined that there was a likelihood of continued movement and recommended additional study including subsurface borings, slope inclinometers, piezometers, surveyed reference points, and periodic monitoring. The results of our Phase I study are contained in our report titled "Phase I Reconnaissance Evaluation, Carlyon Beach/Hunter Point Landslide" dated March 18, 1999.

Changes Since Phase I Report

Ground movement within the landslide area has continued since our Phase I report. During our field activities we have noted the appearance of new surface features such as cracks and scarps and ongoing movement on existing features. Uplift of clay along the beach at the toe of the landslide has resulted in erosion of the normal beach material such as sand and gravel, and distress of concrete sea walls in the form of uplift and rotation. Damage to residences within the landslide has continued with additional ground movement.

SITE DESCRIPTION

GENERAL

The landslide extends for about 2,800 feet along the shoreline of Squaxin Passage and is roughly oval in plan view, with the long axis oriented slightly north of west. The eastern limit is near Hunter Point. The western limit is near a large ravine that bisects the Carlyon Beach community. The slide area extends from seaward of the existing shoreline to the upland of the peninsula and ranges between 700 and 900 feet in width. The limits of the landslide are shown in Figure 2.

The active landslide area encompasses portions of Mariner and Crestridge Drives Northwest, Broadview Court and Hunter Point Road. Additionally, the landslide encompasses all of Jibstay and Bowline Courts. The southern limit of the landslide approaches Broadview and Seahurst Courts and Lookout Drive. The landslide includes both developed and undeveloped residential properties, together with associated roadway and utility systems. A smaller, active landslide is located outside of the study area on the east edge of Hunter Point.

SURFACE CONDITIONS

Topography

The Carlyon Beach/Hunter Point landslide is located on a north-facing slope overlooking Squaxin Passage in southern Puget Sound as shown in Figures 1 and 2. The upland area south of the landslide is broad and nearly level. The landslide headscarp intersects the upland ground surface at about Elevation 165 feet mean sea level (MSL). The toe of the landslide appears to be at or near sea level.

A steep slope is located along the southern limit of the landslide. This slope is approximately 15 to 25 feet high and typically inclined at 50 to 60 degrees from the horizontal. Recent displacement near the top of this slope has created a near vertical headscarp ranging from 1 to 15 feet in height. This steep slope extends both east and west beyond the limit of the currently active landslide area as shown in Figure 2.

The west third of the landslide comprises residential properties of the Carlyon Beach community. Slopes within the developed area are generally between 7 and 20 degrees from the horizontal with the exception of the headscarp which is inclined at 45 to 70 degrees. Topographic features within this area have been modified by development activities.

The region east of Carlyon Beach is sparsely developed. The topography in this area is characterized by a series of topographic benches inclined at approximately 5 to 20 degrees to the south. These benches are separated by shallow depressions, called grabens, and by steep slopes inclined at 60 to 80 degrees from the horizontal. Access roads and residences within this area have been severely damaged by ground movement. Our field mapping methods are described in Appendix A.

Vegetation

Vegetation within the landslide affected area of the Carlyon Beach community consists of grass lawns and landscaped areas. The landscaped areas are generally separated by mature and sub-mature, second-growth coniferous and deciduous trees and patches of blackberries. Vegetation in the sparsely developed area east of Carlyon Beach consists of a second-growth forest of coniferous and deciduous trees with an understory of sword fern, blackberry, and salmonberry.

Some of the mature coniferous trees within the landslide area are butt-bowed (concave upslope). Many trees are tilted both up and down slope at angles ranging up to 20 degrees from vertical. The trunks of these trees are normally vertical and straight in areas not affected by ground movement. Shallow slope movements, including soil creep, can disturb vertical tree growth, resulting in a bowed base.

Seepage areas and standing water areas (sag ponds) within the slide area are vegetated with hydric plants such as water tolerant grasses, rushes, and algae that indicate saturated soil conditions over a substantial portion of each year.

Cultural Development

The area encompassed by the landslide was undeveloped forestland prior to human habitation. In addition to the streets and houses previously discussed, waterlines, electrical power, telephone, cable television and septic utilities are present in Carlyon Beach and the less-developed area to the east. Private water supply wells are located on two properties immediately east of Carlyon Beach and both have been damaged beyond repair by ground movement. Subsurface water mains within the slide area in Carlyon Beach have been damaged and are not currently operating. Electrical power, telephone and cable television lines have all been damaged to varying degrees in affected areas.

GEOLOGY

Regional Geology

We evaluated regional geology by reviewing available geologic maps and documents. Geology within the area of study is the result of several periods of glaciation in the Puget Sound region. The geologic units present at the site were deposited during two or more glacial and/or interglacial episodes, the most recent occurring roughly 13,000 to 15,000 years ago and referred to as the Vashon Stade of the Fraser Glaciation. Glaciers extended as far south as the southern portion of Thurston County during the Vashon Stade.

According to the Coastal Zone Atlas of Washington (CZA, 1980), near-surface deposits associated with Vashon glaciation in the vicinity of the study area include glacial till overlying undifferentiated Pleistocene sediment. The CZA shows the upland plateau in the study area to be underlain at relatively shallow depths by Vashon till -- a dense, nonsorted, nonstratified deposit of silt, sand, gravel and cobbles that has been compacted by the weight of several thousand feet of glacial ice. However, this till was not encountered in our borings.

The sediment mapped as undifferentiated Pleistocene sediment in the CZA consists of three distinct strata: an upper 0- to 60-foot-thick unit of sand, a middle 75- to 220-foot-thick unit of silt and clay, and a lower unit of sand and gravel of undetermined thickness.

Most of the slope extending from the upland plateau to Squaxin Passage in the Carlyon Beach area is mapped as having intermediate slope stability in the CZA. A historic landslide is mapped in the center of the currently active landslide feature.

Site Geology

Subsurface conditions at the site were evaluated by conducting a reconnaissance-level evaluation of surficial geologic materials and by advancing 22 borings completed as slope inclinometer casings and/or ground water observation wells. The locations of the completed borings are shown in the Site Plan, Figure 2.

Dense to very dense silty fine to medium sand was encountered in borings drilled in the upland area near Hunter Point Road. The sand was encountered from ground surface to depths between 15 and 50 feet below ground surface and is exposed in the face of the headscarp north of Hunter Point Road. This sand correlates to the upper zone of undifferentiated Pleistocene deposits.

The relatively high density of the upper sand material suggests that it has been consolidated by glacial ice. Silt interbeds at the base of this unit suggest a conformable contact between the upper sand and the underlying glacially consolidated silt and clay. Based on these observations, we interpret the upper sand to be glacially consolidated advance outwash. The advance outwash sand also occurs as a relatively thin veneer in the active portion of the landslide. The advance outwash sand unit was not encountered in borings drilled within Carlyon Beach and appears to have been eroded away in this area. The advance outwash sand was encountered in borings completed in other areas of the slide.

Stiff to hard, silt, silty clay and clay underlie the advance outwash sand in the eastern portions of the study area and are exposed at ground surface in Carlyon Beach. This unit is about 140 feet thick in the western portion of the site and increases to over 200 feet thick in the eastern portion. The silt and clay were deposited in a deep fresh water lake that formed when glacial ice dammed the northern outlet of Puget Sound and have been glacially consolidated. The silt and clay unit appears to be massive and without well developed depositional structures. However, undisturbed samples break along parallel sub-horizontal planes, indicating partings that may represent bedding or stress relief structures. This unit is exposed in many areas at the toe of the landslide where it is quite sheared and broken, probably as a result of past episodes of ground movement.

Sand and gravel were encountered beneath the silt and clay unit. The contact between these units ranges between Elevation +10 MSL in boring SI-6 and Elevation -100 MSL in PB-12. The sand and gravel unit has been glacially consolidated and likely represents deposits from an older glacial or interglacial period. The upper surface of this unit appears to slope downward to the northeast, as shown in Figure 5.

The distribution of surface and near-surface geologic units within the study area are indicated on interpretive cross-sections A-A', B-B', and C-C' (Figures 3, 4, and 5). The unit contacts shown are based on borings and mapping completed by our firm. Contacts should be considered approximate due to the variable thickness and character of these deposits.

HYDROLOGY
Precipitation

We reviewed historical precipitation data recorded by the National Weather Service at the Summit rain gauge station located approximately 15 miles southwest of the site. The existing data span the 58-year record from 1941 to the present. We compared cumulative precipitation from water years 1993 through 1998, and the first six months of 1998-1999 to the 58-year average annual cumulative total (see Figure 6). A water year spans a 12-month period beginning in September and ending in August. Our evaluation revealed that precipitation during the past five water years exceeds the 58-year average by 3 to 26 percent. In addition, precipitation during the first 6 months of water year 1998-1999 exceeds the previous maximum six-month cumulative rainfall by 65 percent.

Surface Water

The primary surface water features within the landslide area include small drainages, sag ponds, and seeps. The drainages originate from seepage areas and sag ponds within the landslide area and terminate in Squaxin Passage. Seepage and surface water are concentrated into a creek immediately east of the residence at 9700 Hunter Point Road, with a discharge of approximately 10 gallons per minute (gpm) on March 11, 1999. Seepage and surface water near the east end of Mariner Drive and Jibstay Court collect into a small creek, with a discharge of approximately 5 to 10 gpm on March 11, 1999. The flow in these drainages is variable and can significantly increase in volume during periods of intense precipitation. Some flow is also diverted and infiltrated into open ground cracks where the drainages cross these features.

Sag ponds and saturated ground are found within grabens located immediately north of Hunter Point Road in the sparsely developed area of the landslide. Seepage zones are present throughout the northern half of the landslide area.

Other surface water features include the residential storm drain system and ditches within Carlyon Beach. The residential storm drain system collects surface water from streets and some residential downspouts, and discharges into Puget Sound. One section of ditch on the western side of the intersection of Crestridge and Mariner Drives has been routed within a 12-inch diameter corrugated metal half-round pipe. This section of half-round pipe crosses directly over a major ground crack.

Ground Water

Ground water occurs in three zones in the area of the landslide. A shallow, unconfined aquifer occurs in the advance outwash sand, located south of the central portion of the landslide. This aquifer appears to be up to 50 feet thick in the area of Hunter Point Road, as determined by data from boring SI-9, and appears to thin rapidly to the west. Shallow ground water occurs at depths between 3 and 11 feet below ground surface in the unconfined aquifer. A summary of ground water measurements is presented in Table B-2 in Appendix B. The direction of ground water flow in the shallow aquifer is assumed to be toward the northeast into the landslide.

A thin, confined zone of ground water was observed in several locations where the landslide plane was encountered. Piezometer P-5 was constructed to intercept this zone. Ground water measurements conducted in P-5 suggest that the potentiometric surface within the slide plane is approximately 50 feet higher than the failure zone, indicating that ground water within the slide plane is confined.

A deep, confined aquifer was encountered beneath the silt and clay unit. This aquifer is composed of sand and gravel and is confined by the overlying clay. Ground water in several deep piezometers appears to rise 60 to 80 feet above the base of the clay and corresponds to an elevation of about sea level. It does not appear that this aquifer is hydraulically connected with either the landslide failure plane or the shallow aquifer due to the thickness and extremely low hydraulic conductivity of the intervening silt and clay unit.

A surficial zone of weathered silt that has been regraded during site development in the Carlyon Beach area also contains shallow, unconfined ground water. It is likely that this zone is only marginally hydraulically connected to the shallow sand aquifer and does not represent a significant source of ground water to the landslide.

DISCUSSION

GENERAL

The landslide continues to move as of the date of this report. Since our investigation began in mid-March 1999, we have noted an increase in the length and offset of numerous ground surface fractures and increasing damage to certain residential structures. The offset on some cracks in road surfaces has increased and some new cracks have appeared. We have noted continued uplift of the beach at some locations during our work on-site. Disturbance to the beach appears to be restricted to a zone extending only a few tens of feet offshore of the existing bulkheads or shoreline.

Continued movement of the slide mass has occurred along the immediate shoreline area within the Carlyon Beach development, at the McCarthy, Fisher, Hughes and Dillon residences in the central and eastern area of the slide and adjacent to Hunter Point Road in the south-center of the headscarp. The residences noted have been evacuated for some time. The headscarp has increased from about 5 to 8 feet in height in mid-March to more than 15 feet in height by late April at a location roughly 500 feet east of the 90 degree turn in Hunter Point Road. The scarp line has regressed to within about 3 feet of the north shoulder of Hunter Point Road. We have provided recommendations to the County regarding relocating Hunter Point Road. The County has taken action to address this issue.

In the last few weeks, we have been requested to examine potential slide damage evidence at a number of residences in near proximity to, but outside of, the known slide area. At this time, none of the indicated damage can be attributed directly to ground motion associated with the landslide. However, the residents reporting the damage indicate that the observed changes have occurred since the landslide became active. As of this report date, no residences have been added to the County's list of damaged or evacuated properties since March 15, 1999.

Surface Features

Surface cracks that indicate tension or lateral extension of the ground are generally oriented perpendicular to the direction of ground movement and parallel to slope contours. We observed this pattern of ground cracking around the upper perimeter of the slide and at several locations within the slide mass. The tension cracks separate small to large intact blocks of soil that sometimes have back-tilted surfaces, resulting in a series of steps or benches. Frequently, the cracks border areas that drop sharply below the elevation of the surrounding ground surface. These features are called grabens. Ponds often develop on the bench surfaces, or in grabens, especially where the downset exposes water-yielding soil. Numerous features of this character are noted on Figure 2.

We observed shear features within the slide mass that are the result of differential movement between two masses of soil. Along the shoreline, we observed blocks or zones of soil overriding areas that are apparently moving more slowly or are stationary, resulting in pressure ridges and bulges. Many of the houses we observed within the slide area appear to be twisted or tilted as a result of the differential ground movements within the landslide.

In our opinion, based on our observations, the greatest magnitude of ground movement has occurred in the central portion of the slide. We expect that future movement will continue this trend.

Survey Data

We analyzed ground surface survey data to determine the direction and magnitude of movement at the Carlyon Beach-Hunter Point landslide. This data, obtained by SCA Engineering of Lacey, Washington, includes three consecutive weekly survey measurements of approximately 50 control points between March 29 and April 15, 1999. Thirty-three of the 50 control points are located within the landslide area. The displacement measured includes the direction and horizontal distance of movement and the elevation change (vertical) for each control point. The estimated accuracy for all points is ± 0.10 feet for both horizontal and vertical measurements. The survey data, displacement measurements, and control point locations are provided in Appendix D.

The measured horizontal displacement ranged between 0.04 and 1.40 feet within the landslide. The maximum horizontal displacements occurred along the waterfront in the central portion of the landslide (control points 121, 122, 123 and 124). The majority of the control points have a direction of movement oriented between North 37 degrees East and North 25 degrees East. However, the total range in the horizontal direction of movement varies between North 87 degrees West and South 72 degrees West. Cumulative vertical displacement ranged from 0.01 to 0.43 feet. The maximum vertical displacements occurred near the headscarp of the landslide (control points 115 and 116).

In summary, the greatest measured vertical movement has occurred near the headscarp and the greatest measured horizontal movement has occurred near the toe of the landslide. The area of apparent maximum displacements is located south and southeast of the main landslide headscarp (control points 108, 109, and 110).

There has been less than 0.05 feet of horizontal displacement and less than 0.07 feet of vertical displacement at the control points to the west, south and east of the landslide area. However, it should be noted that these displacements are well within the margin of error for the data and may or may not represent actual ground movement.

Slope Inclinometers and Poor Boys

We installed slope inclinometer casings in nine borings to monitor ground movement within and adjacent to the landslide area. The four slope inclinometer casings located within the active landslide area crimped or sheared off within a few days of installation. Repeated inclinometer surveys were performed to detect changes in the alignment of the remaining five casings located outside of the active landslide area. Each survey was compared to the initial baseline reading to detect the magnitude and direction of differential movement within the soil mass penetrated by the casing. The slope inclinometer survey data are summarized in Appendix E.

Poor boys were installed within most of the slope inclinometer casings and three 2-inch diameter well casings. The poor boys consist of a 1.5-foot-long, 1.5-inch-diameter PVC pipe set at the base of each casing and suspended from the surface by a ¼-inch diameter rope. The poor boys were lifted toward the surface to detect crimping or shearing of the casing. Soil movement had not excessively deformed the casing if the poor boy could be lifted to the surface. If the poor boy encountered a crimp in the casing while being lifted, the depth to the crimp was recorded as a possible failure plane or zone of deformation within the soil mass. Poor boys were also lowered from the top of the casing, allowing the observer to identify additional zones of deformation.

Slope inclinometer casing within the west portion of the active landslide in SI-2, SI-3, SI-4, and SI-5 crimped at depths of about 32, 26, 46, and 66 feet, respectively. Slight crimping was also observed in SI-4 and SI-5 at depths of about 24 and 26 feet, respectively. The PB-8 casing in the east portion of the active landslide crimped at depths of about 55, 77, and 105 feet and the casing in PB-12 crimped at depths of about 49 and 90 feet. No discernible crimping has been observed in PB-10.

Repeated slope inclinometer surveys at SI-1, SI-6, SI-7 and SI-9, which are located south of the landslide headscarp, are inconclusive. In our opinion, the magnitude of the apparent movement in these casings lies within the margins of accuracy for slope inclinometer surveys, and is probably not indicative of movement. Only initial baseline readings have been measured at SI-11.

Slope Stability

The slope stability analyses were completed using the computer program XSTABL developed by Sunil Sharma of Interactive Software Designs, Inc. This program has the capability of modeling slope stability for a wide range of slope and failure surface geometries with multiple subsurface layers and ground water levels. The purpose of this modeling was to evaluate different failure mechanisms and sensitivity to various soil, ground water and slope configurations as well as evaluate the effectiveness of the remedial options.

The model identifies the most critical failure surface for specified topography, subsurface conditions, soil properties, and ground water conditions. The stability of the soil mass is reported as a factor of safety, which is the ratio of resisting forces to driving forces. A factor of safety of 1.0 indicates that the resisting forces are equal to the driving forces and that the soil mass is at marginal stability. Factors of safety less than 1.0 indicate that the resisting forces are smaller than the driving forces and that the soil mass is unstable. Factors of safety greater than 1.0 indicate that the resisting forces are greater than the driving forces and that the soil mass is stable. Based on the current standard of practice, a factor of safety greater than 1.3 is typically desired in designing landslide repairs; however, lower factors of safety and higher risk levels are sometimes acceptable for very large landslides.

We developed representative cross sections (see Figures 3, 4 and 5) based on surface topography and interpretation of the boring data. Soil units were interpreted from boring data. The ground water piezometric elevation was interpreted based on data collected from the piezometer and surface seep elevations.

The procedure used in the analysis was as follows:

1. **Estimate the shape and depth of the failure plane.** The shape and depth of the failure plane was estimated based on the poor boy data, the geologic conditions and our experience in evaluating landslide mechanisms. In this case, the overall slope of the ground surface in the area of the landslide is very flat (approximately 11 degrees). The impact of the shape of the failure plane, circular or wedge, is relatively minor. The analysis indicated very little difference in the factor of safety using the two methods.

   Typically, the failure plane occurs at the upper or lower contact between the glacially consolidated silt/clay and sand units in the Puget Sound area. In the Carlyon Beach/Hunter Point landslide, the geometry of the stratigraphic contacts does not appear to match the landslide geometry. The poor boy data indicate a slightly inclined failure plane within the silt and clay unit. The poor boy data also indicated several other slide planes within the slide mass which are indicative, in our opinion, of flexural displacements within the slide mass. Based on our evaluation of the poor boy movements and the size of the heaved area on the beach, we selected a 3- to 4-degree inclined failure plane that daylights approximately 10 feet below the beach level.

2. **Estimate the ground water regime at the time failure occurred.** The significant information needed for the analysis is the hydrostatic pressure that exists along the failure plane. Based on the piezometer that was specifically installed on the estimated failure plane, the ground water regime was assumed to be hydrostatic above the failure plane with a water level near and subparallel to the ground surface. The ground water piezometric surface used in the analysis is shown as the upper water table in the cross sections in Figures 3 and 4.

3. **Back-calculate shear strength parameters that yield a factor of safety of 1 for the conditions estimated to exist at the time of failure.** Our analyses indicate friction angles ranging from 15 degrees to 16.5 degrees, with no assumed cohesion.

4. **Compare the back-calculated shear strength parameters with the values obtained from the laboratory testing and empirical relationships for the soil types.** The friction angle calculated along the failure plane is considerably less than the strength values calculated in the laboratory testing. This is considered to be due to the intact structure of the soil tested. Even the residual (resheared) direct shear tests indicated higher strengths, possibly indicating that the soil tested is not the sheared material existing at the slide plane. The friction angles used in our analysis are in good agreement with typical residual values used for the clay units deposited during the Vashon glaciation. Laboratory test results are contained in Appendix C.

5. **Refine the assumed shear failure surface, back-calculated shear strength parameter and ground water regime to obtain a best-fit model for the slide event.** The back-analysis was modified slightly by varying the sensitivity of the failure plane inclination and the residual friction angle. The back-calculated failure plane is shown in Figures 3 and 4. It should be noted that there are probably other subparallel failure planes within the landslide mass which are also undergoing shear displacements. Specifically, poor boy No. PB-8 indicates movement at a lower elevation than the assumed failure plane (Appendix E). Additional data may be required to refine the failure plane location. It is our opinion that the failure plane assumed in the back-analysis is probably adequate to evaluate the various remedial options. Computer program output is contained in Appendix F.

6. **Use the back-calculated information to model future performance for the remedial design options.** The back-analysis and corresponding increases in the factors of safety for the various remedial options are described subsequently. Discussion of the details of these options and estimated costs are presented in the Conclusions section, below, and in Appendix G.

CONCLUSIONS

In our opinion, the current activity at the Carlyon Beach/Hunter Point landslide is the result of reactivation of a large, ancient landslide feature. It is our opinion that the direct cause of the reactivation is a general increase in regional ground water recharge, particularly in the area south of Hunter Point Road, resulting from several successive years of above-normal rainfall. The landslide appears to be primarily translational in nature with localized backward rotation of some of the individual slide blocks, particularly in the area of the headscarp. The slide blocks appear to lose integrity with increasing downslope movement. The area of greatest movement appears to occur in the central portion of the landslide, north of Hunter Point Road. This appears to be the result of high ground water flow into the headward portions of the landslide from the shallow advance outwash sand aquifer.

In our opinion, the landslide will continue to move until the pre-recent slide equilibrium condition is restored, either by natural processes (normal, or below-normal rainfall) or through engineered means. Our stability analysis indicates that the factor of safety which existed prior to the current movement was only slightly greater than 1.0. Increased ground water flow into the landslide mass has reduced the factor of safety less than 1.0, resulting in activation of the ancient landslide features.

The landslide movement is characterized by relatively slow ground motion. Measured horizontal ground movement in the central portion of the slide was about 1 foot measured over a period of about three weeks between April 1 and April 23, 1999. Ground movement at the eastern and western limits was much less during this time period. A significant portion of the overall recent ground movement appears to have occurred shortly after the period of intense precipitation experienced in early February 1999. While this rate of movement does not appear to represent a direct threat to life, it is of sufficient magnitude to cause major damage to structures and utilities. As ground movement continues, we expect further disruption to streets and utilities. Ultimately, unless the landslide movement ceases, we expect that all structures within the landslide area shown in Figure 2 will experience minor to major damage. Localized sloughing of sandy soil on the face of the headscarp, or possible headward regression, has the potential to severely damage Hunter Point Road and limit access to the Hunter Point community.

It is our opinion that implementation of remedial measures will increase the overall factor of safety for landslide movement. It does not appear that any single remedial option, if implemented, will increase the factor of safety above a value of 1.3, and additional movement may be expected to occur at least for a period of time. Therefore, multiple and redundant remedial measures are recommended if stabilization actions are undertaken.

SUMMARY OF REMEDIAL OPTIONS

SHORT-TERM

Short-term actions already undertaken by Thurston County staff or recommended by GeoEngineers include the following:

- Identify unsafe conditions and recommend evacuation of certain structures within or adjacent to the area of movement. Thurston County undertook this action in February and March, 1999. At this time, affected homes within the area of landslide movement have been evacuated.
- Re-align the eastern portion of Hunter Point Road to the south to reduce the risk of road failure resulting from sloughing of the headscarp in this area. We understand that the realignment is in progress by Thurston County Roads and Transportation.
- Evaluate and remove at-risk trees in populated areas to reduce the hazard to life safety. Severely tilted trees should be removed.

- Install emergency shutoff valves in water lines within Carlyon Beach to reduce the risk of water line failure and addition of water to the landslide.

LONG-TERM

General

The long-term stability of the landslide can be improved by increasing the available forces to resist landslide movement or by decreasing the driving forces that are causing the movement. Resisting forces can be increased by lowering of ground water levels within the slide mass and at the slide plane (increases effective stress and friction), and by constructing a gravity buttress at the toe of the slide. Driving force reductions can be accomplished by regrading the landslide headscarp to remove weight from the head of the slide and by reducing the overall steepness of the slope.

An evaluation of possible remedial measures is summarized in the following sections. These measures could be used individually or in combination to improve the stability of the slide mass and potentially arrest landslide movements over the long term.

For each option, we have provided a general description of actions required for implementation, an estimated cost for implementation, and an estimate of the factor of safety achievable once the option is implemented. In each option we have assumed that all surface water within and adjacent to the landslide is collected and routed to the beach. This action includes draining areas of ponded water and directing stormwater runoff through pipes or ditches (that do not cross disturbed ground) to Puget Sound. We estimate that pond drainage will cost approximately $73,000 and stormwater drainage improvements will cost about $4,000 per household (approximately $152,000 total).

Several of the options require the purchase and demolition of some or all of the homes in order to implement the remedial action; we have included those costs in our assumptions. Land and improvement values were obtained from Thurston County Development Services and an average cost was used in estimating purchase values. We have also assumed that certain costs will be required of each homeowner to conduct repairs on individual structures and repair infrastructure (roads and utilities) in order to reoccupy the dwellings. A summary of tasks necessary to implement each option is included in Appendix G.

It should be understood that there is a low likelihood that a single remedial option will achieve stability of the entire landslide. The application of multiple and redundant measures will be required to achieve stability of the landslide. It may be possible to stabilize portions of the landslide by applying remedial options in selected locations.

Furthermore, even if the major slide mass is stabilized, additional adjustment and ground movements will occur within the disturbed portion of the slide mass. These longer-term adjustments could occur for years after stabilization of the primary shear plane.

Small measures such as reducing surface water ponding by regrading or directing roof, yard and street drainage into a tightlined storm water system will help improve stability to some extent, but larger-scale efforts are needed to result in significant, long-term improvements to slope stability.

No matter what remedial option is selected, we recommend conducting long-term, periodic monitoring of currently stable areas behind the headscarp to evaluate possible movement. The monitoring program may include slope inclinometer readings, surface reconnaissance, ground water level monitoring, survey of reference points and structural inspections. Some aspects of the monitoring could be performed by residents or County staff after being trained in proper data collection. The aspects of the monitoring program can be determined following discussions with residents and County staff.

No Action

In our opinion, under the no action alternative, there will be continued intermittent movement of the Carlyon Beach/Hunter Point landslide as long as above-normal precipitation continues and elevated shallow ground water conditions exist near the slide area. Left unattended, the landslide will likely expand to include the entire ancient landslide feature. Movement is anticipated to recur in all or part of the slide mass every few decades in response to cyclical increases in precipitation, resulting in renewed damage to houses, roads, and utilities. We expect that the steep headscarp will regress to the south and will cause additional damage to houses on Lookout Drive and Hunter Point Road, and to Hunter Point Road itself. We anticipate that this regression will occur as blocks 50- to 100-feet wide become detached behind the current headscarp. Ultimately, it may be necessary to evacuate additional residences adjacent to the current landslide area if no action is taken.

Ground Water Controls

Lowering the ground water elevation increases slope stability by increasing friction forces acting on the slide plane while decreasing the weight and driving forces acting on the slide mass. The major advantage associated with the dewatering option is the relatively low impact to existing residences, streets, and utilities. The relatively low impact is exclusive of the streets and utilities that must be replaced or repaired regardless of other remedial actions that may be implemented if the area is to be used for residential purposes in the future. Affected homeowners will need to conduct structural repairs prior to reoccupying their homes. Disadvantages associated with this option include a high initial cost and the necessity for ongoing maintenance and operation.

We modeled the landslide with the computer program XSTABL to estimate the effect of lowering ground water elevation on slope stability. Topography, subsurface geology, and soil properties used in the model are the same as those presented previously within the Slope Stability section of this report.

Ground water recharge to the slide appears to be occurring in the headscarp area along Hunter Point Road from the advance outwash sand aquifer generally located south of the slide mass. We estimate that lowering the ground water piezometric surface by 20 feet below its present elevation in the advance outwash aquifer, south of the headscarp, increases the factor of safety to about 1.2. This provides a marginal factor of safety against landslide activity. Lowering the ground water piezometric elevation by 40 feet below its present elevation in the advance outwash aquifer increases the factor of safety to 1.3.

We evaluated the following three options to lower ground water elevations within the slide mass in an attempt to increase stability:

Extraction Wells. A system of approximately 160 extraction wells placed in a roughly east-west line south of the headscarp would be required to lower the water table elevation 40 feet in the shallow sand aquifer. The purpose of these wells would be to intercept ground water upslope of the landslide headscarp and reduce the quantity of ground water entering the landslide. We estimate that implementation of this option would cost between $5.1 and $5.6 million. Ongoing operation and maintenance would cost roughly $50,000 per year.

The extraction wells would be constructed of 6-inch-diameter casing and would be screened between 10 and 50 feet below ground surface. Electric submersible pumps would be used to extract water and would be connected to piping to convey the water to Puget Sound. Pump controls and a stand-by electrical generator would be housed in a building constructed for this purpose. Several residential wells would need to be redrilled, if they were to be used for future potable supply, due to the lowering of the ground water piezometric surface.

Cutoff Wall. A soil cement cutoff wall installed south of the headscarp is an alternative to dewatering wells to reduce the quantity of ground water entering the landslide. The wall would be constructed using a series of large auger borings to mix cement with subsurface soil to form a relatively impermeable below-ground wall. The wall would need to be installed approximately 5 to10 feet into the competent silt and clay unit to adequately reduce the flow of ground water. We anticipate that the wall would be approximately 1600 feet long and extend from about Lookout Drive on the west to the east end of Hunter Point Road. Ground water behind (south) the wall would be controlled by the installation of a 10- to 15-foot-deep drainage trench that would passively collect water from areas upgradient of the wall and discharge the water in tightlines to Puget Sound.

The factor of safety achievable using the cutoff wall is estimated at 1.3 -- similar to the dewatering well scenario. We estimate that costs associated with installing a cutoff wall and drain would be between $4.0 and $4.6 million.

Drains. A system of trench and slot drains within and adjacent to the landslide mass would remove near-surface ground water from the landslide mass and dewater the failure plane somewhat, increasing frictional resistance. The maximum placement depth of the drains is restricted by construction constraints and is generally limited to about 25 feet. Maximum dewatering depth is limited by the spacing of the drains and by the maximum placement depth of the drains. The drain system would be most effective if placed along the toe of the slide where the slide plane can be intercepted using standard excavation methods.

Using slot drains alone, the factor of safety can be increased to approximately 1.1. We estimate that the cost for implementing this alternative would be on the order of $5.8 million.

Structural Controls

Regrading. Regrading the landslide headscarp area increases stability by decreasing the driving forces acting on the slide. The advantages associated with this option include a high potential of success and minimal projected yearly maintenance. Disadvantages include the high initial costs associated with massive earthwork projects, the need for off-site disposal of excavated material, and the need to move or demolish as many as 30 residences within the regraded area. Regrading would move the top of the slope approximately 500 feet south to achieve a 12.5 percent overall grade.

An estimated factor of safety of 1.3 or better can be achieved through the application of this option. We estimate that 3 million cubic yards of material would require removal and offsite disposal. The destruction of a number of currently unaffected houses along with a high estimated implementation cost of $39 million render this option infeasible.

Toe Buttress. Construction of a buttress system across the landslide toe at approximately Elevation +20 will increase resistance to movement on the slide plane and improve drainage to help stabilize the landslide mass. We evaluated a buttress composed of a toe key to increase shear strength across the slide plane.

We evaluated a 3,000-foot-long buttress extending from Carlyon Beach on the west to Hunter Point on the east. This buttress would be composed of well-graded quarry rock and would be keyed into competent soil beneath the slide plane, requiring excavation below sea level. The buttress dimensions would be 20 feet wide at the base, 80 feet wide at the top and 30 to 45 feet deep.

Advantages associated with this option include a significant increase in the estimated factor of safety to 1.4. The buttress would act as a drain and help dewater the landslide, would form a stable base for construction of new building foundations, and would require minimal yearly maintenance. Disadvantages include the very high initial cost associated with a massive earthwork project ($27.5 million), removal and replacement of approximately 15 houses, the need for off-site disposal of excavated material, possible hazardous working conditions due to unstable slopes and the need for a large quantity of imported rock.

Drilled-Pile Wall. A drilled-pile wall would be installed along the toe of the landslide at about Elevation +20 MSL to increase resistive forces along the shear plane. The drilled piles would be constructed by drilling 5-foot-diameter shafts to a depth of approximately 2.5 to 3 times the slide thickness. Typical spacing of the piles is between 2 and 3 times the diameter of the individual piles. Drill cuttings would be removed from the drill holes and replaced by concrete reinforced with steel. Installation of the piles would require the demolition of approximately 15 homes along the waterfront. These homes could be rebuilt following stabilization.

Approximately 240 piles would be required along the toe of the landslide. Calculating the factor of safety for a pile wall requires a detailed design and modeling effort that is beyond the current scope of our services. Using nominal values within the ranges presented above, the cost for construction of a drilled pile wall would be on the order of $10 to $15 million.

LIMITATIONS

We have prepared this report for use by Thurston County Development Services. The data and the report may be made available to other parties, as the County deems appropriate, but our report, conclusions and interpretations should not be construed as a warranty of subsurface conditions or future behavior of the landslide.

This report is based on conditions present as of April 23, 1999. Data collection and field observations are continuing and additional information could result in modifications to our conclusions or recommendations.

Our services have been provided over a relatively short time period and the results of our evaluations should be viewed in light of this constrained approach. The remedial cost estimates presented in this report must be considered as preliminary and conceptual in nature. The estimated costs given in this report may differ from actual costs. We recommend that our interpretations, conceptual remedial options and preliminary cost estimates be refined as remedial options are selected and as more data becomes available.

Within the limitations of scope, schedule and budget outlined in our contract with Thurston County, our services have been performed in accordance with the standard of care and skill ordinarily exercised by other geotechnical professionals in the area at the time the report was prepared. No other warranty or conditions, express or implied, should be understood.

Thank you for the opportunity of working on this project. Please call if you have questions regarding this report or require additional information.

Very truly yours,

GeoEngineers, Inc.

William E. Halbert
Senior Hydrogeologist

for
Jon W. Koloski
Principal

WEH:JWK:ja:vc
Document ID: 7318001R.DOC

Attachments

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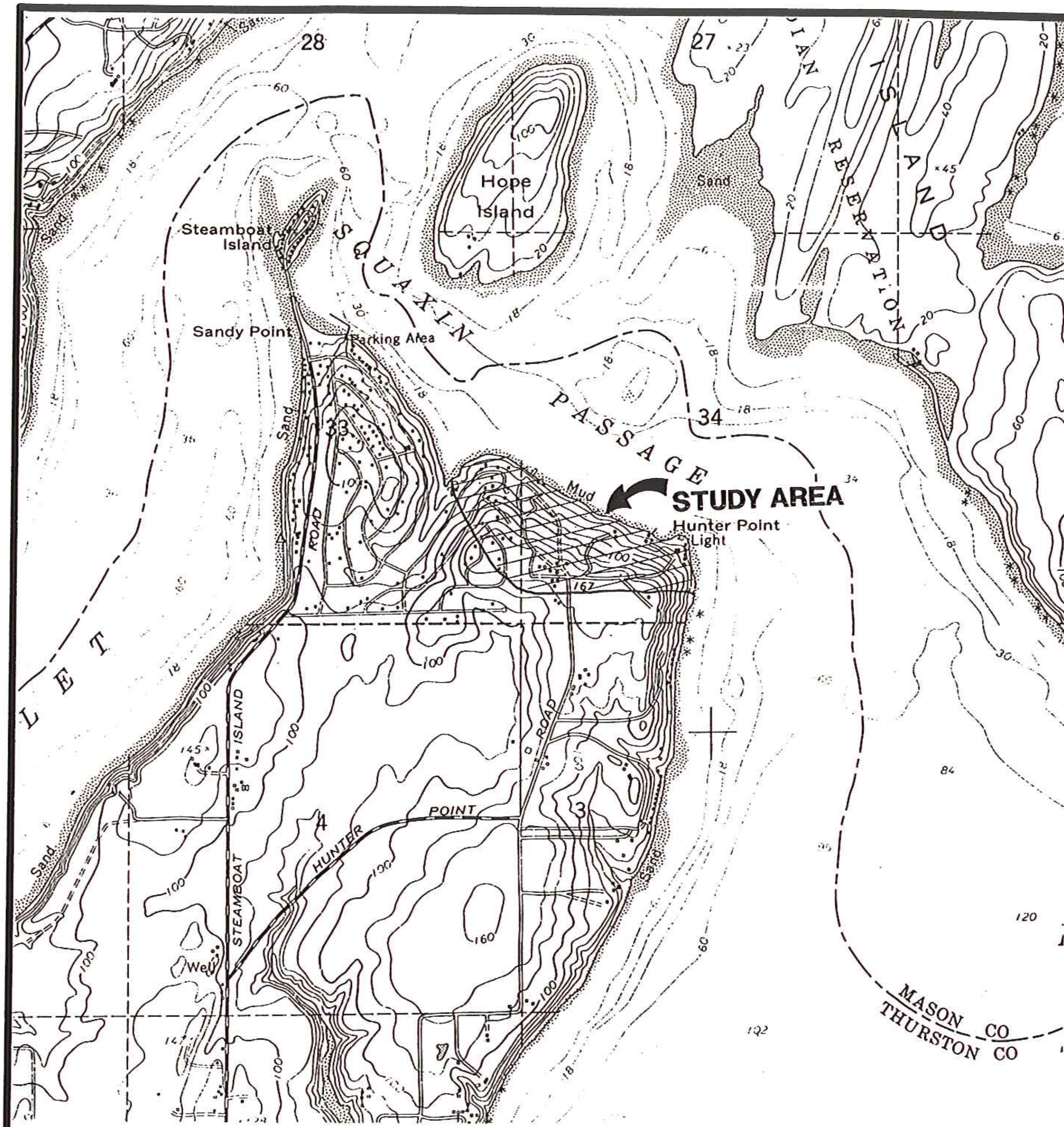

0 2000 4000

SCALE IN FEET

CONTOUR INTERVAL 20 FEET

N

Reference: USGS 7.5' topographic quadrangle map "Squaxin Island, Wash." photorevised 1968.

GeoEngineers	VICINITY MAP
	FIGURE 1

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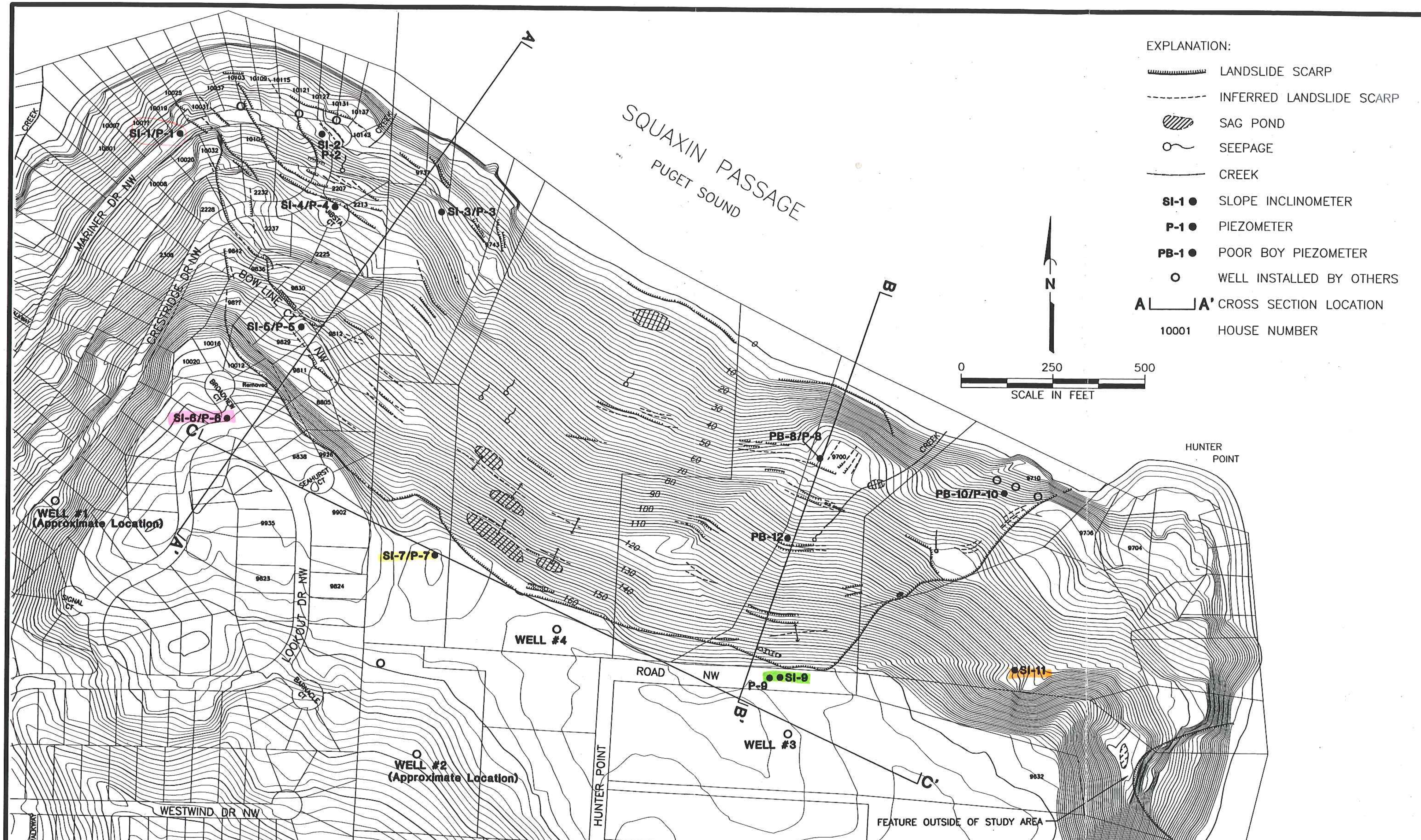

Note: 1. The locations of all features shown are approximate.
2. Landslide features as of 04/07/99.
3. Slope indicator and piezometer locations are shown at one location when the two boring locations were less than 10 feet apart.

Reference: Drawings provided by Thurston GeoData Center, March 1999.

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SITE PLAN

FIGURE 2

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