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Hydrogeologic Characterization of a Rural Landfill for Compliance Operation and Closure Design

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INTRODUCTION

In 1983 King County initiated a solid waste site characterization and ground-water monitoring program at six landfill sites. One active site, which has been in operation since the 1950s, was evaluated in detail. This small rural landfill is located in southeast King County at the site of an old sand and gravel borrow pit near Hobart, Washington (Figure 1). The overall site is approximately 51 acres in size, of which about 27 acres are overlain by solid waste. Solid waste covers most of the developed areas at the site and ranges in thickness from less than 1 to more than 30 ft. The elevation of the site ranges from a maximum of 651 feet in the north to a low of 625 feet in the southeast.

The landfill is currently being operated as a nonconforming facility as determined by the Washington Department of Ecology Minimum Functional Standards (MFS) (WAC 173-304). Closure objectives include correcting existing environmental concerns and bringing the landfill into compliance with the MFS.

The major environmental concern is the ground-water quality impacts to the shallow aquifer. These impacts include elevated concentrations of total dissolved solids and volatile organics. Detected volatile organics have included benzene, ethylbenzene, and toluene.

This paper presents an overview of the hydrogeologic studies performed for King County Solid Waste Division in support of remedial measures designed to eliminate any potential health risks and bring the site into compliance with the MFS (Sweet, Edwards & Associates, 1986).

FIELD INVESTIGATIONS

Extensive field investigations were begun at the site in August 1985 and included: beneficial use surveys, test drilling and monitoring well construction, permeability testing, ground-water sampling and testing, and streamflow gaging.

Beneficial Use

A search of Washington Department of Ecology files and a field survey of existing wells adjacent to the landfill were completed to evaluate current beneficial use of ground water near the landfill. A total of 20 wells was identified during the investigation.

The beneficial use survey identified four domestic wells completed in the shallow aquifer and 16 domestic wells completed in the deep aquifer. Nine wells are located to the north (upgradient) of the landfill, and 11 wells are located south (downgradient) of the site. Included in the 16 deep wells are four public supply wells or wells which provide water to more than one household. These four wells are all located south or downgradient of the landfill.

Drilling

A total of 29 borings and 7 test pits have been completed during this investigation. The locations of the borings and test pits are shown on Figure 2. Six borings (SW-2 through 7) were drilled through the solid waste to determine fill thickness and degree of saturation. The borings ranged in depth between 21 and 60 ft, and all borings were completed with 3/4-in. piezometers.

Twenty shallow borings were drilled and completed with 2-in. PVC monitoring wells. The borings ranged in depth from 15 to 72 ft and terminated in undifferentiated glacial sediments of Vashon age or older.

Three deep borings were drilled and completed with monitoring wells in the deep aquifer. The depths of these borings ranged from 130 to 180 ft.

Permeability Testing

Rising head permeability tests were conducted in three monitoring wells to evaluate the *in-situ* permeability of the older undifferentiated deposits at varied depths and locations across the site. The method of Hvorslev (1951) was used for interpreting the water level recovery versus time data.

A 24-hr pump test was also conducted in monitoring well MW-14 on April 24-25, 1986, to evaluate the hydraulic conductivity of the shallow glacial outwash aquifer. Continuous ground-water levels, specific conductance, pH, and temperature were monitored in the pumping well and an observation well throughout the

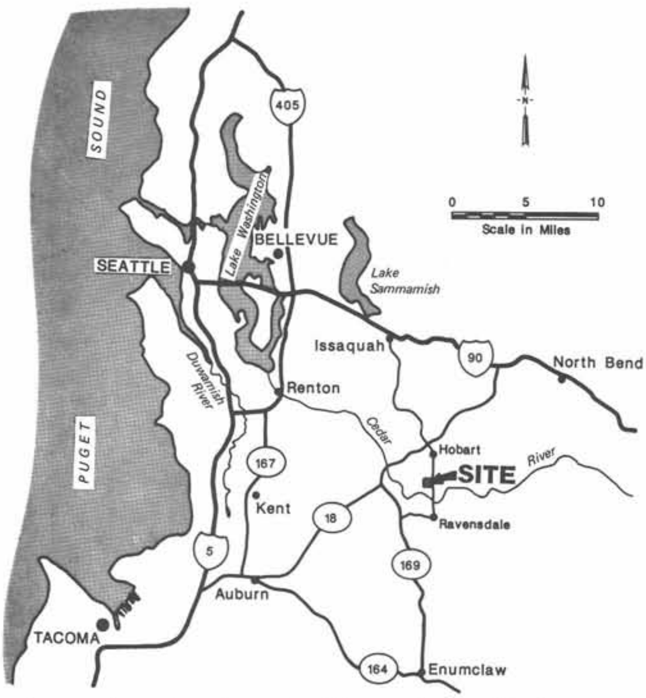

Figure 1. Location of the landfill near Hobart, Washington.

test. In addition, periodic water-level measurements were collected from six other observation wells throughout the pumping test. The well was pumped at approximately 30 gpm, and the discharge water was piped to the borrow pit located approximately 1,800 ft northeast of the well. An in-line carbon filtration system was used to intercept and remove volatile organics in the ground-water stream prior to its discharge into the borrow pit.

Ground-Water Sampling

Ground-water samples were collected from each onsite well screened in the shallow and deep aquifers and from offsite domestic wells in the vicinity of the landfill. Samples were analyzed for primary and secondary drinking water standards, total organic carbon (TOC), and total halogenated organics (TOX). In addition, water samples collected from downgradient and upgradient shallow and deep monitoring wells were analyzed for priority pollutants, including volatile organics, base-neutral and acid extractables, and a pesticide/PCB suite. Additional water samples were collected and analyzed for volatile organics from five downgradient shallow monitoring wells. Four offsite domestic wells (three downgradient and one upgradient) were also tested for volatile organic compounds.

Streamflow Gaging

A diversion canal approximately 3 ft deep and 6 to 8 ft wide is located along the northern edge of the site (Figure 2). The canal was constructed by the City of Seattle to divert poor quality water from Walsh Lake to the Cedar River, below the City's water-supply intake.

Beginning in October 1985, stream flows were measured at five to six stations along the diversion canal on a monthly basis. The purpose of the gaging was to determine potential seepage losses through the canal into the shallow water-table aquifer.

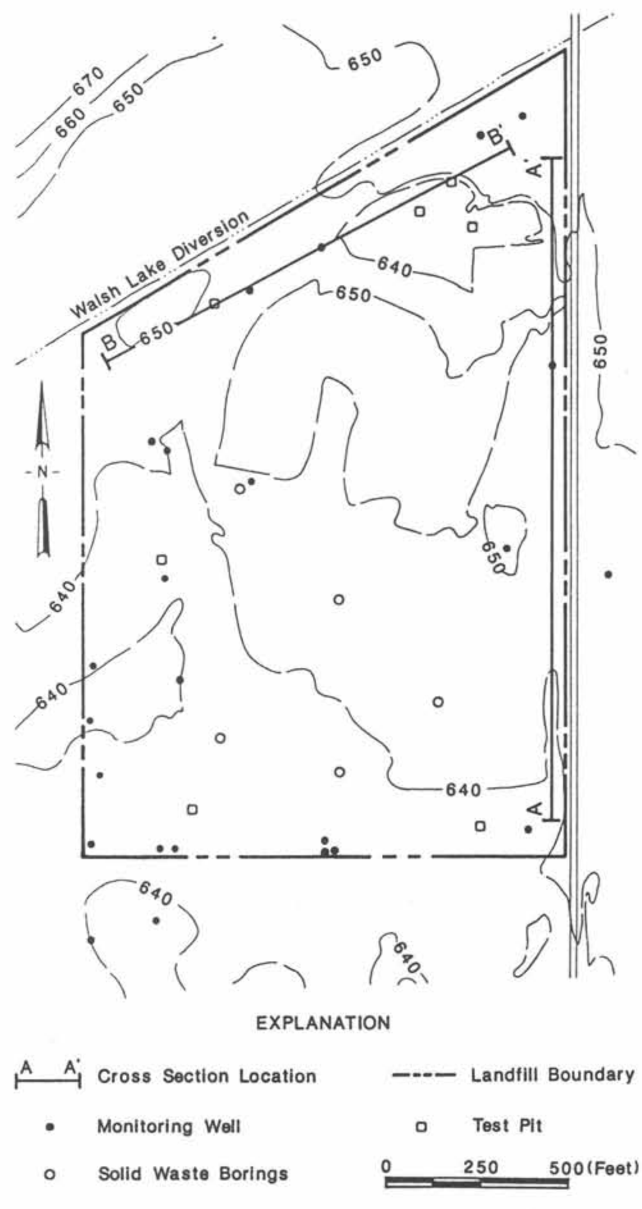

Figure 2. Major features of the landfill and field investigation sites.

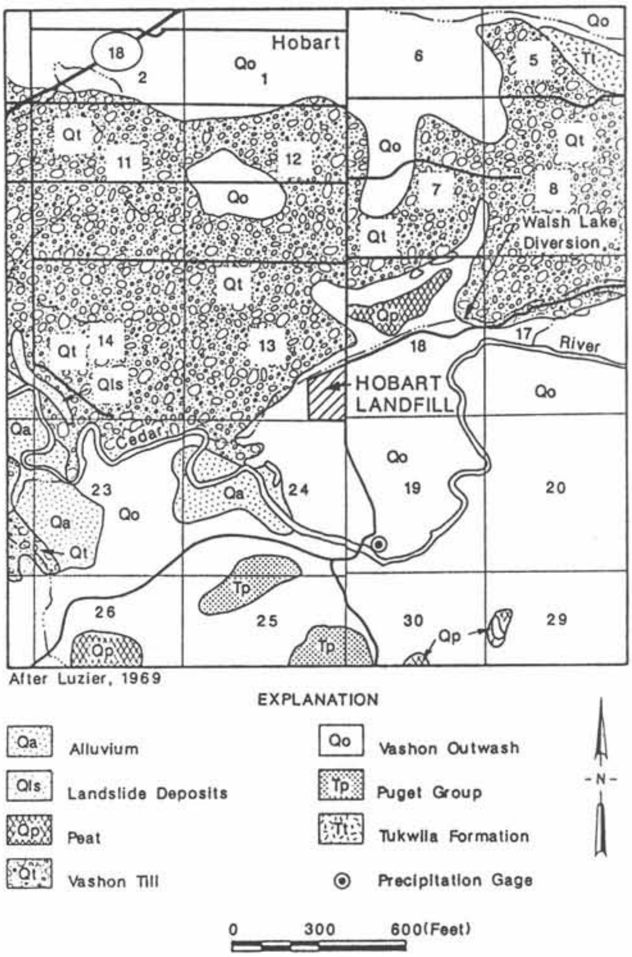

Figure 3. Generalized geologic map of the Hobart landfill area.

GEOLOGY

Regional Geologic Setting

The Hobart landfill is located in the east-central part of the Puget Sound Basin. The geology and physiography of the Puget Sound Basin are the result of a number of complex geological processes over a long period of time.

The geology of the study area is primarily influenced by regional bedrock structure, glacial erosion, and glacial deposition. The principal bedrock outcrops in the Hobart vicinity include undifferentiated sediments of the Puget Group and the Tukwila Formation (Figure 3). Pleistocene glaciers advancing south from Canada eroded the valley bedrock, creating a broad trough which was then filled with glacial deposits as the glaciers retreated.

Vashon glacial drift dominates the surficial geology of the site vicinity. A typical glacial sequence includes the advance outwash (sands and gravels deposited in front of the advancing glacier), till (highly compacted, "concrete-like" material which was overridden by the glaciers), and recessional outwash (sands and gravels deposited on the till by retreating glaciers). In addition, an advancing or retreating glacier may deposit thick sequences of silt and clay in proglacial lakes (lacustrine deposits).

Recent alluvium has been deposited in stream and river channels. Isolated pockets of peat in lowlands are common, as are both large and small landslide deposits.

Site Geology

The glacial stratigraphy encountered at the landfill site is complicated by the older undifferentiated deposits which underlie the most recent glacial deposits of Vashon age. The top of the older undifferentiated deposits (peat/silt) have been carbon dated as 40 ka (Sweet, Edwards & Associates, Inc., 1986) whereas the Vashon glacial period is approximately 15 ka.

Subsurface geologic units at the Hobart landfill site consist of Vashon glacial outwash and till underlain by formations that make up older undifferentiated glacial deposits. The geologic materials encountered have been subdivided into eight identifiable units that are listed below from youngest to oldest.

Recessional Outwash	
Glacial Till	Vashon Drift
Advance Outwash	
Peat/Silt Deposits	
Silt/Clay	Older undifferentiated deposits
Silty Sand	
Older Till	
Gravelly Sand and Silt (Perching Unit)	

A brief description of each unit is given below. The stratigraphic position of these units is shown in cross sections A-A′ and B-B′ (Figures 2, 4 and 5).

Recessional Outwash

Recessional outwash (Vashon age) consists of brown and gray, fine to coarse sands and gravels with scattered cobbles and boulders. The unit is characteristically well bedded, well graded, and poorly compacted. The deposits are continuous beneath the site around the perimeter of the landfill and range in thickness from 5 to 33.5 ft.

Till

The Vashon till at this site consists of a very dense, well graded, silty sand and gravel. The unit was penetrated only in borings at the north end of the site and ranges from 5 to 7 ft in thickness. The till was absent in all other borings and appears to be lenticular and limited in areal extent.

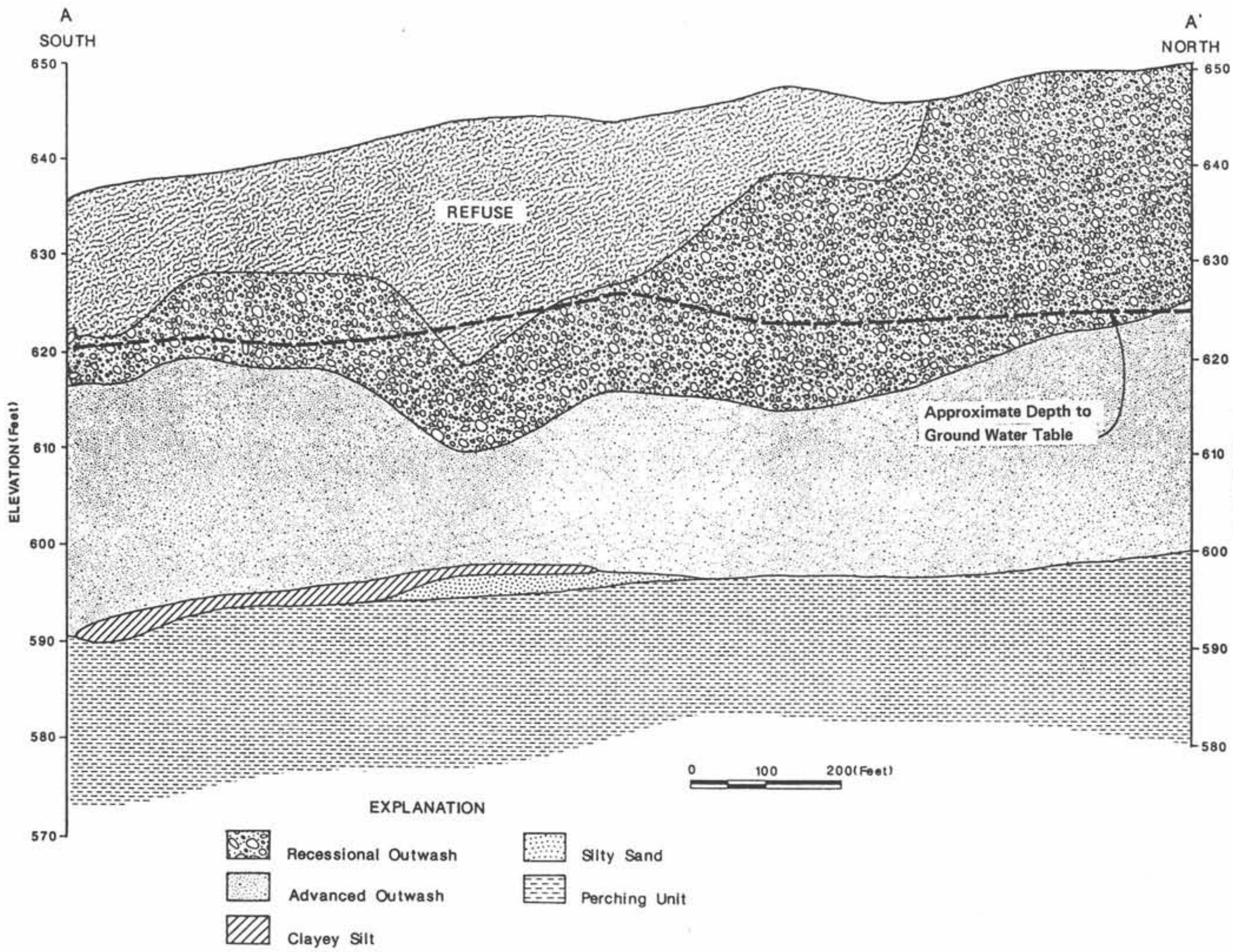

Figure 4. Geologic cross-section A-A′. See Figure 2 for location of section.

Advance Outwash

The advance outwash is the most varied formation encountered with respect to color, texture, and grain-size distribution. Typically, the formation consists of a gray sandy silt to a gravelly silty sand and is moderately dense, well to poorly graded, and commonly well stratified.

Lower parts of the formation in the northern area of the site consist of a coarse, poorly consolidated sandy gravel, contain little to no fines, and represent a higher energy deposit than is described above.

The basal 5 ft of the advance outwash is commonly brown or orange and similar in appearance and texture to the perching unit. The lithologic characteristics are typical over much of the site, and the unit is thought to represent reworked older undifferentiated deposits.

Peat/Silt Deposits

The peat/silt unit is restricted to the northwest corner of the site. The lithology consists of a dark brown, moderately firm, peaty silt. The known thickness ranges from 1.5 to 5 ft. It occurs immediately below the advance outwash and overlies an older till.

Silt/clay

The silt/clay unit is restricted to the southeast corner of the site. The unit's thickness ranges from 1 to 2.25 ft, and it occurs immediately below the advance outwash deposits and overlies the perching unit. The unit consists of a gray to orange, firm, sandy clayey silt.

Silty Sand

The silty sand unit is restricted to the southeast corner of the site. The lithology consists of a yellow to brown, fine to medium sand with trace silt and gravel.

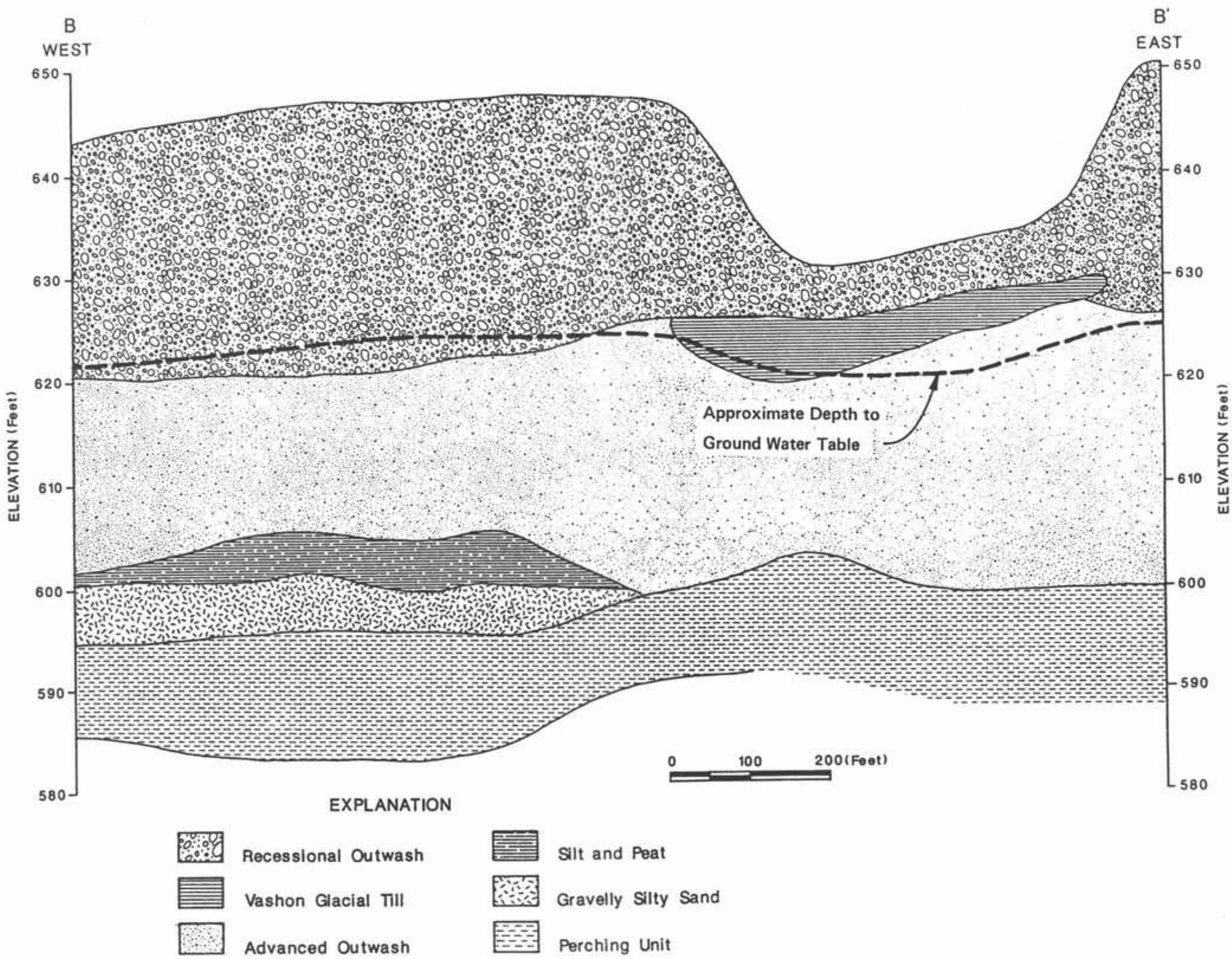

Figure 5. Geologic cross-section B-B′. See Figure 2 for location of section.

It is moderately dense, slightly friable and between 2 and 3 ft thick. The unit occurs below the silt/clay unit and the advance outwash.

Older Till

An older till unit was encountered in the northwest and southwest areas of the site. The lithology consists of a dense, well graded, silty gravelly sand. The till generally ranges in thickness from less than 1 to 10 ft and overlies the perching unit.

Perching Unit

The perching unit consists of an oxidized, dark to moderate yellowish-orange, dense, well graded, silty, gravelly sand. From field observation and laboratory analysis, the formation contains a significant proportion of fines and exhibits permeabilities in the range 1.8 x 10^{-6} to 7.2 x 10^{-7} cm/sec. The unit was identified in all of the deep test holes, is approximately 10 ft thick, and exhibits decreasing oxidation with depth. The perching unit's consistent distribution and low permeability is a significant factor in the selection and design of site remedial measures.

The upper surface of the perching unit exhibits a shallow uniform dip to the southwest, suggesting a preglacial erosional surface. A highly weathered zone was observed in the upper 5 to 10 ft of the perching unit, indicating an ancient soil profile.

GROUND-WATER CONDITIONS

Ground-water elevations were obtained from wells and piezometers screened in advance or recessional outwash deposits or in deeper undifferentiated deposits. Water levels were measured twice monthly between October of 1985 and August of 1986. On the basis of water-level and geologic data, two flow systems were identified, a shallow perched-water table aquifer and a deeper unconfined regional aquifer.

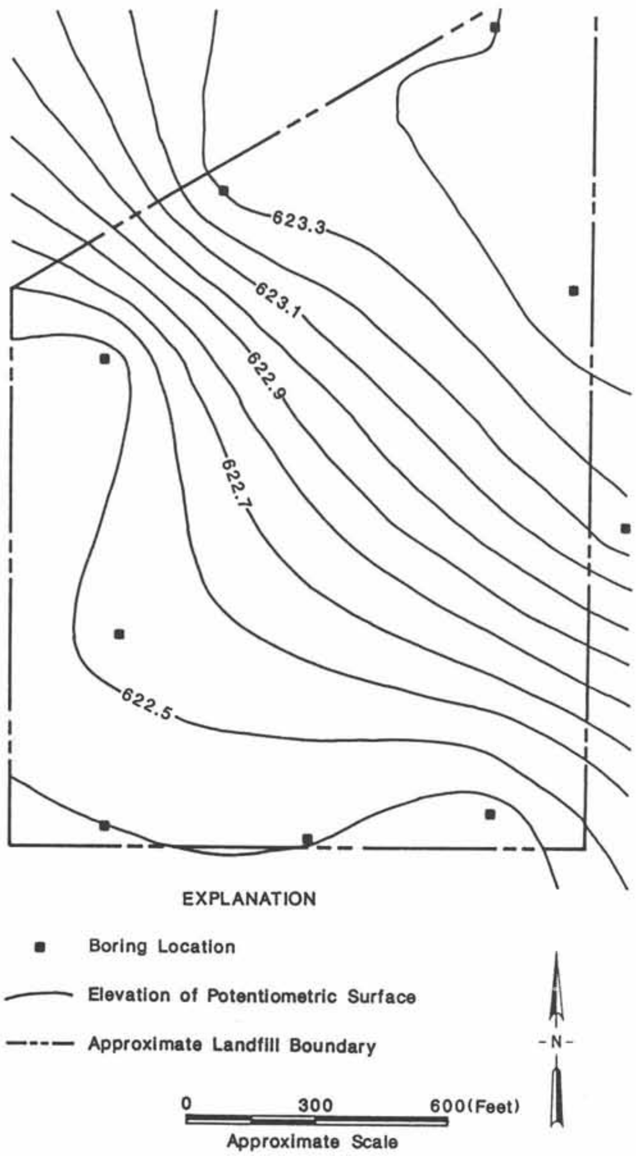

Figure 6. Potentiometric map of shallow aquifer October 1985, Hobart landfill.

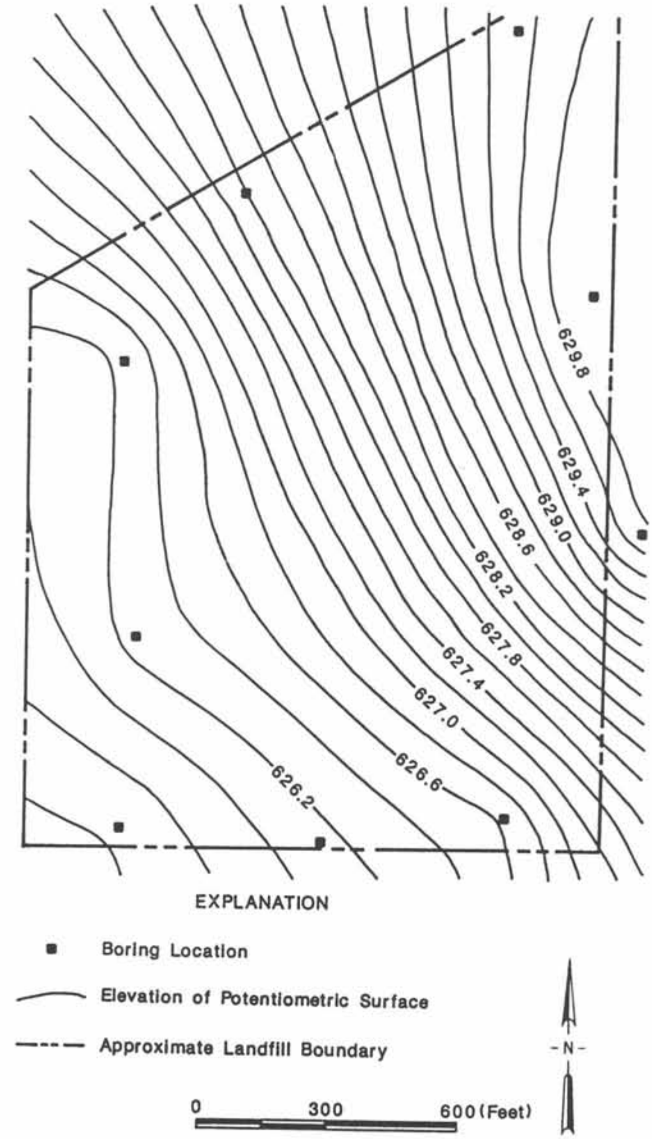

Figure 7. Potentiometric map of shallow aquifer January 1986, Hobart landfill.

Shallow Aquifer

The shallow water-table aquifer is continuous beneath the site and perched on the older undifferentiated deposits. The shallow aquifer occurs within glacial outwash deposits with static water levels ranging from 3 to 30 ft below ground surface. The saturated thickness of the shallow aquifer ranges from about 40 to 50 ft. Offsite monitoring indicates that the aquifer extends to the south, east, and west of the site. Direct precipitation upon the outwash sand and gravels and streamflow infiltration from the diversion canal are considered the two major sources of recharge to the shallow aquifer.

Potentiometric maps indicate that flow in the shallow aquifer is generally to the southwest. Seasonal fluctuations in flow directions and gradients are depicted in Figures 6 and 7. Ground-water levels measured in four solid waste piezometers indicate that seasonal saturation of the fill occurs at the south end of the landfill. It appears the solid waste reaches a maximum saturated thickness of 3 ft near boring SW-2. Stream-flow measurements suggest that seepage from the diversion canal is contributing to the saturation of the solid waste. Localized seepage losses of as much as 3 cfs were measured in the canal approximately 300 ft northeast of the landfill boundary.

Analysis of pump test results indicates that the shallow aquifer is very permeable. Transmissivities computed for the shallow aquifer indicate values of 150,000 to 300,000 gpd/ft. The specific yield computed from the pump test results was 0.20, which is typical for a water-table aquifer in these materials.

Regional Aquifer

A deeper ground-water flow system exists at a depth of 130 to 150 ft beneath the site. The aquifer occurs in undifferentiated sand and gravel material representing older glacial or interglacial deposits. Static ground-water levels range from 100 to 110 ft below ground surface. The deep aquifer appears to be unconfined. Recharge to the deep aquifer is possibly through percolation from the shallow aquifer and from sources outside the project area. Ground-water discharge from this aquifer is probably to the Cedar River approximately 1 mi south of the site. Water-level data indicate that locally the ground-water flow is toward the west.

GROUND WATER QUALITY

Regional ground-water quality for southwestern King County has been described by Luzier (1969) as generally good. Total dissolved solid contents are usually low, and most constituents are not present in concentrations high enough to cause significant problems, although iron and manganese are often higher than the secondary drinking water standards of 0.3 ppm and 0.05 ppm, respectively. The data presented by Luzier (1969) indicate that natural levels of iron can be expected to range between 0.01 and 4 ppm and levels of manganese from 0.01 and 0.8 ppm.

Ground-water quality in the monitoring wells was evaluated, and results show that the shallow water-table aquifer has been locally impacted by landfill operations. The deeper regional aquifer is being monitored both on- and offsite, and testing results to date indicate the landfill has not affected this aquifer.

Offsite Water Quality

Inorganic analyses of water samples from offsite domestic wells did not identify any significant impacts, except in one well west of the site, where concentrations of manganese ranged from 15.4 to 23.2 ppm during two winter sampling runs. A subsequent spring sample indicated manganese concentrations had decreased to 0.08 ppm, suggesting a seasonal influence on manganese concentrations.

Organic constituents of concern have not been generally detected in offsite private wells, with a few exceptions. Traces of diethylphthalate have been detected in water samples from two downgradient wells on single occasions. Methylene chloride was also detected in two of five samples from a downgradient well and in one of five samples from another downgradient well, at levels less than 20 ppb. This is felt to represent sample contamination in the laboratory. Also, traces of toluene (levels below 10 ppb) have been found in some samples collected from three wells (two downgradient and one upgradient).

Onsite Water Quality

Impacts on ground-water quality in the shallow aquifer are greatest along the western and southwestern margin of the site. Specific conductance values range from 6 to 25 times background levels (background is 60-80 μmhos/cm). Elevated concentrations of iron and manganese ranging from 15.4 to 32.1 ppm and 4.7 to 29.2 ppm, respectively, were found in nine monitoring wells.

Volatile organics detected in downgradient monitoring wells included benzene, ethylbenzene, and toluene. Benzene was detected in ten wells at concentrations less than or equal to 15 ppb. The current Recommended Maximum Contaminant Level (RMCL) for benzene is 5 ppb. Chlorobenzene was detected in four wells at concentrations up to 5.4 ppb. In addition, several chlorinated alkanes and alkenes (1,2 dichloroethane, 1,1-dichloroethane, 1,1,1-trichloro-ethane, 1,1,2-trichloroethane, and tetrachloroethylene) were detected in single wells, generally on isolated occasions, at levels less than 10 ppb. Toluene and ethylbenzene were found in a number of wells at concentrations as high as 150 ppb and 11 ppb, respectively.

RISK ASSESSMENT

Of the organic constituents detected in the sampled wells, the toluene and ethylbenzene components do not appear to pose a risk at this time since levels of concern (RMCL) are 2,000 ppb and 680 ppb, respectively. Levels of diethylphthalate detected in samples taken from two domestic wells are well below levels associated with human health concerns. The sporadic occurrence and generally low levels of the chlorinated volatile organics make it difficult to draw precise conclusions regarding their presence or source. Average concentrations of these volatile organic compounds for each well are below those defined as health concerns and are erratic in occurrence. The isolated nature of the detections casts doubt on any presumed average concentrations. A preliminary conclusion is that these constituents pose little or no hazard in the shallow aquifer.

The greatest potential risk is from benzene, which occurs in several wells at levels equal to or in excess of the proposed or pending 5 ppb RMCL. Detections have been noted only on or near the site, and no benzene has been detected in offsite domestic wells. Therefore, there is no obvious immediate health hazard. However, the County, wishing to take no chances, elected to implement remedial measures for controlling contaminant migration and reducing benzene concentration at the site.

REMEDIAL ASSESSMENTS/MEASURES

A three-dimensional finite difference model developed by McDonald and Harbough (1984) was used to evaluate landfill closure design alternatives. The goal of the modeling was to identify an extraction well pumping scheme which would reduce ground-water levels to at least 5 ft below the solid waste fill, consequently eliminating direct ground-water contact with the waste and minimizing leachate generation and offsite migration.

Three basic closure plans were simulated with the model. These included a well field to simply dewater the area and/or use of a slurry wall which would partially or totally surround the site.

For all simulations it was assumed that the entire landfill was covered by a low-permeability cap which allowed as much as 5 percent leakage from precipitation. A number of runs were made for each simulation using different scenarios including different pumping rates, number of pumping wells, and precipitation rates. Initial modeling results showed that a partial slurry wall did not appreciably lessen the amount of pumping required relative to no slurry wall. Therefore, only the continuous pumping and complete slurry wall approaches were fully developed and evaluated.

The first alternative modeled was continuous pumping and no slurry wall. This scenario consisted of several pumps clustered in the southwest portion of the site and running continuously to create a cone of drawdown extending beneath the landfill. The artificially created water-table level would be maintained several feet beneath the solid waste. Several pumping schemes were modeled, including those that utilized five pumps at 500 gpm for a combined total of 2,500 gpm or five pumps at 350 gpm for a total of 1,850 gpm. Initial pumping periods of 60 to 75 days were modeled, with maintenance pumping at a lower rate following. A removal of as much as 270 million gallons of water was required to pull down the water table prior to maintenance pumping operations. Although this alternative would have the least, initial, in-place capital expenditures, effluent treatment costs were estimated to be $720,000/yr.

A second alternative incorporated a slurry wall constructed to completely surround the solid waste fill portion of the site and penetrate into the perching unit. A total of four runs was made for this simulation. Two runs simulate one 500-gpm well in the southwest corner of the site with 2 and 8 in. of rainfall recharge, while the other runs simulated three 250-gpm wells in the same area. A transmissivity value of 150,000 gpd/ft was used in all runs to reflect the effect a slurry wall would have in reducing recharge and the saturated thickness of the aquifer. A pumping period of 60 days was selected from trial runs which indicated that any additional pumping time would result in unnecessarily large drawdowns. Modeling results indicated that maintenance pumping requirements with a slurry wall would be about 5 to 25 gpm. The initial ground-water volume requiring removal during the initial drawdown would be 11 million to 29 million gallons. Construction costs would be high for this alternative, but yearly pump and slurry wall maintenance and water treatment costs would be around $60,000.

On the basis of the results of the modeling effort and the comparative cost analysis, closure designs that include a slurry wall appear to offer the best conditions for achieving the goal of lowering the water table and controlling leachate migration. Total estimated construction costs are between $1.5 million and $2.0 million.

While the slurry wall design is under development, there remains a potential for impacts to the shallow aquifer from landfill operations. Therefore, a water-well replacement program was implemented in March 1986 to provide new wells for local residents whose wells were drawing from the shallow aquifer. Four new wells were drilled into and sealed in the deep regional aquifer.

ACKNOWLEDGMENTS

Deborah P. Lambert of the King County Solid Waste Division provided many valuable suggestions and useful comments, as well as assistance throughout the preparation of this manuscript.

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Hydrogeologic Investigation for Closure of the Grandview Landfill

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INTRODUCTION

A reconnaissance of the Grandview landfill was conducted April 14-16, 1986. Surface geology was mapped, and areas were identified as potential soil location sites for cover or cap material, outcrops of basalts, or solid waste disposal areas. In addition, landfill operational practices were observed.

From this initial site investigation, locations were selected for 40 onsite test pits. The test pits were dug on April 22 and 23, 1986, using a backhoe. Information gathered from the test pits was used to estimate vertical thickness of soil units.

Located adjacent to the landfill is the City of Grandview Sewage Treatment Plant. Sewage treatment consists of surface ponds, located in depressions in the basalt, and an aeration process using irrigation sprinkler systems. Treated sewage water was sprinkled on top of the basalt and part of the landfill. This practice has caused the formation of several surface ponds at the landfill site. The City of Grandview has altered sewage sprinkling to exclude the landfill area and minimize impacts to ground water.

PHYSIOGRAPHY

The Grandview landfill site covers 38 acres and is located in the lower Yakima River valley, south of the Yakima River, and approximately 2 mi south of the City of Grandview (Figure 1). Farther south lies the escarpment of the Horse Heaven Hills. The City of Prosser lies approximately 7 mi east of the site.

The topography of the Grandview landfill is relatively flat. The site is characterized by small ridges and shallow valleys. Generally, the ridge tops are composed of basalt and the valleys are filled with soil. Many of the valleys have been flooded because of sewage treatment practices.

The climate of the region is arid. Summers are generally hot and dry, and winter has the wettest month. National Oceanic and Atmospheric Administration data for Sunnyside, Washington, indicate the average annual temperature is 59° F and the normal annual rainfall is 6.7 in. Regional climate affects the legal requirements for the regulation of solid waste facilities.

GEOLOGY

The lower Yakima valley is in the west-central part of the Columbia Plateau province, which is an area underlain by the Columbia River Basalt Group. Covering approximately 100,000 sq mi, the Columbia River Basalt Group is a massive flood basalt which has been subdivided into five formations. Figure 2 shows the generalized stratigraphic column.

The basalts are overlain by sedimentary units that are Pleistocene to Holocene in age. In the vicinity of the Grandview landfill site, the sedimentary units include alluvium, loess, catastrophic flood gravels, landslide deposits, and soil. Alluvium consists of primary stream deposits of silt, sand, and gravel. Loess deposits are composed of wind-blown silt and fine sand. Coarse gravels and sand were deposited by waters from catastrophic flood events emanating from the Pleistocene glacial Lake Missoula to the northeast. Landslide deposits are unstratified and poorly sorted clay, silt, sand, and gravel. Unconsolidated sediments are composed of silty sand, sand-silt mixtures, silt, and very fine sand.

Subsurface data were obtained from reports about Washington Department of Ecology (DOE) water wells (Figure 3). The relations of the basalt and overlying sediments are shown in cross-sections A-A′ and B-B′ (Figure 4).

The site geology of the Grandview landfill is characterized by basalt overlain by relatively thin sand and gravel soil units. The basalt has been mapped previously as Saddle Mountains Basalt (Myers and others, 1979). The topography of the site is controlled by the outcropping of basalt. At the surface, the basalt appears blocky with abundant vertical fractures. Structural integrity of the deep basalts is unknown. Although the basalt has the ability to retain surface water, as shown by the existence of many ponds on the site, water from the ponds may be infiltrating at slow rates.

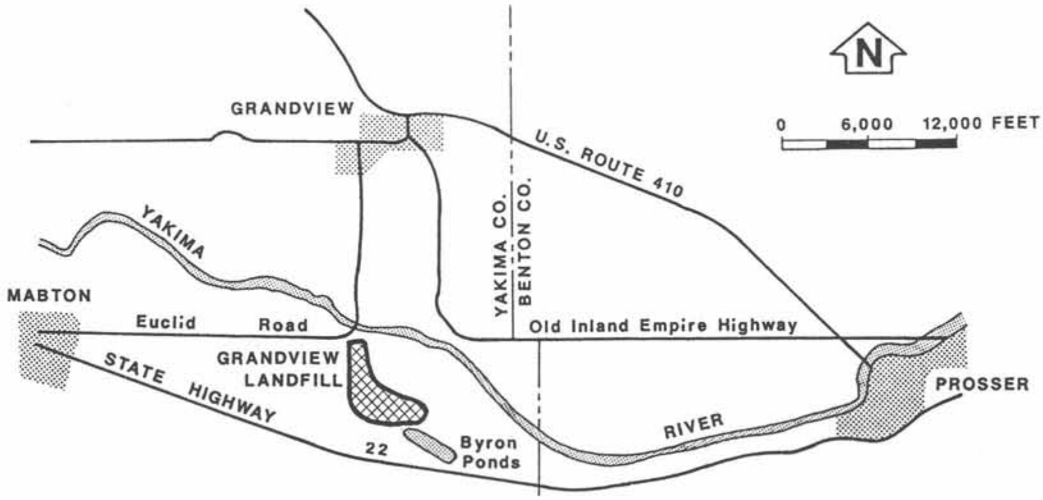

Figure 1. Location map for Grandview landfill.

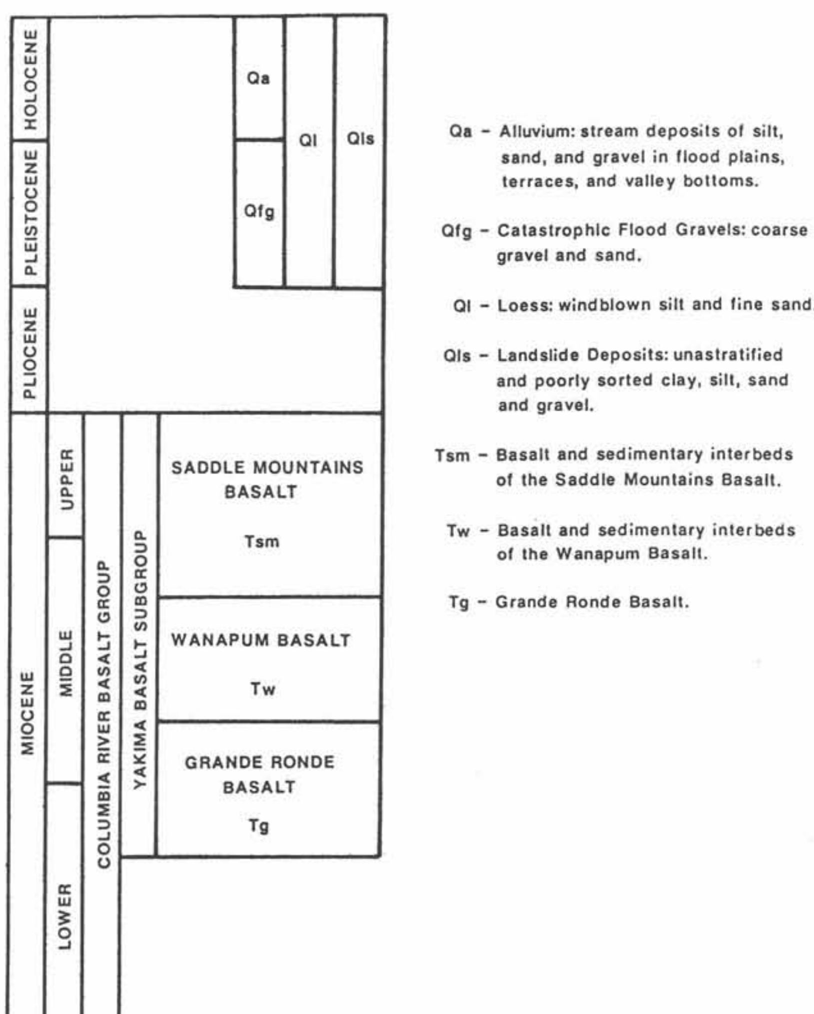

Figure 2. Stratigraphy of lower Yakima River valley.

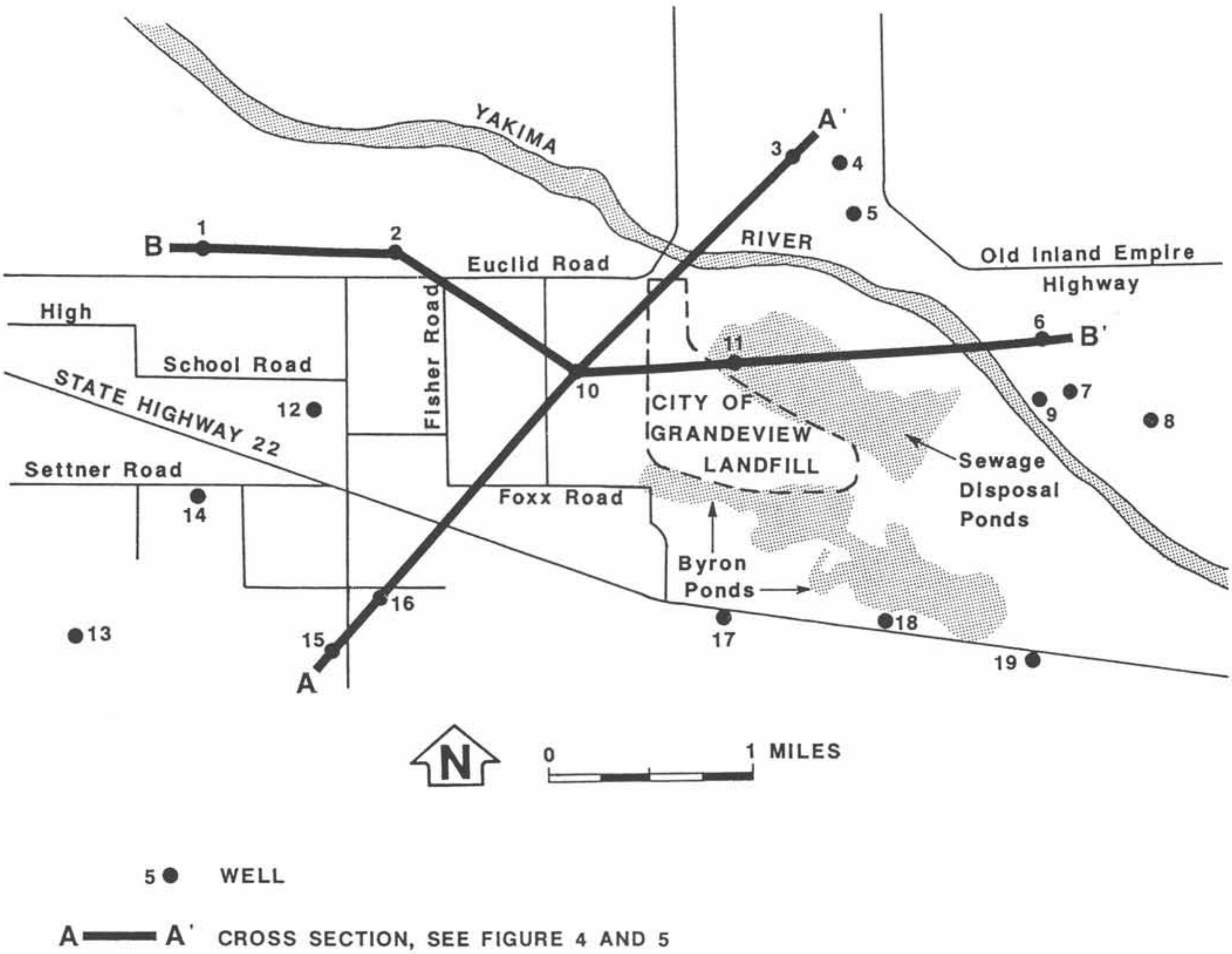

Figure 3. Water wells in the Grandview landfill area.

The surficial units at the site are composed of unconsolidated deposits of a silty fine sand overlying clayey, silty fine sand, gravel, gravelly sand, silty gravelly sand, and boulders and cobbles of basalt. The silty fine sand and clayey silty fine sands are the soil units most suitable for use as cover material for landfill closure.

The soil occurs mainly in the shallow valleys as small pockets throughout the site. The southwest corner of the property contains the largest deposit of soil.

The total estimated volume of soil available for cover is 200,000 cy. The estimate was made using a planimeter and an estimate of the vertical thickness. The data used to estimate the vertical thickness included logs for the test pits excavated during this study, six monitoring well logs, and 25 test pit logs from a 1982 GeoEngineers investigation for a proposed penitentiary. Due to the varied nature of the unconsolidated deposits, thickness estimates are conservative; however, distribution of soil types is such that the available material should be treated as a composite soil of moderate permeability (1×10^{-3} to 1×10^{-5} cm/sec).

Of the estimated 200,000 cy, 85 percent is located within the area of the full-circle sprinkler system (Figure 6). Approximately 60 percent of the soil is located in the southwest corner of the site. Much of the total volume is presently saturated with water due to sprinkling practices; thus removal may be difficult. In addition, the presence of large basalt boulders and small basalt outcrops will impede soil removal, reducing the total material available for use as cover.

OCCURRENCE OF GROUND WATER

As infiltrating precipitation saturates the waste material, high concentrations of inorganic and organic substances can be leached from the waste. The volume of leachate produced is a function of the amount of water percolating through the landfill, which in turn is

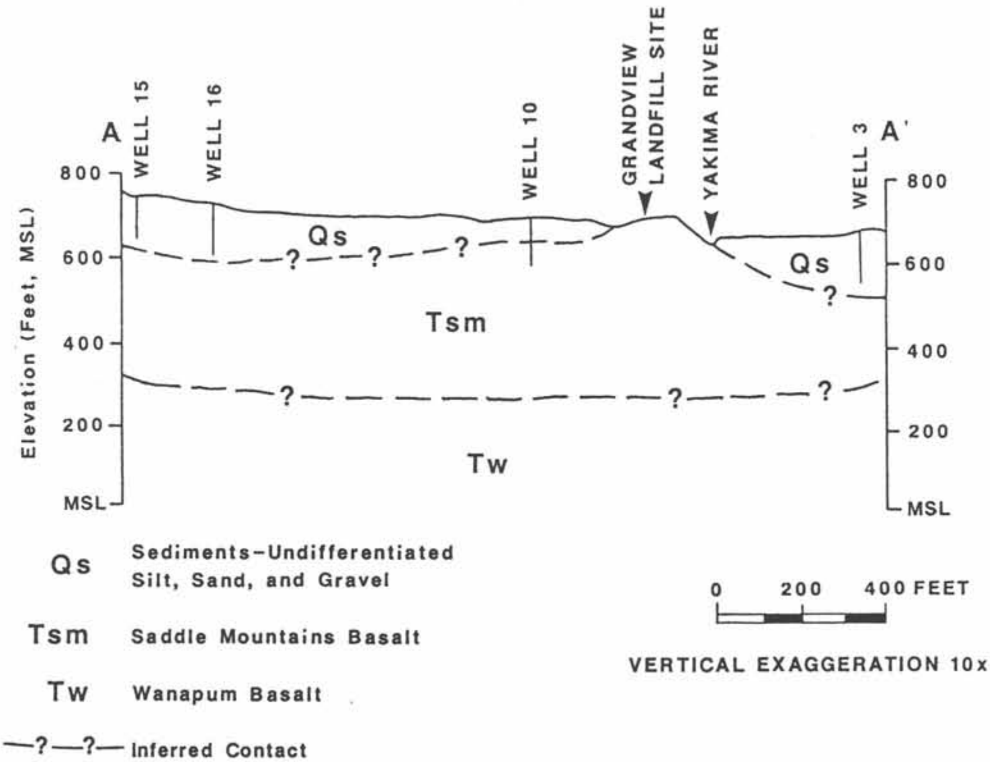

Figure 4. Geologic cross-section A-A′ for Grandview landfill; see Figure 3 for cross-section location.

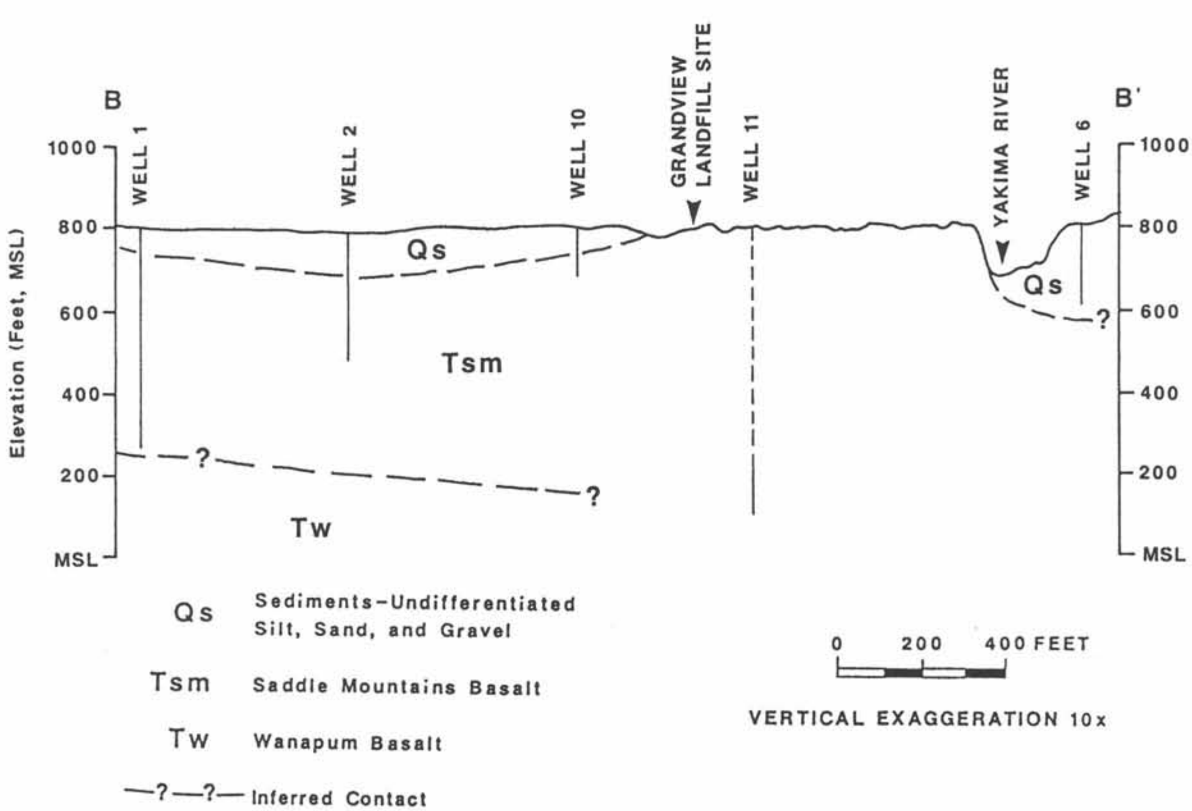

Figure 5. Geologic cross-section B-B′ for Grandview landfill; see Figure 3 for cross-section location.

dependent on a number of interrelated climatological and soil conditions.

In arid climates such as that of Grandview, low precipitation is a relatively minor contributor to leachate generation; however, the presence of shallow ground water within the waste may be a significant contributor to the generation of leachate. As leachate is developed, the potential for contaminating both local and regional ground-water supplies is increased, depending upon the flow systems and contaminant transport characteristics in the geologic formations underlying the site.

Ground-water movement in eastern Washington can be divided into regional, intermediate, and local flow systems. The regional and intermediate flow systems are within the deeper basalt units. Recharge areas include mountain ranges on the perimeter of the Columbia Plateau and, to some extent, the upper elevations of the plateau and upper Yakima River valley. The discharge areas are the larger regional drainage systems, lakes, and reservoirs. Local flow systems are superimposed on the regional and intermediate systems. Recharge and discharge are controlled by local topography and geologic conditions. In the Grandview area the upper part of the local flow system is believed to be locally perched on or within the shallower basalts and is greatly influenced by onsite activities.

Regional/Intermediate Ground Water

The Yakima River appears to be a local and possibly an intermediate ground-water system discharge area. U.S. Geological Survey stream gaging stations are located upstream at Cle Elum and downstream at Kiona. Gephart et al. (1979) present U.S. Geological Survey and U.S. Bureau of Reclamation data that indicate a net inflow from ground water along the Cle Elum-Kiona reach, although local conditions are not defined at this time.

Construction of a potentiometric surface map was not practicable due to insufficient data on well construction and uncertainties in water-level data. Wells are typically 200 ft or deeper, and aquifers are generally confined. To the south of the Grandview landfill site is the escarpment of the Horse Heaven Hills. DOE water-well report data indicate ground-water flows from the topographic high areas in the Horse Heaven Hills toward the Yakima River.

Recharge to ground water in the Grandview landfill area is mostly from local precipitation, deeper ground water discharging in the area, and artificial infiltration. Local precipitation, although sparse, infiltrates and travels down topographic gradient toward the Yakima River. Gephart et al. (1979) cited evidence that inferred a general upward movement of ground water from deeper units.

Shallow Ground Water

Shallow ground water of the local flow system at the Grandview landfill is heavily influenced by local precipitation and artificial recharge. Six piezometers were installed in the shallow system during the 1982 GeoEngineers investigation (Figure 6). Depth to water was monitored during this study's field investigation.

Over the period of the fieldwork (April 14 to April 23, 1986), a general decrease in water levels was observed (Table 1). This drop may be a seasonal effect. Two of the piezometers (MW-4 and MW-7) experienced unusual water-level responses. MW-4 is located next to a pond where the water level was lowered by pumping the water into Byron Ponds. The monitoring zone in MW-4 is 1.5 to 11.5 ft below the surface. MW-7 (near Byron Ponds) was influenced by the discharging water from the same pond. The water was discharged from a rubber hose 7 ft upgradient from MW-7.

Monitoring information from GeoEngineers (1982) indicates a north-northwest flow direction for the shallow ground-water system. However, ponding may influence and/or obscure the actual direction of ground-water flow.

SEWAGE TREATMENT PRACTICES

The sprinkling practice and sewage effluent ponding control the water level in many of the ponds. Figure 6 shows the position of the landfill sewage treatment ponds as well as the track of the sewage treatment sprinkler. Sewage treatment practices apparently also affect the perching of the shallow local flow system.

Sewage effluent irrigation occurred directly on landfill material. The irrigation sprinkler traversed landfill material in the south-central area of the property. The potential for leachate generation was increased from these practices. If the water level in the ponds near MW-4 rises to a high level, it impedes the movement of the irrigation sprinkler. To continue operation of the sprinkler, the pond water was then pumped over a small ridge into Byron Ponds, a wildlife refuge.

MINIMUM FUNCTIONAL STANDARDS

The Washington Department of Ecology Minimum Functional Standards (MFS) for Solid Waste Handling (WAC 173-304) include the construction and groundwater monitoring requirements. The MFS state that a minimum of three downgradient and one upgradient wells should be installed. Water quality should be monitored to assess the impacts from landfill operations. Water-quality degradations are determined statistically by comparing upgradient to downgradient water-quality data.

The MFS require a low-permeability liner be used for continued operation of the landfill. Typically, liners are

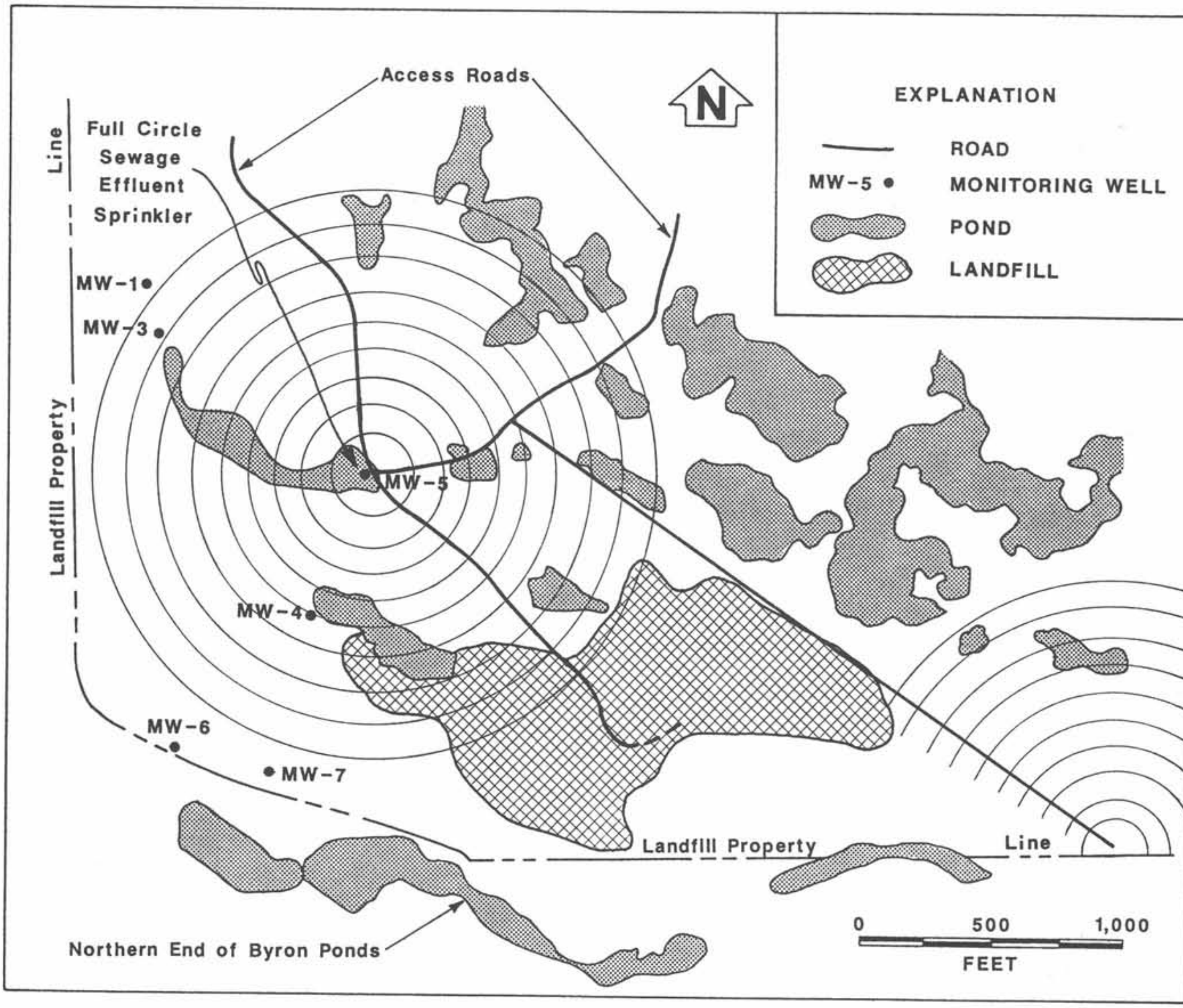

Figure 6. Site map of the Grandview landfill.

Table 1. Piezometer depth to water, Grandview landfill

	Piezometer (zone (ft)	Monitoring date 4/14	4/16	4/21	4/23
MW-1	28-38	13.61	13.69	13.68	13.83
MW-3	4-14	6.23	6.26	6.53	6.81
MW-4	2.5-11.5	1.48	1.82	2.23	2.36
MW-5	3-13	2.16	2.26	2.37	2.58
MW-6	9-19	3.06	3.11	3.27	3.31
MW-7	9-19	8.34	7.99	.18	8.34

of fine-grained materials that have hydraulic conductivity less than 1 x 10^{-7} cm/sec. The onsite basalt may also be suitable for a liner if the fracture hydraulic conductivity can be shown to be less than 1 x 10^{-7} cm/sec. In arid environments (locations with less than 12 in. of annual precipitation) the DOE may waive liner requirements provided the bottom of waste is no less than 10 ft above the seasonal high water table and the vadose zone (unsaturated zone above water table) is monitored. Because of the complex nature of unsaturated flow in basalts, vadose zone monitoring may not be practical or manageable.

In addition, a close-out cap or cover is required at closure. As with a liner, fine-grained materials with low permeability are necessary. In arid environments, hydraulic conductivities of 1 x 10^{-5} cm/sec are required for the cap.

At the Grandview landfill, the operation of the sewage treatment facility directly affects the ground-water monitoring program, liner requirements, and cover materials. The intent of reduced hydraulic conductivity standards for cap and liner material in arid environments is to have a more practical and manageable system, while avoiding degradation of ground-water quality. The sewage effluent irrigation practices jeopardized this intent. The additional water from this irrigation may have already increased leachate production and increase the risk of contamination of ground water.

CONCLUSIONS

This project was the first step in the landfill closure process as required by the MFS. Issues were identified and recommendations were made on the next phase of the hydrogeologic investigation.

The second phase should include the installation of at least four monitoring wells. The hydrogeologic zones of interest include the shallow ground water as well as the basalt aquifers. Ground waters and surface waters should be sampled quarterly to determine background levels of constituents of interest. The parameters for analysis are defined in the MFS. Additional parameters may be defined at a later time.

In addition, test boring should be completed to determine if refuse has been saturated with shallow ground water. Information from the test borings may be used with a moisture balance model to determine leachate-generating potential.

Data from the monitoring wells and perhaps the test borings can be used to determine the direction of ground-water flow in the shallow aquifer. Hydraulic gradients estimated from monitoring data can be used with aquifer test results to help estimate ground-water travel times.

The aerial sprinkling of sewage effluent and the disposal of solid waste are incompatible site uses. Saturation of existing solid waste or future waste expansions could increase leachate generation and other environmental problems.

The application of sewage effluent has affected the shallow ground water. Formation of surface ponds has occurred in the low areas in the basalt. Water levels in monitoring wells located near a surface pond rose and fell in concert with the level in the ponds. The potential for leachate migration was greatly increased by pumping the surface ponds into the adjacent Byron Ponds.

Based on the findings of this investigation and a revised Solid Waste Management Plan (R. W. Beck and Associates, Inc., 1986), the City of Grandview has been able to alter its sewage sprinkling operations to exclude the landfill area and minimize the potential for ground-water contamination.

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Ground-water sample collection for a temporary drive point well screen, King County. Photograph by Denise E. Mills.

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Evaluation of Pollution Potential and Monitoring Strategies for Eight Landfills in Island County, Washington

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INTRODUCTION

From September 1984 through May 1986 the Island County Health Department conducted a ground-water quality assessment of eight landfills and one industrial disposal site. This paper describes the evaluation of relative pollution potential for the eight landfills and development of a comprehensive monitoring strategy. The project was funded by the Washington Department of Ecology 205J program. The objective of the project was to assess the relative pollution potential of each site and design monitoring programs such that limited funding for monitoring ground water could be spent wisely.

STUDY AREA CHARACTERISTICS

Geographic Setting

Island County is located in the Puget Lowland at the eastern end of the Strait of Juan de Fuca. It includes Whidbey and Camano islands, a total area of approximately 210 sq mi. Whidbey Island is 40 mi long, and Camano Island is 15 mi long (Figure 1). No point on either island is more than 2.5 mi from marine water due to the irregular shape of the shorelines. Rolling uplands characterize the land surface, and hilltop elevations typically range from 100 to 300 ft above sea level, although some areas reach elevations from 400 to 600 ft. The shorelines are generally backed by steep slopes or cliffs. The sites under investigation are all located in upland areas or on their sloping margins. Seven landfills are located on Whidbey Island, while one landfill is on Camano Island.

Climate

The climate of Island County is characterized by dry summers and wet winters. The temperature varies from a January average of 38°F to a July and August average of 61°F. The mean annual temperature is 50°F. The central and northern parts of Whidbey Island and part of Camano Island are within the rain shadow of the Olympic Mountains. This results in an average annual rainfall of 18 to 20 in. (Figure 2). The rain shadow begins to lift at Greenbank, so rainfall on the southern part of Whidbey Island is well over 30 in./yr and increases with land surface elevation (Easterbrook et al., 1968). The three southernmost study sites are on the boundary of or outside the rain shadow. The prevailing winds in the county are from the northwest in the summer and the southwest in the winter. Strong winds are not common. The Strait of Juan de Fuca modifies this general pattern over northern Whidbey Island, increasing the wind strength and shifting the direction to the west and northwest.

HYDROGEOLOGY

Island County is underlain by a complex sequence of glacial and interglacial materials deposited during the Quaternary period (approximately 11,000 yr to 1.6 Ma). Older metamorphic bedrock is present on the north end of Whidbey but has not been identified near or underlying any of the sites under investigation. Most of the county is located in a downdropped regional structural block (Marysville Low) filled with as much as 2,000 ft of sediment (Stoffel, 1981). Most of this sediment reflects the cyclic glacial activity of the Pleistocene Epoch.

During the Quaternary, several continental glaciers advanced and retreated across the county (Crandell et al., 1958). The most recent was the Vashon Stade of the Fraser Glaciation (Armstrong et al., 1965). The Vashon ice was more than 5,000 ft thick in the vicinity of Island County, and its passage left a thick sequence of glacial drift divided into three major units:

- Vashon recessional outwash (sand and gravel)
- Vashon till (compact sandy silt and gravel)
- Vashon outwash (sand and gravel)

During periods of maximum glacial retreat, clay, silt, and sand accumulated as interglacial deposits.

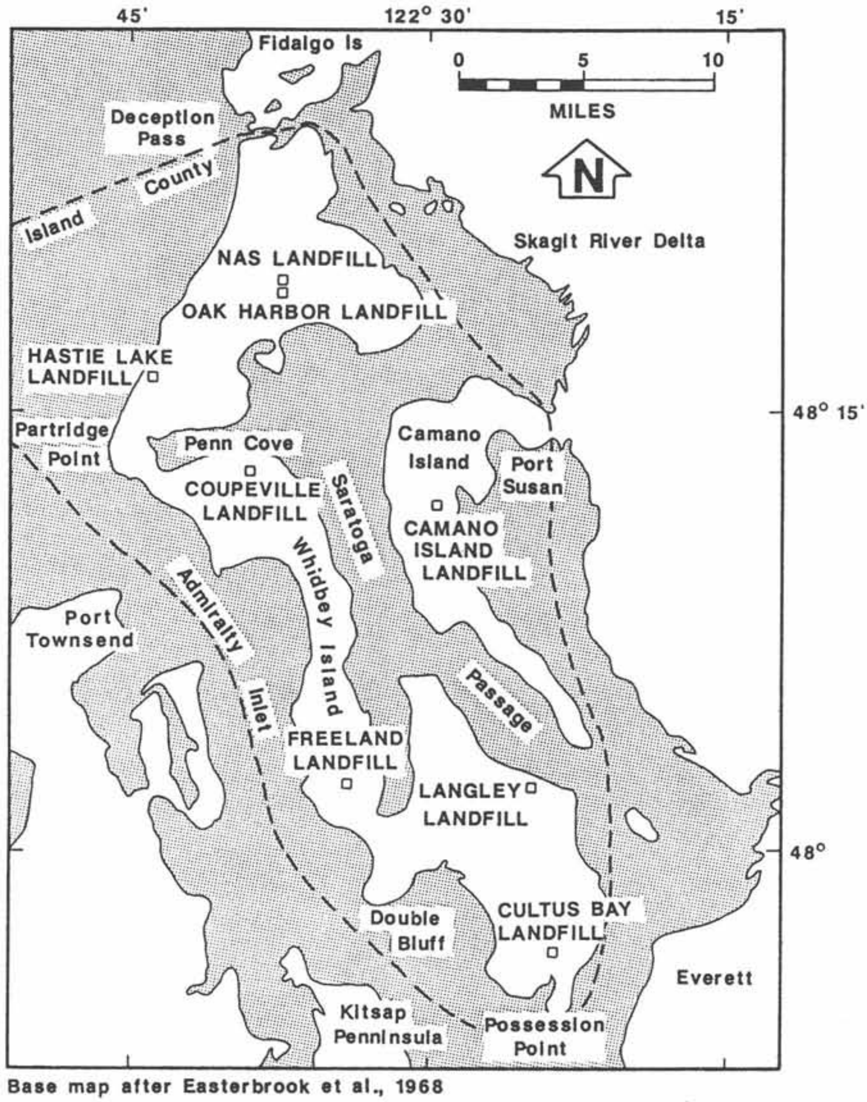

Figure 1. Locations of landfills studied in Island County.

Ground water in Island County is typically withdrawn from the coarser glacial deposits, while the till and interglacial deposits serve as aquitards. Three principal aquifers are present and were considered in evaluating the potential for pollution at each site as well as the design of the monitoring strategy.

The first aquifer encountered below the ground surface is saturated recessional outwash perched on the Vashon till. This perched aquifer is of limited areal extent and is not developed for beneficial use at any of the sites under investigation (Figure 3).

The next aquifer encountered is the basal portion of the Vashon advance outwash. This is the shallowest major aquifer in use near the sites under investigation and is the most likely to be affected by the landfills. Precipitation infiltrates through the unsaturated upper portion of the advance outwash, especially where the till is thin or absent, to the underlying low-permeability transition beds. Ground water occurs in an unconfined condition in this aquifer. In many areas, water is also found perched on silt layers (lake deposits) within the outwash.

The deepest aquifer identified as being in use in the areas under consideration has been named the "Sea Level Aquifer" (Cline et al., 1982). This aquifer occurs between 30 ft above and 200 ft below sea level. Its piezometric level is commonly within 30 ft of sea level and above the base of the overlying transition beds, indicating confining conditions. In some areas near the coast, pumping has drawn the water level down below

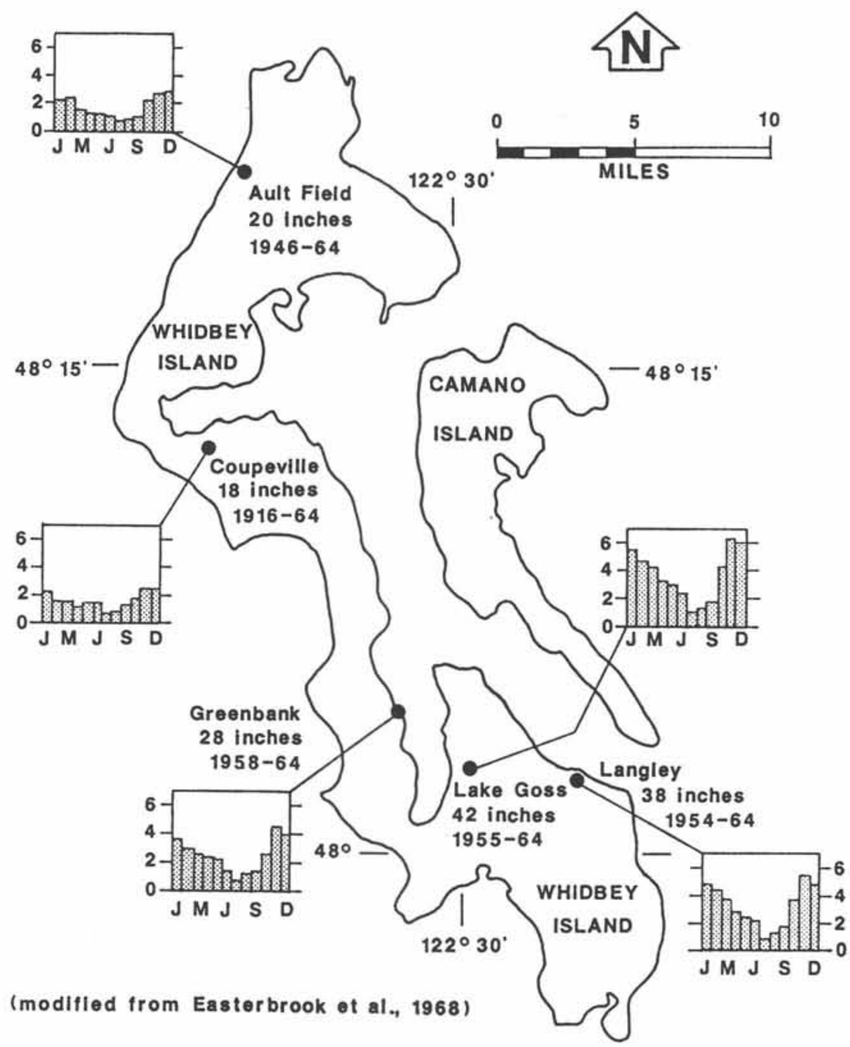

Figure 2. Precipitation at five sites in Island County.

sea level, creating the potential for sea-water intrusion. The "Sea Level Aquifer" is the most heavily exploited aquifer in the county because it provides sufficient yields for water supply and agriculture.

Ground-Water Development

Ground water is the primary source of potable water in Island County. About 84 percent of the ground water withdrawn from wells is used for household purposes. The remaining 16 percent is used for irrigation and industry. Estimated ground-water pumpage in 1981 was 4 cfs for Whidbey and 1 cfs for Camano Island (Sapik et al., 1987).

Personnel of the Island County Health Department identified 248 wells in use within 1 mi of the eight sites included in the study. The shallow and perched aquifers are primarily used by wells in upland areas, but the largest volumes are generally withdrawn from the sea-level aquifer.

Water Quality

Very little ground-water quality information is available for any of the landfill sites. Data obtained by Island County Health Department personnel indicate that, in general, the chemical quality of regional ground water near the sites under investigation is good. However, ground water at several of the sites is known to have high iron and manganese concentrations, which may be naturally occurring or due to impacts from landfilling operations.

Most areas of the county appear to have moderately hard to hard water. Significant differences in the water quality between the deep, shallow, and perched aquifers are not apparent with the available data. Ground-water contamination from salt-water intrusion poses a problem in Island County but does not appear to be a problem near most of the sites studied.

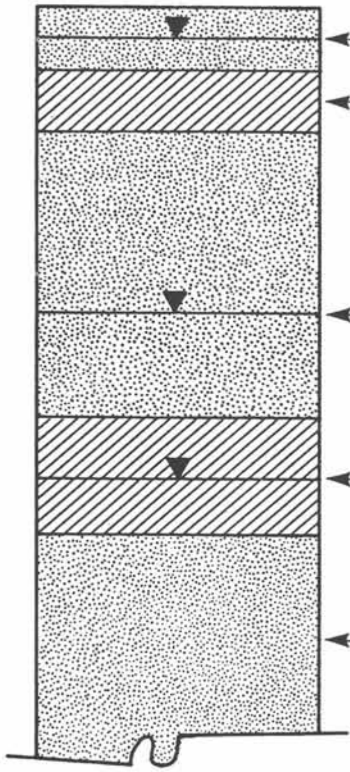

Figure 3. Hydrostratigraphic relations in Island County.

LANDFILL LEACHATE GENERATION

The first step in evaluating a landfill's potential for contaminating ground water is to estimate the amount of leachate generated by the landfill. As infiltrating precipitation saturates the waste material, high concentrations of inorganic and organic compounds can be leached from it. The volume of leachate produced is a function of the amount of water percolating through the waste, which, in turn, is dependent on a number of interrelated climatological, vegetative, and soil conditions that were evaluated using the water balance method.

For seven of the landfill sites under investigation, the Thornwaite and Mather method was used to estimate the water balance and subsequent potential leachate generation (Fenn et al., 1975). The absence of vegetation at the Naval Air Station (NAS) site negated usefulness of this method. Experience has shown that it is conservative but reasonable to estimate a 50-percent infiltration rate where vegetation is absent. The water balance method is based on the relation between precipitation, evapotranspiration, surface runoff, and soil moisture storage. Since a precise knowledge of all these factors is rarely possible and field measurement is difficult, they have been estimated for this study from known site conditions and published data.

As Table 1 illustrates, all of the landfills generate leachate. As discussed earlier, differences in leachate generation between sites are related to extent of vegetative cover, annual precipitation, and area underlain by waste.

It should be emphasized that the calculated values for leachate generation represent simplified conditions for the sole purpose of comparing sites.

WASTE CHARACTERIZATION

The information on history, operation, and waste types in this report is based on research by the Island County Health Department. Many of these land burial sites began as burning dumps in the 1950s and were located, for convenience, in gravel/sand pits. This was typical waste disposal practice for rural areas at the time. Only the NAS and Coupeville disposal sites were operating at the time of this investigation, although the Oak Harbor site was operating sewage sludge lagoons and the Freeland site was used as a restricted landfill and recycling center.

All of the sites had received domestic/municipal types of solid waste. At least four of the sites had reportedly received sewage sludge and/or septic pumpage. Some liquid industrial wastes, including cleaning sol-

Table 1. Summary of annual leachate generation for Island County

	Acres underlain by waste	Annual precipitation (in.)	Leachate discharge (gpm)	Leachate volume ($x10^4$) (gal/acre/yr)	Calculated annual leachate generation (gal/yr)
Naval Air Station	6.13	20	3.2	27	1,660,000
Oak Harbor	15.0	20	2.6	9.2	1,373,000
Hastie Lake	3.0	18.6	0.4	7.3	222,000
Coupeville	7.4	18.6	1.7	12.2	905,000
Camano Island	2.0	18.6	0.5	12.0	241,000
Freeland	2.4	28	0.7	14.5	370,000
Langley	2.2	38	1.9	45.0	989,000
Cultus Bay	2.5	38	1.6	34.6	866,000

vents and waste oil, reportedly were disposed of at several sites.

The NAS and Coupeville sites are the only landfills for which estimates of annual waste volumes received exist. Since most of the closed sites were burning dumps during much of their history, the in-place waste volumes are generally low at these sites.

SITE EVALUATION

A comprehensive site evaluation was performed for each of the eight landfills. (For details, refer to Sweet, Edwards & Associates, Inc., 1986). The evaluations included preparation of conceptual ground-water models for each site (such as cross-sections, geological maps, and potentiometric maps). Geology was field checked and water levels measured in existing supply wells where possible.

POLLUTION POTENTIAL

Given limited funding resources for monitoring and a large number of sites, the first step in developing a ground-water monitoring strategy is to determine the pollution potential for each of the landfills under investigation. The main factors governing the pollution potential of a specific site include:

- Leachate discharge
- Age and type of facility
- Type of waste
- Pollutant mobility to saturated zone
- Beneficial use

Pollutant mobility within the saturated zone is also an important consideration. However, hydrogeological analysis indicated that all sites possessed sufficiently similar hydrogeologic characteristics to preclude pollutant mobility within the saturated zone as a meaningful comparative criterion.

A numerical system was developed to determine the relative pollution potential of the eight sites. The five listed factors were assigned a relative rank for each site. The ranks are defined as (1) low, (2) moderate, and (3) and (4) high potential for pollution. The ranking of each factor is described in the following sections and is based on experience in similar studies and knowledge of the hydrogeology and landfill operations of the county. Additionally, each factor is assigned a multiplier that is based on the perceived importance of the factor. The multipliers may vary between 1 and 5 as described in the following sections. The sum of the products of the assigned ranks and multipliers for each landfill defines its Pollution Potential Rating.

The objective of this study has been to establish an approach for allocating Health Department resources available for landfill ground-water monitoring. The system we developed does not determine if a site is polluting ground water. It attempts to define the relative potential for ground-water contamination of the eight landfills and thereby which landfill should be monitored with the available resources. The system should be considered flexible and be modified as additional information is required.

Leachate Discharge

The greater the leachate discharge at a waste disposal facility, the greater the potential for exceeding safe concentrations of pollutants in drinking water. On the basis of the moisture balance analysis for the sites (Table 1), each landfill was ranked as follows:

Pollution-Potential Rank	*Leachate Discharge*
1	0-1 gpm
2	1-2 gpm
3	>2 gpm

Because leachate discharge is one of the most influential factors, it was assigned a multiplier of 5.

Age and Type of Facility

The age of a landfill affects the concentration of contaminants which might be generated and detected by monitoring. Older, inactive landfills commonly have exceeded their peak potential for leaching contaminants from the waste. Older sites where waste burning was practiced also tend to exhibit lower concentrations of selected contaminants due to the buffering action of burned residue.

Rank	*Age of Facility*
1	Old, closed burning dumps
2	Recently active or restricted sites
3	Active

While age is a factor to be considered, it is relatively minor with respect to other pollution potential factors and was assigned a multiplier of 1.

Type of Wastes

Not all wastes pose the same hazard to public health. Ideally, waste facility operations should screen and regulate the type of wastes accepted and prevent the improper disposal of dangerous or hazardous waste. In practice, this is difficult to achieve. However, small rural facilities that serve small communities and individuals typically take in refuse with less pollution potential than facilities that serve industrial operations or large municipalities.

Waste disposal facilities which are limited to demolition debris and wood waste are less of a hazard than those which receive a wide variety of other wastes. Wood wastes often include treated wood products which might contain preservatives classified as hazardous. Sites which have received both wood wastes and municipal and industrial wastes are of particular concern because the wood disintegration process generates chelates. Chelates increase the subsurface mobility of other contaminants (particularly toxic metals).

All the sites investigated received domestic and municipal wastes. Some of the small rural sites have received limited amounts of industrial waste (that is, dry-cleaning fluid). Other sites regularly received industrial and municipal sludges. Therefore, sites are ranked on the reported portions of industrial effluent, municipal sewage, and hazardous waste received.

Rank	*Waste Type*
1	Domestic, municipal waste only
2	Domestic, municipal waste with small fraction of industrial/municipal sludge
3	Domestic, municipal waste with large fraction of industrial/municipal sludge
4	Hazardous waste

This pollution potential factor has been assigned a multiplier of 3.

Pollutant Mobility to Saturated Zone

Geologic materials above the water table (vadose zone) commonly serve to remove pollutants from downward percolating waters. Pollutant attenuation is affected by numerous mechanical, biological, and chemical processes. Mechanical factors important in mobility of pollutants within the vadose zone include the thickness of unsaturated sediments, filtration, and sorption. Filtration and sorption are functions of the type of soil materials, particularly texture and grain size. All the sites under study are underlain by similar materials (sand and gravel). Therefore, material type is not an important consideration in the relative pollution potential of the eight sites.

The thickness of unsaturated sediments is important in that the greater the distance the pollutant must travel through unsaturated materials, the longer the time of migration and the greater the opportunity for other attenuation processes to affect the pollutants. When the water table is shallow (for example, 5 or 10 ft deep), there is little opportunity for attenuation before the pollutants reach the ground water. Where the unsaturated zone is thick (for example, greater than 50 ft), a considerable amount of attenuation can take place, substantially reducing the amount of pollutant reaching the water table.

Rank	*Depth to Ground Water*
1	>50 ft
2	10 to 50 ft
3	<10 ft

This pollution potential factor was assigned a multiplier of 3.

Beneficial Use

The distance to and number of wells and surface water near a waste disposal site must be considered when evaluating pollution potential because:

(1) Improperly sealed or constructed wells can serve as conduits for contaminants reaching the ground water.

(2) Wells provide drinking water supply to the public.

(3) Surface waters are a potential source of public exposure to contaminants.

In order to address the full range and degree of beneficial use, a separate ranking system was established for this pollution potential factor.

Sub-Rank	*Beneficial Use Considerations*
1	Nearest well >1,000 ft downgradient
2	Nearest well 100-1,000 ft downgradient
3	Nearest well <100 ft downgradient
1	<5 wells within 1 mi downgradient
2	5-10 wells within 1 mi downgradient
3	>10 wells within 1 mi downgradient
1	Perennial surface-water body >2,000 ft downgradient
2	Perennial surface-water body 200-2,000 ft downgradient
3	Perennial surface-water body <200 ft downgradient

Assigned sub-ranks for each site and beneficial use consideration are presented in Table 2. Totals range from 4 to 7 and define the overall rank for beneficial use.

Rank	*Total Beneficial Use Sub-rank*
1	<5
2	5 to 6
3	>6

Due to its importance, beneficial use was assigned a multiplier of 5.

Pollution Potential Summary

Each pollution potential factor was evaluated at every site. Table 3 presents the rating for all eight sites under study. The higher the rating, the greater the pollution potential of a given site. Table 4 lists the sites in order of priority for monitoring.

MONITORING STRATEGY

The monitoring strategy we developed is an approach for implementing monitoring program(s) at each of the eight landfills under study in a cost-effective manner. The major factors to be weighed in the development of a monitoring strategy include:

(1) Pollution potential

(2) Basic data requirements

(3) Cost

Pollution potential was discussed in detail in the preceding section. Any monitoring strategy for Island County must first address pollution potential as the basis for determining priority of action.

Basic-data requirements refer not only to the data obtained from a monitoring program, but also to the data or information necessary to properly interpret the monitoring data. Therefore, at some of the landfill sites where hydrogeologic data are lacking, the monitoring program is, in part, a data-collection program.

The direct cost for implementing a monitoring program is heavily influenced by the hydrogeology of a specific site. The depth of monitoring wells, the number of aquifers, and ground-water flow characteristics all influence the cost of a monitoring program. For example, a landfill with a single shallow aquifer and well defined unidirectional ground-water flow is relatively inexpensive to monitor, whereas a site with radial flow and multiple aquifers at great depth could be orders of magnitude more costly to monitor.

Cost was not considered in establishing whether or not a site should be monitored. However, cost in conjunction with the available Health Department resources has been considered with respect to the recommendations for implementing each site monitoring program.

Washington Department of Ecology Minimum Functional Standards (MFS) for monitoring detailed in WAC 173-304-490 are applicable to owners and operators of landfills. The monitoring strategy presented here has been structured for the Health Department for the purpose of most efficiently identifying hazards to public health. The strategy presented does not include all the elements included in the MFS; however, the strategy is structured to allow incorporation of individual site monitoring programs into MFS-mandated programs with little or no duplication of effort.

Monitoring Program Development

The objectives of this monitoring program are to:

- Obtain samples representative of *in-situ* ground-water quality.
- Use monitoring and analysis methods that provide reproducible results through quality assurance and training of personnel.
- Develop a monitoring program consistent with MFS.

Table 2. Beneficial use considerations

	WELLS DOWNGRADIENT						Surface-water downgradient					
	Distance from site			Number of wells								
	<100 ft*	100-1000 ft*	>1000 ft*	<5 within mile	5-10 within mile	10> within mile	Perennial body <200 ft	Perennial body 200-2000 ft	Perennial body >2000 ft	Total sub-rank	Rank	Factor rating (5xmultiplier)
Naval Air Station		2				3			1	6	2	10
Oak Harbor		2				3		2		7	3	15
Hastie Lake		2			2				1	5	2	10
Coupeville	3				2				1	6	2	10
Camano Island		2		1					1	4	1	5
Freeland			1			3	3			7	3	15
Langley	3			1					1	5	2	10
Cultus Bay		2			2			2		6	2	10

* Based on interpretation of existing ground-water flow data

Table 3. Landfill pollution potential rating

Pollution potential factor	Naval Air Station	Oak Harbor	Hastie Lake	Coupeville	Camano Island	Freeland	Langley	Cultus Bay
Leachate discharge	15	10	5	10	5	5	10	10
Age and type of facility	3	3	1	3	1	2	1	1
Type of waste	12	9	6	9	3	3	3	3
Pollutant mobility to saturated zone	3	3	3	3	3	9	9	3
Beneficial use	10	15	10	10	5	15	10	10
Total rating	43	40	25	35	17	34	33	27
Monitoring priority	1	2	7	3	8	4	5	6

Table 4. Pollution potential summary

Monitoring Priority	Site	Pollution Potential Rating	Monitoring Priority	Site	Pollution Potential Rating
1	Naval Air Station	43	5	Langley	33
2	Oak Harbor	40	6	Cultus Bay	27
3	Coupeville	35	7	Hastie Lake	25
4	Freeland	34	8	Camano Island	17

Monitoring programs were developed for each site incorporating alternatives for well placement. Each program addresses:

- Where to monitor
- What to monitor
- When to monitor
- How to monitor
- Cost of monitoring

Where to Monitor

Ground-water and contaminant flow occur within a three-dimensional system and, therefore, monitoring locations must be defined both areally (site location) and with depth (aquifer locations).

Site Locations

Proper monitoring site locations are critical to achieving the goals of the monitoring programs. However, when sufficient data are lacking, a substantial amount of time and money are at risk regardless of the approach used in selecting site locations.

The principal factor in locating monitor wells is the direction of ground-water flow. Monitor wells should be located downgradient and as near as practical to the contaminant source. Where possible, drilling through garbage to install a monitoring well should be avoided. Federal Solid Waste Standards (40 CFR 247) dictate a minimum of three monitoring wells downgradient from the waste. The newly promulgated state MFS (WAC 173-304) also require at least three downgradient wells.

In order to adequately determine whether or not ground-water quality changes over distance and has been impacted by landfill operations, it is necessary to establish a background monitoring well upgradient of the contaminant source.

Aquifer Locations

To achieve early contaminant detection and minimize pollution impact, it is preferable to monitor the shallow or uppermost aquifer beneath the contaminant source. At some sites the shallow aquifer is perched and has little or no beneficial use; it may be in hydrologic connection with deeper, more developed aquifers. In these situations, it may be necessary to monitor two or more aquifers.

What and When to Monitor

The MFS for solid waste facilities (promulgated November 1985) specify minimum requirements for testing of ground-water samples. Site monitoring wells must be sampled quarterly for the life of the facility, including the closure and post-closure periods. The constituents to be tested quarterly in ground water are specified in WAC 173-304-490 and listed in Table 5. In addition to the parameters specified in the regulations, monitoring of total halogenated organics (TOX) is also recommended. TOX is a good indicator of volatile organics.

Specific procedures for evaluation of water-quality data are also included in WAC 173-304-490. The site owner/operator must maintain a water-quality database for each site. The water-quality data from each quarterly sampling run must be statistically evaluated (Student's T test) to see if there is a significant increase in constituent concentration (or decrease in pH) in any downgradient well(s) as compared to the site background well(s). If there is a significant increase in water-quality constituent parameters, all monitoring wells must be re-sampled.

Under the MFS, the County Health Department must decide what further investigation will be needed to resolve instances of apparent ground-water contamination, including monitoring for organic contaminants. In most cases, a specific sampling program will be needed to determine if corrective action is needed to protect public health. The frequency of sampling, constituents to be tested, and other technical issues are usually best decided with input from ground-water quality experts from regulatory agencies or private consultants. Figure 4 shows monitoring steps required under the MFS.

Table 5. Water-quality parameters required by the Washington Department of Ecology to be tested quarterly

Parameter	Testing Location
Temperature	Field
Conductivity	Field and laboratory
pH	Field and laboratory
Chloride	Laboratory
Nitrate as nitrogen	Laboratory
Nitrite as nitrogen	Laboratory
Ammonia as nitrogen	Laboratory
Sulfate	Laboratory
Chemical oxygen demand	Laboratory
Total organic carbon	Laboratory
Dissolved iron	Laboratory
Dissolved manganese	Laboratory
Dissolved zinc	Laboratory
Total coliform	Laboratory

How to Monitor

Monitoring ground water requires specialized facilities, equipment, and procedures.

Facilities

Access to the ground-water system is via wells. Establishing whether or not waste disposal operations are impacting ground-water quality generally requires the use of specially constructed monitoring wells. Monitoring wells provide for accurate water-level measurements, collection of representative water samples, and quality assurance/quality control. Only through the use of properly installed monitoring wells can it be determined which aquifer is being monitored and if that aquifer is effectively isolated. A disadvantage with monitoring wells is the high cost of installation.

The use of existing wells (typically, domestic wells) in the site vicinity has two advantages: they have no related installation costs, and they allow assessment of the quality of water the public is actually consuming. However, existing wells have several major disadvantages, including:

- It may not be possible to relate ground-water contamination at a private well to landfill operations.
- If public supplies are contaminated, it is generally too late to take remedial action.

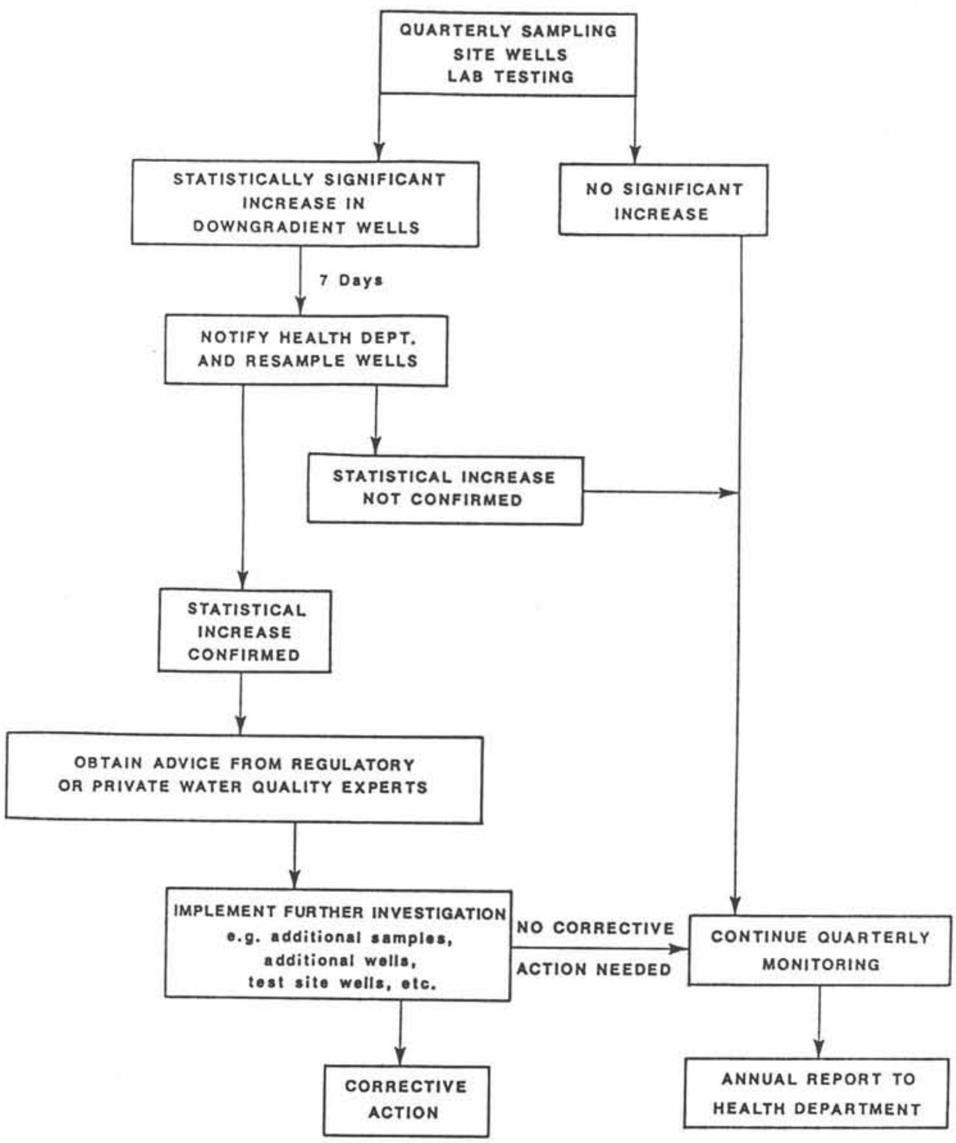

Figure 4. Solid waste site ground-water monitoring flow chart.

- Water levels and water quality results are rarely representative of ground-water conditions. Uncertainty exists as to what aquifer is being monitored because of lack of well construction data.
- Existing domestic wells will not meet MFS for monitoring of landfills.
- Existing domestic wells are usually not ideally situated with relation to the site; therefore, more wells are required to detect a contaminant plume.

In addition, the use of existing supply wells for monitoring requires:

(1) A means of access and permission of owner to access.

(2) Drawing sample from tap, before the water has passed through a pressure tank and/or water conditioner. (Samples taken after pressurization or treatment do not represent true aquifer conditions.)

(3) Drawing samples after a long non-pumping period, which must be determined specifically for each well.

Proper installation of specially constructed onsite monitoring wells requires the use of experienced personnel, proper equipment, materials, and procedures. The depths of the borings will range from 50 to 200 ft, or a minimum of 15 ft into a saturated zone. Access sufficient to accommodate two 40-ft-long rigs (drill rig and pipe truck) is necessary. At some locations, road construction will be required. Access to water for drilling will be necessary.

Monitoring Equipment and Procedures

Proper sampling and field testing equipment are critical for effective monitoring. Likewise, adherence to proper monitoring procedures is an absolute necessity to obtain consistent and reliable results. County Health Department staff were supplied and trained in the use of appropriate monitoring equipment.

Monitoring Costs

In order to efficiently allocate the Health Department's available resources, a cost estimate for implementing a monitoring program was developed for each site (Table 6). Program A assumes installation of monitoring wells, and Program B assumes use of existing wells only. Table 6 takes into account existing monitoring wells at the Coupeville and Freeland landfills.

CONCLUSIONS

Clearly, as Table 6 illustrates, ground-water monitoring is a costly enterprise. On the basis of the results of this study, the Island County Health Department has a better perspective of the pollution potential and, consequently, the health hazard posed by each of the islands' landfills. With this information, the Health Department can focus its limited resources in the areas of greatest need.

The Naval Air Station's landfill (Pollution Potential 1) is currently under investigation. Five ground-water monitoring wells were installed under an Initial Assessment Study (Landau Associates, Inc., 1984), and two additional ground-water monitoring wells were installed during 1987. Elevated levels of chromium and iron have been detected beneath the landfill. Results from the initial monitoring indicate that further characterization of the site is necessary.

Table 6. Estimated monitoring costs; costs will decrease somewhat with increasing volume of laboratory analyses. 200 / 100, double completion well

Site	Monitoring priority	Monitoring well depths (ft)	Total program installation costs ($)
Naval Air Station[a]	1	150	0
Oak Harbor	2	100 100 100 100	27,000
Coupeville[b]	3	200/100 200/150 200/150 200/100 200/100	69,000
Freeland[bc]	4	100 100 100	18,600
Langley	5	50 50 50 150	22,000
Cultus Bay	6	150/100 150/100 150/100	34,000
Hastie Lake	7	200 200 200	28,900
Camano Island	8	150 150 150	20,600

[a] Naval Air Station is a federal facility and not subject to Department of Ecology or county Health Department regulations.

[b] Suitable dedicated monitoring wells already exist on site. Well depths in this table are for additional required wells.

[c] Program B not recommended for this site.

In 1987 the Environmental Protection Agency installed six monitoring wells at the Oak Harbor Landfill (Pollution Potential 2). Preliminary inorganic analytical results indicate elevated concentrations of iron and manganese beneath the landfill (Flood, 1987).

In 1986-87, the county installed a ground-water monitoring system at the Coupeville Landfill (Ecology and Environment [E&E], 1987b) (Pollution Potential 3) as part of its efforts to obtain its 1987 operating permit under the new MFS. The monitoring system consists of 12 wells, and monitoring is conducted quarterly by the Health Department. Elevated concentrations of iron, manganese, increased specific conductivity, and elevated coliform levels have been detected in some wells.

The Health Department has been monitoring two wells constructed at the Freeland Landfill (Pollution Potential 4) since the conclusion of this study. Concentrations of both manganese and iron have exceeded maximum contaminant levels. The EPA, through its contractor, E&E, conducted a site investigation of the Freeland Landfill and recommended no further investigation under Superfund (E&E, 1987c).

The only other site to receive attention since the conclusion of this study was the Hastie Lake Landfill (Pollution Potential 7). E&E also conducted a site investigation for EPA at this landfill, including sampling and testing of seven nearby existing domestic wells. E&E concluded that no ground-water contamination was evident (E&E, 1987a).

The increasing awareness of the vulnerability of ground-water supplies to contamination by older landfills has placed a significant burden on local health departments responsible for protecting public drinking water supplies. However, as this study and the subsequent follow-up investigations have shown, the application of a rational approach to assessing pollution potential can go a long way in alleviating a major part of the burden.

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Hydrogeologic Investigation of the Reichhold Chemicals, Inc., Facility in Tacoma, Washington

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CH2M HILL, Inc.

INTRODUCTION

Reichhold Chemicals, Inc. (Reichhold) owns and operates a manufacturing facility on about 52 acres in Tacoma's Commencement Bay industrial area between Hylebos Waterway to the northeast and Blair Waterway to the southwest (Figure 1). The plant site is on relatively flat terrain that generally has less than 5 ft of topographic relief. This area was constructed in the early 1950s by hydraulically filling the tideflats with dredge spoils from adjacent waterways. Shallow subsurface sediments below the site represent a complex deltaic depositional environment.

Since the Tacoma plant began operations in 1956, a variety of chemical and chemical-related products have been manufactured there. Reichhold has submitted a revised Resource Recovery and Conservation Act (RCRA) Part A and B Application to the U.S. Environmental Protection Agency (EPA) and the Washington Department of Ecology (DOE) in October 1986. This application included descriptions of plant management units known at the time of the submittal and pertinent analytical information available at that time about the site and closure issues.

As part of an agreement among Reichhold, EPA, and DOE, Reichhold submitted a plan and schedule for completing an assessment of the site hydrogeology and upgrading the existing ground-water monitoring program. Upon completion of this assessment, a plan for ground water and soil remediation will be developed.

This paper is a case study of an ongoing site hydrogeologic characterization investigation. The focus is on the results as well as the methods used to characterize the hydrogeology of a complex tideflat/deltaic environment. The approach must satisfy RCRA ground-water requirements in federal regulations 40 CFR Parts 264, 265, and 270. The ultimate goal of the site hydrogeologic characterization is to provide sufficient hydrogeologic data to design an effective ground-water monitoring system(s), evaluate corrective action alternatives, and obtain closure status for the facility.

SITE INVESTIGATION ACTIVITIES

The site investigation included the review of state well records, existing regional geologic literature, and local hydrogeologic investigation reports. Onsite activities consisted of soil sampling, installation and sampling of monitoring wells, ground-water elevation measurements, slug tests, pump tests, monitoring of tidal influences, and surface-water and sediment sampling.

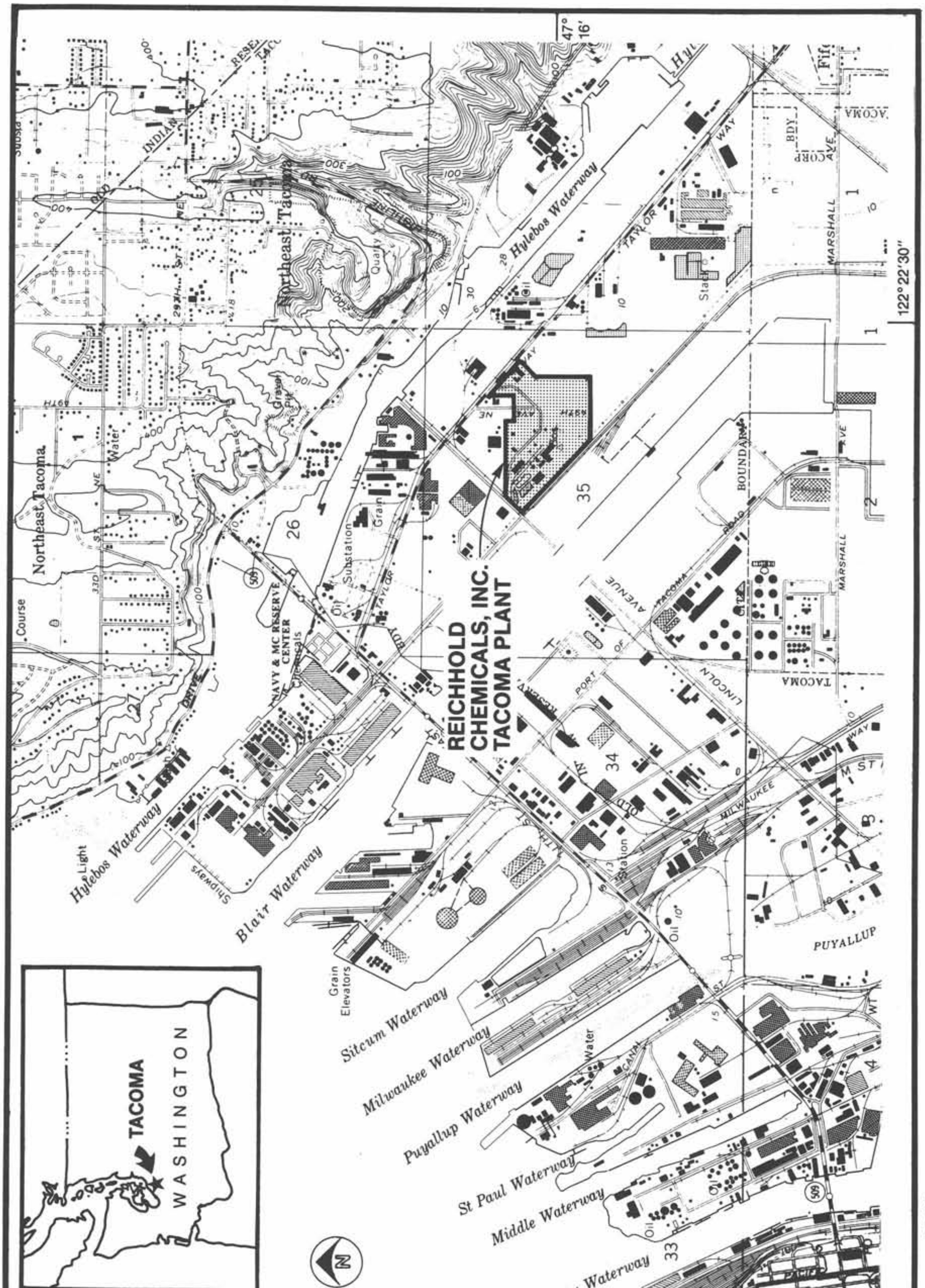
Literature Review

The review of existing geologic and hydrogeologic literature, both local and regional, provided inexpensive subsurface information. In reviewing DOE well records, it was determined that ground-water production wells nearest the Reichhold facility are approximately 1 mi away; they tap the regional aquifer at 800 to 900 ft below ground surface and would not likely be affected by past waste management and industrial activities.

Soil Sampling

The purpose of soil sampling was to characterize the stratigraphy of the upper surficial soil and evaluate the nature and extent of soil contamination. The areas sampled were those where waste disposal activities had been known to exist during the operational period of the Reichhold facility (1955 to 1988). In general, continuous soil samples were obtained from the ground surface through the upper surficial soils to the top of the uppermost confining layer. Waste was not expected to be found below this layer. The upper aquitard, which is characteristically a plastic silt deposit with organic matter, differs dramatically from the overlying granular dredge-fill material. The aquitard was therefore easily identifiable during drilling and sampling. The depth to the aquitard varied from 4.5 to 13.5 ft below ground surface.

The chemical analysis of soil samples consisted of field screening for site-specific indicator parameters, pollutant analyses (modified RCRA Appendix VIII constituents), and DOE waste designation tests. Bulk soil



Source: USGS
Tacoma North 7.5 Minute Quadrangle
Washington
1:24,000-Scale Series
(Topographic) 1981

Figure 1. Location of study area, Tacoma.

samples were collected for batch and column leach testing.

The field screening analyses were conducted onsite by CH2M Hill's close support laboratory (CSL). The CSL used a gas chromatograph temporarily installed at Reichhold's laboratory to obtain fast turn-around analysis (1-2 days).

The purpose of the CSL analyses was to gather chemical data at the time of drilling and sampling in order to assess the need for additional sampling. Every other sample interval (1.5 ft) in each boring underwent CSL analysis. The short turn-around time of the CSL analyses significantly reduced the time needed to complete the site investigation by avoiding the delays associated with sending samples to outside laboratories (turn-arounds of 30 days or longer). In addition to guiding the field drilling program, the CSL provided analyses of discrete sample intervals for the indicator parameters in contrast with the composite samples tested in the pollutant analyses.

Monitoring Well Sampling

The hydrogeologic investigation involved the installation of 51 monitoring wells in phases from September 1986 to June 1987 to upgrade a network of 20 wells installed in November 1985 by Shannon & Wilson, Inc. During drilling, soil samples were collected for physical property testing (grain size analyses, Atterberg limits, moisture content, organic carbon, and falling-head permeability testing). Figure 2 shows locations of the monitoring wells and well clusters; only the wells discussed here are labeled in the figure.

Monitoring wells were installed in a first phase for purposes of upgrading the existing monitoring well network and further characterizing the hydrogeologic system. After review of data from the original and first-phase monitoring wells, wells were installed during a second phase starting in May 1987 for offsite plume delineation and additional characterization. Ground-water sampling was conducted on a quarterly schedule in all wells following completion of the first phase of well installations. The parameters analyzed were those selected for soil chemical analyses and those required in 40 CFR Part 265, Subpart F.

Ground-water elevation data were collected monthly from the monitoring well network consisting of both onsite and offsite wells (Figure 2). Measurements were made on a monthly basis for the first year in all three aquifers, starting in January 1987. In addition, surface-water elevations across the site and Blair Waterway were measured at 8 to 10 staff gages. Continuous ground-water elevation monitoring was performed in January 1987 with electronic recording equipment to evaluate tidal influences on vertical and horizontal hydraulic gradients. Generally, coastal aquifers hydraulically connected to the ocean will exhibit sinusoidal ground-water level fluctuation in response to ocean tides. As the tidal influences move inland, the magnitude of the fluctuations decreases or is attenuated, and the lag time increases. Lag time is defined as the time between the occurrence of a high or low tide in the ocean and the time the maximum or minimum effect reaches an observation point.

The hydrogeologic characterization work provided the necessary information for the selection of two aquifer test sites.

The MW-14 well cluster location was selected for aquifer testing because of the greater-than-average thickness of both the upper and lower aquitards. The MW-10 well cluster was selected for testing because the upper and lower aquitards were thinner than average in this area. Two extraction wells were installed in the intermediate aquifer in June 1987 for the aquifer tests. The wells are designated as EW-1 and EW-2 and are located approximately 10 ft from well clusters MW-10 and MW-14, respectively (Figure 2).

The objectives of the tests were to evaluate the degree of hydraulic interconnection between the shallow, intermediate, and deep aquifers and to obtain aquifer hydraulic coefficients (apparent transmissivity and apparent storativity) in the vicinity of the MW-10 and MW-14 well clusters (Figure 2). The results are important for further ground-water assessment evaluation and corrective measure design.

To perform the aquifer tests, each extraction well was pumped for about 48 hr. At the conclusion of the 48-hr period the pump was turned off, and water-level recovery data were collected for about 24 hr.

In addition, chemical analysis of the surface water and sediments from the onsite drainage ditches was performed to assess surface contaminant migration pathways. This investigation is not complete and will not be discussed further.

INVESTIGATION RESULTS

Regional Geology

Briefly, consolidated rocks in the Puget Sound area form the bedrock or basement complex upon which thick sequences of sediments have been deposited. The bedrock is the Eocene Puget Group, a series of consolidated sandstones, shales, coals, and conglomerates. The depth to bedrock at the Reichhold site is unknown (Walters and Kimmel, 1968). Glacial and nonglacial sediments consisting of sand, gravel, silt, and clay were deposited on the basement complex and extend to depths greater than 1,200 ft (Walters and Kimmel, 1968).

Marine deposits consisting of primarily very fine sediment overlie the glacial material. The lower reaches of the Puyallup River were inundated by seawater until the recession of the last glacier. During this period, the

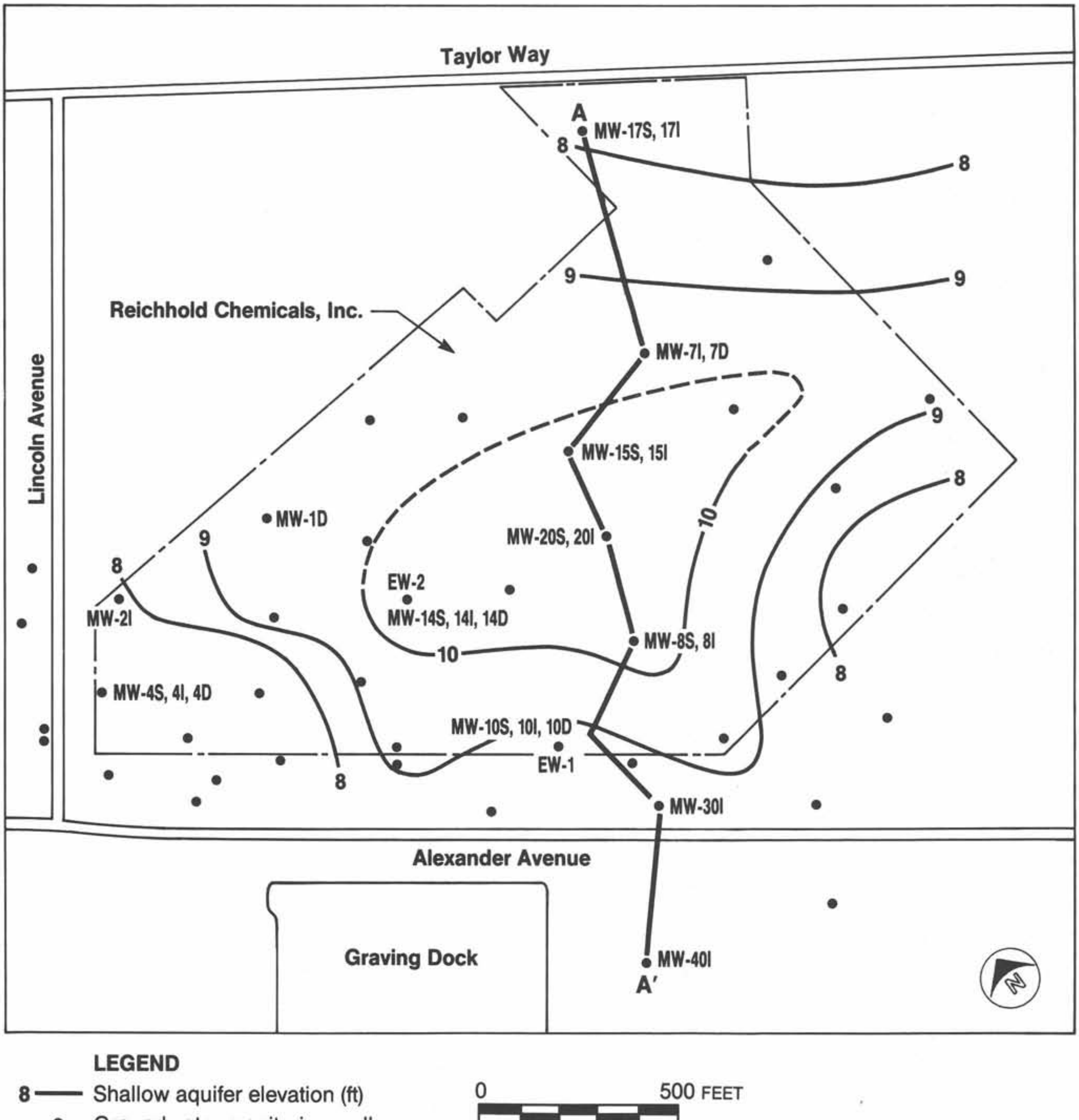


Figure 2. Location of monitoring wells and water-table contours (January 1986).

fine river sediments were deposited under marine conditions (Walters and Kimmel, 1968). These deposits are as much as 400 ft thick and grade upward into deltaic deposits.

The uppermost sedimentary unit is that formed by the buildup of the Puyallup River delta. This unit of sand, silt, and silty sand dominates the geology of the site area and ranges up to 100 ft thick.

The youngest sediments in the site area are the result of dredging the Blair and Hylebos industrial waterways. Dredged material placed on shore in 1951 and 1952 and again in 1956 covers the site to an average depth of 10 ft. The fill forms a continuous layer of material that ranges from silty fine sand to gravelly sand.

Regional Ground-water Flow System

Regional ground-water flow patterns are controlled by geology, topography, and the location of regional ground-water recharge and discharge areas. The principal regional aquifer system consists primarily of the extensive glacial material underlying the marine deposits. The marine deposits hydraulically separate the regional system from the surface alluvial deposits, that is, the shallow or local ground-water flow system. The western slope of the Cascade Range and foothills is the primary regional ground-water recharge zone; Puget Sound is the probable discharge zone.

Wells constructed in the regional aquifer are 800 to 950 ft in depth (Walters and Kimmel, 1968). The presence of flowing wells completed 800 ft below ground surface 1.0 mi southeast (Kaiser Aluminum and Chemical) and approximately 1 mi northwest (City of Tacoma) of the Reichhold site indicate the upward movement of ground water near the regional discharge area. Deep downward migration of ground water is not expected to occur in the regional discharge area because of the upward vertical gradient. Therefore, deep aquifer ground-water supplies such as those being used by the previously mentioned Kaiser and City of Tacoma wells have not been, and probably will not be affected by past disposal practices at Reichhold.

Site Hydrogeology

Site Geology

Subsurface soils data collected from borings completed to a depth of about 50 ft indicate the presence of shallow, apparently continuous, alternating layers of sand and silt under the site. Similar sequences have been identified beneath other industrial sites located along the peninsula occupied by Reichhold (Dames & Moore, 1981, 1985; Hart, Crowser and Associates, Inc., 1980, 1975). These sequences of sand and silt are interpreted as three near-surface aquifers and two near-surface aquitards. (An aquitard is generally defined as a stratigraphic unit of low permeability.) The three aquifers are referred to as the shallow, intermediate, and deep aquifers, and the two aquitards are referred to as the upper and lower aquitards.

Hydrogeologic cross-sections were constructed from subsurface data. The location of a representative cross-section is shown in Figure 2, and the cross-section is presented in Figure 3. The varied thickness of the hydrogeologic units, as depicted in the cross-section, is consistent with the deltaic depositional environment of the area.

The shallow aquifer is unconfined and is defined as the saturated material below the water table and above the upper aquitard. The thickness of the shallow aquifer and the overlying unsaturated zone varies seasonally, from a maximum thickness in the winter and spring, when the water table is at or above ground surface in some areas, to a minimum during dry conditions in the late summer and fall.

The shallow aquifer is the uppermost geologic unit encountered at the site. Grain-size analyses performed on material from the shallow aquifer confirmed the field classifications as fine to medium sand and silty sand. The material is primarily dredge spoils from the Hylebos and Blair waterways deposited during 1951, 1952, and 1956. In some areas, mechanical fill consisting of sandy gravel and cobble mixtures occurs at the surface in the upper 2 to 3 ft. The average thickness of the shallow aquifer across the site is less than 10 ft.

The upper aquitard is the uppermost natural depositional unit at the site. Grain size and Atterberg limit analyses performed on samples of aquitard material confirmed the field classification as silt or organic silt. Zones of organic material and peat, as well as interbedded silt, silty sand, and sandy silt are common.

Borehole data indicate that the upper aquitard separates the shallow and intermediate aquifers at each well location. At one location the aquitard is only inches thick, and it ranges to a maximum thickness of 15 ft; typically, the aquitard is 5 ft thick across the site.

The intermediate aquifer consists primarily of fine to medium sand and silty sand. Zones of interbedded sand, silty sand, and silt within the aquifer are common. Shell and wood fragments are also present. Gradational changes in silt content occur in the lower portion of the aquifer. Thus, in some locations there is a transitional zone rather than a sharp boundary between the intermediate aquifer and lower aquitard. The aquifer ranges in thickness from 4 ft to approximately 25 ft.

A lower aquitard appears to separate the intermediate and deep aquifers at the site. The aquitard generally consists of silt, organic silt, and clayey silt, with scattered very fine sandy silt and peat interbeds. Zones of organic material and shell fragments are common. The data indicate that the lower aquitard ranges in thickness from 4 to 15 ft.

The deep aquifer was encountered below the lower aquitard and consists primarily of alternating fine to medium sand and silty sand, with some silt interbeds. Scattered zones of shell fragments and wood are present. The total thickness is not known; however, regional studies indicate that sand might reach a thickness of 80 ft or more in the Reichhold vicinity (Hart, Crowser and Associates, Inc., 1980; Shannon & Wilson, Inc., 1985). The deep wells installed at the Reichhold site penetrate about 10 to 15 ft into the deep aquifer (approximately 50 ft below grade). Similar sequences of sand and silt at approximately the same elevations have been encountered in other nearby site investigations (Dames & Moore, 1981, 1985; Hart, Crowser and Associates, Inc., 1980, 1975). Therefore, it is possible that the lateral extent of the three near-surface aquifers is continuous to the Blair and Hylebos waterways. The waterways were excavated to a depth such that hydraulic communication exists between the three near-surface aquifers, as shown in Figure 3 and as demonstrated by the presence of tidal influences in the intermediate and deep aquifers.

Ground-Water Flow System

Horizontal flow direction:

The interpretation of local shallow ground-water elevation data shows a ground-water divide between the Hylebos and Blair waterways (Figure 2). The inferred flow lines generally show ground water moving from the center of the peninsula to the waterways located on either side. The ground-water flow direction in the intermediate aquifer is generally to the west across the site, with a component of flow directly to the graving

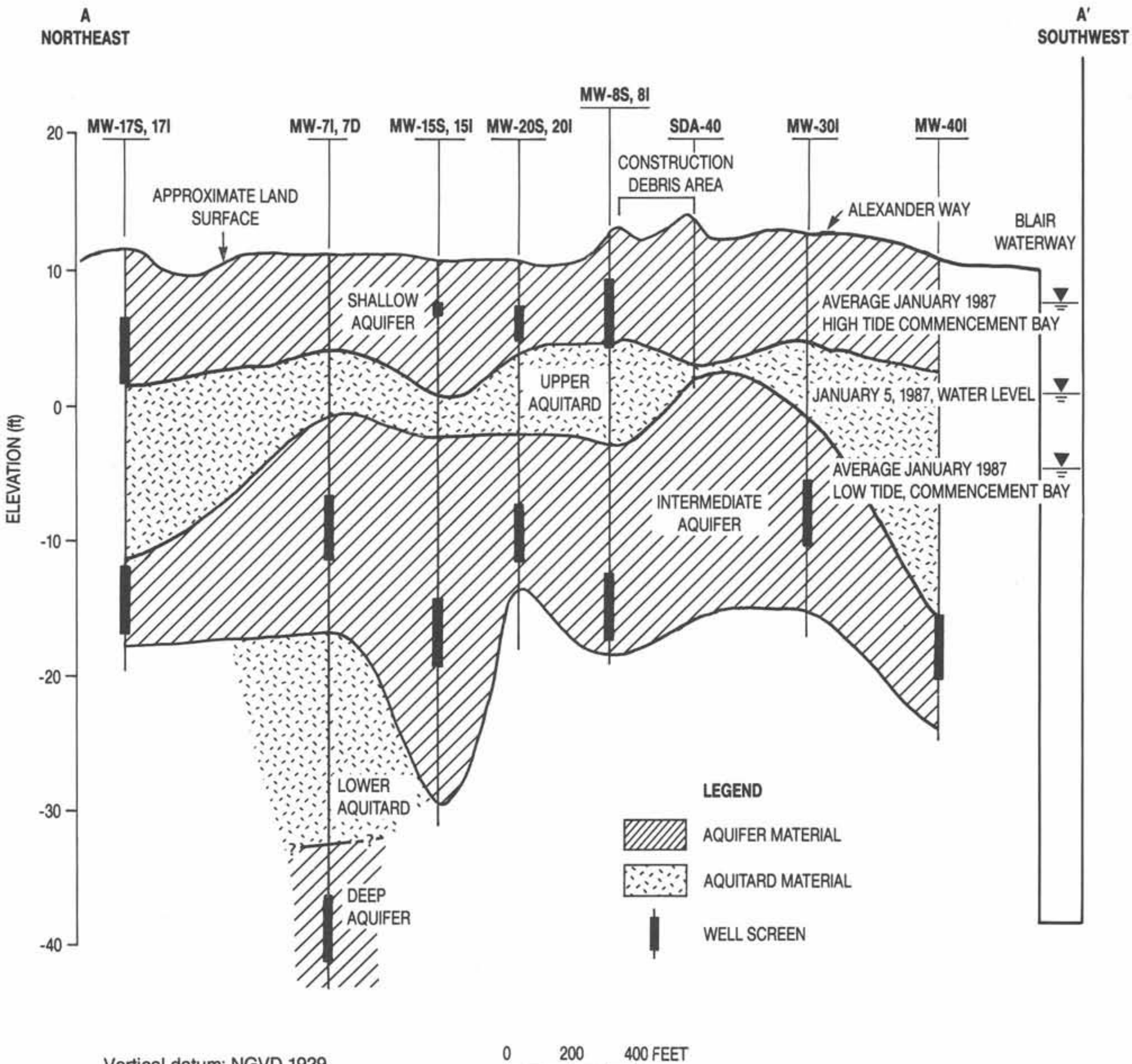


Figure 3. Hydrologic cross-section A-A′; see Figure 2 for location of cross-section.

dock (Figure 2) and the Blair Waterway along the southern boundary of the site. The deep aquifer flow direction is northerly across the site.

Horizontal hydraulic gradients in the shallow aquifer range from approximately 17 to 70 ft/mi (0.0032 to 0.0133 ft/ft) across the site. Gradients vary seasonally, and, because the water table is close to the surface, they are highly influenced by precipitation, land surface conditions, and topography. Features such as obstructed drainage ditches and surface depressions acting as precipitation catch basins, paved areas and buildings that prohibit or inhibit infiltration, and leaking storm sewer pipes will affect shallow aquifer gradients. Current data from shallow aquifer wells indicate that the range of seasonal water-level fluctuations is approximately 3 ft.

Ground water in the intermediate aquifer generally occurs in a confined condition. This is evidenced by the potentiometric surface (water-level elevations) extending above the top of the intermediate aquifer and into the upper aquitard and shallow aquifer. During seasonal low ground-water levels, unconfined conditions can occur at some locations.

Ground-water levels in the intermediate aquifer, as measured on January 5, 1987, were approximately 4 to 9 ft below the ground surface. The horizontal hydraulic gradient of the ground water ranges from approximately 4 to 18 ft/mi (0.0008 to 0.0034 ft/ft). The ground-water flow in the intermediate aquifer appears to be in a northwesterly direction across the site toward Commencement Bay, on the basis of the January 5, 1987, water levels. Current data indicate an average seasonal fluctuation range of approximately 2 ft in the intermediate aquifer.

The ground water in the deep aquifer also appears to be under confined conditions as indicated by the potentiometric surface 20 to 30 ft above the top of the deep aquifer. The ground-water levels in the deep aquifer were approximately 7 to 9 ft below the ground surface. The horizontal hydraulic gradient ranges from approximately 3 to 21 ft/mi (0.0006 to 0.004 ft/ft). The ground-water flow in the deep aquifer appears to be in a northerly direction across the site toward Commencement Bay. The current deep aquifer water-level data indicate an average seasonal fluctuation range of approximately 3 ft.

Vertical flow direction:

Ground-water level elevation data indicate that the vertical flow directions are consistently downward between the shallow and intermediate aquifers. Between the intermediate and deep aquifers, the vertical flow direction is generally downward, although all well clusters with both intermediate and deep wells have at least one record indicating upward flow across the lower aquitard. These vertical flow reversals between the intermediate and deep aquifers are still being evaluated.

Downward vertical gradients on January 5, 1987, range from 0.14 to 0.40 ft/ft across the upper aquitard and from 0.01 to 0.05 ft/ft across the lower aquitard. An upward gradient was recorded between MW-4I and MW-4D at 0.025 ft/ft during this period.

Tidal Influences:

The shallow aquifer water level in well MW-14S showed no tidal response over a 2-day continuous monitoring period.

Tidal effects, monitored over a 3-day period, in the intermediate aquifer at well MW-21 displayed a maximum water level fluctuation of approximately 0.2 ft. The fluctuations were not sufficient to cause either horizontal or vertical gradient reversals between the shallow and intermediate aquifers.

The greatest tidal influence was recorded in monitoring wells completed in the deep aquifer. Ground-water fluctuations of more than 1 ft in monitoring wells MW-1D and MW-14D were recorded. These wells were chosen because they are located approximately along the same flow line in the deep aquifer, an arrangement that simplified evaluation of tidal influences on horizontal flow gradients.

The January 1987 data indicate that a reversal in horizontal hydraulic gradient occurs daily in the deep aquifer between wells MW-20 and MW-14D. The reversal occurs when the water-level elevation in well MW-14D drops below the water-level elevation in MW-1D. Because MW-14D is closer to Blair Waterway than is MW-1D, the magnitude of the tidal influence is slightly greater and the lag time is slightly less. Thus, after a high tide, water levels in MW-14D start declining slightly sooner and at a slightly faster rate than water levels in MW-1D. At some point in the tidally induced decline, the water level in MW-14D drops below that in MW-1D, thus creating a reversal in the horizontal hydraulic gradient.

The Blair Waterway tidal fluctuations were greater than 10 ft during the monitoring period. A tidally induced vertical hydraulic gradient reversal was not observed among the three near-surface aquifers during continuous monitoring of MW-14S, 14I, and 14D. However, because of the similar heads in the intermediate and deep aquifers and the magnitude of the tidal fluctuations in the deep aquifer, vertical gradient reversals (from downward to upward) are likely, particularly along the southwest property line closest to the Blair Waterway.

A graving dock (Figure 2) is located on Blair Waterway across Alexandria Avenue southwest of the site. A dewatering system encompasses the graving dock, which, when in operation, dewaters the dock and the surrounding area. When in use, the graving dock dewatering system will likely influence ground-water levels on the Reichhold property.

Aquifer Testing

Slug tests:

Slug test values of hydraulic conductivity were calculated, as follows:

	Hydraulic Conductivity Ranges (cm/sec)	
	Hvorslev's Method (1951)	Hazen's Equation
Shallow aquifer	$6x10^{-5}$ - $6x10^{-3}$	
Interme-diate aquifer	$1x10^{-4}$ - $9x10^{-3}$	
Deep aquifer	$6x10^{-4}$ - $4x10^{-3}$	
Avg. all aquifer material		$3x10^{-4}$ - $3x10^{-2}$

For an assumed effective porosity of 0.20 (unitless) for the site aquifer material, horizontal ground-water flow velocities at the site range from 0.001 to 1.1 ft/day across the site. The large range in flow velocities is caused by the variation in hydraulic conductivity and flow gradients across the site.

Pump Testing:

Extraction well EW-1 is completed in the intermediate aquifer approximately 15 ft from well cluster MW-10. Well EW-1 was pumped at 0.5 gpm for 48 hr in September 1987. During the pump test the water level in MW-10S (shallow aquifer) gradually declined throughout the test period (Figure 4). This decline in water level likely reflects the seasonal drop expected during that time of the year. The shallow aquifer water level in well MW-10S showed a maximum tidally induced fluctuation of approximately 0.07 ft during the aquifer tests. This tidal influence was not detected in the shallow aquifer during the January 1986 monitoring. These results show that the shallow aquifer was not significantly affected during the test at this location and that there is no direct evidence of interconnection between the shallow and intermediate aquifers.

The water level in MW-10D decreased about 0.5 ft during the pumping phase of the test and increased by about 0.5 ft during the recovery phase. This change in water level during the pumping and recovery phases of the test indicates that there is direct hydraulic connection between the intermediate aquifer and the zone in which MW-10D is completed.

Tidally induced water-level fluctuations in wells MW-10I and MW-10D are also shown in Figure 4. The tidal responses in both wells are almost identical in magnitude (about 0.2 ft) and in the time of occurrence. Thus, the sand unit in which well MW-10D is completed exhibits hydraulic characteristics that resemble those of the intermediate aquifer more than those of the deep aquifer.

Extraction well EW-2 is completed in the intermediate aquifer near well cluster MW-14. Well EW-2 was pumped at 1.6 gpm for 48 hr in August 1987. No noticeable change in the shallow or deep aquifer water levels was observed at the MW-14 well cluster during the EW-2 pumping test. This indicates that the degree of hydraulic interconnection between the three aquifers at this location is insignificant under these conditions (Figure 5).

The aquifer hydraulic coefficients of transmissivity (T) and storativity (S) were calculated for the intermediate aquifer by using the data obtained from tests EW-1 and EW-2. A single observation well completed in the intermediate aquifer was used for each aquifer test. The two tests were analyzed with the leaky artesian aquifer analytical solution (Walton, 1970) and an analytical solution to determine the apparent distance to a single recharge boundary, as developed by R. W. Stallman and discussed by Lohman (1979). The solution presented in Walton (1970) estimates apparent transmissivity and storativity. The solution presented in Lohman (1979) also provides an independent estimate of apparent transmissivity and storativity.

The results of the pump test data analysis indicate an average value of 61 gpd/ft for the apparent transmissivity (T) in the vicinity of the MW-10 well cluster. The average apparent storativity (S) at the same location is 5×10^{-4}. These average values represent drawdown and recovery results from using both Walton (1970) and Lohman (1979) methods. The average apparent T and S values in the vicinity of the MW-14 well cluster are 277 gpd/ft and 1×10^{-3}, respectively.

CONCLUSIONS

The results of the hydrogeologic characterization investigation in the Tacoma tideflat area shows a complex ground-water flow system that requires a sophisticated monitoring well network to evaluate the nature and extent of potential contaminants and comply with RCRA guidance and regulations. Alternating layers of sand and silt encountered during drilling are interpreted as three near-surface aquifers separated by two aquitards. Ground-water flow velocities and directions vary considerably across the site. The shallow aquifer data indicate an unconfined ground-water divide in the central area of the site, and very slight tidal influences. The intermediate and deep aquifers are influenced by tides, and the data indicate that the deep aquifer experiences horizontal gradient reversals during at least part of the seasonal cycle.

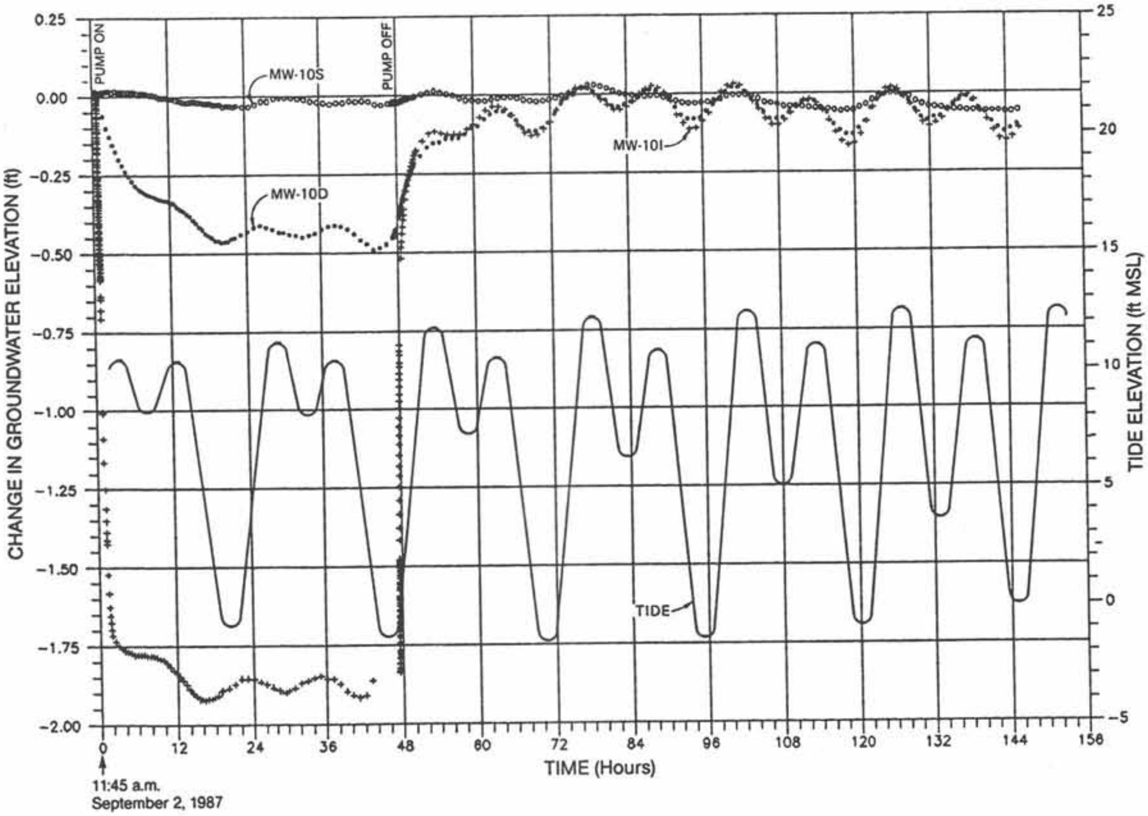


Figure 4. Aquifer test results, well EW-1; see Figure 2 for well location.

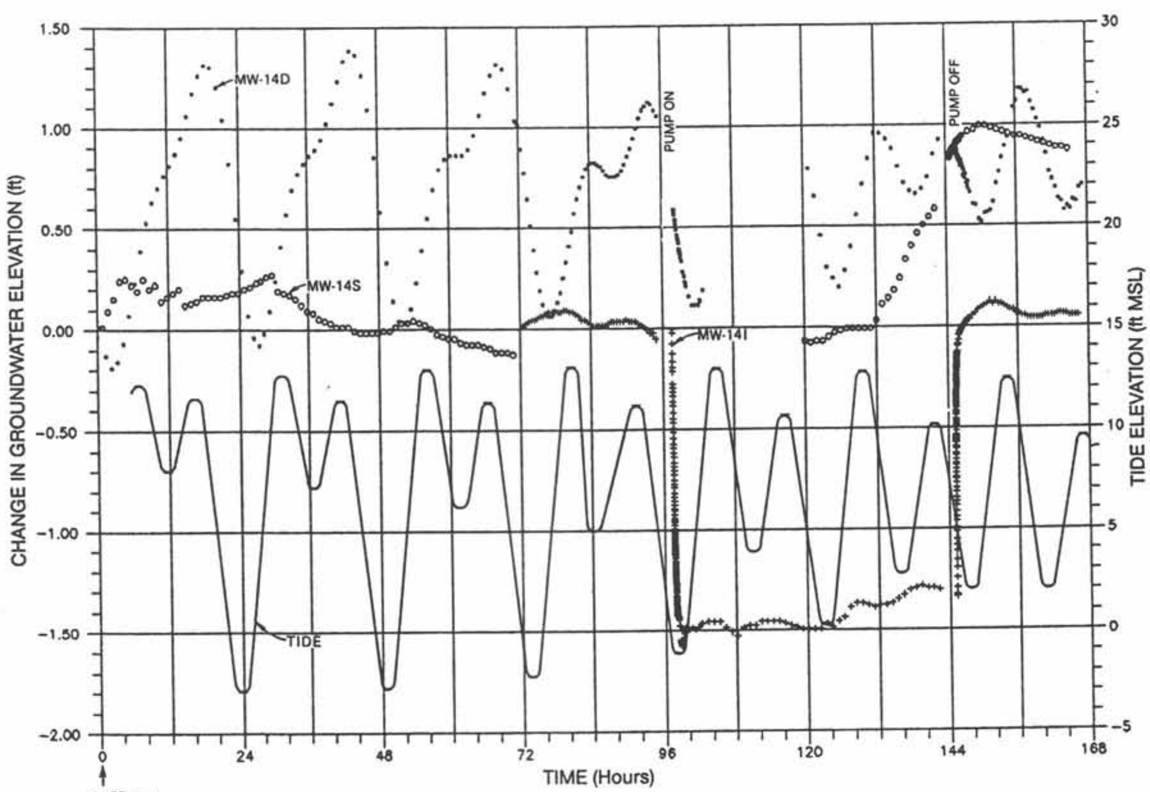


Figure 5. Aquifer test results, well EW-2; see Figure 2 for well location.

The complexity of this tideflat/deltaic depositional environment has required the installation, sampling, and chemical analysis of 71 monitoring wells and numerous soil borings over a 2-yr period. Additional ground-water monitoring wells might be required to fulfill regulatory requirements and provide additional information. The results of this investigation are being used by Reichhold to design ground-water monitoring systems and evaluate corrective action alternatives for the existing contamination. Compliance and corrective action monitoring, as required by RCRA, will be performed at selected existing wells and new wells to monitor the effectiveness of the corrective actions that are eventually implemented.

ACKNOWLEDGMENTS

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Hydrogeologic Evaluation of the Snohomish County Regional Landfill

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INTRODUCTION

In general, plans for landfills built prior to 1985 did not include comprehensive ground water evaluations as part of their preliminary designs. Since enactment of the State of Washington Minimum Functional Standards (WAC 173-304) in 1985, geotechnical assessments, including those for ground-water conditions, are required for the siting of new landfills.

In the early 1980s, the City of Snohomish recognized that a new landfill was needed in Snohomish County to provide a facility to handle solid waste for at least the next 20 yr. Existing landfills were being closed or at near capacity. Therefore, the Snohomish County Solid Waste Division initiated a landfill selection process for a new regional landfill. Thirty sites potentially suitable for a landfill were initially evaluated in Snohomish County. After further evaluation of seven of the most favorable sites, the county determined that the 440-acre site adjacent to the existing Cathcart Landfill was most suitable for development. The site is located approximately 3 mi south of Snohomish and directly west of the existing Cathcart Landfill (Figure 1).

PURPOSE AND OBJECTIVES

The principal objectives of the geotechnical investigation were to determine the location, extent, and thickness of geologic units and to define ground-water conditions as needed for site development.

An overall site investigation was first completed to determine subsurface conditions beneath the proposed 440-acre site. Following the completion of the overall site evaluation, the study then focused on an area selected for initial development and located along the northern margin of the site (Figure 2). Emphasis was given to defining the geotechnical characteristics of the till and outwash deposits in order to develop a conceptual design for the new landfill. Boundaries for the area to be developed were approximated. An investigation of the proposed soil borrow area, to estimate the types and volumes of earth materials for use as daily landfill cover materials completed the site investigation.

The site is on the south flank of the Snohomish River valley. Ground-surface elevations at the site range from 200 to 574 ft. above mean sea level. Site drainage is directed north across the site toward the Snohomish River floodplain, which is approximately 20 ft above mean sea level.

SITE INVESTIGATION

Between November 1985 and February 1986, 38 hollow stem auger borings and 3 air rotary borings were drilled. Nineteen of the 38 hollow stem auger borings were completed as piezometers, 6 as monitoring wells, 7 as permeability testing wells, and 6 were abandoned. All three air rotary borings were completed as monitoring wells. The location of each boring is shown in Figure 2. In addition, 68 test pits were excavated and sampled.

Rising head permeability tests were conducted in each of the seven permeability testing wells to evaluate the *in-situ* permeability of the till at various depths and locations in the initial development area. Each test was initiated by causing an instantaneous change in water level in the 2-in.-diameter well through sudden removal of approximately 3/4 gal of water. Recovery of the water level with time was then measured. Hvorslev's Analytical Method (1951) was used to interpret the data collected from each well.

Geophysical surveys supplemented the exploratory boring program in the investigation and evaluation of the proposed borrow area. Resistivity soundings were completed by Geo Recon International for the purpose of identifying the bottom of the till overlying outwash deposits. At the sites where vertical resistivity soundings were accomplished, relatively good agreement was found between the electrical and boring data.

During exploration of the initial development area and the proposed borrow area, a total of 48 disturbed soil samples were collected for mechanical sieve analysis. Selected bulk samples were also collected from the test pits and tested for compaction characteristics and remodeled permeability. The elevation for each

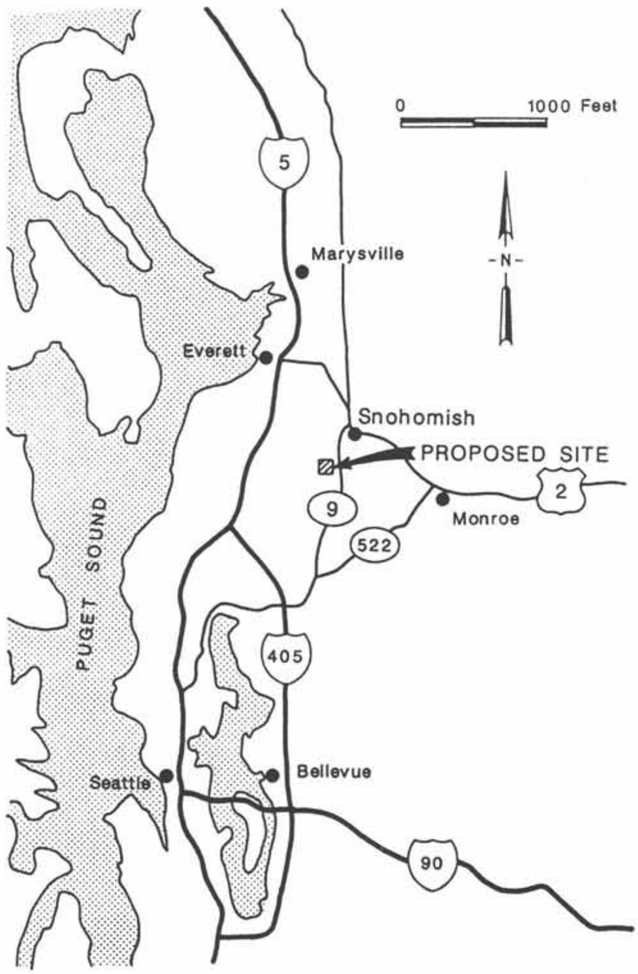

Figure 1. Location of Snohomish County Regional Landfill.

piezometer, monitoring well, and ground surface of each test pit was surveyed to the nearest .01 ft. Measurements of depth to water in each piezometer and monitoring well were converted to ground-water elevation and used to further define the hydrogeology beneath the site.

HYDROGEOLOGY

The information obtained from the 41 borings and 68 test pits was combined with geologic mapping to determine the overall hydrogeologic framework of the site and, in greater detail, the proposed development area.

Geology

Four basic geologic units were identified beneath the proposed landfill site: Tertiary sandstone bedrock, advance outwash sand and gravel, till, and recessional outwash sand.

The lowest stratigraphic unit is sandstone bedrock (of Tertiary age) which underlies the till and was penetrated at depths ranging from 0 to 130 ft. Sandstone was found at or very near the surface on the eastern part of the site; it dips to the west.

Pleistocene pre-Fraser and Fraser glaciation age outwash deposits overlie the sandstone. The relationship between these units is shown on geologic cross-section A-A′ (Figure 3). Interglacial lacustrine silt locally overlies the sandstone throughout much of the study area. The thickness of the silt ranges from 10 to 30 ft where present in the borings. Overlying the silt are poorly sorted advance outwash deposits that vary from moist to saturated and range in thickness from 7 to 55 ft. In eastern parts of the site, the advance outwash deposits are locally absent; otherwise, these deposits thicken to the west. A transitional zone, consisting of oxidized, gravelly sandy silt, commonly separates the advance outwash deposits from the overlying till.

Vashon till mantles the vast majority of the site with thicknesses typically greater than 30 ft on the west and thinning to the east. Typically, the deposit consists of an upper weathered silty sand horizon, 1 to 30 ft in thickness, underlain by very hard, compact, unweathered, fine sandy silt. The till was found to be moist to wet and locally saturated in the southwest corner of the site due to local discharge of ground water from the underlying artesian advance outwash aquifer. Horizontal permeabilities of the till ranged from 5.9×10^{-6} to 8.7×10^{-7} cm/sec.

Recessional outwash deposits overlie the till and occur in isolated pockets and ridges throughout the site. The deposits consist of sand and gravel and rarely exceed 10 ft in thickness. Generally the material is very moist to wet and grades into the underlying weathered till.

GROUND-WATER OCCURRENCE

Ground water is present in all four geologic units beneath the project site. The Tertiary sandstone aquifer and the advance outwash aquifer are part of a regional flow system that is recharged from sources outside the project area. Superimposed upon the regional flow system are perched ground-water systems contained locally in the till and recessional outwash sands. The perched ground water is recharged primarily from local infiltrating precipitation and surface drainage.

Tertiary Sandstone Aquifer

Ground-water elevations showed that the aquifer is unconfined (water-table conditions) and the flow direction is to the north and the Snohomish River valley. The Tertiary sandstone is not an important regional aquifer and is capable of yielding only small amounts of ground water to wells (Robinson & Noble, Inc.,1972).

Advance Outwash Aquifer(s)

Geologic and water-level data indicate that two saturated zones, separated by an unsaturated layer, are present within the advance outwash deposits. A distinct separation of the upper and lower saturated zones exists in the western part of the site where ground water oc-

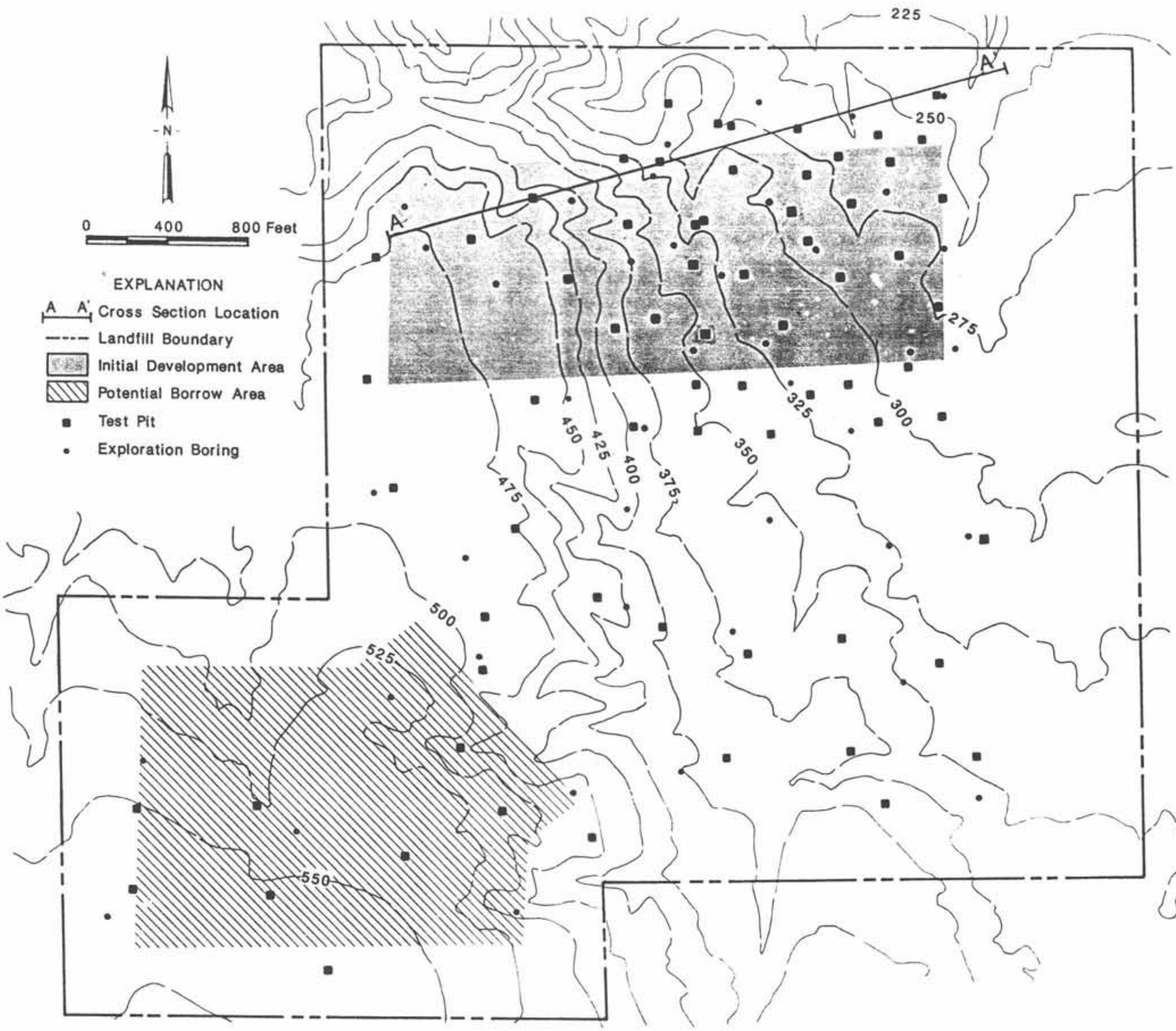

Figure 2. Location of exploratory borings and plan of Snohomish County Regional Landfill site development.

curs in the upper part of the transitional zone. Ground-water elevations in the transitional deposits to the west are more than 80 ft higher than elevations measured in the lower advance deposits. Ground-water elevations in the central and eastern parts of the site are similar. There is no evidence of an unsaturated layer between upper and lower parts of the advance outwash aquifer.

Static water levels in the lower advance outwash aquifer are more than 130 ft below ground surface along the western margin of the site. In this area the aquifer exhibits water-table conditions. To the east, the character of the aquifer changes to a confined aquifer with significant artesian head. In the central and eastern parts of the site, ground-water elevations in the advance outwash aquifer range from 1 to 50 ft above the base of the overlying till. Along the eastern margin of the site, the potentiometric surface of the advance outwash aquifer exceeds ground elevation. This aquifer would yield large quantities of water to wells, although no downgradient beneficial uses of water from this aquifer currently exist.

Perched Ground Water

The till is an important hydrogeologic unit beneath the project site. The low permeability of the till restricts downward flow of infiltrating precipitation, resulting in minor amounts of perched ground water in the weathered till and recessional outwash sands.

Till also acts as a confining unit to the underlying advance outwash aquifer over much of the eastern third of the site. The area near the southeast corner of the site is poorly drained and supports standing bodies of water. It

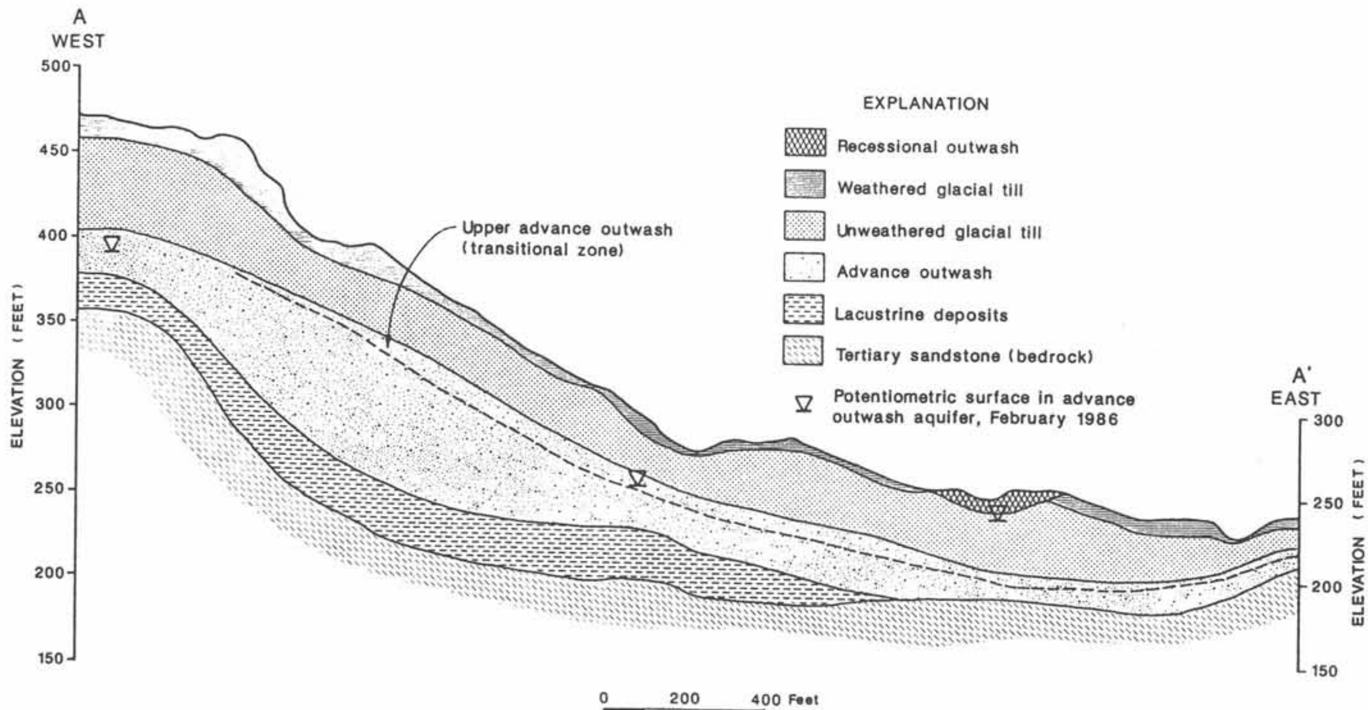

Figure 3. Geologic cross-section A-A′ at the Snohomish landfill; see Figure 2 for location of section.

appears that local discharge of ground water from the underlying artesian advance outwash aquifer has locally saturated the till and possibly breached the surface.

The recessional outwash deposits contain scattered, isolated pockets of ground water. The deposits are primarily free draining, with ground water discharging as seeps and springs that contribute to surface-water runoff that eventually drains north off the site.

Ground-Water Movement

Ground-water movement beneath the site is controlled primarily by the site topography. Ground-water elevation data indicate that the ground-water flow direction in the advance outwash aquifer is similar to that in the bedrock aquifer, northeast toward the Snohomish River valley (Figure 4). Ground-water flow direction(s) in the discontinuous perched ground-water system could not be determined with the hydrogeologic data collection for this study. However, as noted, perched ground water most likely reaches the surface drainage system via seeps and springs throughout the site.

In the eastern area of the site, ground-water discharge from the advance outwash aquifer occurs where till is absent or thin and where the outwash deposits terminate laterally against bedrock. Discharge from this aquifer is also likely to occur north of the site where outwash deposits are exposed in the incised drainages and creeks (Minard, 1981). Wet, poorly drained, and "swampy" areas in the southwest parts of the site represent another discharge zone for the aquifer.

The Minimum Functional Standards which relate to landfill siting (WAC 173-304-460) require that the bottom of the solid waste be more than 10 ft above the seasonal high ground-water level. In the initial landfill development area, where artesian conditions are present in the advance outwash aquifer, a significant thickness of overlying till (aquitard) is present. This will provide a 10-ft minimum separation between the saturated aquifer conditions and solid waste. However, because the advance outwash aquifer is under artesian pressure, static water levels in piezometers and monitoring wells would rise to elevations less than 10 ft from the top of the till (bottom of solid waste). For this reason, a minimum 10-ft aquitard separation was recommended as the factor influencing site design and not the potentiometric surface elevation of the advance outwash aquifer. A variance issued by Washington Department of Ecology for this alternative design concept would be required for the aquitard separation to be an integral part of the landfill design.

GEOTECHNICAL CONSIDERATIONS

Information developed during the investigation confirmed the suitability of the site for development as the new regional landfill. However, a number of hydrogeologic factors will influence the engineering design of the site. Geotechnical constraints to be considered during developmental planning and design include the following:

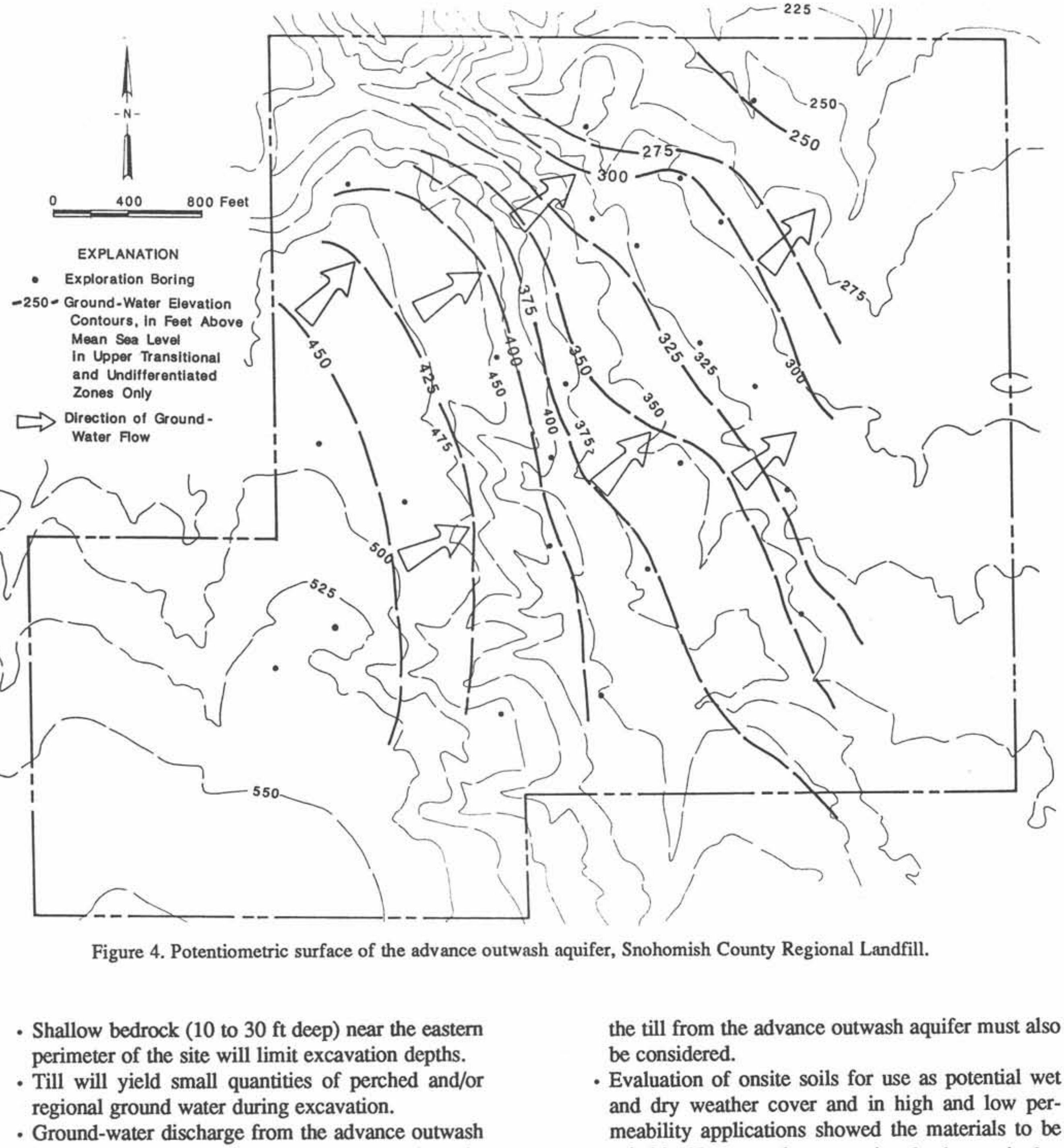

Figure 4. Potentiometric surface of the advance outwash aquifer, Snohomish County Regional Landfill.

- Shallow bedrock (10 to 30 ft deep) near the eastern perimeter of the site will limit excavation depths.
- Till will yield small quantities of perched and/or regional ground water during excavation.
- Ground-water discharge from the advance outwash aquifer occurs in the northern, eastern and southeastern areas of the site. In poorly drained areas, swampy and wetland conditions may occur.
- Depth of excavation near the eastern margin of the site will require careful consideration. In these areas, the till is thin, and the potentiometric surface of the advance outwash aquifer is higher than or slightly below ground surface. If the till is locally irregular or thinner than expected, it could be accidentally breached. Upwelling and uplift pressure exerted on the till from the advance outwash aquifer must also be considered.
- Evaluation of onsite soils for use as potential wet and dry weather cover and in high and low permeability applications showed the materials to be suitable. However, the excavation depths required to reach these deposits may not be economically acceptable.

LANDFILL DESIGN RECOMMENDATIONS

A regional landfill can be sited adjacent to the existing Cathcart site, subject to appropriate engineering measures to mitigate impacts on ground and surface water. As a result of the geotechnical investigation, the following design recommendations were submitted to the county's engineer:

- Excavation plans should be based on the thickness and distribution of till across the site in order to determine the potential for utilizing the *in-situ* till as an additional liner system.
- Discharge of perched ground water and seepage from the till must be controlled to successfully construct soil or membrane liner systems.
- The till should not be excavated in the eastern section of the site where it is less than 20 ft in thickness and where the potentiometric surface of the underlying artesian advance outwash aquifer is at or above ground surface.
- Along the western and northern margins of the site, where the potentiometric surface is below ground surface in the artesian aquifer, excavations should not extend below the potentiometric surface or to a depth which would reduce the till thickness to less than 20 ft.
- Localized areas in the eastern portion of the initial development area may have a high potential for discharging larger volumes of water through the till that presently exists. Where the till is thin or absent and where the piezometric surface is above ground surface, ground water will have to be collected and directed elsewhere.

For final design, further evaluation of the till is required to characterize and locate the till sources that are best suited for landfill development.

Sweet, Edwards & Associates, Inc., recommended that several test excavations be completed in the initial development area to determine ground-water inflow into each test area. The test excavations would need to be at least 100 x 100 ft in order to expose enough till and evaluate potential ground-water flow and geotechnical constraints on landfill design and construction. The findings from these pilot studies on ground-water inflow would assist the engineer to design temporary and permanent drain systems and to determine the feasibility of constructing the landfill cells.

SUMMARY

A pre-design phase of geological exploration for the proposed Snohomish County Regional Landfill was completed in February 1986. The results of the investigation indicated that the proposed site of the new Snohomish County Regional Landfill is underlain by four distinct geologic units which are, from youngest to oldest, recessional outwash, Vashon till, advance outwash, and sandstone bedrock. Vashon till mantles the vast majority of the site and has thicknesses typically greater than 30 ft to the west and thinning to the eastern margins of the site. Recessional outwash overlies the till in isolated pockets and ridges. Advance outwash directly underlies the till on the western portion of the site, and sandstone bedrock underlies the till on the eastern portion.

Ground water is present in all four geologic units. Isolated pockets of perched ground water were found in the permeable recessional outwash, although most of the deposits were unsaturated. The weathered till, which is significantly more permeable than its unweathered counterpart, was generally saturated, and recharge to this unit is thought to be largely from infiltrating precipitation. Ground water was also encountered in the low permeability unweathered till, although permeability testing of wells reflects the overall low measured permeabilities. Horizontal permeabilities of the till ranged from 5.9×10^{-6} to 8.7×10^{-7} cm/sec.

The regional aquifer occurs in the advance outwash deposits beneath the site. The character of the aquifer changes from a water-table aquifer, where static water levels are in excess of 130 ft below ground surface along the western margin of the site, to a confined aquifer with significant artesian heads along the eastern margin of the site. Where the advance outwash deposits are under artesian pressure, the till is acting as a confining layer. The regional aquifer will yield large quantities of water to wells, although no downgradient beneficial uses of water from this aquifer currently exist.

The geotechnical investigation confirmed the suitability of the 440-acre site for development as a regional landfill site. However, it may be that social and economic factors will influence its future development.

ACKNOWLEDGMENTS

The authors thank the Snohomish County Solid Waste Division for allowing publication of the results of this study. In addition, the authors thank Parametrix, Inc., for the opportunity to provide geotechnical consulting services on the Snohomish County landfill siting studies (Sweet, Edwards & Associates,Inc., 1984), and the Regional Landfill Siting Study (Sweet, Edwards & Associates, Inc.,1986). The help and support given by the staff of Sweet, Edwards & Associates in preparing the manuscript is also acknowledged.

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Overview of Subsurface Remediation at a Major Superfund Site: Western Processing Site

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INTRODUCTION

The Western Processing Co., Inc., facility is located in the southeastern part of the Puget Sound lowland near Kent in the flood plain of the Duwamish and Green River, sec. 1, T. 22N., R. 4E. (Figure 1). Since 1982, the site has undergone numerous multi-firm and -disciplinary studies to assess the nature and extent of on- and off-site contamination. An extensive and unique subsurface field program was developed and implemented in 1986 to characterize subsurface conditions for subsequent design of remediation measures. The Western Processing facility is an excellent example of the costly, multi-faceted, and highly technical nature of a Superfund site clean-up. Presented below are discussions of the historical and regulatory framework, site description, subsurface geologic and hydrogeologic conditions, site characterization, field program for development of remediation measures, and a synopsis of the remedial action program.

Western Processing was operated as an industrial waste processing and recycling facility from 1961 to 1983. During this period, reported operational activities included:

- Processing and recycling of animal by-products and brewer's yeast;
- Recovery of heavy metals and waste solvents;
- Reclamation of flue dust, metal finishing by-products, and ferrous sulfide;
- Reprocessing of pickle liquor;
- Neutralization of acids and caustics;
- Production of zinc chloride and lead chromate by chemical recombination;
- Electrolytic destruction of cyanide; and
- Handling of other miscellaneous industrial waste products including battery chips.

From 1952 to 1961, the Western Processing site was leased to the U.S. Army for use as an anti-aircraft facility. In 1961, the site was returned to its original owner and sold to the Western Processing Company, Inc. This company operated an industrial waste recycling facility until the early 1980s. During May 1982, the U.S. Environmental Protection Agency (EPA) conducted a stream survey around the site. Twenty-six priority pollutants were found in the surface water there, all of which were subsequently found on-site. During June 1982, the Municipality of Metropolitan Seattle (METRO) sampled Mill Creek upstream and downstream of Western Processing; a marked increase in heavy metal content, notably zinc, was evident. At this time, the company had Interim Status as a storage facility for hazardous materials as regulated by the federal Resource Conservation and Recovery Act (RCRA). On the basis of the preliminary findings, the EPA issued an order under Section 3013 of RCRA to require the owner to conduct such monitoring as would be reasonable to ascertain the nature and extent of hazard to human health or the environment presented by the site. Since the site owner had declared himself unable to carry out the necessary monitoring, a court order was obtained to enable the EPA to conduct further investigation of the site.

National Priority Listing

In the latter part of 1982, the EPA listed the Western Processing property for cleanup under the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA or "Superfund") by placing the property on the National Priorities List (NPL). As part of the NPL listing process, EPA issued an administrative order requiring Western Processing, Inc. to cease all operations and to begin remedial cleanup of the site. The property has had extensive attention since listing on the NPL, including projects under the direction of the EPA and its contractors, the Washington Department of Ecology and the METRO. The results of investigations conducted in 1982 and early 1983 by EPA and their subcontractors have identified 73 contaminants in soil samples, including 20 metals and 53 organics. In ground-water samples, 46 priority pollutants, including both metals and organics, were identified (Aldis and others, 1984).

Emergency Response and Phase I Clean-up

In April 1983, the EPA initiated an emergency response, including a sampling program, for the removal

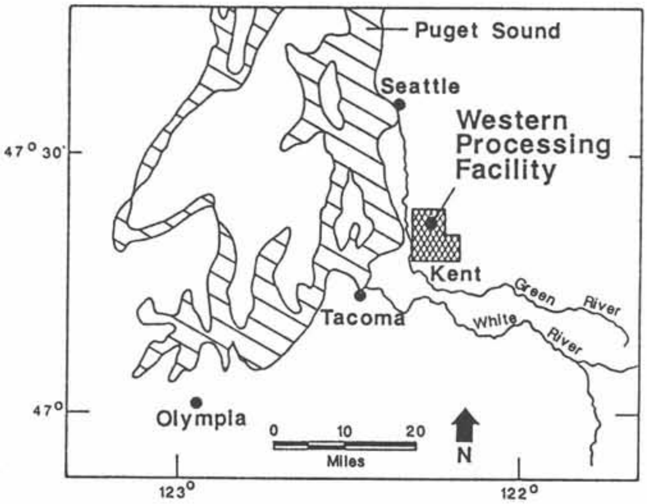

Figure 1. Location of the Western Processing Co, Inc. site in relation to Puget Sound.

of drums and impounded liquids from Area I (Figure 2). Also during 1983, the Washington Department of Ecology completed a project to control storm-water run-on and -off from Area I. This project included stockpiling of contaminated sediments, some grading and paving, and constructing berms around Area I. These emergency responses were followed in 1984 by surficial cleanup of Area I (referred to as Phase I of the remedial action) by Chemical Waste Management, Inc. under contract to certain of the Potentially Responsible Parties (PRPs).

Phase II Remedial Actions

"Phase II" of the remedial action at Western Processing, currently in progress, addresses delineation of subsurface contamination. The Phase II work will be accomplished by a PRPs subgroup known as the Consenting Defendants, within the framework of a Consent Decree negotiated with the EPA and the Washington Department of Ecology. This private party action will be one of the largest to date in the United States. Discussed in this paper is an overview of subsurface Phase II remediation activities conducted and presently being performed at the Western Processing site. Presented below are the site description and geologic and hydrogeologic setting. Also presented is a synopsis of Phase II remedial activities, including discussion of some of the innovative techniques used to provide sufficient physical and chemical information for overall soil remediation, and the subsurface ground-water remedial action program which combines both passive and active methodologies for the control, containment, and remediation of contaminated soil and ground-water.

SITE DESCRIPTION

The Western Processing site is officially defined as the Western Processing Property (Areas I and VII), which occupies approximately 12.4 acres, and Off-Property Remedial Action Areas (Areas II-V and VII-X). A site layout of each area is presented in Figure 2. Mill Creek (also known as King County Drainage Ditch No. 1) runs parallel to the northwest corner of the site and between Areas VIII and IX. An Interurban Trail occupies a former railroad right-of-way paralleling the eastern boundaries of Areas I, III and IX and the western boundary of Area X. A drainage ditch borders the eastern boundary in Area X in conjunction with a high-voltage power line and the Olympic Pipeline, which carries petroleum products. An active Burlington Northern Railroad line also runs parallel to the east side of the site.

At the time Phase II remediation activities were initiated, Area I was occupied by several operational

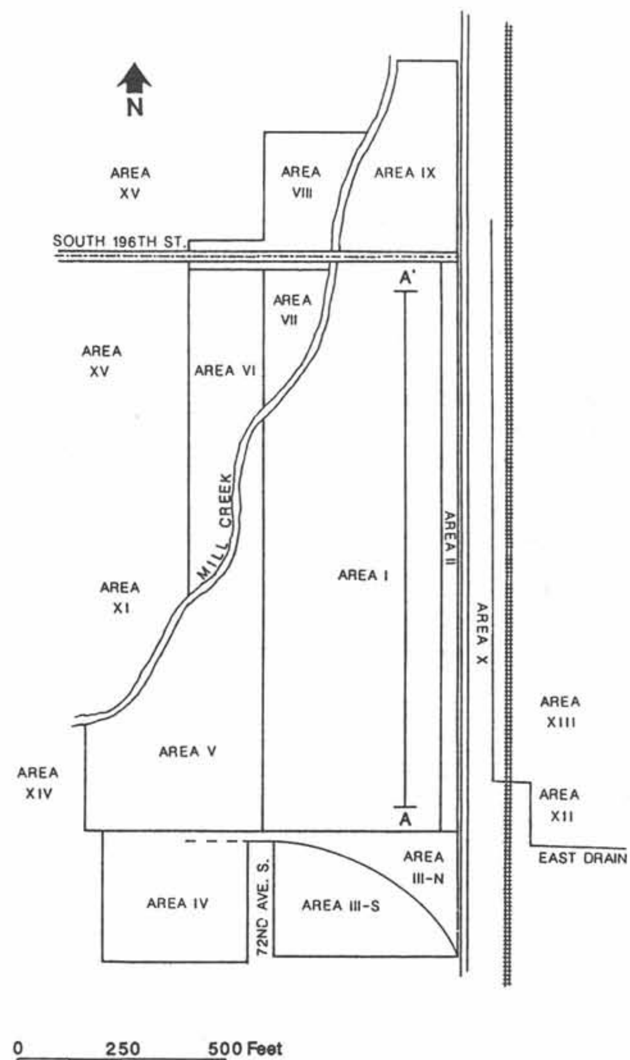

Figure 2. Layout of the Western Processing site showing Western Processing property (Areas I and VII) and Off-property Remedial Action Areas (Areas II through V and VIII through X).

facilities, a mobile dioxin chemical destruction system and tank, a mobile water treatment plant, one synthetic-lined impoundment, and several areas covered with asphalt paving and concrete slabs. Areas II through X were relatively undeveloped and unoccupied.

GEOLOGY

The Western Processing site is situated in the southeastern part of the Puget Sound lowland, a broad, fairly level glacial drift plain that is dissected by a network of deep marine embayments (Luzier, 1969). The site is located in the Green River floodplain, formerly a marine embayment that has been filled with sediments since the end of the last glaciation, referred to as the Vashon Glaciation of Pleistocene age. The valley is bounded by glacial drift plain uplands to the west and east. The general geologic history of the valley and vicinity is discussed in Luzier (1969) and Mullineaux (1970).

The geologic history of the area is summarized as follows:

- Deposition of sedimentary and volcanic rocks which comprise the Puget Group of Eocene age. These rocks were deposited in a subsiding coastal plain which occupied the Puget Sound lowland area and form the base of the unconsolidated glacial and nonglacial deposits in the area.
- Deposition of glacial meltwater outwash deposits reflecting glacial advance into the western Washington area and subsequent deep erosion of the valley. These deposits are chiefly composed of sand and gravel; dense, compacted till was deposited in the upland areas.
- Formation of a deep marine embayment with concurrent glacial retreat and deposition of a thick accumulation of sediment derived from the glacial drift uplands. These sediments are typically coarse sand and gravel near the mouth of the Green and Cedar rivers; they become finer grained with distance toward the north and west in the area of the site.
- Deposition of the Osceola Mudflow which flowed down the former White River valley from Mount Rainier and into the embayment, covering much of the older alluvial deposits. These deposits do not directly underlie the site.
- Deposition of sediments from the White and Green rivers until the White River was rerouted to the south in 1906. Only the north-flowing Green River occupies the valley today, and it is located approximately 4,000 ft west of the site.

The site area is underlain, from oldest to youngest, by the Puget Group, older undifferentiated glacial and interglacial deposits, Salmon Springs Drift, undifferentiated glacial deposits of Vashon age, and White River alluvium.

The Puget Group forms the base of the unconsolidated glacial and nonglacial deposits. Outcropping at the north end of the valley and along the northeastern valley wall, this bedrock unit is projected to be at a depth of greater than 800 ft below the existing valley floor (Hall and Othberg, 1974).

The older undifferentiated glacial and interglacial deposits of Pleistocene age overlie the Puget Group. Composed of thick sequences of silt and sand with subordinate layers of sand and gravel, these deposits do not appear to extend across the valley.

The Salmon Springs Drift of Pleistocene age, which overlies the older glacial and interglacial deposits, flanks both the east and west sides of the valley. Consisting mainly of sand and gravel, most of these deposits occur above sea level; their base is approximately at the elevation of the valley floor.

The Vashon undifferentiated glacial deposits comprise the surficial deposits in the upland areas. These deposits typically consist of sand and gravel with recessional outwash near the surface. The recessional outwash deposits are underlain by a significant thickness of dense till which overlies advance outwash sand. Estimated to be about 100 to 200 ft thick, these deposits occur above sea level and the existing valley floor.

The White River alluvium, of late Pleistocene and Holocene age, comprises the valley fill deposits. These deposits consist predominantly of sand, silt, and clay with scattered layers of sandy gravel to depths of more than 360 ft below ground surface.

The site is immediately underlain by fill and postglacial unconsolidated deposits of laterally discontinuous lenses of sand, gravel, silt, and clay of the White River alluvium. On the basis of published geologic and hydrogeologic data and previous and present geotechnical and assessment investigations, it is known that localized fill deposits are typically present to depths of about 8 ft below ground surface. However, as much as 24 ft of contaminated fill has been encountered locally. The fill is underlain by unconsolidated, stratified, finer grained soils (silty sand, silt, clay, and peaty organic-rich soils) with subordinate coarser grained soils (silty sand and sand) to a depth of about 50 ft below ground surface (Figure 3). Two distinct laterally discontinuous silty clay layers ranging from about 5 to 10 ft thick are encountered at depths of less than 15 ft and at 30 to 40 ft below ground surface. The shallow silty clay stratum is restricted to the northern part of the site. Between about 50 and 110 ft below the site, poorly graded fine to medium sand with little to some fine material predominates; subordinate discontinuous clayey silt lenses and sandy gravel with sparse shell fragments are also present. The sandy gravel is typically present as a discontinuous lens less than 2 ft thick at a depth of about 108 ft (Ecology and Environment, 1983). Below that depth are finer grained, stratified, discontinuous, interbedded lenses of clayey silt and clay with subordinate

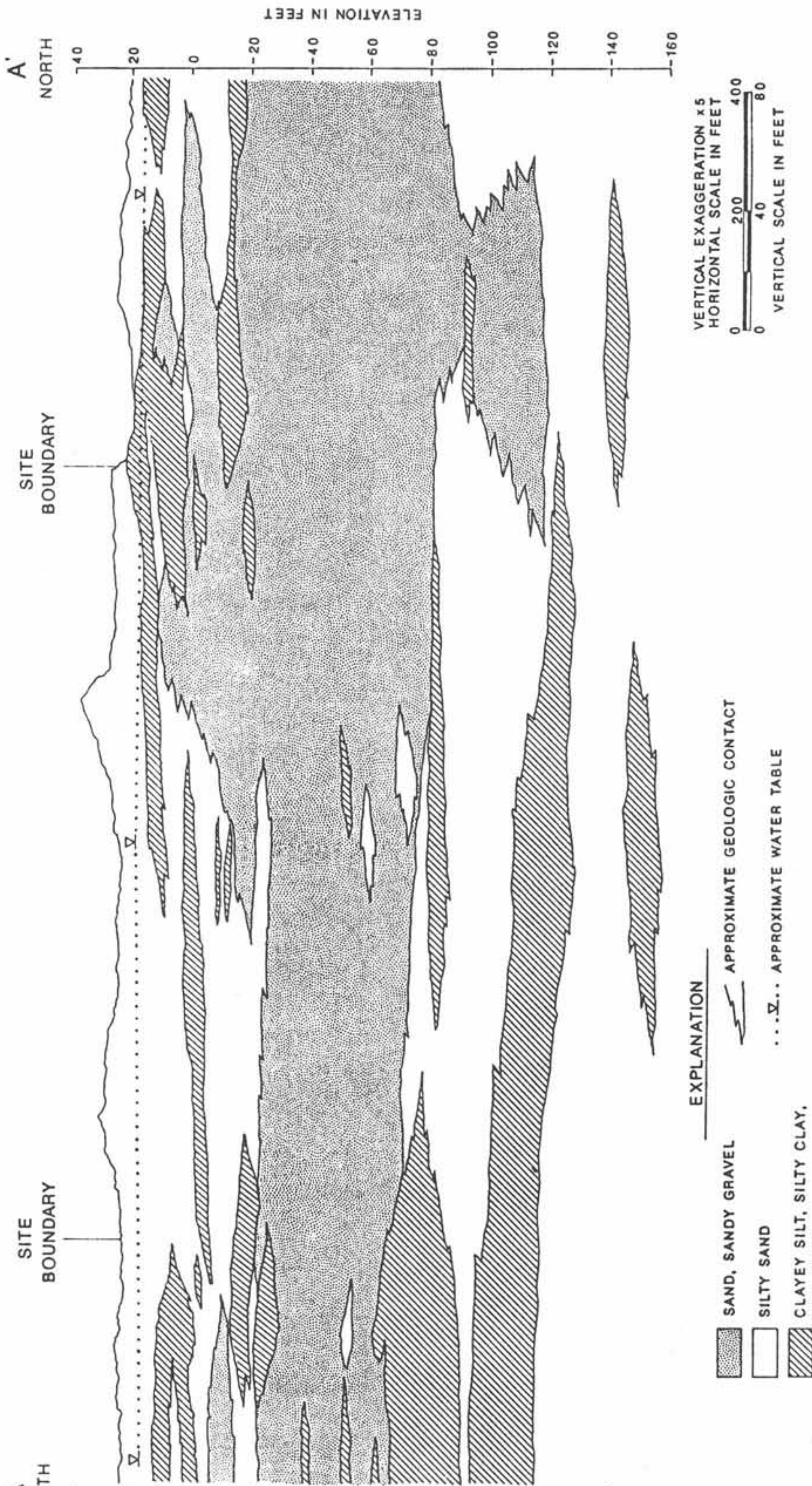

Figure 3. Hydrogeologic cross-section trending north-south through the central part of the Western Processing site.

silty sand, silty clayey sand and sand (Ecology and Environment, 1983).

HYDROGEOLOGY

The study area has cool, dry summers and mild, cloudy, and rainy winters. The dry season occurs from about April through September; the wet season generally lasts from October until March. Average annual precipitation ranges from about 40 in. within the floodplain near the site to about 80 in. near the headwaters of the Green River east of the site.

Ground water near the site occurs at shallow depths under water-table conditions within the upper part of the White River alluvium and under confined conditions within a deeper glacial (or alluvial) aquifer. Although the presence of a confining layer (aquitard) has not been confirmed directly beneath the site, the existence of such an intervening stratum is evident both west and east of the site where numerous deep, flowing (artesian) wells are present. The piezometric surfaces in these wells, which develop the deeper aquifer, are at a higher elevation than that of the upper alluvial aquifer, indicating upward vertical flow gradients; reported artesian levels are tens of feet higher than ground surface. The regional ground-water flow direction in the upper alluvial aquifer is north-northwest at an estimated gradient of 0.002 ft/ft. Seasonal fluctuations are significant, ranging from about 2 ft to several feet.

Recharge for the deep aquifer is from the upland areas east and, to a lesser degree, west of the Green River valley. Recharge to the upper alluvial aquifer is primarily by direct infiltration of precipitation within the valley and to a lesser degree from the upland areas and periodic contributions from streams during high stage periods.

Site ground-water conditions in the upper alluvial aquifer are characterized by mounding in the central portion of the site (Figure 4). Ground water is encountered at depths ranging from about 3 to 12 ft below ground surface. Conditions of the water table indicate that ground-water flow directions are roughly radial and downward. Gradients are relatively steeper in the northwest part of the site adjacent to Mill Creek, which hydraulically influences the shallow flow regime and acts as a local discharge zone. In wells installed near Mill Creek and screened at different depths, vertical downward gradients are observed to a minimum depth of about 30 ft (Figure 4). Mounding is likely caused by a variety of factors, including on-site ponding at drainage outlets on the site's periphery and variations in soil conditions. Below 30 ft in depth, upward vertical flow gradients on the order of 0.02 ft/ft are apparent.

The upper alluvial aquifer can be classified as a Class IV aquifer—that is, as a potential, but unlikely source of drinking water (Bureau of National Affairs, 1983).

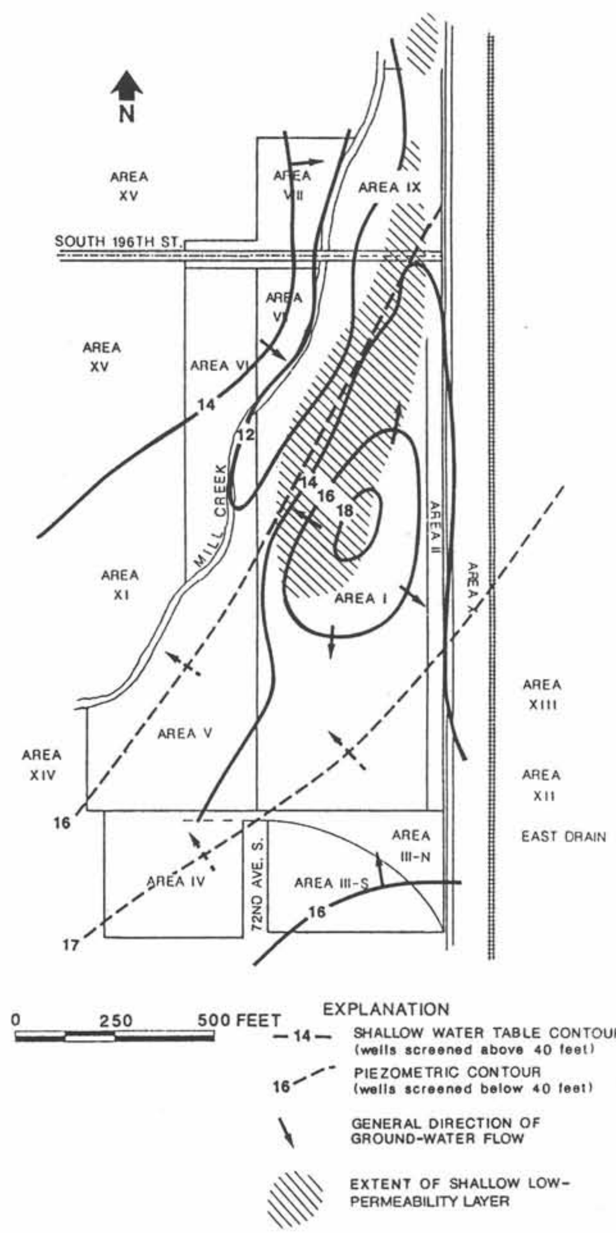

Figure 4. Site layout showing shallow water table and deeper peizometric contours. Also shown is the general direction of ground-water flow and lateral extent of shallow low-permeability layer.

The deeper aquifer could possibly be a potential source of ground water. Thus, it would be protected by the same means as similar sources and classified as a Class II aquifer.

The hydraulic conductivity of the finer grained soils within 50 ft of ground surface is estimated at about 3 x 10^{-4} to 3 x 10^{-3} cm/sec or 1 to 10 ft/day on the basis of pumping and slug test data. The conductivities of coarser grained soils at depths below 50 ft are estimated at about 3 x 10^{-3} to 3 x 10^{-2} cm/sec or 10 to 100 ft/day.

PRE-PHASE II REMEDIAL ACTION PROGRAM

Prior to initiation of Phase II subsurface remedial activities for soil and ground water, additional physical and chemical information for development of certain aspects of the Phase II Remedial Action Program pertaining to soil remediation was provided by a multi-disciplinary team assembled to achieve the following objectives:

- Sample and analyze soil and waste materials to develop chemical data. The data generated would be used to prioritize removal of contaminated materials and evaluate alternative remedial techniques;
- Develop correlations among visual appearance of soil and waste materials and contamination;
- Determine the locations of buried materials such as drums, utilities, and process lines; and
- Determine the extent of surface and subsurface contamination on adjacent properties.

Geophysical Survey Program

The results of the geophysical surveys were successfully used to locate, excavate, and sample soil and water from a wide variety of subsurface features including buried drums, subsurface tanks, buried utility corridors, and process lines.

Approach

The geophysical surveys performed included a magnetometer (MAG) survey, including both total field and vertical gradient, a shallow electromagnetic induction (EM) survey, and a ground penetrating radar (GPR) survey. MAG and EM data were collected on 10-ft centers, while GPR lines were run in other orientations and spacings where required to gather more site-specific data.

The MAG measures variations and disturbances in the Earth's total magnetic field strength created by ferromagnetic objects (that is, iron and steel). The magnetic gradiometer has greater lateral resolution than that of a total field survey. Thus, buried objects which appear as a single composite total field anomaly can often be individually identified on vertical profiles or maps. This greater spatial resolution thus reduces the area of search at individual anomalies.

The EM measures the apparent conductivity of the soil to a depth of about 20 ft and responds to electrical conductivity contrasts such as buried pipes. In addition, variations in soil conductivity can be assessed, notably in the presence of contaminated ground-water plumes.

The GPR was used in conjunction with the MAG and EM to characterize shallow subsurface anomalies and to refine the data generalized from the other systems to locate and track buried utilities. The radar energy reflected from various subsurface materials depends on the contrast in electrical properties (which are dependent upon mineral constituents), density, and water content of those materials.

Combining the results of these geophysical surveys proved to be a very cost-effective and powerful interpretive tool for focusing and minimizing subsequent exploration and remediation activities. A discussion of these techniques and how they operate is given by Lepic and others (1987).

Results

To allow for characterization of anomalous surface conditions, EM terrain conductivity and in-phase component maps, total magnetic field and vertical gradient maps, and a radar target map were developed. Compositing of these maps in conjunction with reference to historical documentations identifying past site land-use activity was then used to assist in the subsurface exploration and chemical testing program. As a result, a total of 39 anomalous regions were identified.

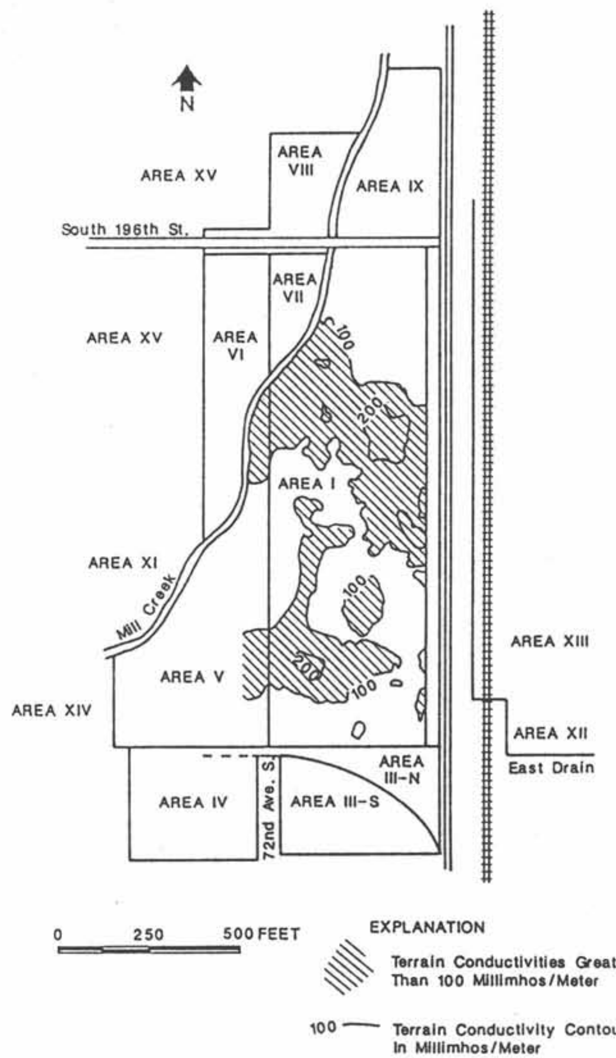

Figure 5. Terrain conductivity map showing areas of anomalously high terrain conductivities which may represent ground-water contamination and/or areas of high metal concentrations in the underlying soil.

A summary of the terrain conductivity (EM) data for Area I is presented in Figure 5. Areas of terrain conductivity greater than 100 millimhos/m are considered anomalous. Large areas of the site were determined to fall within this category. These areas of high terrain conductivity may represent shallow layers of soil saturated with ground water of high specific conductance or locations of buried conductive wastes and debris. General terrain conductivity conditions and the signature of a utility trench containing a buried steel pipe are shown in Figure 6. The geophysical signatures (EM-31 inphase and vertical magnetic gradient contour maps) of an area where several drums were buried in the southern part of the site are presented in Figure 7. Finally, the radar signature of a buried 6-in.-diameter steel pipe is shown in Figure 8.

Analytical Program

Upon completion of the subsurface geophysical surveys, the drilling and excavation with chemical testing of selected soils was initiated. Two hundred and seven shallow soil borings were made, 92 test pits were excavated, and selected soils were subsequently sampled for chemical testing. The investigation program resulted in the collection and analysis of 1,515 samples of soil and non-soil materials. All samples were forwarded to one of three laboratories for specific analysis. Two off-site laboratories provided priority pollutant analyses on 272 samples. One of the offsite laboratories performed solid waste leach procedure (SWLP) testing on 127

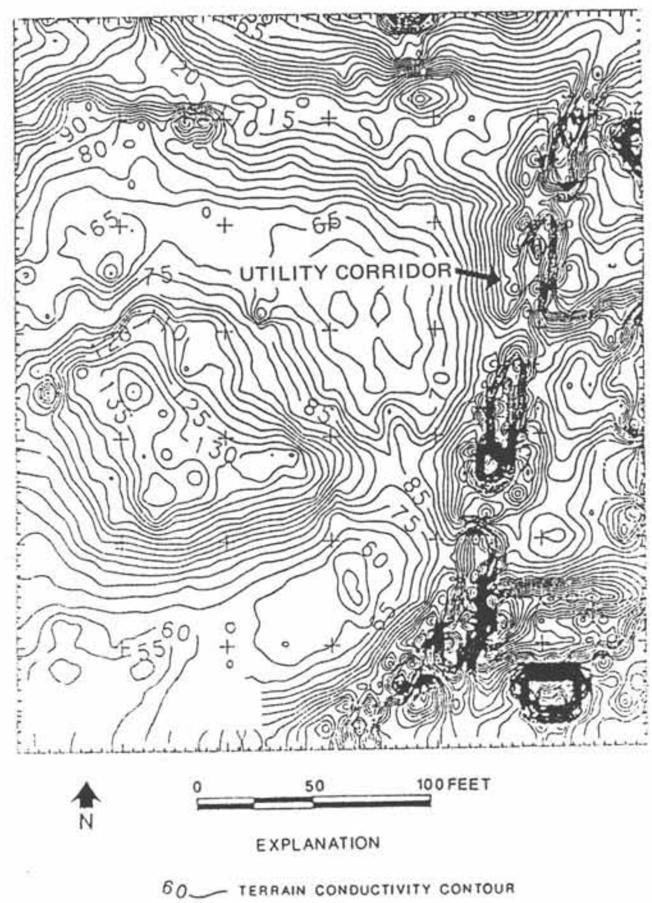

Figure 6. General terrain conductivity contour map showing a signature of a buried utility trench containing a steel pipe.

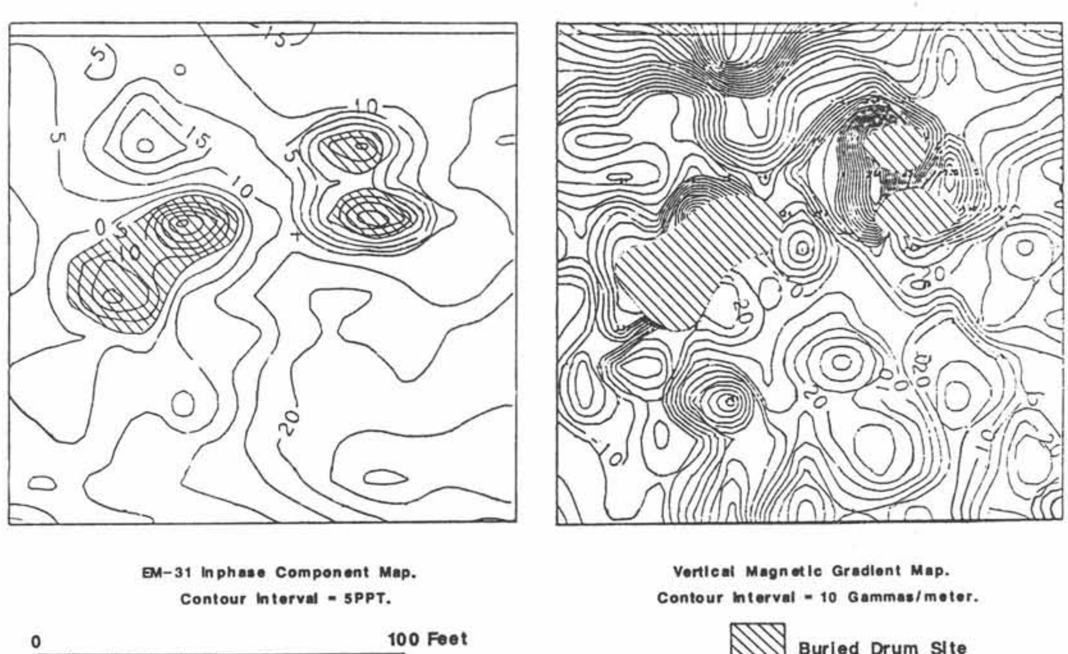

Figure 7. Vertical magnetic gradient and inphase component maps showing anomaly signatures at an area containing multiple buried drums in the southwestern part of Area I.

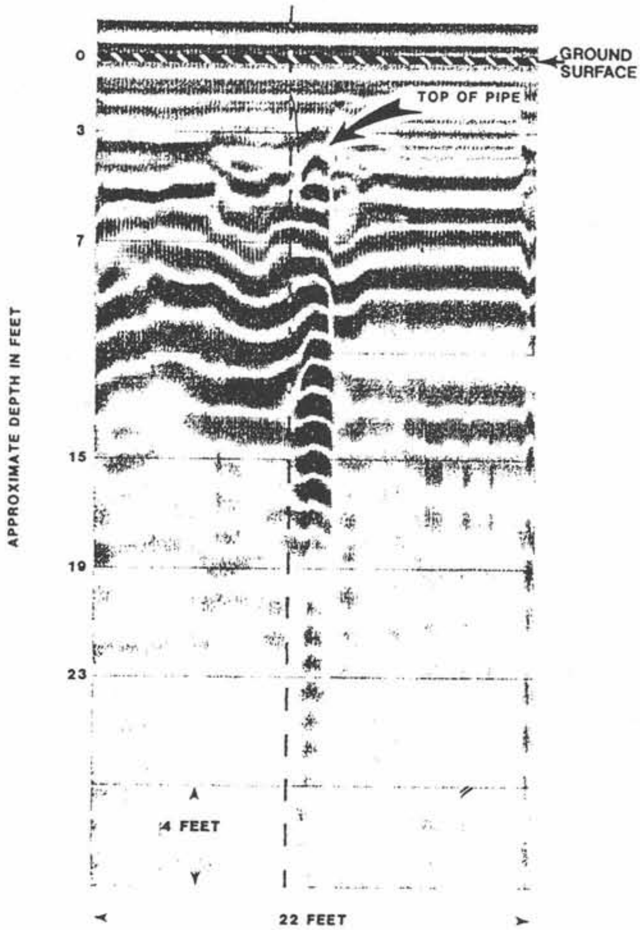

Figure 8. Ground-penetrating radar profile showing geophysical signature of a buried 6-in.-diameter pipe.

samples. Both offsite laboratories performed all analytical work in conformance with a detailed quality-assurance program developed specifically for this project. In addition, an onsite laboratory was set up to provide rapid screening of soil samples for 26 chemical constituents. This greatly reduced total analytical costs as well as the time required to respond to problem areas and overall site characterization.

On-site laboratory

The onsite laboratory consisted of a sample-processing facility and an analytical facility. This design was selected to reduce the possibility of sample contamination. Samples were delivered to the processing facility for weighing, extraction, and digestion. The resulting sample extracts were then transferred to the analytical facility for analysis. This facility contained two Hewlett-Packard Model 5809A gas chromatographs fitted with automatic sample and data processors; a Waters High Pressure Liquid Chromatograph coupled to a WISP auto sampler, a Waters Model 490 multi-wavelength detector, a Perkin Elmer Model LS-4 UV-fluorescence Spectrophotometer with automatic data processing by two Spectra Physics Model 4290 integrators; a Perkin Elmer Model 2300 Atomic Absorption (AA) Spectrophotometer; and Dionex Model 4002i ion chromatograph. Except for the AA, these instruments were fully automated and ran continuously for the duration of the project. The onsite laboratory was designed to screen 20 samples per day for inorganic and organic contaminants, although daily processing peaked at 40 samples. Samples were analyzed for the 26 chemical constituents listed in Table 1. The constituents are grouped according to the analytical equipment used by the onsite laboratory.

The onsite laboratory provided analytical results the day following sample collection. To achieve this rapid turn-around and still be able to process the desired number of samples, the onsite laboratory used screening analyses. Generally, these methods were simple modifications of existing EPA procedures. In some cases, new analytical screening methods were developed specifically for the contaminants anticipated to be present at the site. Rigorous quality-assurance testing was carried out to assure that the onsite testing

Table 1. On-site laboratory screening parameters for soil samples

Method	Parameter
Ion chromatography	* Cadmium * Chromium (hexvalent) * Chromium (total) * Copper Cyanide (total) * Lead * Nickel * Zinc
Headspace analysis chromatography	1,1-dichloroethane Chloroform Ethylbenzene Methylene chloride Tetrachloroethylene Trans-1,2-dichloroethene Trichloroethene Toluene 1,1-dichloroethene
Gas chromatography	* PCB aroclors (total) + Benzo(a) pyrene
High pressure liquid chromatography	* Oxazolidone 2,4-dichlorophenol Phenol 2,4-dimethylphenol bix(2ethyl-hexyl)phthalate

* Denotes analysis was performed on-site and off-site

+ Denotes analysis was only performed on on-site properties provided reliable analytical results. The quality-control program included blank and duplicate samples, analytical replicates, matrix spikes, and reference materials. The blank and duplicate samples represented an additional 20 percent of the total investigation samples. Duplicate samples were presented to the laboratory in a totally blind fashion.

Data management

Approximately 300,000 units of analytical data were generated. These data were entered into a custom-designed relational database management system developed in the R:BASE System V database environment. Each of the three laboratories entered analytical data directly into menu-driven modules of the database system which were designed and provided for their use. Analytical results were reported using floppy discs or telephone modem on a daily (screening) or weekly (priority pollutant/SWLP) basis. Onsite microcomputers were used to track sample information, including sample location, depth, and chain of custody. The computers were also used to perform data reduction and sorting and report generation. In addition to daily reports of analytical results, the data were reported in ad-hoc configurations as needed to the study team, the client, and representatives of EPA and Department of Ecology. For instance, the previous day's results could be sorted and reviewed to identify locations where additional samples were needed. These systems were invaluable to "fast-track" an intensive investigation of this nature.

During the investigation, the results of the screening analysis were reviewed to determine appropriate locations for additional borings and test exploration sites. The computer data were sorted into desired categories, and three-dimensional contour plots of the screening data were prepared. This information served both as a tool in this decision-making process and a method of providing a graphical display of interim data to the EPA and Department of Ecology. Examples of the three-dimensional plots for cadmium and chromium are shown in Figure 9.

PHASE II REMEDIAL ACTION PROGRAM

Several effective means of dealing with soil and ground-water contamination problems exist. With the variations in subsurface geologic and hydrogeologic conditions and the vast array of contaminants present in the subsurface at the site that must be dealt with, the preparation of a subsurface remedial action program is made very difficult. Thus, much study is undertaken prior to any decision regarding which alternative or combination of alternatives are employed for containing, controlling, and remediating contaminated ground water. Essential to the development of an appropriate and successful containment, control, and remediation program is a thorough understanding of the subsurface geology and hydrogeology and ground-water flow regime.

A combination of both passive and active forms of containment is typically employed. Passive components of an overall containment system are those physical elements that act as high-impedance barriers to the flow of ground water. They are typically constructed of materials exhibiting very low intrinsic permeability relative to the *in-situ* soils in or on which they are placed. The components of an active containment system include those that modify the flow of ground water by artificially creating ground-water sinks or sources. The artificial sinks or sources are typically established via pumping, pipe transport, and recharge using extraction or recharge wells and infiltration trenches. Such systems effect contaminant isolation through gradient manipulation.

The remedial action program for the Western Processing site consists primarily of two major interrelated activities: (1) the physical removal of contaminated soil and wastes, and (2) the flushing of contaminants from both soils and ground water beneath the site. An overall plan is summarized in Figure 10.

Because the contaminants reported in soil and ground water from beneath the site have migrated to considerable depths, total removal to background levels would require, in part, removal of soils to depths considerably below the water table and would necessitate an extensive dewatering and ground-water treatment program. Thus, an alternative program was developed to incorporate both passive and active containment and remediation strategies.

Soil Removal

The soil remediation program includes removal of buried wastes and soil from all areas where filling of topographic lows and excavations occurred while the site was in operation. The surficial soils (±-1 ft, depending on the extent of contamination) will also be removed from the remainder of the site. These areas will be backfilled with clean fill material. The site will then be regraded and a roller-compacted concrete cap emplaced. Soils contaminated by migration of contaminants leached from their respective sources will remain. To allow removal of contaminated sediments from the course of Mill Creek, the creek will either undergo dry construction by diversion via a temporary diversion pipeline or be remediated using wet construction methods.

Contaminant Flushing

A pumping system will be installed to flush contaminants from the soil and ground water (Figure 11). To allow water from the surrounding and underlying upper aquifer to be drawn up through the contaminated soil, a 50-ft hanging cut-off wall will be installed along the perimeter of the site. As this relatively cleaner water

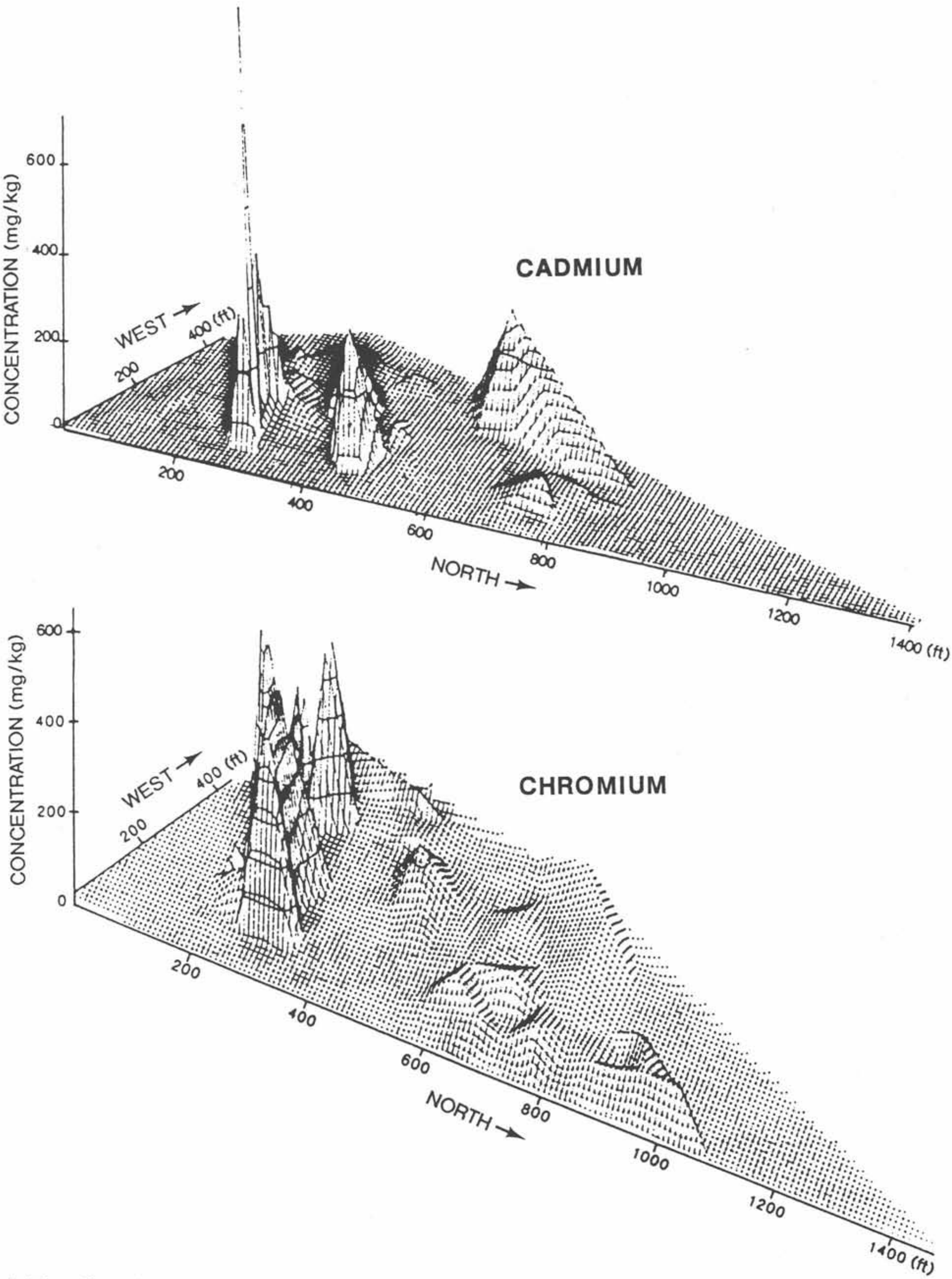

Figure 9. Three-dimensional plots for cadmium and chromium concentrations for surficial soils across the entire Western Processing site.

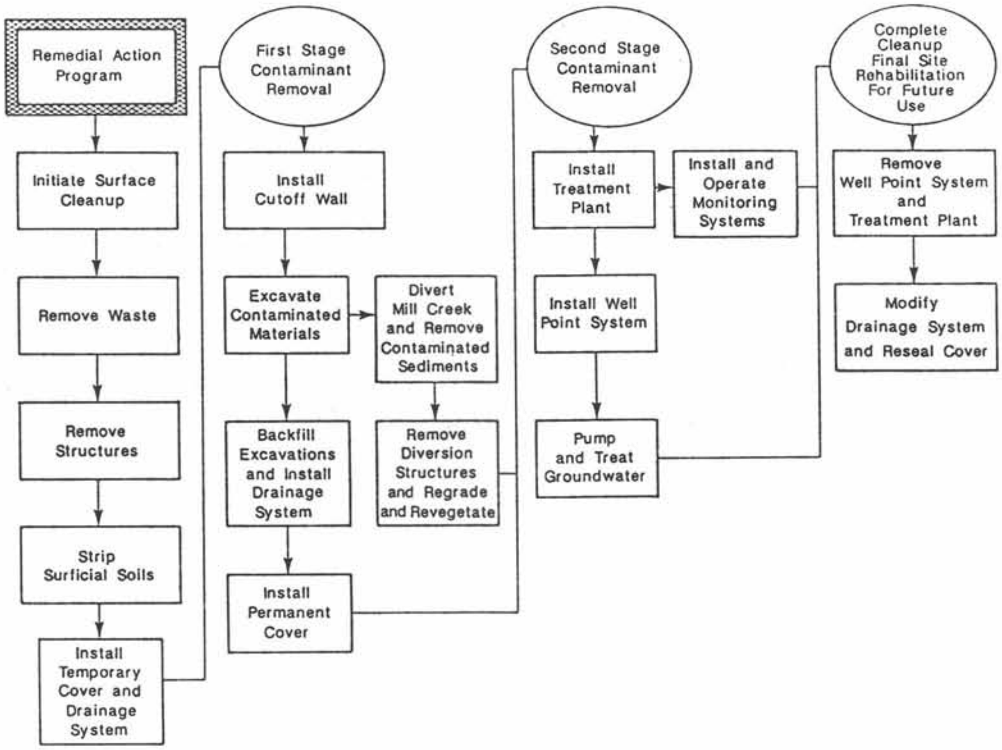

Figure 10. Flow diagram outlining the overall remedial action program.

passes through the site soils, mobile contaminants will be flushed from the remaining contaminated soil. The cut-off wall will be constructed prior to initiation of the flushing program. A soil-bentonite slurry with a permeability of 1×10^{-7} cm/sec or less will be used to construct the wall. Prior to final design, the effects of site-specific contaminants on the wall materials and long-term permeability will be evaluated. In addition, on-site testing will include viscosity, density, and monitoring of slurry preparation during construction.

Within the slurry wall area, a well point extraction system will be installed. A series of well points on

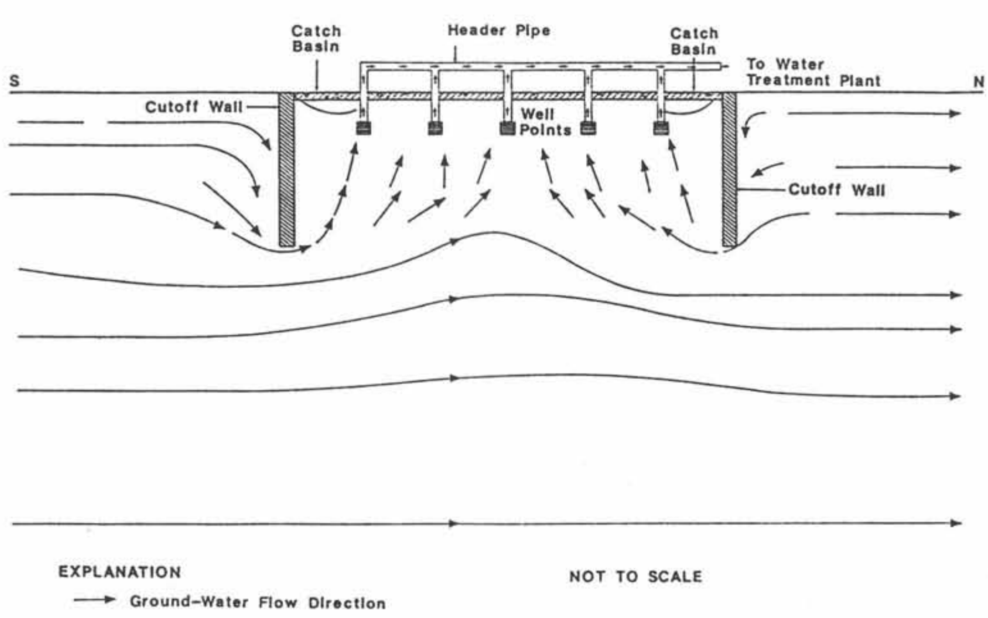

Figure 11. Schematic diagram showing proposed pumped system to accomplish the flushing of contaminants from the soil and ground water.

20-ft centers will be installed through the cap to an approximate depth of 15 ft below the water table. The purpose of the well point extraction system is twofold: (1) to remove contaminated ground water from beneath Area I and a portion of Area IX; and (2) to allow for the option of injection of chemical solutions which can be used to either enhance the leaching process or, during later stages, immobilize the contaminants. The well point extraction system will include approximately 209 well points, three header pipes, three combination pump and vacuum units, including appurtenances in a pumphouse facility and discharge piping to the extraction water-treatment facility at Area VII. All well points will be manifolded to a header pipe. A centrifugal-vacuum pump combination will pump ground water through the manifold system and to the water-treatment facility.

An extraction system will also be installed to remove and treat ground water in selected locations where the earlier presence of relatively high concentrations of trans 1,2-dichloroethylene have been reported. Upon completion of this phase of activity, selected wells may be incorporated into the barrier well extraction system.

The barrier well extraction system will be used to establish a hydraulic barrier to regional ground-water flow in the depth range of 40 to 70 ft. This system will be approximately located along the western perimeter of the site, west of Mill Creek. An alternate system is also planned so as to provide a reversal of ground-water flow at this 40- to 70-ft interval. This system will consist of approximately 6 pumping wells and pits, 12 piezometers, gravity discharge pipelines to Area VII, and a valve vault at Area VII. An alternate barrier well system is located east of Mill Creek and will consist of 11 pumping wells and pits, 14 piezometers, discharge piping systems to the center of Area I and to Area VII, and a valve vault.

To augment the removal of contaminants using ground-water extraction systems, an infiltration trench system will be installed in Areas I, II, V, and IX. This system will consist of about 13,000 linear feet of infiltration trench about 3 ft in depth, distribution pipelines, and other appurtenances such as pumps, valves, and controls. The purpose of this trench system is essentially to infiltrate treated water from the ground-water extraction systems into contaminated soils above the water table. In addition, this system may also be used to distribute chemical solutions for enhanced leaching of contaminated soils.

The flushing program is planned as a 5-yr operation, after which not all contaminants will have been completely removed, but significantly reduced concentrations of many of the organics are anticipated. Testing of ground water will be periodically performed to monitor on- and off-site water quality, flushing, and treatment efficiency, pumping volumes, and ground-water behavior. Approximately 70 wells will be periodically sampled.

The pumped water will be discharged to a treatment plant to be constructed on the site. Water treatment will include metal precipitation, air stripping, and activated carbon as necessary to meet effluent requirements. Treated water will be discharged in accordance with an appropriate NPDES permit, while materials removed from the water will be disposed of at a regulated hazardous waste disposal facility or in another approved manner. Following completion of the ground-water flushing program, pumping and treatment facilities will be dismantled and removed, and the site will be sealed by asphalt and ready for productive use.

ACKNOWLEDGMENTS

I have been periodically involved with the Western Processing site since April 1983. Significant information pertinent to the Phase II Remedial Action program was provided by numerous sources. The analytical program discussed under the Pre-Phase II Program was developed and performed by Farr, Friedman & Bruya, Inc., of Seattle, Washington. The geophysical survey program discussed was developed and performed by Northern Technical Services, Inc., Redmond, Washington. I thank my wife, Lydia, for assistance with manuscript preparation and Tony Carmouche of Engineering Enterprises, Inc., for preparation of the technical illustrations.

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Coastal and Marine Engineering Geology

Kim L. Marcus, Chapter Editor

Coastal and Marine Engineering Geology: Introduction

KIM L. MARCUS
Dames & Moore

The peoples of what is now known as Washington have always used the abundant water resources of the state for their livelihood, transportation, and recreation. The 2,341 mi (Scott et al., 1986) of salt water coastline and thousands of miles of navigable rivers are used by thousands daily, and these are where abundant wildlife live. From ancient to modern times the many rivers that dissect the land and the ready access to the land from the sea have been focal points for civilization. And as focal points they have been modified by the peoples of the area, and the edges of the land-water interface have been adjusted. The early filling of low spots or narrowing of channels for the placement of nets or dredging to make transportation easier have given rise to the wholesale adjustment of river mouth. In some cases these changes have created the need to artificially recharge beaches where natural sediment sources have been disrupted by the damming of rivers, and the need to build massive jetties and breakwaters to facilitate navigation.

This dynamic relation between the peoples of the area and the coastal environment has at times worked well and at times not. The modifications made to the shorelines have on occasion been made with little knowledge or thought of the outcome. As modifications have been made, lessons have been learned, and that is what this chapter is about: some observations of and the experiences with coastal processes.

The paper by Thomas Terich addresses the need to understand the implications of modifying or living within the coastal zone of Puget Sound. He identifies data gaps that need to be closed, and he proposes that accumulated information should be applied pragmatically where possible.

One of the fundamental building blocks of living in this coastal interface zone is the understanding of physical processes that affect it. Maury Schwartz discusses one of the most important of these processes: net shore-drift. As engineering geologists, we are trained observers who can turn observations into practical solutions. In his paper, Schwartz provides insights into the relation of beach morphology to the determination of drift cell dynamics. The central concept is that source areas, transport zones, and deposition or terminus sites are all interdependent.

Richard Galster's paper on jetties, breakwaters, and boat basins relates coastal processes to various features of design and siting of these most common of coastal structures. While these structures are mostly manmade, Galster's description of the Ediz Hook, presented in another paper, shows what modifications and studies have been performed in order to preserve a natural spit.

In the paper by Steve Fuller and his coauthors, the Holocene stratigraphy at the mouth of the Snohomish River is assessed in order to understand the potential impact on future development. This study was undertaken for the U.S. Navy in its bid to put a Navy homeport in Everett. Detailed geologic, biologic, oceanographic, and hydrologic studies were performed to understand the coastal processes. Knowledge obtained in this investigation will have far-reaching implications. Few sites in the Washington region have been or will continue to be so thoroughly studied. The paper presented herein is a small piece of the overall picture, but it is one of the key elements in the homeport investigation.

Wholesale modifications of coastal areas are virtually a thing of the past; today they are faced with the expenditure of large amounts of money and years of delay in the preparations of Environmental Impact Statements. Not so in the past, as is evidenced in the last paper in this chapter. Richard Galster reports on the history and engineering geology of three navigation canals built in the early part of the 20th century in the Puget Sound region. The Lake Washington Ship Canal, Port Townsend Canal, and Swinomish Channel constitute the only true navigation canals in the state. Large-scale construction jobs such as these contrast dramatically with present-day projects such as the U.S. Navy's homeport. Advances in technology have reduced construction time, increased worker safety, and enabled geologists to understand site conditions in much greater detail than ever before.

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View to the west of the rubble mound breakwater at Blaine. Photograph by R. W. Galster, June 12, 1987.

Floating breakwater protecting the former boat basin at Friday Harbor, San Juan Island. The marina has subsequently been enlarged, and a new concrete floating breakwater has been constructed. Photograph by R. W. Galster, May 1984.

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Structural Erosion Protection of Beaches along Puget Sound: An Assessment of Alternatives and Research Needs

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The coast of Puget Sound is mainly vertical. Shorebluffs with narrow pebble and cobble beaches at their bases prevail. Coarse beaches and cold water discourage sun and water bathers; nonetheless, waterfront property around the sound commands premium prices.

Puget Sound and its shores are the products of at least four episodes of glaciation over the past 2 m.y. Great tongues of ice pushed down the Fraser River canyon in British Columbia into the Puget Sound lowland. The scouring action of the ice dug deep, narrow channels. As the ice retreated, massive amounts of till, sand, and clay were laid down from Olympia to Blaine. Alternating layers of these sediments are exposed along shorebluffs, revealing our glacial legacy.

The beaches of Puget Sound are derived from the erosion of the glacial sediments that line the shore (Downing, 1983); therefore, most beaches of the region are composed predominantly of pebbles and cobbles. The finer grained sediments are carried into quiet coves and offshore. The beach cobbles and pebbles are transported along the shore only by the strongest storm waves. Some large immovable boulders seen along the shore mark the former positions of the retreating shorebluffs.

Three shore configurations are common to Puget Sound. Most common is the narrow pebble-and-cobble beach backed by high glacial bluffs; many of these are more than 20 m high (Figure 1). Wind, water, and gravity attack the bluff, driving eroded sediments to the beach below. The dry backshore is commonly strewn with drift logs and flotsam deposited during the coincidence of storm waves and maximum tides. Homes on these high bluffs take advantage of commanding water views.

Another common configuration of the shore is the low shore bluff, 1 to 3 m high, aproned by a narrow beach that is usually submerged at ordinary high tides. These shores are attractive for development, and property owners are usually quick to install a seawall, hoping to reduce further erosion of the bluff and to give a "cosmetic" edge to the waterfront margin of their land.

The third and less commonly encountered shore type is the low depositional spit with a sandy and pebbly beach backed by bays or wetlands. These shores are developed quickly since they offer the optimum beachfront living; however, they are also vulnerable to erosion and flooding. Their stability depends upon an uninterrupted supply of sediment from updrift sources. Blockage of the sediment supply makes these shores prone to erosion and flooding (Figure 2).

Seldom does the shore erosion problem around Puget Sound and Georgia Strait result in homes falling into the sea or massive failures of the shorebluffs. Erosion is generally not spectacular; rather, it is slow and steady. Three common hazards facing the waterfront property owner are beach erosion, storm-surge flooding, and shorebluff failure. Each presents various levels of risk dependent upon oceanographic, meteorologic, and geologic conditions. The instinctive response to these hazards is the installation of a wall between the land and water. The fundamental problem with this approach is the placing of a rigid immobile structure in a highly mobile environment—the shore. Seawalls and bulkheads are generally perceived by coastal residents to be guaranteed protection against coastal erosion and storm damage (Burton, 1969). They become a type of insurance property owners are willing to invest in. Knowles and Terich (1977) found that more than 66 percent of the property owners at Sandy Point, which faces the Strait of Georgia, believed waves and beach erosion to be a minor problem, in spite of several homes having been damaged by high waves and drift logs just 2 years earlier. This kind of perception prompts shorefront property owners to attempt a less expensive "do-it-yourself" approach to erosion protection (Downing, 1983).

Proper design and engineering of a concrete seawall are key to its longevity. Griggs (1988) found that well-designed concrete seawalls installed along California's central coast required less maintenance and provided a greater measure of protection to coastal properties than did riprap. Riprap was found to slump into the beach sand and in some instances disappear.

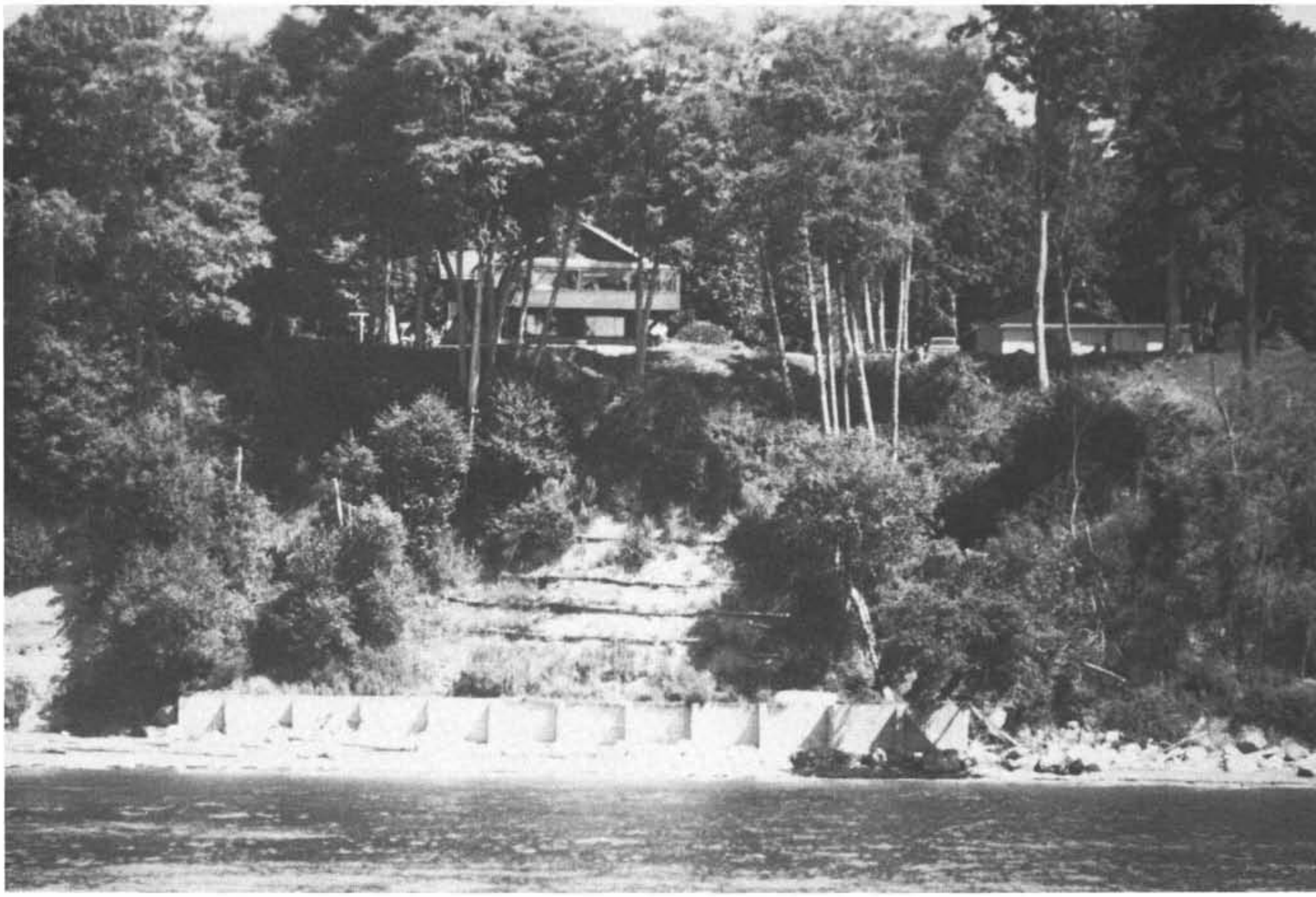

Figure 1. Typical high shorebluff with erosion protection.

Prior to the passage of the Shoreline Management Act of 1971, very little information regarding the design of such structures was available. Today, more information is available from federal, state, and local agencies, but the the degree to which this information is actually used by the public is not known.

It should also be noted here that not all the seawalls around the shores of Puget Sound are installed for erosion control. Some simply serve as the "cosmetic edge" to waterfront property. These walls are commonly seen in calm, sheltered bays and are long-lived because erosional forces are at a minimum.

Cities and counties around Puget Sound and Georgia Strait have permitting authority over the installation of erosion control structures. Under certain conditions, state and federal agencies also participate in the permit process. Permits are granted on a case-by-case basis. The long-term consequence of this practice has been the regional "hardening" of the shore. More than 50 percent of King County's shoreline, including Vashon Island, is shrouded behind some type of erosion control works.

The collective impact of these structures on the physical and biological condition of the shore is not fully known. Much has been written and is known about the effects of structures that are prependicular to the shore, but much less information is available regarding the impact of structures parallel to the shore. Prominent coastal geologists and engineers give conflicting arguments and evidence (Dean, 1988; Pilkey, 1988). One point of view is that seawalls compound beach erosion by accelerating littoral currents and preventing sediment exchanges between the beach and uplands. Others argue with equal vigor that there is no evidence to support such claims. These arguments are likely to continue for some time; however, there is little doubt that seawalls do interrupt, if not totally prevent, sediment exchanges between the shore and uplands. The effect of sporadic seawalls along stretches of beach where there are several sources of sediment, such as rivers, cliffs, and dunes, is relatively small.

The beaches of Puget Sound and Georgia Strait, however, tend to be short and segmented into drift cells

Figure 2. Seawalls and riprap protection at homes on a low depositional beach.

(Jacobsen and Schwartz, 1981). Each drift cell usually has just one sediment source, which in most places is an eroding bluff. If this sediment source is blocked by a seawall, the entire cell is deprived of sediment, which leads to the degradation of the beach. The beach profile narrows and steepens as the beach "buffer" is lost (Terich, 1987). This allows waves to attack the shore more easily. Property owners are then forced to install additional erosion control structures as the erosion "domino" progresses along the shore within the drift cell.

The low depositional shoreforms such as spits and bars are particularly vulnerable to beach erosion and flooding. These features require a continuous uninterrupted flow of littoral sediments. When the sediment supply diminishes, the threat of damage by flooding and wave attack significantly increases. Some of the most massive concrete seawalls and piles of riprap are found along these shores. The effects of these structures are not entirely understood. Komar (1977) found where riprap was sporadically placed along the eroding shore of Siletz Spit, Oregon, waves flanked the riprap structure, severely eroding unprotected neighboring properties. More recently, it has been shown that the depth of flanking erosion is proportional to the length of the erosion protection structure (McDougal et al., 1987).

The regional loss of beaches in Puget Sound and Georgia Strait also impacts the general public. Shorefront beaches lost result in expensive public restoration projects. Birch Bay in Whatcom County is an example of this situation. Net sediment transport here flows counterclockwise from south to north (Figure 3). Homes and erosion structures built on the upper beach forced a seaward shifting and narrowing of the foreshore. Groins installed on the beach have interrupted the natural northward flow of the littoral sediment to the public beaches to the north. Over the years, Whatcom County has spent thousands of dollars in coastal flood and erosion control along the northwestern shore of the bay in an attempt to alleviate the problems (Bauer, 1975).

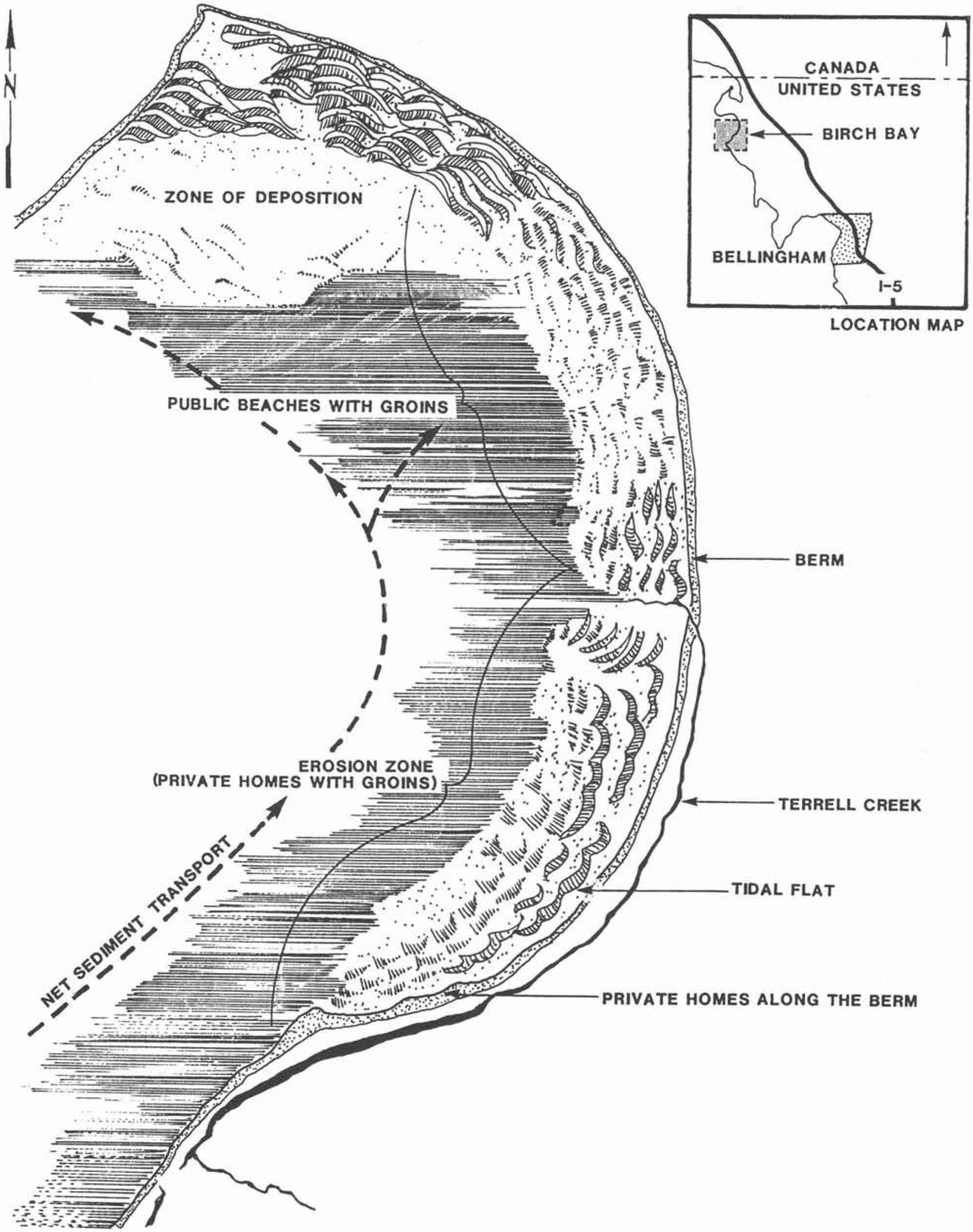

Figure 3. Birch Bay in Whatcom County. Net sediment transport flows from south (bottom) to north (top) in a counterclockwise motion. Homes built on the upper berm along the southeastern part of the shore have "starved" the public beaches to the north of beach sediment.

PLANNING ALTERNATIVES

There are several good reasons for attempting to stabilize the shore. Jetties, seawalls, bulkheads, and riprap may be necessary to maintain channels for navigation safety, port and shipping terminals, and marinas and to provide storm protection in urban areas. However, most shore stabilization in Puget Sound is not for the public good, rather for the protection of private property (Terich, 1987). Some protection structures are necessary. However, the efforts of one property owner to protect shorefront land can have harmful effects downdrift. Some riprap and seawalls are placed along the shore not as a defense to some overwhelming erosion or flooding problem, but in response to the "works" of updrift neighbors. This is particularly true in Puget Sound where the beach segments are limited to very few sediment sources and interruption of sources starves downdrift beaches. Landowners do indeed have a right to protect their land from erosional loss; however, great care must be taken to insure that their actions have minimal impact on shorefront neighbors.

Shoreline development policies should seek some alternatives to the installation of hard erosion protection structures. It may be beneficial to encourage, where possible, a wide development setback from the water's edge (Stevenson, 1986). While this does not solve the erosion or flooding problem, it at least provides some distance from the problem. Where wide setbacks are not possible, or where the erosion is too accelerated, some type of structural protection is necessary.

The granting of permission to build an erosion protection structure could be "coupled" with a provision for beach nourishment. This sediment would help protect the structure while adding to the beach sediment prism for the benefit of neighboring properties. Beach nourishment is useful in conjunction with some type of structural protection (Yanggen, 1981). The one-time supplementation of beach sediment will not solve a long-term beach erosion problem, but it at least provides some remedy. Long-term maintenance would require a regional erosion management plan, which might call for periodic sediment nourishment for the beach.

RESEARCH NEEDS

Shoreline development and management policies in Puget Sound and Georgia Strait are fraught with one common recurrent problem: the lack of adequate oceanographic and geologic data to support engineering design and planning policies. Most data are derived from secondary sources or inferred from studies conducted in similar environments.

Wave data, fundamental to any type of shoreline development decisions, are virtually non-existent for Puget Sound and Georgia Strait. Wave heights and periods have been estimated or hindcast for specific sites and periods, but no reliable recorded data exist. The geographic diversity of the water bodies in the sound and strait precludes economical widespread wave measurements. However, actual recorded wave data measured in the more open waters of Georgia Strait, Admiralty Inlet, and Puget Sound would be extremely valuable to engineers, coastal planners, and researchers.

Some progress has been made in understanding the physical dynamics of beaches in the region; however, many more studies and basic data are needed. Documentation of beach profile changes under varying storm or wave conditions would be very useful. Our knowledge of sediment transport directions and rates has improved (Wallace and Schwartz, 1988), but data on sediment transport rates and volumes are still insufficient.

Finally, there is a dearth of information on relative shorebluff erosion rates throughout the region. Keuler (1988) provided some estimates for the Admiralty Inlet area. His data have been extrapolated to many other sites where shorebluff retreat information is lacking. More studies of this type are needed throughout the region. In addition, laboratory and field studies of the impacts of structures on all types of beaches are needed.

SUMMARY

Washingtonians celebrate the beauty and utility of Puget Sound and Georgia Strait as valuable natural resources. The beaches are important not only as recreational resources, but also as buffers for the uplands from wave attacks and flooding. These same beaches are showing signs of degradation and shrinkage. Much of this loss can be attributed to the sealing off of natural sediment sources by the proliferation of seawalls, bulkheads, and riprap lining the water's edge. Shoreline planning and management alternatives to the installation of structural erosion protection should be encouraged. One beneficial step would be a wide development setback. Where a wide setback is not possible or pre-existing developments are threatened by erosion, structural protection may be necessary. In these instances, the installation of a structure should be "accompanied" by artificial nourishment of the eroding beach. These efforts can help to restore and preserve the beach.

Finally, shoreline planning and management policies could be significantly improved through a better knowledge of fundamental coastal dynamics of the region. Obviously lacking are data on wave conditions, beach dynamics, and shorebluff erosion rates.

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Net Shore-Drift in Puget Sound

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INTRODUCTION

Coastal geomorphologists and engineers have utilized the concepts of drift cells and net shore-drift directions for many years. It is agreed that a drift cell has an origin at a source of sediment supply (a river or cliff-beach-backshore erosion), a central transport zone, and a terminus at a sediment sink or site of deposition. Net shore-drift is the predominant, long-term direction of drift through a cell.

What is not generally agreed upon, however, is how to best determine the direction of net shore-drift. Methodology, and therefore the results, vary considerably among workers in this field.

One school of thought advocates determination of drift direction from wave hindcasting, or the use of sediment tracers or traps. The other school of thought (Hunter et al., 1979; Jacobsen and Schwartz, 1981) relies on long-term geomorphic indicators such as spit development, log-spiral or headland bay beaches, sediment accumulation and erosion at groins and other obstacles, direction of stream diversion, beach width and height, sediment size gradation, uniquely identifiable sediment, nearshore bar development and orientation, and changing bluff or cliff morphology along the coast. This latter group of coastal workers feels that hindcasting is prone to misinterpretation based on the quality of the data and calculations employed and that short-term use of sediment tracers or traps only shows the direction of drift for the period during which the study was conducted. The following examples illustrate the differences in the approach and results of the two schools of thought.

Published wind data for the offshore region along the central coast of Washington indicate predominant winds from the northwest (Jamison, 1974a, 1974b). Wave hindcasting and constructing wave orthogonals, from that base, would lead to a determination of net shore-drift in a southerly direction along most of the Pacific coast of the state. In contrast to this conclusion, however, is a study based on a combination of geomorphic indicators that shows that net shore-drift is to the north along most of the coastal sectors, with only short reversals in the wave shadow of headlands, wave-refraction areas behind islands, or where the coastal orientation changes to east-west from the general north-south trend (Schwartz et al., 1985). This determination is borne out by a similar study by Plopper (1978) and by a U.S. Army Corps of Engineers report (Keislich, 1981) which state that progradation is greater on the south side of two large jetties at an inlet in the central portion of this coast. Terich and Levenseller (1986) also document considerable evidence in favor of northerly net shore-drift.

In a marine atlas, produced under the aegis of the Washington Department of Ecology, net shore-drift along the Cherry Point sector of Whatcom County is depicted as to the north in both winter and summer (Anderson et al., 1974). The direction was determined by hindcasting from wind-roses recorded at five weather stations in Puget Sound and then constructing wave orthogonals to find the drift patterns. Sediment tracers (pebbles) also were transported mainly to the north in this sector during a 2-month-long series of observations made in connection with a proposed graving-facility permit application. And yet, the Sandy Point spit that is the terminal deposition site of the Cherry Point sector drift-cell extends precisely in a southern direction (Jacobsen and Schwartz, 1981). A marina inlet on the west (sound) side of the spit is constantly filled by sediment drift from the north, and the downdrift southern tip of the spit is starved of sediment and eroding.

To apply the concepts advocated here, graduate students at Western Washington University qualitatively determined the directions of net shore-drift and limits of drift cells in Puget Sound and the Strait of Juan de Fuca, using geomorphic indicators. Each student investigated a coastal county, and the findings were reported to the Washington Department of Ecology for use in updating the Coastal Zone Atlas of Washington. These studies were performed by Steven C. Bubnick, Clallam County (1986); Bruce Taggart, Kitsap County (1984); Dana G. Blankenship, Mason County (1983); David M. Hatfield, Thurston County (1983); Brad D. Harp, Pierce County (1983); Michael Chrzastowski, King County (1982) (Figure 1); Ralph Keuler, Skagit County (1979), and Edmund Jacobsen, Whatcom County (1980). The findings from this work were strictly qualitative. The next step in the study of net shore-drift in Puget Sound and the Strait of Juan de Fuca was to determine the quantity of

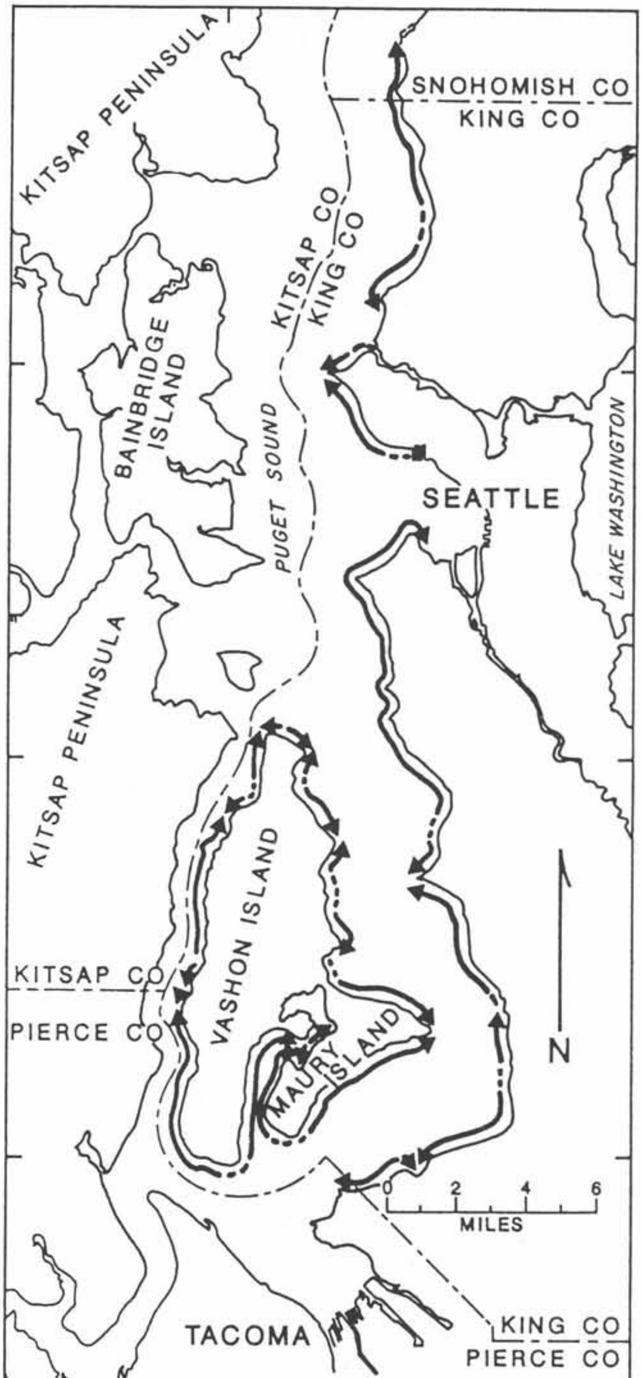

Figure 1. Modified map of King County net shore-drift. After Chrzastowski (1982).

sediment moving at various locations within the study area.

There are several ways to quantify the rate of net shore-drift—the various techniques varying along the same philosophical lines as described above for determining the direction of net shore-drift. We chose the most direct, expedient, and geomorphic means. In our studies (Schwartz and Wallace, 1987; Wallace, 1987) net shore-drift rates were determined for 26 locations within Puget Sound and the Strait of Juan de Fuca (Table 1; Figure 2). Three methods were used to obtain these drift rates: (1) field measurement of sediment accumulation at drift obstructions; (2) extrapolation of spit growth using aerial photographs and historical maps; and (3) evaluation of maintenance-dredging volumes at navigation channels.

METHODOLOGY

Net Shore-Drift Direction

Principles

Our study was founded on some basic underlying principles that have often been overlooked in drift studies. One of these principles was the distinction between prevailing wind and waves and predominant wind and waves. Although prevailing and predominant are usually defined, in general use, as synonyms, they each have a distinct meaning when used in coastal studies.

In coastal studies, prevailing is used to mean the most common wind and wave direction, while predominant is used to mean the winds and waves which have the most effective influence on any particular stretch of coast due to fetch or velocity considerations.

Another important principle employed was that of the drift-cell or drift-sector of sediment transport along coasts. This concept maintains that along rocky and irregular coastlines, sediment transport is compartmentalized, even under high wave-energy conditions, so that movement can usually be measured in terms of kilometers.

The idealized coastal cell or drift-sector is defined as consisting of three different areas. One is an area of erosion, which supplies the sediment necessary for shore-drift. The second is an area of transport where wave energy moves the sediment along until it reaches the end of the cell. The third area is an area of deposition. Deposition occurs whenever wave energy decreases below the level necessary to move a given sediment size. It should be recognized that because of the great variability of drift direction, very few drift-sectors have absolute boundaries marking their beginning and ending. Usually sector boundaries consist of broad, generalized zones. There are, however, exceptions to this.

Another very important distinction was made between net shore-drift and seasonal drift. Wind direction is quite varied, and the direction of shore-drift often coincides with the prevailing wave direction for any season of the year. Some previous shore-drift studies have acknowledged this and have published their conclusions in terms of seasonal shore-drift directions (Anderson et al., 1974). However, knowing the seasonal

Table 1. Quantification Site Location Parameters (after Wallace, 1987); site numbers are keyed to Figure 2
Waves: Direction of predominant wave approach (compass direction)
Fetch: Effective fetch distance (km). Cell: Drift cell length (km)
Rate: Net shore-drift rate (cu m/yr)

Site	Waves	Fetch	Cell	Rate
1. Neah Bay Breakwater	E	4.0	4.0	860
2. Ediz Hook	W	51.5	11.8	9,170
3. Dungeness Spit	W	unlimited	26.0	14,200
4. Travis Spit	NE	30.0	5.6	2,182
5. Port Townsend Marina	SE	8.6	6.0	1,190
6. Pope & Talbot Mill	SW	8.0	26.0	77
7. Kingston Ferry Terminal	SE	12.5	1.2	2,082
8. Twanoh Park Boatramp	SW	6.9	8.1	215
9. Vaughn Bay Spit	S	8.6	3.9	2,013
10. Steamboat Island Spit	SW	5.4	4.0	328
11. Cooper Point Spit	S	10.0	7.7	808
12. Zittel's Marina	N	4.8	0.5	95
13. South Foss Tug Jetty	S	1.5	0.6	81
14. North Foss Tug Jetty	S	1.5	0.3	95
15. Carr Inlet Naval Range	SW	6.4	2.8	634
16. Nearns Point Spit	SW	6.4	0.7	93
17. NW Fox Island Bridge	SW	7.6	1.5	30
18. SE Fox Island Bridge	SE	7.6	2.7	40
19. Des Moines City Marina	SW	17.7	2.9	4,912
20. Keystone Harbor	SW	13.7	9.6	4,550
21. Mariner's Cove	SW	14.3	1.5	213
22. Skyline Marina	SW	48.3	10.0	830
23. Sandy Point Spit	NW	145.0	13.3	2,115
24. Birch Bay Village Marina	W	40.0	2.9	600
25. Semiahmoo Spit	W	40.0	6.5	8,210
26. Point Roberts Marina	SE	48.0	3.3	3,552

shore-drift provides little usable information, as the predominant (net) drift direction is not given in such studies. Net shore-drift then is the direction in which the majority of the sediment moves over a long period of time, in spite of any lesser, seasonal, movement in the opposite direction.

It is also important to realize that a greater amount of sediment may be transported in one extreme 10- or 20-yr event than under the more normal conditions which prevail between such events.

Since shore-drift is so varied with respect to direction, time, place, duration, and amount, the task of determining net shore-drift requires methods which take these variables into consideration. Total reliance on engineering methods such as artificial tracers, sediment traps, and the calculation of wave heights based on wind data, is fraught with hazards. None of these methods takes into account all the variables of drift, especially the possibility that the predominant movement of sediment may be caused by relatively rare, extreme events (Hunter et al., 1979). These methods, when used by themselves, may determine only seasonal drift, not net shore-drift. The more complicated the coastline, the less likely that these methods will accurately predict net shore-drift direction.

In our study, long-term indicators of regional drift, especially morphological indicators, were relied upon most heavily because they take into account the many variables involved in net shore-drift. Concrete physical evidence that can be seen was stressed, rather than abstract calculations which tended to remove the observer from the observed. The natural processes which form coastal landforms respond to all the variables, while the human mind may overlook some of these in its attempt to describe nature in the language of mathematics. These long-term indicators were supported, at times, with more local, short-term indicators of drift. It should be emphasized that no one method is, of and by itself, conclusive for a net shore-drift direction. To be certain that a net shore-drift direction, rather than a seasonal shore-drift direction, is determined, many different indicators must be used. The resulting data for an entire region should fit together like so many bits of a puzzle, with each bit of data reinforcing another, to give a sensible picture of the whole.

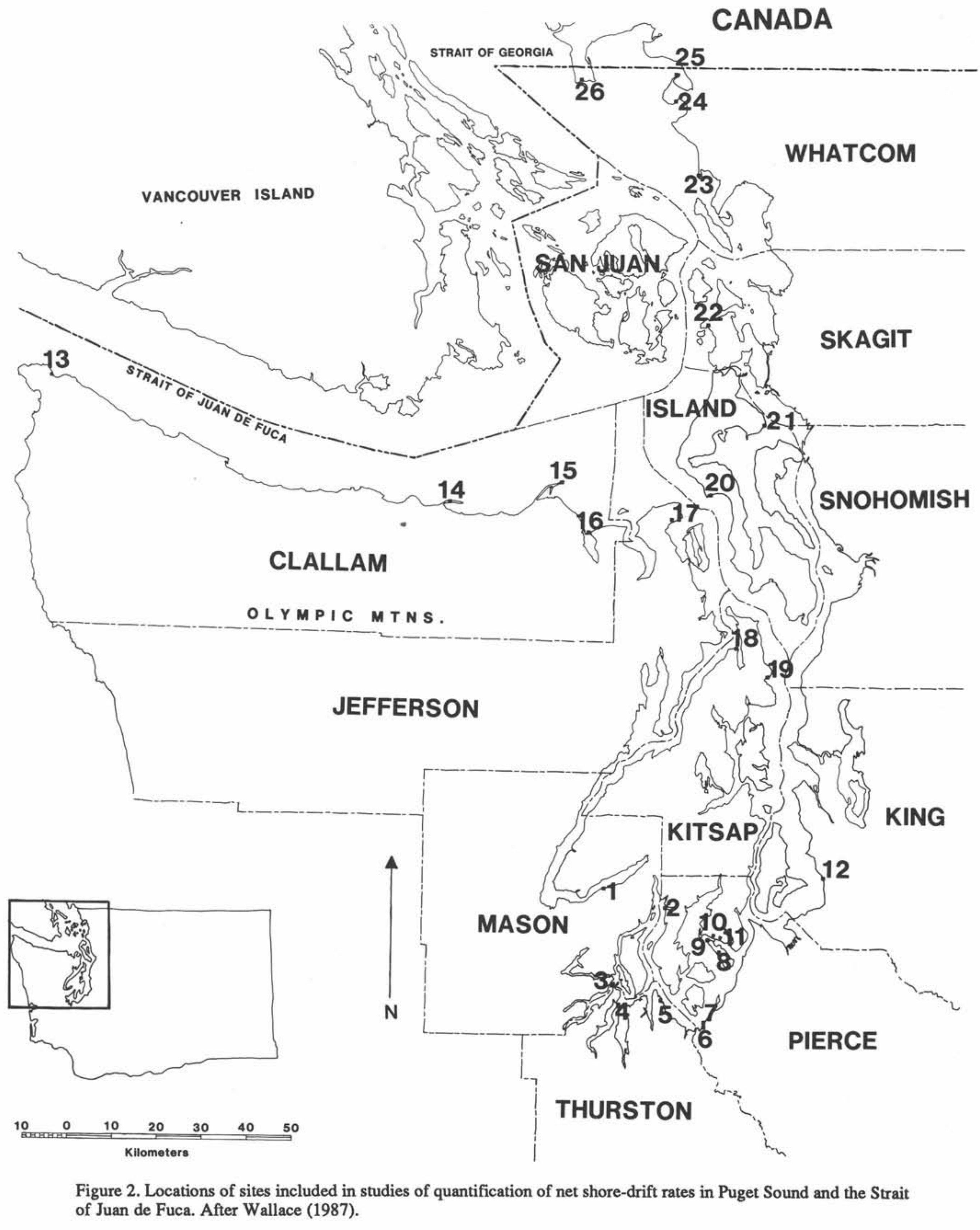

Figure 2. Locations of sites included in studies of quantification of net shore-drift rates in Puget Sound and the Strait of Juan de Fuca. After Wallace (1987).

Fetch

Fetch is a factor in long-term drift which is commonly underemphasized in coastal studies. Fetch is the distance of open water that the wind can blow across without encountering any interfering landmass. Given winds of equal velocity and duration, the greater the fetch, the larger the wave that can be generated. This holds true up to a limiting distance, after which any increase in fetch will not cause a significant increase in wave height.

On intricate coasts, where there are great differences in fetch, fetch can become the most important factor in the generation of those waves which have a significant effect on the coast (Bird, 1984; Engstrom, 1978). A good correlation exists between the greatest fetch and net shore-drift direction, as shown by other indicators.

Wind

Wind data, under ideal conditions, would be a very reliable indicator of drift direction, since the waves that drive shore transport are dependent on the velocity, duration, and direction of the winds. However, wind data in much of the Puget Sound area cannot be totally relied upon. There are several reasons for this. Recording stations are scarce, relative to the intricacy of the coast. The span of time covered in records is relatively small, from a geological standpoint, so that extreme events that may be responsible for the majority of sediment transport could be missing. The placement of instruments near manmade structures or inland from the coast can cause an inaccurate reflection of the velocity and directions of the wind actually affecting the coast. If these drawbacks are taken into consideration, though, wind data can be an acceptable long-term drift indicator when used in conjunction with other morphological indicators, including fetch.

Spit Growth

It is well known by coastal geomorphologists that the accumulation landform known as a spit grows in the direction of net shore-drift (Evans, 1942). Before a spit is visible above the water surface, a submarine platform is built out from the mainland in the direction of net shore-drift. This period of platform building is followed by a period of spit building. A spit, generally smaller than the platform on which it is built, accumulates seaward from the mainland in such manner as to rise up above the surface of the water. As with the platform, a spit is built with sediment supplied by net shore-drift and therefore also grows in the direction of net shore-drift. Platform building and spit building episodes alternate in a random fashion (Meistrell, 1972). Spits form near or at the end of drift sectors and are, perhaps, one of the most reliable long-term indicators of net shore-drift direction.

Log-Spiral Beaches

A log-spiral beach is another good long-term indicator of drift direction. A log-spiral beach, also called a headland bay beach, is defined by Yasso (1965) as "a beach with a seaward concave plan shape that lies in the lee of a headland." The headland forms a wave shadow when the predominant waves approach behind it. In the shadow, wave refraction and, to lesser extent, diffraction and reflection, account for most of the wave action. These bending waves in the shadow zone sweep up the coast from roughly the opposite direction of the predominant waves on the windward side of the headland, with an area near the headland as a general pivot point for the waves. This action causes the sediment on the lee side of the headland to move toward the headland, forming a beach plan that is seaward concave, with the radius of curvature increasing with greater distance from the headland. Sediment in the shadow zone increases in size with greater distance from the headland, with a corresponding increase in beach slope.

The beach plan closely approximates a log-spiral curve which has the equation $n = e^{\cot a}$, where n is the length of a radius vector from the log-spiral center, a is the angle between the radius vector and the tangent to the curve at the intersection of the beach and the radius vector and is constant for a given log-spiral, and e is the base of naperian logarithms. This curve has been tested for fit on several headland bay beaches and found to fit well. The beaches tested included Spiral Beach, Sandy Hook, New Jersey; Halfmoon Bay Beach, and Drakes Beach and Limantour Spit Beach, the last two north of San Francisco, California (Yasso, 1965). Many other similarly shaped beaches are found in North America and other parts of the world, including such diverse places as Australia, Africa, Ceylon (Sri Lanka), Malaya (Malaysia), and England (Davies, 1973). This landform has been used to predict net shore-drift directions through the use of satellite photos of remote parts of the Earth. Birch Bay in Whatcom County is a good example of a log-spiral beach.

Object Interrupting Shore-Drift

If any object large enough and secure enough to impede shore-drift over a long period of time is placed across the beach and nearshore, more or less perpendicular to the shore, accumulation of sediment will occur on the updrift side, while the downdrift side will experience erosion due to sediment starvation (Bird, 1984). An obstacle to drift will cause the beach updrift to widen because of sediment accumulation, and the beach downdrift of the obstacle to narrow. Also, the level of the beach on the updrift side of the obstacle will be raised above the level of the beach on the downdrift side. The larger and the more permanent the obstacle, the more reliable it is as a long-term indicator. For example, a riprap loading platform that rises 7 m above water level and reaches well out into the nearshore zone is a more reliable indicator of net shore-drift than a small concrete groin 30 cm high and 3 m long, which would most certainly indicate only seasonal drift. It must be remembered, however, that a small obstacle will have a relatively greater effect in impeding shore drift on a low-energy coast as compared to its effect on a high-energy coast. At Sunrise Cove on Lummi Island, a groin composed of rocks slightly less than a meter high had been constructed across the foreshore of a beach in a sheltered position. This has caused not only accumulation of sediment on the updrift side of the structure, but it has also initiated significant erosion along bluffs downdrift from it.

Not all obstacles to shore-drift are artificial or constructed. For example, rocky headlands may act as barriers to net shore-drift and impede it as, or more, effectively than manmade structures. Natural obstacles are especially reliable when in a series, forming a stepped coastline (Hunter et al., 1979).

Stream Diversion

Stream diversion offers an easily recognized long-term indicator of net shore-drift. If the conditions are correct, the mouths of streams will be diverted in the direction of net shore-drift (Bird, 1984). As beach and longshore sediment is transported to the updrift bank of the stream faster than the stream can carry it away, some of it gradually accumulates there. It is by this method that some stream mouths have been diverted great distances in the net shore-drift direction. The mouth of Terrell Creek, at Birch Bay, has been diverted 3 km to the northeast by net shore-drift in that direction. There are exceptions to this indicator, however. Some very small stream mouths are not actually diverted in the direction of net shore-drift. The beach sediment merely drifts along at its own rate, while the stream continually adjusts itself to the shortest route to sea level, and runs across the top of the beach sediment. In other cases, these very small streams may even reflect seasonal variation in shore drift. The diversion of larger streams always appears to be in the direction of net shore-drift. This net shore-drift indicator has been found to be useful in other drift studies (Hunter et al., 1979).

Beach Width

Beaches tend to widen, develop one or more berms, and develop a larger backshore in their downdrift direction. This is due to the fact that in most places, one direction of shore-drift predominates over the long term and that accumulation generally prevails at the end of drift cells while at the beginning of drift sectors, erosion is usually occurring.

This was found to be true for many of the beaches in Puget Sound, but the actual widening may be quite gradual or partially obscured by the input of further sediment, to the point where it might not be noticed when walking along the beach. In this case, aerial photographs or actually measuring the beach width will lend greater accuracy.

Sediment-Size Gradation

Sediment size generally decreases in the direction of net shore-drift due to decreasing wave energy (Bird, 1984; Davies, 1973; Self, 1977; Sunamura, 1972). As waves move down a coast, coarser particles, which require more energy than fine particles to transport them, must rely on the less frequent, higher energy waves. Fine sediment can be moved by more frequent, lower energy waves. As a result, fine particles tend to out-run the coarser particles. Typically, a sector that showed this type of particle-size gradation would have a beach at the start of the drift sector composed of predominantly boulders and cobbles with gravel and sand, while downdrift, this would gradually change to a beach of mostly cobbles, with some sand and gravel, and then to a beach of mostly gravel with varying amounts of sand.

This ideal situation, though, can often be obscured by the fresh input of sediment from eroding bluffs or by local reversals in size because of other causes. The sediment gradation may appear to reverse in sections of beaches because of changes in wind direction. Although this usually occurs locally, large lengths of beach were seen to reverse temporarily in gradation after extreme storms. These apparent "reversals", in most instances, occurred when a layer of sand or gravel was washed over coarser materials. It usually was quickly removed and could only pose a problem in determining shore-drift if insufficient field studies are done. In Whatcom County, there were also other noted exceptions to the usual fining of sediment in the direction of net shore-drift. If the terminal end of a drift sector is much more exposed to high-energy waves than the beach updrift from it, or has been sediment-starved due to the intervention of man, such as at Sandy Point, the fine materials will be transported away, leaving behind the coarser sediment which is more resistant to erosion and transportation.

Another rare exception to the pattern of particle transport was found on the northwest corner of Lummi Island, where waves from predominant winds moved all sediment sizes south, and waves from the prevailing winds moved the finer sediments north.

As mentioned, these exceptions are unusual; if appropriate caution is used, sediment-size gradation can be a good long-term indicator of net shore-drift. A good example of the usual size diminution in the drift direction can be found on the mainland near Lummi Island, where the net shore-drift direction is to the south from Cherry Point down to Neptune Beach. Another case in point is the log-spiral beach at Birch Bay, where the sediment size decreases going from Point Whitehorn on the south to Birch Bay Village on the north.

Bluff Morphology

This concept (Keuler, 1979) was of great use in Puget Sound. Coastal bluffs at the beginning of the drift sector generally show evidence of strong erosion and commonly of landsliding. Generally, the slopes are nearly vertical and completely bare of vegetation. The beach under the bluff is narrow enough to at least allow storm waves to erode the bottom of the bluff. Through the drift sector, in the direction of net shore-drift, the bluffs gradually become more vegetated and their slopes less steep. Finally, at the end of the sector, the slopes take on the more gentle appearance of subaerially weathered terrain. Here, the gentle bluffs are well-vegetated, and a broad accumulation-beach is generally found at their bases. As with all net shore-drift indicators, there are complications with application. For instance, an intervening eroding headland may start the entire bluff morphology sequence over again, although it may only be midway through a drift sector.

Identifiable Sediment

When the source of readily identifiable beach sediment can be found, it is possible to use the direction that the sediment has drifted as an indicator of net shore-drift direction (Bird, 1984). Whether the identifiable sediment is natural, as in the case of an unusual rock type, or artificial, such as bricks from a dumping site, the sediment should move in the direction of net shore-drift and be found downdrift of its source. The longer the sediment has been drifted, the better net shore-drift indicator it is. For example, in some drift studies artificially colored sediment is added to beaches and its movement traced for a few days or weeks. Obviously, if the sediment has been drifting for years or centuries, it would be more likely that the net shore-drift direction was being observed, rather than a brief seasonal variation in shore-drift direction.

Nearshore Bars

Under some conditions, large sand bars are built by wave action so that they angle away from the shore. These nearshore bars can be valuable aids in determining the direction of net shore-drift. However, considerable care is required because of the great variety of bar forms. For further details on nearshore bar morphology the interested reader is referred to two fine reports on the subject (Greenwood, 1982; Greenwood and Davidson-Arnot, 1979).

Quantification of Drift Rates

Sediment Accumulation at Drift Obstructions

In order to measure the volume of sediment accumulated updrift of an obstruction (jetty, pier, marina), it was necessary that the obstruction be oriented essentially perpendicular to the coastline, and to assume it to be a total barrier to the alongshore transport of sediment. There is undoubtedly a small amount of sediment that moves into deep water off the seaward tip of an obstruction, but it was not possible in our study to quantify this amount. The accumulated, updrift, sedimentary prism was measured as follows: (A) from the seaward tip of the obstruction to the back of the pre-accretionary beach; and (B) perpendicular to this, from the obstruction, updrift, to the far end of the prograding sediment wedge. The thickness of the wedge (C) was determined by surveying the elevation difference between the lower foreshore level and the landward upper edge of the prograded beach. The beach face was treated as a planar surface in this profile.

The approximate volume of the sediment wedge was calculated by: (A)x(B)x(C)x(.25). The .25 reduction factor is used to approximate the length and width "wedge" shape of the accumulated sediment within the "boxed" volume of length, width, and height. The volume divided by the age of the obstruction equals the accumulation rate (that is, net shore-drift rate), in cubic meters per year.

Spit Growth

Using aerial photographs (scale: 1 in. = 200 ft) taken in 1970 by the Seattle District, U.S. Army Corps of Engineers, spits (Travis Spit, Cooper Point Spit, Sand Point Spit, Semiahmoo Spit, and Nearns Point Spit, among others) were examined, and individual areas of the spit and platform were calculated using a digitizing board. Nautical charts and topographic maps with well-defined features and bathymetric contours were used in conjunction with the aerial photographs to obtain elevations for spits and platforms. Field surveys confirmed subaerial spit elevations above mean high water. From these data spit volumes were calculated. Aerial photographs, of the same scale, taken in 1985 or 1986 were analyzed in the same manner to determine changes in the total volume of each spit. From the difference in volumes and the time interval between photographs, an average volume-per-year rate was calculated.

Dredging Volumes

Maintenance-dredging records are good sources of information for determining net shore-drift rates at navigation channels. These channels generally have a specific depth, length, and width that must be maintained for safe navigation. Records of dates and amount of sediment removed can be compiled to calculate net shore-drift rates.

At the entrance channel to the marina at Birch Bay Village, in Whatcom County, this method was used in conjunction with sediment accumulation updrift of an obstruction, where sediment bypassed the west jetty and subsequent dredging of the channel became necessary (combined volume, 600 cu m/yr).

Fetch Distance

Schou (1952) found that in protected coastal areas, the direction of net shore-drift is most often determined by the direction of maximum fetch. The waterways of southern Puget Sound (Figure 2) are oriented in a general southwest-northeast trend. However, the coastline of the southern region is very irregular, with many headlands, coves, bays, and islands. Fetch distances and directions are varied because of these irregularities. Prevailing air masses move northward through the Chehalis River valley into the southern region (Blankenship, 1983).

Fetch distances observed in the southern region of Puget Sound were, with the exception of the Des Moines City Marina, all less than 10 km. The mean annual net shore-drift rate for 11 sites (sites 1-11 in Figure 2) in the southern region, excluding Des Moines, is 400 cu m/yr. The relatively small drift rates are related to shorter fetch distances.

In contrast to southern Puget Sound, the central region waterways are generally oriented east-west. The main body of water is the Strait of Juan de Fuca, through which wind and waves approach from the west.

The central region is subdivided into west-central and east-central regions. The west-central region includes four sites (13-16 in Figure 2) along the southern coast of the Strait of Juan de Fuca. Drift rates at Ediz Hook and Dungeness Spit, 9,000 and 14,000 cu m/yr, respectively, represent the largest rates in the study area. Both sites are within drift cells having fetch distances in excess of 50 km, among the largest in the Puget Sound area.

The east-central region includes six sites (17-22 in Figure 2) located east of the Strait of Juan de Fuca. Four have fetch distances in excess of 12.5 km. The mean annual net shore-drift rate for this region is 1,500 cu m/yr.

The northern region is dominated by air masses moving in from the west and northwest through the Strait of Georgia. The four sites (23-26 in Figure 2) in this region all have fetch distances greater than 40 km. Consistent with relatively long fetches, these sites have a relatively large mean annual net shore-drift rate of 3,500 cu m/yr.

Regression analysis also supports the fetch distance-rate relation. If there is no relation between fetch distance and drift rate, computation using regression equations of a critical value (r), with a confidence level of 95 percent, should have a value of 0.33 or less. Data on fetch distance was used to arrive at a value for (r). The critical (r) value was 0.69; thus the null hypothesis was rejected and an increase fetch distance-increase drift rate relation was inferred.

Available Sediment

The rate of net shore-drift is intimately related to the amount of sediment made available to the littoral zone. The marine coasts of Puget Sound and the Strait of Juan de Fuca have a preponderance of easily eroded glacial deposits. However, human intervention along the shore has interfered with the natural supply of sediment from the bluffs to the beaches. Evaluation of available sediment requires a close look at coastal stratigraphic units, with special consideration for the number and type of shore defense structures in a given area.

Shore defense structures are becoming a part of nearly every coastal property owner's landscape in Puget Sound. As development continues, natural sediment supplies are restricted or eliminated. As a result, net shore-drift rates are drastically reduced. In the southern region of Puget Sound, along the northwest shore of Hale Passage, the smallest net shore-drift rates in the study were recorded (30 and 50 cu m/yr). There are easily erodible, 5- to 8-m-high bluffs fronting the shore (Harp, 1983) and fetch distances of 7.5 km. Similar conditions exist at Vaughn Bay Spit, with regard to fetch distance, cell length, and bluff morphology; however, the annual drift rate is two orders of magnitude greater than those along Fox Island. The difference lies in the extent of shore defense structures. While the coast updrift of Vaughn Bay Spit has relatively few bulkheads and groins, the shore along a 4-km stretch on either side of the Fox Island bridge is completely bulkheaded to protect waterfront property.

Drift Cell Length

The length of a drift cell plays a role in the observed rate of net shore-drift. Although each cell is constrained by its own geographic orientation with regard to wind, waves, and sediment supply, mean drift rates show an overall increase with increased drift cell length. This is most evident in less developed areas where there is simply more shore exposed to wave action in longer drift cells and, therefore, the potential for erosion and transport is greater.

Data on cell length was also analyzed using regression. If the null hypothesis is assumed to be correct (no relation between cell length and drift rate) the critical value (r) should once again be less than 0.33, to be within a 95 percent confidence interval for the data set. Analysis yielded an (r) value of 0.53. Although not as high as the critical value for fetch distance, this, nonetheless, appears to indicate that there is a relation between the length of a drift cell and the rate of net shore-drift.

Drift cells ranged in length from 0.3 to 26.0 km. Four groups were delineated, based on drift cell length: Group 1, ≤2 km; Group 2, ≤4 km; Group 3, ≤10 km; and Group 4, >10 km. Of the 26 sites, seven were within drift cells of 2 km or less. The mean drift rate in cells of this length was 400 cu m/yr. Within drift cells 2 to 4 km long, there were eight sites, these had a mean drift rate of 1,500 cu m/yr. Seven sites were within drift cells 4 to10 km long, and the mean rate for these sites was 3,000 cu m/yr. In drift cells exceeding 10 km there were four sites; these show a mean drift rate of 6,000 cu m/yr. These data support the trend of increased drift rates associated with increased drift cell length, as was suggested by the critical value test.

SUMMARY

Some generalized statements about the direction of net shore-drift in Puget Sound and the surrounding region can be made. For the most part, the shores are aligned in a north-south orientation, winds are mainly northerly and southerly, and the predominance of waves depends mainly on the fetch distance for a particular drift cell. It might be said that in the southern sector net shore-drift is mostly to the north, in the northern sector mostly to the south, and mostly to the east along the Strait of Juan de Fuca shore. These are the generalities; however, a perusal of the collected data on this subject will reveal many exceptions to the rule. For specific details on a county-by-county basis, the interested reader is referred to the following Washington Department of Ecology publications: Whatcom County (Jacobsen and Schwartz, 1981), Mason County (Schwartz and Blankenship, 1984), King County (Schwartz and Chrzastowski, 1984), Pierce County (Schwartz and Harp, 1984), Thurston County (Schwartz and Hatfield, 1984), Kitsap County (Schwartz and Taggart, 1984); Clallam County (Schwartz and Bubnick, 1985).

As with net shore-drift direction, net shore-drift rates in Puget Sound and the Strait of Juan de Fuca vary considerably. The range in rates is due to interaction of the following: fetch distance, available sediment, and drift cell length.

Relatively large segments of undefended coastline, under the combined influence of westerly waves, large fetch distances, and relatively long drift cells, exhibit transport of the largest volumes of sediment recorded in this study—eastward, along the south coast of the Strait of Juan de Fuca (west-central region). Northern Puget Sound exhibits the second highest mean annual transport volume. This is due in part to fetch distances in excess of 40 km, combined with predominant, northwesterly wind and waves entering northern Puget Sound through the Strait of Georgia. The east-central region is not directly subjected to westerly waves like those sites along the south coast of the Strait of Juan de Fuca. However, the mean fetch distance for sites in this area is 17.6 km, and the overall drift rates are an order of magnitude greater than those in southern Puget Sound. Southern Puget Sound, with waterways oriented along a southwest-northeast trend, is affected primarily by southerly waves and relatively small fetches to the south, southwest, and southeast. Smaller volumes of sediment are transported in southern Puget Sound due mainly to limited fetch exposures arising from the many islands, bays, and headlands in the region.

Due to contrasting shore-drift rates, development in the southern portion of Puget Sound is less likely to cause a major interruption of sediment transport than development in the two central and the northern regions.

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Geologic Aspects of Jetties, Breakwaters, and Boat Basins in Western Washington

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DEFINITIONS AND SITING

Breakwaters and jetties are the most common major coastal structures. They are constructed to facilitate navigation or to protect marine docking facilities. The terms, however, are often confused. A jetty is a structure whose principal axis is more or less normal to the general trend of the shoreline, its purpose being to control accumulation of shoals at a harbor or inlet entrance or river mouth. Breakwaters are structures whose principal axis is more or less parallel to the shoreline, although closure sections can be normal to the shoreline. Its name describes its function: to still the wave action and provide protection for a port mooring area. Both types of structures attempt to control coastal geologic process at the sea-land interface. Some are remarkably successful, others fail or require extraordinary maintenance. This paper considers the breakwaters and jetties of the Washington coastal areas and the influence of the underlying geology and prevailing coastal processes in various areas on the siting, design, and relative success of such structures.

One should keep in mind that many older structures have been sited for political and/or hydraulic reasons, in many instances with little attention paid to geologic processes. For the major jetties at the mouth of the Columbia River and at Grays Harbor, it has been a case of experimenting with the prototype. For some breakwaters, existing port locations dictate breakwater positioning, and some of those breakwaters have less than adequate foundations that have required expensive designs and maintenance. Ideally, a breakwater-boat basin combination consists of a rubble mound, pile or pile and whale structure located on a reasonably firm and stable sea floor in fairly shallow water. The boat basin is excavated as necessary, and the excavated material is used for fill on land upon which port-related facilities are constructed. However, for the most part, ports are sited for reasons other than geology, and the engineer and geologist must work with what is there. The availability of deep-water protected harbors on Puget Sound and other inland waters has minimized the requirement for large breakwaters and jetties, a happy situation that is a direct result of Pleistocene glaciation and, in some places, postglacial littoral drift. Major breakwaters are listed on Table 1.

The terms mean lower low water (MLLW), which is the usual zero elevation reference used in coastal engineering, and mean higher high water (MHHW) are used throughout this paper without further explanation. The reader should be aware that these elevations relative to mean sea level (MSL) vary from place to place along coastal waters.

SELECTION OF TYPE OF STRUCTURE

A great variety in design of breakwaters and jetties is found in the Washington coastal zone. However, these structures fall into three general categories: rubblemound, timber pile, and floating. In the rubblemound category (rubble being quarried rock) is an infinite variety of construction, from an unzoned structure graded with stone of a few tens of pounds to 10 tons or more, to a structure having a sand and gravel core flanked by several zones of protective stone in which rock size increases outward. There is also an infinite variety of timber pile structures. Some are reinforced with quarried rock; others are tied together with whales (horizontal timbers). Floating breakwaters may be constructed of various materials: timbers, foam-filled tires, or concrete.

Rubblemound structures require a firm foundation or an extensive gravel and rock mat if any longevity of the structure is to be expected. Such structures occupy a greater space and are usually more expensive to build than are timber pile structures, but they have greater longevity when properly designed for the expected wave climate and constructed of adequate quality and sized rock. Floating breakwaters require concrete block or pile anchors; thus bottom conditions and depth continue to be important. The selection of breakwater type is an engineering decision ideally based on proper assessment of wave climate, hydrographic conditions, and geologic conditions. Geologists are therefore involved in three areas of jetty-breakwater construction: foundations, including determination of relative ease of pile installation; rock quality characteristics; and long-term influence of these structures on adjacent areas.

Table 1. Major western Washington breakwaters. Type: RM, rubblemound; P, pile; FC, floating concrete; RRP, rubble-reinforced pile

Location	Type	Length (ft)	Crest elev. (ft, MLLW)	Year constructed/ rehabilitated	Foundation conditions
Anacortes (Cap Sante)	RM/P	500/2 @470	14	1958, 64	bay mud, glaciomarine drift
Anacortes (marina)	P/FC	3,000/2 @100,75	16	1980	glaciomarine drift, bay mud
Bellingham (Squalicum)	RM[1]	3,900/1,500	15/17	1959/80	soft bay mud, deltaic marine sand
Blaine	RM	2,500	15	1957	marine sand, glaciomarine drift
Blake Island	RM	720	18	1975	marine sand
Des Moines	RM	2,750	20	1969	marine sand, gravel/glacial till
East Bay (Olympia)	FC	700	-	1983	bay mud
Edmonds	RM	1,850/1,000	18.5	1962/68, 85	marine sand and gravel
Everett	RM	12,550	14	1911, 63, 74	very soft deltaic clay and silt
Friday Harbor	FC	1,600	--	1984	bay mud
Kingston	RM	1,040	19	1967	marine sand and gravel
Nahcotta (Willapa Bay)	RM	1,500	15	1958	loose bay sand, dense marine sand
Neah Bay	RM	8,000	18	1944, 56, 80	marine gravel, sand, bedrock
Point Roberts	RM	600, 2@300[2]	23/20	1978	marine gravelly sand
Port Angeles	RRP	1,026	17	1959	marine sand, gravel, wood debris
Port Orchard	FC	2,350/450	--	1974	soft bay mud/glacial till
Port Townsend	RM	2,600	18	1964	marine sand
Quillayute (La Push)	RRP	1,020/260	9	1960	estuarine gravel and sand
Semiahmoo	FC	3,300	--	1981	soft bay mud, marine sand, gravel, very soft, deltaic sandy silt
Sequim Bay (John Wayne)	RM	1,800	16	1985	marine sand
Shilshole Bay (Seattle)	RM	4,440	20	1958/61	marine sand and gravel
Tacoma (yacht basin)	slag rubble	2,500	20±	N/A	unknown, probably marine sand and gravel
Tokeland (Willapa Bay)	RM	1,250	15±	1958	marine sand
Westhaven Cove (Westport)	RRP/RM	1,020/750/2,400	17	1950/65/79	marine sand

[1] Rubblemound section of breakwater extension rests on torpedo net at elevation +2 ft, which rests on fine rock fill over a gravel foundation pad.

[2] Protective structures consist of two jetties between MLLW and MHHW and an offshore breakwater.

JETTIES

General

Major jetties have been constructed and maintained at the mouth of the Columbia River and entrance to Grays Harbor during most of the 20th century. Smaller structures have been built at the mouth of the Quillayute River. There are few jetties on the inland waters, however. The abundance of natural deep-water harbors, the large number and more limited extent of littoral drift cells, and the lower energy wave climate all contribute to this paucity. Short jetties have been constructed at the mouth of Keystone Harbor, at Point Roberts boat basin (discussed later), at the south end of the Port Townsend Canal, and at the Swinomish Channel. Most other jetties are very short, privately owned features flanking small, private boat basin entrances. Some of the latter do not appear to have been well-engineered.

The effect of the ocean jetties is of greater significance because of the greater dynamics and greater littoral exchange along the ocean coast. These will be considered in greater detail.

Columbia River Jetties

Although only the North Jetty at the Columbia River mouth lies in Washington, it is not possible to logically discuss the area without consideration of the South Jetty, which is on the Oregon side of the river. Changes in the mouth of the Columbia have been documented for nearly 200 yr (Figure 1). The mouth of the river lies between Point Adams, a low sand spit on the Oregon side, and Cape Disappointment, a bedrock headland including North Head on the Washington side. Prior to jetty construction, numerous changeable shoals known as Clatsop Spit extended northwest from Point Adams, and equally changeable shoals known as Peacock Spit extended southwest from Cape Disappointment (Hickson and Rodolf, 1950) (Figures 1A and 1B). The latter spit frequently merged with a convex-seaward ocean bar known as Middle Sands (Figure 1C). The position of the entrance channel shifted from north to central to south, and though such shifting is expected to have been cyclic, documentation prior to jetty construction is not adequate to prove the cyclic character or determine the cycle period (Galster, 1987). These highly mobile shoals, whose motion was a function of seasonal littoral drift changes, ocean storm activity, and detritus moving down the Columbia River, were great hazards to navigation.

Construction began on the South Jetty in 1885 and was completed in 1889. It was a rubblestone structure extending on a convex-northward alignment 4.5 mi WNW from Point Adams across the shoals of Clatsop Spit (Hickson and Rodolf, 1950; Kidby and Oliver, 1965) (Figure 1D). The stone was dump placed from a wooden trestle to elevations of +12 ft at the landward end to +4 ft at the seaward end. Four groins were placed on the north side, but no foundation matting was provided. Jetty construction immediately resulted in the channel's deepening and shifting to the central part of the river mouth, a shift which continued northward. Between 1903 and 1913 the South Jetty was extended 2.5 mi seaward (Figure 1F).

Between 1913 and 1917 the North Jetty was constructed. Initially it was about 2.3 mi long, extending southwest from Cape Disappointment across the shoals of Peacock Spit (Figure 1G). This was also a rubblestone structure constructed to elevations of +26 to +32 ft (MLLW) by dumping from a wooden trestle and with the rock resting directly on the ocean floor. With completion of the two jetties, the width of the mouth of the river was effectively reduced from 6 mi to 2 mi, maintaining an entrance channel generally in excess of elevation -40 ft .

By 1931, however, both jetties had seriously deteriorated. The outer 2.75 mi of the South Jetty had been reduced to a 200-ft-wide rock reef at low water, and the outer portion of the North Jetty had suffered a similar fate. Reconstruction of the South Jetty to about elevation 26 ft was completed in 1936, although the outer 300 ft ravelled back during a following normal winter. The outer end was impregnated by a hot mixture of sand and asphalt, which completely filled the voids to low water. The treatment was ineffective, and a concrete terminal block was constructed 3,900 ft landward of the end of the original (extended) structure. Stone size was increased to an average of 10 tons with a maximum of 25 tons (Hickson and Rodolf, 1950). Further rehabilitation was required in the mid-1960s and again in 1982. To date, an estimated total of 8.7 million tons of stone have been used to construct and maintain the South Jetty (Galster, 1987), the outer 3,900 ft of which now lies about elevation -10 ft. In addition to destruction of the outer part of the North Jetty, some undermining had taken place by erosion on the estuary side. The jetty was rebuilt, except for the outer end, in 1939-1940 and again in 1965. An estimated total of 3.3 million tons of stone has been utilized in construction and maintenance of the North Jetty. Jetty A and four groins were constructed between 1932 and 1938 to arrest further northward migration of the channel into Baker Bay and further undermining of the North Jetty (Figure 1H).

The extraordinary efforts to construct and maintain these structures is a result of several factors. The normal sea conditions make placement of foundation mats on the shifting sand foundation nearly impossible; however, the destruction of old jetties through settlement and rearrangement of stone by storm waves has provided a better foundation for the reconstructed jetties. The movement and placement of larger (15-25 ton) stone also had to await development of more powerful construction equipment. Thus the reconstructed jetties are built of larger, better placed stone and rest on a more durable foundation.

The effect of the jetties, in addition to providing a stable, essentially dredge-free channel, has been far-reaching. To the south, Clatsop Spit and a nominal 3,000-ft width of beach has accreted to the Clatsop Plains since construction of the South Jetty due to the semi-annual littoral drift reversal (Kidby and Oliver, 1965; Galster, 1987). On the Washington side the most important effect was accretion between the North Jetty and North Head, rapidly brought about by southward summer littoral drift action, subsequently augmented by wind action, yielding a land surface 12 to 17 ft (and locally higher) above low water. Fort Canby State Park now occupies this land, and the North Jetty is partly drowned in eolian sand. The configuration and positioning of the North Jetty did nothing to reduce seaward accretion along the Long Beach Peninsula, which continues to grow at a nominal rate of 20 ft/yr (Phipps and Smith, 1978). The seaward bar across the Columbia River's mouth rapidly migrated about 1 mi west of its 1885 position with initial construction of the South Jetty, and by 1950 it was 2 mi west of its pre-jetty position and in deep water. The rate of the bar's advance

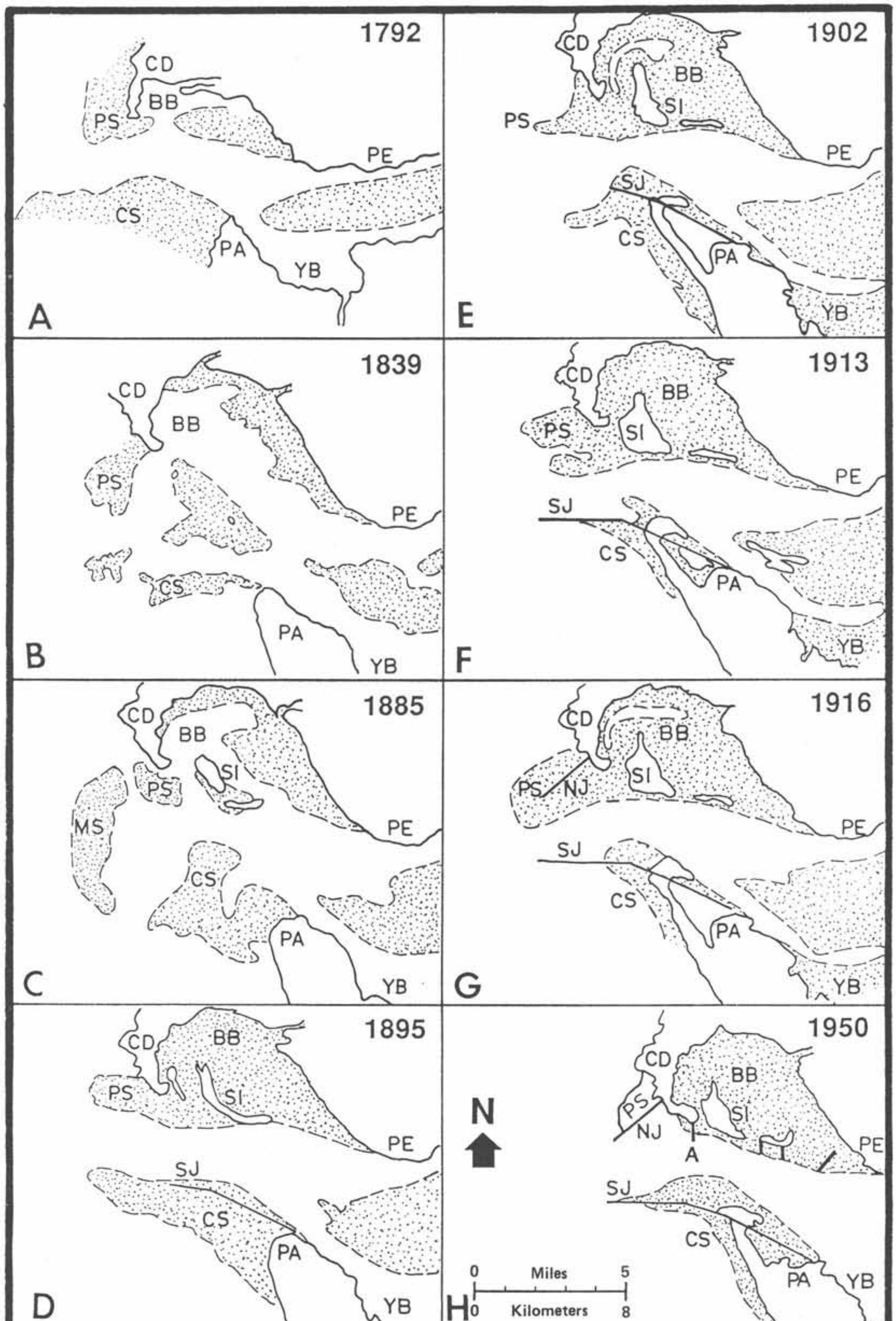

Figure 1. Columbia River entrance changes, 1792-1950. Configuration for 1792 based on a survey by Vancouver; for 1839 based on a survey by Belcher; the others based on U.S. Army Corps of Engineers surveys (after Hickson and Rodolf, 1950). A, Jetty A; BB, Baker Bay; CD, Cape Disappointment; CS, Clatsop Spit; MS, Middle Sands; NJ, North Jetty; PA, Point Adams; PE, Point Ellice; PS, Peacock Spit; SI, Sand Island; SJ, South Jetty; YB, Youngs Bay. Dashed lines represent shoals, solid lines the approximate position of mean high water.

into deeper water would be expected to slow (Hickson and Rodolf, 1950).

Grays Harbor Jetties

No single project has influenced the configuration of the Washington coastline as greatly as the jetty couplet at the entrance to Grays Harbor. The harbor entrance lies between Point Chehalis on the south and Point Brown on the north, and its condition has been recorded for nearly 200 yr (Figure 2). The predominantly northward littoral drift moves an estimated 2.5 million to 5 million cy of material through this area annually (U.S. Army Corps of Engineers, 1982).

Unlike the Columbia River estuary, Grays Harbor is a major sediment "sink". The general sediment regime in the harbor (Scheidegger and Phipps, 1976) and the shore forms responding to the overall dominance of flood tides confirm that the embayment is a sink. The embayment, flanked on the ocean side by extensive spits composed primarily of sand originating from the Columbia River, retains not only detritus from its tributaries, but also that moving northward along the coast.

Prior to jetty construction, the coastline trended about N15°W with no apparent offset between the subaerial spits on either side. A broad, submarine sand spit extended NW to NNW from Point Chehalis, and an even broader submarine spit extended south from Point Brown, which curved tightly into North Bay at the time of the Wilkes Expedition in 1841 (Figure 2B). The two low-water spits tended to constrict the entrance channel. The magnitude and extent of the constriction probably varied greatly, depending on time of year and recentness of heavy storms. The deepest part of the channel favored the Point Chehalis side (and it still does) in response to ebb currents generated by the larger volume of water moving out of North Bay (U.S. Army Corps of Engineers, 1982). This channel shift was probably enhanced by the constant movement of detritus by southwest wave action across the entrance channel nourishing the shallows south of Point Brown. Between 1841 and 1862 the Point Brown shallows had accumulated sufficiently to be shown as subaerial islands (Figures 2B and 2C). The great mobility of littoral detritus along the coastline is evident from these and subsequent (post-jetty) changes.

The South Jetty was constructed between 1889 and 1902; it had a length of 13,784 ft and a crest elevation of +8 ft and was an unzoned rubblemound structure founded directly on mobile marine sands. It was repaired several times during initial construction. Its effect on the coastline was rapid: accretion south of the jetty, diversion of littoral detritus around its seaward end, and continuing detrital movement across the channel to the Point Brown shallows (Figure 2E). By 1935, the jetty had subsided, probably by winnowing, to elevation +6 ft at the landward end and to -10 ft at the ocean end (U.S. Army Corps of Engineers, 1982). It was reconstructed for its full length between 1935 and 1939, again followed by destruction and settlement of a major part of the structure. By the late 1940s accretion south of the jetty had drowned the innermost 4,100 ft in sand. In 1966 the 4,000 ft of jetty oceanward of the resulting changed shoreline was rebuilt to +18 ft using larger stone and zoned construction, and the outermost portion of the structure was allowed to deteriorate; it now lies at elevations between -5 ft and -20 ft.

The North Jetty was originally constructed between 1907 and 1913 to a length of 17,200 ft and a crest elevation of +5 ft. An unzoned rubblemound structure, it was rebuilt to +8 ft in 1916. Accretion north of the jetty produced a 2-mi seaward offset in Point Brown by 1939 and development of the highly migratory, bayward-trending, subaerial Damon Spit on the southeast side of the structure (Figure 2F). This resulted in the detrital drowning of the harborward 9,000 ft of the jetty. Deterioration of the seaward portion of the jetty by 1939 permitted rapid erosion of North Beach, and the outer 8,200 ft was rebuilt in 1941-1942 to elevation +20 ft. Further rehabilitation was done to the outer 6,000 ft in 1975 using larger and better keyed stone (U.S. Army Corps of Engineers, 1982; Galster, 1987).

The jetty couplet has served to stabilize the position of the entrance to Grays Harbor and has created beaches for recreational use behind both jetties owing to the semi-annual littoral drift reversal. A direct correlation of the condition of the North Jetty, erosion along North Beach, and deposition and maintenance dredging in the navigation channels within the outer harbor has been shown (U.S. Army Corps of Engineers, 1982). The outer bar has deepened from 15 to 35 ft and, in the process of deepening, migrated seaward about 5,000 ft. Submarine erosion/accretion studies show continued littoral transport around the jetties and continued movement of material into the harbor, although the jetties appear to have significantly reduced the volume. Within the harbor entrance, accretion continues along the north side in response to the predominant southwest wave climate and predominant flood tidal currents. Scour continues along the south side close to the South Jetty and Point Chehalis in response to the strong ebb currents moving the predominant volume of tidal water from North Bay. The scour has been partly mitigated by construction of six rubblemound groins at Point Chehalis in the 1950s and 1960s and disposal of dredged material inside the South Jetty. Between 1976 and 1982, more than 9 million cy of dredged material was deposited without any appreciable build-up of the bay bottom (U.S. Army Corps of Engineers, 1982). However, deposition of dredged spoils in this area has permitted movement of material into Westhaven Cove on the east margin of Point Chehalis, increasing the need for dredging in this area.

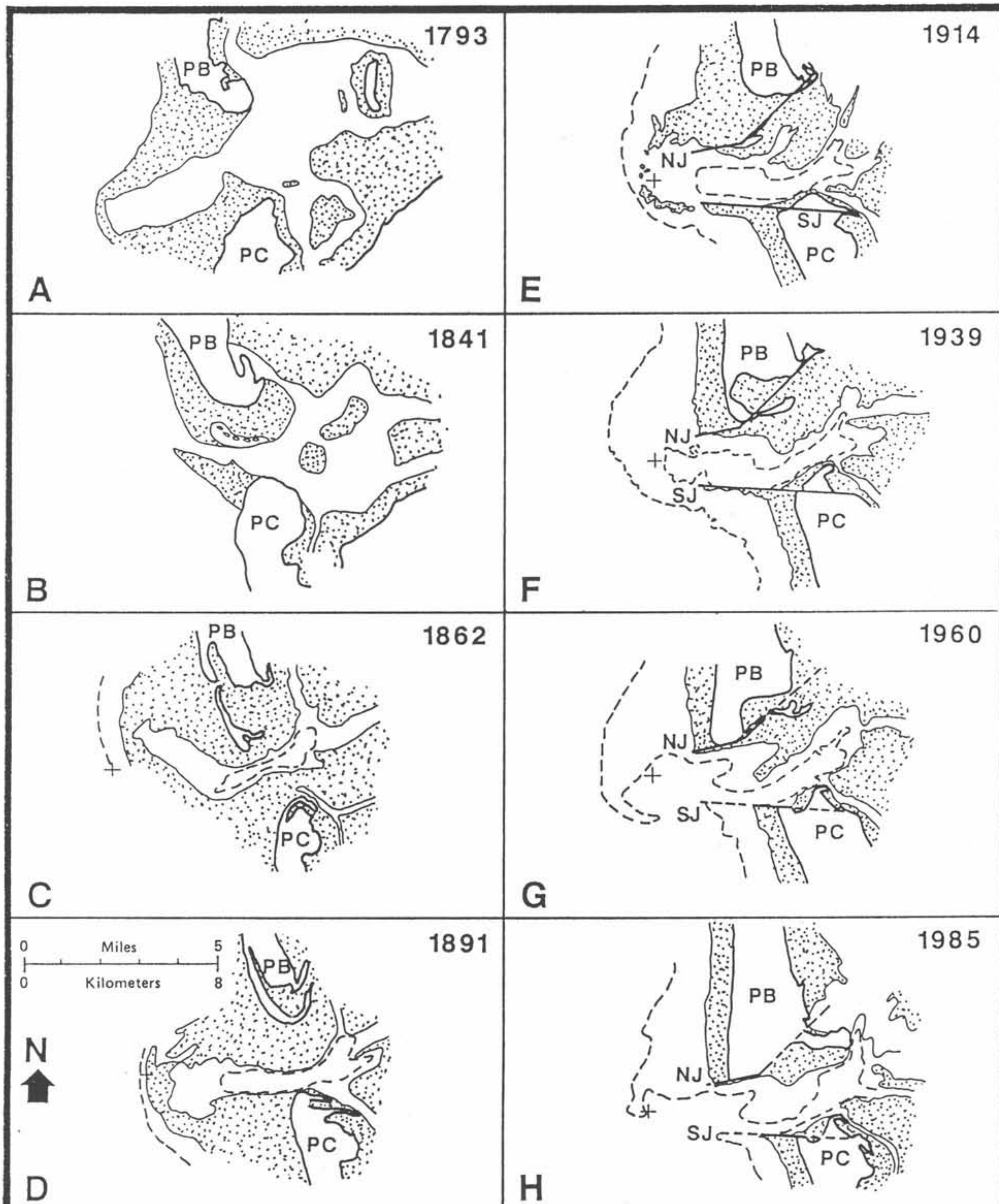

Figure 2. Grays Harbor entrance changes, 1793-1985. Configurations for 1793 are based on the Vancouver survey; for 1841 based on U.S. Exploring Expedition (Wilkes) survey; those for 1862 and 1891 are based on U.S. Coast and Geodetic Survey data. Those for 1914, 1939, and 1960 are based on U.S. Army Corps of Engineers surveys; for 1985 based on a National Oceanic and Atmospheric Administration survey. Stippled area represents area between mean high water and -18 ft. Dashed line represents the approximate position of -36 ft contour. PB, Point Brown; PC, Point Chehalis; NJ, North Jetty; SJ, South Jetty.

Quillayute Jetties

The Quillayute River enters the Pacific Ocean at La Push in a geomorphic position unique among major streams that discharge directly into the ocean. Whereas the Hoh, Quinault, and Queets river discharges, for example, are diverted northward in response to the dominant littoral drift direction, the Quillayute is diverted to the south, partly protected from southwest wave action by a group of near-shore stacks, including James and Rock islands (Figure 3). A narrow cobble-

Figure 3. Mouth of Quillayute River and La Push. View to the northeast on January 22, 1980. Quillayute Spit on the left connects Rialto Beach with James Island (foreground) and Rock Island (left center). Revetment on the spit inhibits breaching of the spit and damage to the boat basin on the opposite side of the estuary at La Push. The distal end of the lower spit is breached in this view. The jetty protrudes from the northwest end of First Beach (right). Photo by U.S. Army Corps of Engineers.

gravel-sand spit, Quillayute Spit, has formed on the right bank (ocean side) of the estuary, ephemerally connecting the right bank headland, in which it is rooted, with James and Rock islands. While the spit appears to owe its origin to wave action from the west and northwest, the material forming the feature originates from limited northward littoral drift and flood detritus from the Quillayute River itself. During some large river floods, the spit has been breached, sometimes near its root at Rialto Beach, and it was probably periodically breached by westerly ocean storms although it is protected from southwest winds. An adequate supply of littoral material permitted rapid healing to its normal form. The last historic record of breaching by the river was in 1876, when a log jam at the river mouth resulted in a breaching near the root. Gradual southerly migration of the outlet occurred, and the outlet reached its present position in 1911.

In 1931, efforts to improve small craft navigation into the estuary were begun by construction of a 1,400-ft-long curved south jetty from the left bank of the river mouth toward James Island. The jetty was a rubblemound structure, constructed of locally quarried graywacke, to an elevation varying from +12 to +15 ft (U.S. Army Corps of Engineers, unpublished reports). It was founded on the mobile littoral gravels and sands which characterize First Beach adjacent to La Push. Simultaneously, a rubblemound dike was constructed on the right bank connecting Quillayute Spit and James Island. This effectively diverted the major source of nourishment that maintained the littoral system feeding Quillayute Spit into deeper water south of James Island. Moreover, the northward littoral drift build-up south of the jetty (First Beach) eventually overtopped the structure and possibly caused shoaling in the lower part of the channel. Additional shoaling occurred adjacent to the boat basin training wall, completed in 1960. The South Jetty was raised to +18 ft in 1957, and the lower part of the channel now appears to be self-maintaining.

Since the 1950s periodic, at times annual, maintenance dredging in other parts of the channel has been required. This material, mostly coarse gravel to sand, is placed on Quillayute Spit. The artificial nourishment has been inadequate, and breaching by ocean storms continues. Efforts to construct a rock revetment on the spit have been only partly successful. Breaching periodically takes place from the ocean side at a point adjacent to the north end of the revetted spit, and the revetment itself continues to progressively deteriorate. The dike on the north side of the river mouth (right bank) has not been repaired since 1958 and apparently has little function in channel maintenance. Its removal might well have beneficial effects on the spit in allowing littoral material to move northward with ease.

MAJOR BREAKWATERS

Grays Harbor

Westhaven Cove

The boat harbor at Westport and the land surrounding it is largely a reclaimed area on the inside of Point Chehalis which did not exist as land 100 yr ago (Figure 4). The basin itself has been dredged from the marine sands deposited as submarine portions of the Point Chehalis spit. It is adjacent to the entrance of the South Bay channel. The sand dredged from the basin now forms the base for port facilities and protrudes north to the edge of the Grays Harbor entrance channel, where it is protected by groins and heavy rock revetment for 1,200 ft along the artificial north end of Point Chehalis in what was 20 to 30 ft of water in 1883, based on a chart for 1886 by the U.S. Coast and Geodetic Survey and a 1985 chart by the National Oceanic and Atmospheric Administration.

Three breakwaters protect the boat basin. The two northerly structures, breakwaters A and B, are combination zoned rubblemound and pile designs with the rubblemound sections rising to +4 ft and the central wood piles to +17 ft. Both structures were completed in 1950, although the original dredged fill connection between them was replaced by a timber pile structure in 1973. Much of breakwater C is of a similar design, though the shallower depth at this site resulted in a smaller rubblemound structure. The curved southern portion of breakwater C is all rubblemound. The entire C structure was not completed until 1965 (U.S. Army Corps of Engineers, 1978b). The littoral dynamics of the nearby ocean coast move sand into Grays Harbor; this accumulation requires dredging of the boat basin at 10-yr intervals.

Strait of Juan de Fuca

Neah Bay Breakwater

The Neah Bay Breakwater is the second longest breakwater structure in the state (Table 1), extending some 8,000 ft between the Cape Flattery headland northwest of the town of Neah Bay eastward to Waadah Island. The breakwater is a rubblemound structure originally constructed between 1942 and 1944. Structure elevations varied between +14 and +18 ft. Construction used rock averaging 4 tons but heavier than 2 tons in an unzoned section. Rock in the original construction is a welded agglomerate which came from the Waatch River quarry a short distance southwest of Neah Bay. Bottom elevations are a maximum of -30 ft. The breakwater rests mostly on marine gravels and sands, but the eastern one-third essentially rests on bedrock.

The structure required some rehabilitation in 1949 and 1959 to repair storm damage, presumably a result

Figure 4. Point Chehalis, Westhaven Cove, and mouth of Grays Harbor, September 15, 1981. View west along the axis of South Jetty showing both former and present positions of the jetty and the accreted lands to the north and south. Groins protecting Point Chehalis are at the far right-center. Point Brown is at the upper right corner. Breakwaters A and B are at far right; breakwater C is the long structure at right center. Photo by U.S. Army Corps of Engineers.

of having used rock that was too small. By 1977 the central half of the breakwater, exposed to the heaviest wave climate, had again deteriorated, due to storm wave action and to the small stone size but partly due to questionable quality of a certain amount of the stones. Testing of stone from the original quarry revealed excellent quality stone was present in part of the quarry; however, the stone did not pass the freeze-thaw test. The ability of the original stone to hold up so well in the strait environment is probably due to the mild climate—and relatively few cycles of freeze-thaw. The 3,900-ft central section was again rebuilt in 1979-1980 using slightly larger armor rock (5- to 10-ton pieces, 75 percent of those larger than 6 tons); the rebuilt section was resting on a "bedding" of the original breakwater stone (U.S. Army Corps of Engineers, 1978a). Stone for the 1979-1980 rehabilitation was obtained from select material at the Mats Mats quarry.

Port Angeles

The small boat basin lies in the southwestern part of Port Angeles Harbor and is protected from heavy storms by Ediz Hook. The basin is situated across the road from high, steep former sea cliffs in glacial drift whose active erosion essentially ceased as Ediz Hook grew to its present length. Marine sand and gravel, mostly originating from the reworking of glacial drift characterize the subbottom to about elevation -30 ft. The sand and gravel are underlain by a hard gravelly clay containing shell fragments that suggest it is glaciomarine drift. Locally as much as 14 ft of wood debris from lumber industry operations overlies the marine deposits. The breakwater is a rubble-reinforced wood pile structure built where bottom elevations vary from -4 ft to -15 ft. The structure is reinforced with rock rubble to MLLW (U.S. Army Corps of Engineers, 1958, and unpublished data). Original exploration for the breakwater and boat basin indicated only sand in the marine deposits. Dredging equipment was sized for this condition, and initial dredging resulted in a lag of gravel and cobbles on the bottom, which required removal by different means.

Port Townsend

The breakwater and boat basin at Port Townsend are situated on the former tidal portion of a bay mouth bar which completely landlocked the small adjacent bay and made it a brackish lagoon. Bottom elevations in the breakwater area varied from MLLW to -6 ft. The main breakwater is at the top of a marine scarp which drops steeply to 7 fathoms. The shallow marine sediments are predominantly loose sand and silty sand and are underlain by denser sands containing shell fragments. The latter sands extend to at least -18 ft and likely deeper. The foundation permitted use of a breakwater design that included a gravel core flanked by quarry spalls and zoned armor section. Armor rock was obtained from Mats Mats quarry and ranged from 1,400 to 4,500 lb (U.S. Army Corps of Engineers, 1962).

Sequim Bay (John Wayne Marina)

Located on the west side of Sequim Bay at Pitship Point, this facility is unique in its curvilinear design and because it was the only boat basin in western Washington excavated by dry-land excavation rather than dredging. The tidal portion of the gravel and sand delta of Johnson Creek which formed Pitship Point had been subject to modification since 1960 when a large amount of gravel-cobble fill was placed to bring part of the delta above high water and a narrow fill was built to MLLW, a distance of about 600 ft. The tidal shelf is underlain by soft siltstone and sandstone bedrock, locally exposed along the sea cliffs behind the backshore. The bedrock surface was mantled by as much as 35 ft of granular littoral and marine deposits that increased in depth seaward across the shelf (U.S. Army Corps of Engineers, 1983).

The breakwater is a zoned rubblemound structure founded on marine sand and gravel generally between elevations +2.0 and -2.0 ft. Rock materials were hauled by truck from the Haller quarry south of Sequim. Upon completion of the breakwater, a temporary dike was constructed across the future entrance channel between the north end of the breakwater and the land, and excavation of the boat basin sediments, which included both granular materials and soft bedrock, was accomplished by backhoe behind this cofferdam. A drainage pumping system was utilized to maintain a "dry" condition for excavation. Excavated materials were disposed of in an engineered waste fill on land (Reid, Middleton and Associates, unpublished data).

San Juan Islands-Strait of Georgia

Bellingham (Squalicum)

The Port of Bellingham lies adjacent to the delta of the Nooksack River, which occupies the northern portion of Bellingham Bay. Channel dredging to provide access to port facilities and a small rubblemound structure date to the early 1930s. The present breakwater and boat basin was partly completed in 1959. A major extension to the structure together with a greatly enlarged boat basin were finished in 1980.

The bay bottom in the breakwater boat basin area is very soft to soft (loose) bay mud consisting of sand, silt, organic debris (including coal and shell fragments) ranging in thickness from 5 to 35 ft (U.S. Army Corps of Engineers, 1975, and unpublished data). The bay mud is underlain by glaciomarine drift consisting of stiff to hard clay and sandy or silty clay with discontinuous layers of gravel; the upper part of this drift has been reworked by postdepositional marine erosion. This marine material appears to extend as deep as elevation -75 ft in the vicinity of the original breakwater. The variations in thickness of the soft bay mud required extraordinary design and construction efforts, especially during work on the breakwater extension. The design included an extensive, thick gravel foundation pad mantled with quarry spalls as the first stage of construction; 6 months later a two-zoned rubblemound structure was built. Pore pressures were monitored by piezometers throughout construction. In spite of these efforts, part of the early construction accomplished during the first stage of work suffered a foundation failure, and the design was modified to reinforce the breakwater foundation using surplus steel torpedo netting sandwiched within the gravel and spall foundation pad. The structure was completed without further incident.

Friday Harbor

The breakwater and boat basin at Friday Harbor lie at the northwestern corner of the harbor, which is nominally 60 ft deep. The basin is constructed across a shallow shelf (0-10 ft deep) and straddles a steep submarine escarpment which drops to elevation -40 to -60 ft in the breakwater area. The basin bottom is characterized by 8 to 10 ft of soft to very soft silt (bay mud) underlain by a nominal 10 ft of firm sand, silt, and clay which is locally underlain by soft silt and clay. The deeper materials are probably marine or glaciomarine deposits. Bedrock is exposed along part of the north shoreline and is locally overlain by glacial drift (U.S. Army Corps of Engineers, 1981).

The bathymetry precluded use of a rubblemound structure at Friday Harbor. A floating concrete breakwater section 150 ft long was salvaged from a breakwater test off West Point at Seattle and utilized in the 1,600-ft-long structure at Friday Harbor. The floating sections are attached to steel H-pile anchors driven into the bottom. No lateral pile support was assumed in the soft bay mud. The northern end of the north breakwater was anchored into bedrock using cast-in-place concrete anchors in holes drilled through the glacial drift.

All of the dredging for the boat basin was in bay mud, which has liquid limits between 41 and 67, plastic indexes between 4 and 30, and water contents in excess of 80 percent. The facility was completed in 1984 and has performed well in this moderate wave climate.

Blaine

The Blaine port area, including the breakwater, was built on filled tidal lands on the northeast side of Drayton Harbor. The Nooksack River apparently once discharged into the harbor area. Loose, marine, silty sands with shell fragments range from 6 to 10 ft thick and are underlain by stiff clay and interbedded silty sand, which may be glaciomarine drift. The marine silty sands thicken seaward and are interbedded with soft plastic silts and clays, possibly of deltaic origin. There is evidence for littoral accretion to the north since construction of the port facility. The present zoned rubblemound breakwater was completed in 1957, founded in part directly on the marine sands and in part on an old timber breakwater.

Semiahmoo

The boat basin and breakwater lie on the Drayton Harbor side of Semiahmoo Spit (Tongue Point). Material transported by the southeastward migration of the spit by storm wave overtopping has lapped on to the soft deltaic sediments, which presumably were derived from the former mouth of the Nooksack River when it discharged into Drayton Harbor. A section on the inside of the spit reveals marine sands with broken shells and some loose to medium-dense gravel to about elevation -15 ft. These are locally overlain by a few feet of very soft, organic bay mud mixed with the marine sands. The underlying deltaic material is very soft, fine sandy silt which yields a strong hydrogen-sulfide odor. Though the upper part of these deltaic sediments may be mixed with marine sands to elevations as low as -30 ft, the soft sandy silt (zero blow-count material) extends to about -50 ft, where it is underlain by stiffer sandy silt mixed with shell fragments indicative of its marine origin. Semiahmoo Spit itself is composed of sand with gravel and cobbles (Rittenhouse Zeman, Inc., unpublished data). The mix of coarser marine and finer deltaic materials on the inside of the spit was the probable cause of dredging problems during boat basin excavation. The limited extent of shallow, firm foundation materials together with the wave climate probably dictated the selection of a floating concrete breakwater. design which has subsequently been built.

Point Roberts

Located near Lighthouse Point on the southwestern corner of the Point Roberts peninsula, this boat basin exhibits some unique features in both design and construction (Figure 5). It was the first curvilinear design in western Washington; its design was created to maintain high water quality within the basin by normal tidal flushing through the narrow entrance so as not to degrade the local fishery (Layton, 1979). The facility is also unique in that two jetties and a breakwater were required to maintain the entrance in this high energy-wave climate, the result of the longest (25 nautical mi) fetch on the inland waters. The structures protect the entrance to the landlocked boat basin, which is behind a natural beach. The beach consists of sand and gravel reworked from erosion of nearby glacial drift of which the peninsula is composed. The rubblemound jetties are constructed between MHHW and MLLW and are founded on littoral materials (sand, gravel, and cobbles) which extend to more than 30 ft below the bottom. The east jetty is partly founded on gravel fill which replaced 6 to 12 ft of unconsolidated clay identified during exploration and removed prior to rubblemound construction. The jetties were placed to prevent shoaling at the entrance channel by seasonally reversing, but predominantly westward, littoral drift. A rock dike to elevation +4 ft connects the east jetty with the breakwater and controls shoaling in the entrance channel, yet permits migration of fish along the shallow shore. The annual quantity of littoral drift was estimated to be between 3,000 and 5,000 cy prior to construction and a by-pass system was devised, creating a preliminary stockpile west of the west jetty from material brought by truck from east of the east jetty. Post-construction monitoring indicates that about 7,000 cy accreted east of the east jetty and 1,500 cy accreted immediately west of the west jetty during the first year of operation. To the west, a net annual erosion of approximately 3,100 cy was observed (Layton, 1987a). The by-pass beach nourishment program has been subsequently increased

Figure 5. Point Roberts breakwater and boat basin, March 2, 1978. View to the northwest, Lighthouse Point on the left. Causeways at Tsawwassen, B.C., are in the background. Photo courtesy of J. A. Layton, Layton and Sell.

to an annual frequency in order to maintain a more natural balance. Between 2,500 and 5,000 cy of sediment are moved annually.

The zoned rubblemound breakwater was constructed on marine sand and gravel where bottom elevations varied between MLLW and -14 ft. The structure crest is at 23 ft. Rock for all rubblemound structures was barged from the quarry at Mats Mats Bay.

The boat basin occupies a former tidal swamp that had been reclaimed for farming by dike construction early in the century. The area was underlain by 20 to 30 ft of dense to medium-dense marine sand underlain by medium dense to hard silt clay with gravel, probably representing till or glaciomarine drift. These materials were locally overlain by a thin layer of peat and swamp deposits.

Most of the basin was excavated by a 16-in. hydraulic suction dredge which was mobilized overland and reassembled in a 1/2-acre pond excavated for the purpose. Thus, most of the basin was excavated in a land-locked situation before the entrance channel through the storm beach was opened. However, during dredging a significant boulder field was found 8 to 10 ft below the land surface over the northern third of the basin and required changes in the method of excavation in this area. The boulder area was ultimately excavated by clam shell. Total dredging was on the order of 1,000,000 cy, most of which was used in grading elsewhere on the marina project.

Anacortes

Small boat basins at Anacortes are situated in the northwestern corner of Fidalgo Bay. The harbor, although less than 10 ft deep, is well protected by Fidalgo Island from southwesterly and westerly storms as well as from northerly winds. However, southerly winds and refracted waves from the north required breakwaters for smaller craft moorage facilities.

The basin bottom and near shore are underlain by bay mud that locally contains logs and other organic debris. This mud varies in thickness from near zero at the shoreline to about 30 ft in some areas and is underlain by a dense (or hard) clayey silt or sandy gravelly till that contains scattered gravel and shells, betraying its glaciomarine origin.

The Cap Sante boat basin occupies a former tidal area used for many years as log storage. Construction required removing many logs and pilings as well as a zone of gravel and boulders from the western part of the basin. The highly varied configuration of the firm ground surface on the shoreline caused some problems. A bulkhead that was constructed to retain dredged fill and that was not founded on firm ground failed as material dredged from the bottom was deposited behind it. The failure circulated dredged material back into the basin.

The north breakwater is a rubblemound structure rooted in the glacial drift of the Cap Sante peninsula and is apparently founded mostly on drift. The bayward thickening of the bay mud probably precluded further rubblemound construction; this is suggested by the fact that the intermediate and south breakwaters are pile structures, though the latter has a short rubblemound closure section on the south end.

The Anacortes Marina facility, completed in 1980, consists primarily of a pile breakwater with three short concrete floating sections to enhance water circulation. The bay mud here thickens seaward to about 18 ft at 1,100 ft offshore (Layton, 1987b). Emplacement of all piles required jetting into the stiff silty clay and silt underlying the bay mud. Dredging of the stiff silt was accomplished (with great difficulty) by using a 10-cy clam bucket. The nearshore portion of the basin was excavated by diking it off at low tide and ripping the stiff silt with a D-8 bulldozer (Osberg, 1987).

Puget Sound

Everett Harbor

The harbor at Everett lies south of where the Snohomish River discharges into Possession Sound (Figure 6). Plans for harbor development date back to 1875, generally consisting of variations of a protective breakwater or training dike located on the right bank natural levee of the major Snohomish distributary and dredging of the distributary channel. In 1893 the Everett Land Company proposed a tidal basin on the Everett waterfront, complete with confining dikes and shiplocks. In 1894 the U.S. Army Corps of Engineers proposed a 17,300-ft-long dike on the major distributary right bank, together with basin and channel dredging inside the protective dike. While the dike and basin were under construction in 1902, excessive siltation in the basin prompted engineers to breach the dike opposite Preston Point in order to provide a more direct outlet for the river's discharge and high detrital load. Subsequently, the dike has been rebuilt and rebreached on several occasions, and various proposals have been made to construct a sill or dike between Preston Point and the downstream limit of the breach to eliminate siltation into the boat basins. None of the proposals has been accomplished, although the Port of Everett in recent years has significantly upgraded the small boat basin and a large filled area on the former tide flats has been made available for commercial use (U.S. Army Corps of Engineers, 1968; Tudor Engineering Co. et al., 1951). The proposed U.S. Navy port anticipates use of a portion of this area together with a portion of Port Gardner off the Snohomish delta.

The northern part of the Everett harbor lies on the Snohomish River delta adjacent to a drumloidal ridge upon which the city is situated. The delta extends about 2.5 mi into Possession Sound with the main distributary channel curving around the north end of the drumloidal ridge and exiting south at Port Gardner. The distal face of the delta drops from a depth of a few feet to the nominal bottom in Possession Sound at a depth of 300 ft. The deltaic sediments are a mixture of soft to very soft clay and silt, loose to very loose silty sand commonly mixed with wood and shell debris. Beneath the breakwater or training dike standard blow counts range from 2 to 8. Liquid limits of the silts and clays range from 30 to 45, and plastic indexes range from 3 to 18. Although exploration has not progressed below about elevation -80 ft, the soft sediments probably extend to the full delta depth, 300 ft.

Early breakwaters were unzoned rubblemound structures founded on layered fascine foundation pads. The dike has been rebuilt several times between 1903 and 1974 due to the structure's having settled into the soft bottom. The 1974 rebuilding and extension used an extensive gravel or quarry spall foundation pad for a zoned rubblemound structure that was constructed in two stages and with 1.5H to 1V side slopes and overbuilt 2 ft to allow for anticipated settlement (U.S. Army Corps of Engineers, 1968, and unpublished data). Siltation within the basin behind the breakwater continues because much of the Snohomish River detrital load passes into or through this channel. Only a permanent upstream barrier that diverts the river discharge into Possession

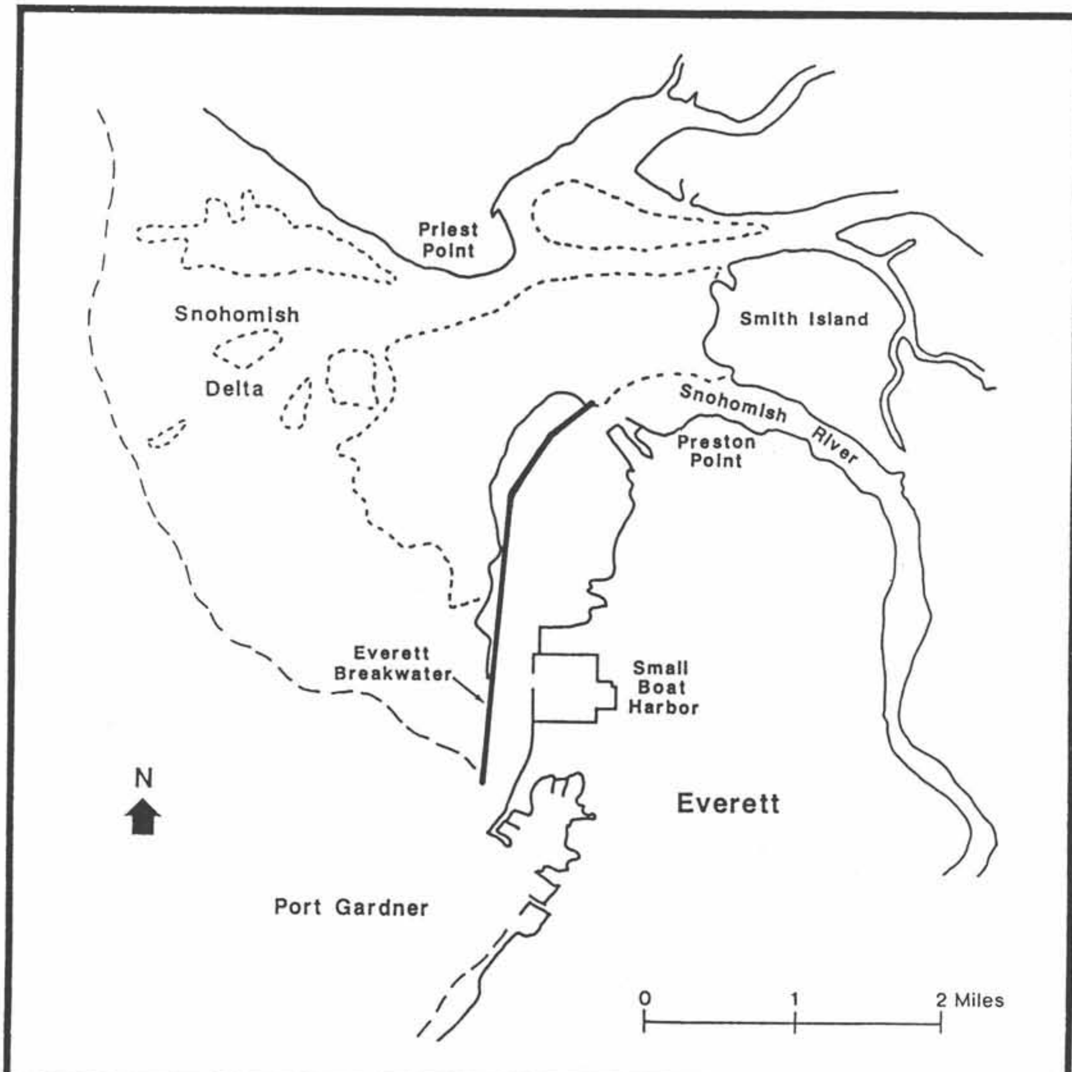

Figure 6. Everett breakwater and the mouth of the Snohomish River. Dashed line shows the landform at -9 ft. Dotted line represents MLLW. The area of proposed U.S. Navy facilities lies south of the small boat harbor and fronts directly on Port Gardner. Taken from National Oceanic and Atmospheric Administration chart 18444.

Sound from a point upstream of the basin will ultimately solve this problem.

Edmonds

The Edmonds boat basin lies behind two zoned rubblemound breakwaters, constructed in tandem near the outer edge of the postglacial marine shelf or terrace about 1,000 ft wide. Beyond the breakwaters, water depths increase rapidly to more than 500 ft. The structures are founded at about elevation -5 ft on dense, medium to coarse marine sand that contains gravel and shell fragments (Hill & Ingman/Dames and Moore, 1960). This material extends to at least -50 ft.

The project lies just north of the till bluff at Point Edwards but apparently encountered no till during boat basin excavation; all excavation took place in marine sand. The northern breakwater was completed in 1962, the south breakwater in 1968. Rock materials were obtained from the quarry at Mats Mats Bay. Apparently a small percentage of the rock used in the south breakwater was of inferior quality; it disintegrated under saltwater exposure. By 1983, enough rock pieces had disintegrated that rehabilitation of the south breakwater was required, that work completed in 1986.

Shilshole Bay

Probably the most exposed breakwater on Puget Sound, that at Shilshole Bay is one of the longest, largest, and of the most complex design section of any of the Puget Sound breakwaters (Figure 7). It rests on

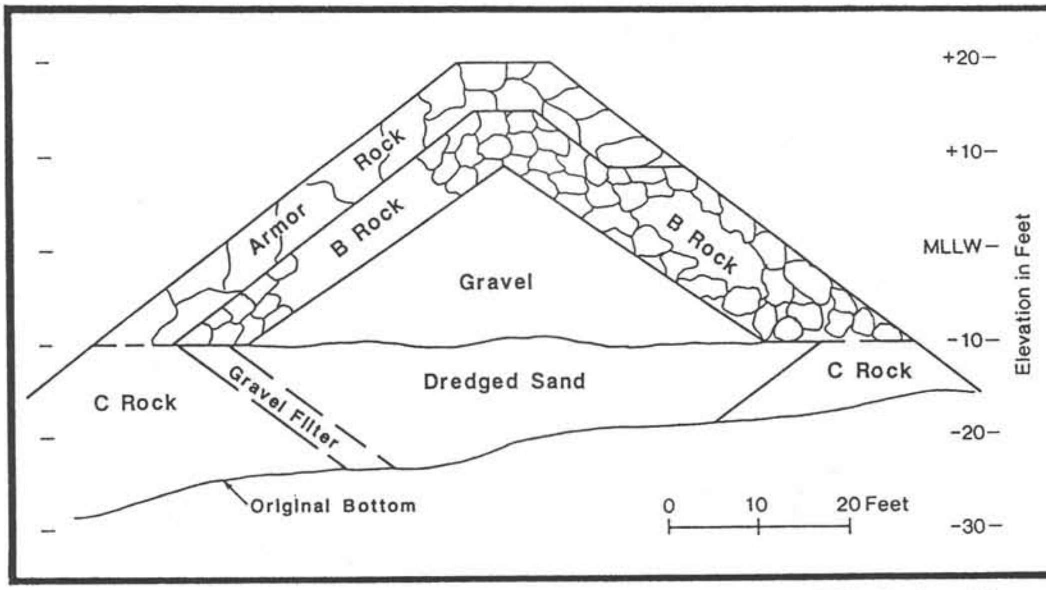

Figure 7. Typical section through the Shilshole Bay breakwater. Placement of dredged sand between C rock submarine dikes was confined to depths greater than -10 ft.

locally gravelly, dense, marine sand apparently derived from local retreat of sea cliffs in glacial drift. The marine deposits overlie an irregular till-lacustrine sediment surface ranging in elevation from -9 to -18 ft on the landward margin of the boat basin to about -50 ft beneath the breakwater. The original bottom elevation beneath the breakwater varied from -15 to -30 ft. Because the wave exposure dictated a stout rubblemound structure, the bottom, where it was below elevation -15 ft, was raised to elevation -10 ft by deposition of dredged sand between two parallel submarine dikes of 2- to 100-lb rubblestone (C rock) placed for that purpose. A gravel core (containing 20 percent sand) was then placed on the dredged material or natural bottom to +9 ft, and the transition rubblestone (B rock), graded between 150 and 1,000 lb, was placed upon the core. Armor rock (1,500 to 3,000 lb) completed the work, which has remained without maintenance since 1958. Most of the rock came from selective operation in the Mats Mats quarry; a small amount was taken from the Gorst quarry near Bremerton. Gravel was obtained from Steilacoom.

Although the dredging of the dense marine sands generally went without difficulty, a problem developed when cemented material was discovered on the bottom. The material proved to be poorly indurated conglomerate, probably ballast jettisoned from sailing ships in the early days, perhaps lumber ships returning to Seattle and other Puget Sound ports for another load.

The north end of the breakwater was extended 240 ft in 1961 to reduce northerly waves from entering the boat basin (U.S. Army Corps of Engineers, 1956, and unpublished data).

Kingston

The Kingston breakwater and boat basin (Figure 8) lie on the north side of Apple Tree Cove where the sea floor is underlain by a considerable thickness of marine sand and gravel. These coarse sediments were largely derived from a reworking of glacial deposits during sea-cliff retreat and mixed with littoral silt, sand, and shell fragments. Locally, wood debris mantled the marine deposits in the boat basin area. Till, probably of Vashon age, generally underlies the marine deposits. The till surface is irregular, ranging from above low water on the shore end of the marina, to elevation -32 ft in the outer channel area, to deeper than -36 ft beneath the breakwater (U.S. Army Corps of Engineers, 1965). The breakwater is a zoned rubblemound structure that has a core of gravel and quarry spalls and two zones of armor rock.

Des Moines

The breakwater for the Des Moines Marina is a zoned rubblemound structure with a gravel core. It is situated on the edge of the postglacial marine shelf developed during sea-cliff retreat over the past 5,000 yr. Nominal foundation elevations are about -10 ft but range from MLLW to -18 ft. The seaward marine slope drops rapidly to more than 600 ft. The shelf is partly underlain by marine sand containing gravel and shell fragments, loose near the surface, gradually becoming more compact with depth. These materials extend to at least elevation -40 ft beneath the breakwater. Shoreward, till, consisting of silty and clayey, fine to coarse sand with gravel, underlies the marine deposits. In the southeastern part of the boat basin, the till surface rose well into the dredge prism, nominally to elevations be-

Figure 8. Kingston boat basin and breakwater at Apple Tree Cove. Note shallow water in foreground, outside dredged basin. State ferry dock lies beyond the breakwater at the center.

tween -5 ft and -8 ft and locally as high as -2 ft beneath the present shore bulkhead (Reid, Middleton and Associates, 1968). At the land-closure end of the breakwater the till was overlain by several feet of organic debris mixed with silty sand.

The main construction problems anticipated revolved around the position of the till surface and attendant difficulty of dredging and pile driving into the till. Engineers recommended ripping the till prior to excavation. However, during construction, all dredging was done using a 24-in. toothed cutter head on a suction dredge, and although tooth attrition was high, the till was successfully dredged, probably largely due to its sandy character and the lack of cobbles and boulders. A single block the size of an automobile had to be removed by means of a clam shell (Anderson, 1987). During test pile work, difficulty was experienced in getting piling to hold because fines were washed out of the till during the jetting process, even though test piles drove easily to 9 ft without jetting (Dames and Moore, 1966).

Piling for floats, shore bulkheads, and a short section of pile breakwater north of the entrance channel were jetted below the till surface. Although two or three piles did rise during an unusually high tide, the problem did not become chronic; nearly all piling is holding well. The difference in pile behavior may be due to variations in the amount of jetting done during test pile investigations and actual construction conditions.

Port Orchard

The floating breakwaters that provide small craft protection at Port Orchard lie adjacent to a narrow marine shelf apparently cut in bouldery till. From the edge of the shelf, the bottom drops sharply to elevations of -25 to -40 ft at the bottom of Sinclair Inlet. Beyond the shelf edge the bottom is underlain by several feet of very soft bay mud. Divers inspecting submarine anchor fastenings reported sinking waist-deep into this mud. Despite the relatively small amount of dredging close to shore, the character of the till required use of a clam shell to complete the needed work.

The very soft mud, together with the water depth, dictated the use of a floating breakwater design in lieu of a rubblemound structure, in spite of the exposed location. The major floating section is anchored with submarine piles, and the short section is anchored to piles that extend above the water surface.

Blake Island

The breakwater at Blake Island State Park lies on a postglacial marine shelf cut in glacial drift lying at about elevation -9 ft. The boat basin and breakwater are sited adjacent to one of the two areas on the island periphery not characterized by coarse gravel and cobbles. The zoned, rubblemound structure is founded on marine sand. A minor amount of settlement was experienced during construction. The boat basin was dredged using clam shell equipment, and most of the dredged material was sand and a minor amount of gravel. Most of the disposal was in deep water about 1 mi south of the island; only a minor amount of land disposal adjacent to the southeast end of the basin took place.

Tacoma

The small-craft basin at Tacoma lies behind the unique (though not engineered) breakwater on the inland waters: the slag waste dump that accumulated for nearly a century from the now-closed Ruston smelter. This rubble-slag breakwater is about 2,500 ft long, 400 to 450 ft wide, and rises to about 20 ft above normal tide on slopes of 1H to 1V and locally steeper. Individual slag pieces vary greatly in size, and wave erosion has oversteepened portions of the pile between low and high water. Foundation conditions beneath the slag are unknown, but the sediments are presumed to be marine sand and gravel derived from retreat of nearby cliffs in glacial drift.

East Bay (Olympia)

East Bay is underlain by a sequence of modern bay muds underlain by fine-grained glacial outwash and lake sediments. The bay mud, consisting of soft to very soft silt, clay with considerable organic debris, and local loose silty sand, is between 10 and 30 ft thick and extends to 20 ft below MLLW in the breakwater area. The underlying outwash has upper and lower units of medium-dense to dense sand and silty sand; the upper unit is less dense. The thickness of the upper unit ranges from 2 ft to more than 60 ft, and the thicker portions are generally beneath the western and southern sections of the bay. Interfingered with the outwash is soft to medium stiff glaciolacustrine silt that contains sandy lenses. The glaciolacustrine silt ranges from less than 15 to more than 60 ft thick. Within the silt is an irregular sand unit which carries ground water under artesian pressure. Head on the aquifer was about elevation +21 ft, although variation coincident with tidal fluctuation suggested that the aquifer is vented to tidal waters. The aquifer appears to slope northwest and to range between elevations -13 ft and -108 ft in the East Bay area. Within the area dredged for the boat basin, the top of the aquifer was a minimum of 30 ft below the maximum dredge depth.

The thickness of bay mud suggested a breakwater design other than rubblemound. A design consisting of seven 100-ft-long floating concrete modules was selected; these are anchored by timber H guide piles embedded in the outwash material beneath the bay mud. Sediment dredged from the boat basin was largely bay mud and some outwash. It was disposed of on adjacent lands behind dikes to provide a substrate for marina-related facilities (U.S. Army Corps of Engineers, 1980).

OTHER BREAKWATERS

A number of additional smaller breakwaters have been built in the inland waters. Several are floating concrete structures—for example, those at Brownsville and Oak Harbor. Some breakwaters are little more than mooring floats attached to piles that extend above the water surface. Others are combinations of various materials. The well-designed pile breakwater at Langley replaces a former floating-tire facility of short longevity (Layton, 1987b). The Brownsville facility has been plagued by sedimentation from Crouch Creek. This may be due to increased erosion possibly occasioned by discharge changes from the urbanizing central Kitsap upland.

At the time of this writing (1987) the Oak Harbor facility includes a breakwater of badly deteriorated barges.

The facility at Flounder Bay occupies a former tidal swamp. The natural beach topped by fill serves as the breakwater, and the entrance is maintained by two short jetties. In more dynamic environments, Sandy Point on the Strait of Georgia and Lagoon Point on Admiralty Inlet have utilized geologically ephemeral landforms (a spit and cuspate foreland, respectively) for protecting their boat basins, adding minor jetty protection to the entrance channel. At Sandy Point, the natural littoral system and heavy storm climate had, in 1987, obliterated much of the entrance protection.

As has been shown in this discussion, many modern well-engineered breakwaters and jetties were preceded by use of materials at hand. Old barges, rafts of old tires, and boom logs have been used as makeshift breakwaters, especially for small port districts and some private boat basins for which financing is difficult to obtain. In many places, advantage was taken of natural landforms.

CONCLUSIONS

Considering the investment in watercraft and docking facilities even in small boat basins, the costs of proper engineering and appropriate geological investigation of foundation conditions and potential project effects on the littoral system are not significant. These case histories demonstrate that understanding the littoral regime contributes to the stability of the protective works at boat basins and harbors.

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Stratigraphy and Holocene Stability of the Snohomish River-Mouth Delta near Port Gardner, Everett, Washington

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INTRODUCTION

The past century has seen the initial development and subsequent large expansion in port and harbor development throughout the Puget Sound region. Development has occurred predominantly on the 11 major river-mouth deltas in Puget Sound. Expansion in port and harbor development has accelerated within the last few decades, mainly in response to national defense and international trade. This paper presents a compilation of engineering geologic investigations on the Snohomish river-mouth delta during the past 10 yr. This work was largely in support of the development and expansion of Port Gardner, Everett, Washington. We describe the stratigraphy and stability of the delta and evaluate how the stratigraphy will influence future development.

GEOLOGIC SETTING

The Snohomish River delta is located in the north-central Puget Sound (Figure 1), part of the Puget Lowland. The Puget Lowland is a complex topographic and structural basin formed during the Quaternary. The lowland has been repeatedly glaciated, resulting in the accumulation of a thick sequence of glacial and nonglacial sediments (Figure 2). The depth to bedrock varies considerably, but near the Snohomish river-mouth delta it is estimated to be about 1,600 ft (Yount et al., 1985).

Ice from the most recent glacial advance, the Vashon Stade of Fraser Glaciation (Armstrong et al., 1965), occupied the Puget Lowland 11,000 to 13,000 yr ago. Ice thickness is estimated to have been about 4,000 ft near the delta during Fraser Glaciation ice maximum (Thorson, 1981).

The topography and geomorphology near Everett are primarily a result of the last glaciation. The overall landscape has changed little since the ice retreated except as related to relative fluctuations in sea level, adjustments in river gradients, and formation of prograding deltas. The low, undulating topography near Everett exhibits wide, flat valleys that support underfit rivers.

Snohomish River-Mouth Delta

The delta complex has been constructed in a re-entrant of Possession Sound (Figure 1). The postglacial delta has been built with sediment supplied by rivers discharging through a gap or breach in a low divide of the bluffs that fringe the re-entrant. The upland bluffs consist primarily of overconsolidated glaciogenic sediments. The mouth of the Snohomish River currently occupies the breach. This breach developed during the northward retreat of ice from this area near the end of the Fraser Glaciation.

During the Fraser Glaciation, sea level was lower than present, and meltwater streams eroded deeply into the landscape, capturing drainages and changing river flow directions as the ice retreated. Since that time, rising sea-level conditions, influenced by both eustatic and isostatic adjustments, have resulted in aggradation of the Snohomish River valley and construction of the river-mouth delta which now covers about 8 sq mi. The delta is prograding as the sediment supply of the Snohomish River exceeds the reworking and transportation ability of the local marine environment.

The surface of the delta is near sea level and relatively flat near shore, sloping more steeply offshore, then easing to the normally gentle slope of the Possession Sound bottom at its outer edges. Transition from nearly flat to steeper slope is relatively abrupt, and the transition from steeper slope to gentle bottom is gradual.

The thickness of these normally consolidated postglacial deltaic sediments is on the order of 20 to 30 ft near the shore and is about 300 ft near the southwestern margin of the delta. In historic time, breakwater construction and dredging and filling activities have modified the near-shore deltaic deposits. Fill has been placed off the west shoreline. Fill consists of wood

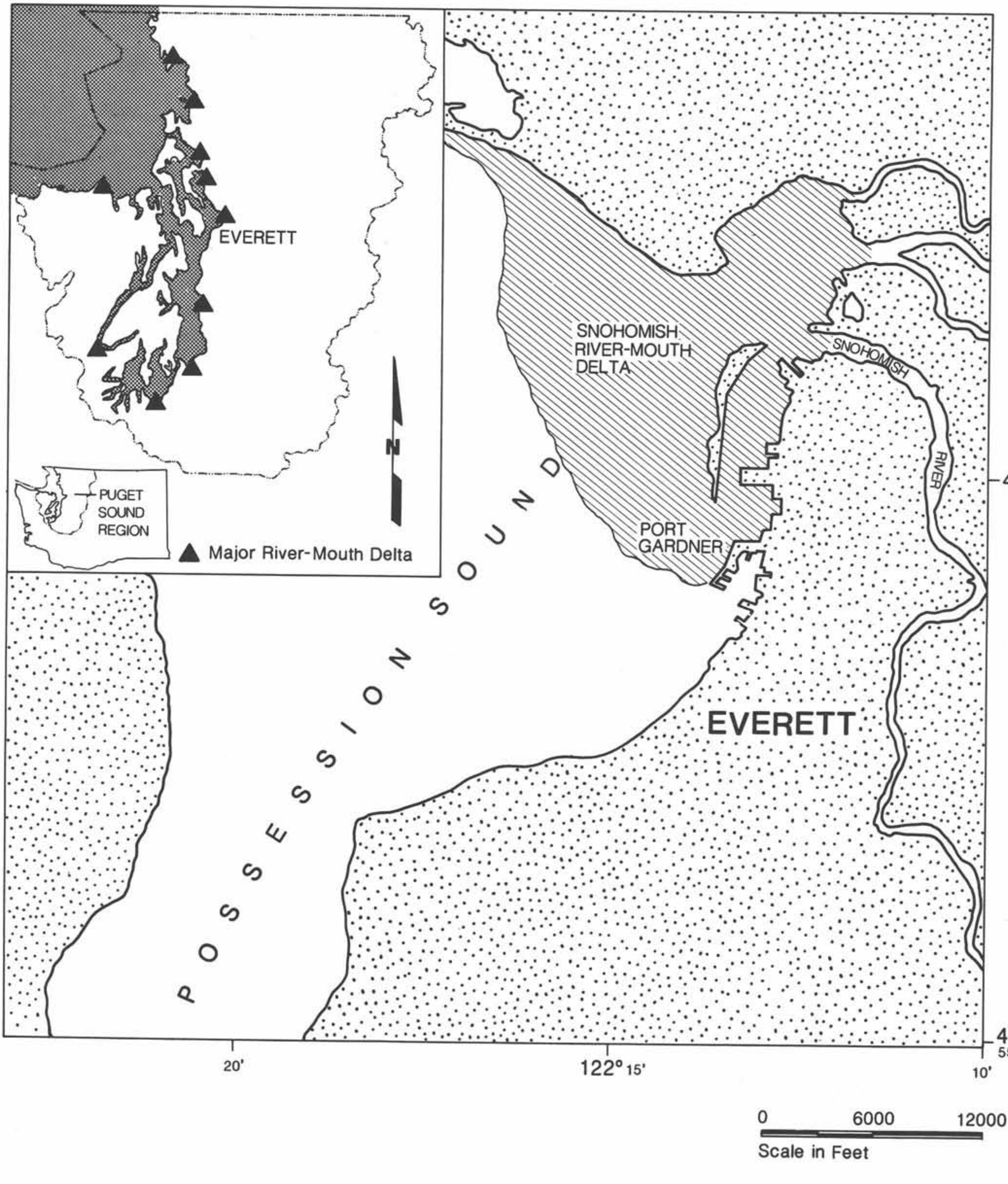


Figure 1. Location of Snohomish River-mouth delta.

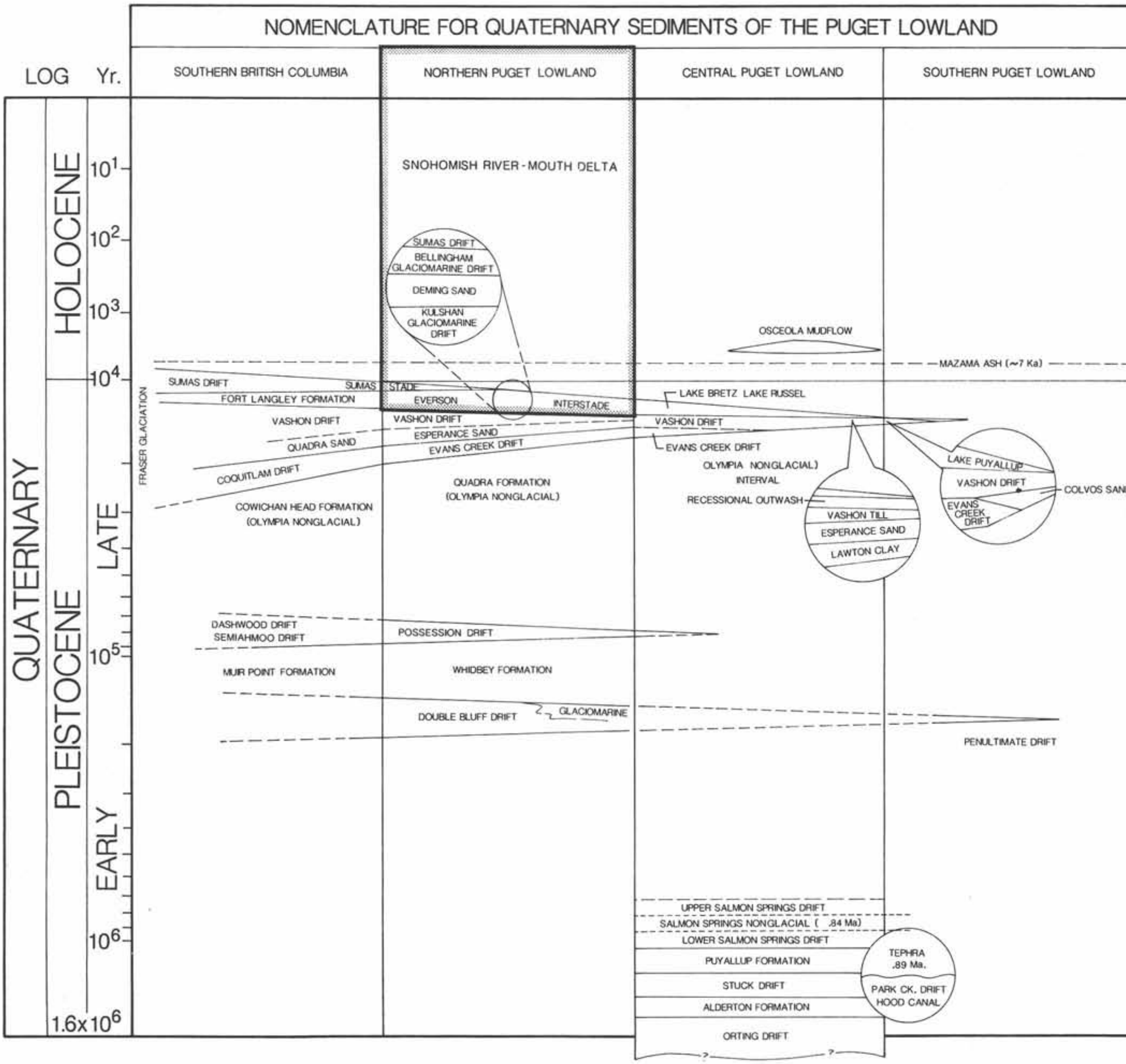


Figure 2. Compilation of generally recognized stratigraphic nomenclature for Quaternary stratigraphy of the Puget Lowland and western Washington.

chips, logs, construction debris, dredge spoils, and other assorted materials. Fill ranges from a few feet to probably more than 30 ft thick.

Stratigraphy

The gross stratigraphy and engineering geologic units defined for the Snohomish delta near Port Gardner are not complex. The units consist of laterally extensive graded sequences of sand, silt, and clay which exhibit internally consistent and generally uniform engineering properties.

Many components of field explorations and laboratory data have been used to identify and distinguish the various engineering geologic units that comprise the delta. Hollow-stem auger drilling and standard penetration sampling, vibracore sampling, and cone penetration testing provided useful and complementary information. The soil samples obtained by drilling were visually examined and classified by particle size, textural characteristics, and gradation. Continuous cone penetration data made it possible to observe the relative consistency, uniformity, and density of the various units. The cone penetration data were particularly useful in identifying potential drainage layers within cohesive soils and unit boundaries for pile design.

The stratigraphy of the delta near the port consists of nine engineering geologic units. Each unit is described in Table 1 in terms of its geologic and engineering characteristics. Sections A-A′ and B-B′ (Figure 3) are typical of the stratigraphic relations at the port.

Stratigraphic Interpretation

The internal structure of a prograding delta reflects its external shape and the processes that formed it. The internal deltaic structure can be divided into three basic components: topset, foreset, and bottomset beds. A typical prograding deltaic sequence is diagrammed in Figure 4.

In the southern and southwestern portion of the Snohomish delta, three recognizable deltaic sequences are indicated by sediment type and internal structure. Figure 5 shows a typical electric cone penetrometer log and the associated engineering geologic units encountered in boreholes. Figure 5 also shows the relation between the engineering geologic units and the internal structure of a prograding deltaic sequence.

The deltaic sequences are a result of a combination of late- and postglacial geologic processes that included shifting of the locus of deposition with time during relative, and possibly episodic, sea-level rise. Each sequence is described in the following paragraphs.

Delta sequence III (Figure 5) rests unconformably on postglacial erosional surfaces and on late-glacial marine sediments. The basal deltaic sequence is generally coarser in texture than subsequent sequences, presumably reflecting the late-glacial or early postglacial environment of lower relative sea level and more voluminous, coarser river bedload. The topset beds of sequence III are predominantly clean sand with fine gravel. The sorting, grain sizes, and stratigraphic position suggest that this sand unit has been winnowed by wave action during a destructive phase of the delta's history.

Delta sequence II exhibits the internal structure and textural characteristics of a classic delta sequence. The position of this sequence with respect to present sea level indicates deposition at a time of relatively lower sea level. Generally, sea-level rise allows for an accumulation of sediment at least twice as large as the amount of sea-level rise. Postdepositional compaction and consolidation associated with loading of the predelta surface can account for sediment accumulation in excess of the amount of sea-level rise. Conversely, postglacial isostatic rebound provides a limiting influence on delta thickness. The minimum elevation of relative sea level at the time of deposition of this sequence is estimated to be about -35 ft MSL. This estimate is in general agreement with published postglacial sea-level curves for the Puget Sound between 7,000 and 10,000 yr ago (Eronen et al., 1987).

The youngest delta sequence exhibits characteristics very similar to those of delta sequence II. A silt unit in the "Sandy Silt I" is likely a channel fill deposit that is not seen elsewhere in the stratigraphic section. A grayish-white 1- to 2-in.-thick coarse silt bed is present near the base of this deltaic sequence. This silt was encountered in numerous borings and identified in several cone probes. On the basis of textural characteristics (Powers and Wilcox, 1965), air-fall distribution maps (Clague, 1981), and relative stratigraphic position (Leopold and Newman, 1986), this silt is tentatively correlated to ash (tephra) from the eruption of Mount Mazama about 7,000 yr ago (Mack et al., 1979). This tephra provides a useful time-stratigraphic marker which allows for the calculation of an estimated average minimum rate of sedimentation for the accumulation of the youngest deltaic sequence of about 0.1 in./yr.

HOLOCENE DELTAIC STABILITY

With the increasing development occurring on the Snohomish river-mouth delta, overall delta stability is a major geotechnical concern. In light of the inherent instability of deltaic (liquefiable) soils and new data related to neotectonics of the Pacific Northwest, an attempt to evaluate the Holocene stability of the delta has been made. Engineering geologic studies included evaluating information related to historic and prehistoric mass wasting conditions on the delta near Port Gardner. Historic and prehistoric mass wasting was evaluated by analysis of bathymetric, geophysical, cone penetrometer, and borehole data.

Mass wasting is a common geologic process in the formation and development of actively prograding deltas. Failures resulting from both static and dynamic conditions are of interest in developing facilities on deltaic sediments. Mass-wasting processes may include slumps, sediment slides, and sediment flows. Most mass-wasting events occur under static failure conditions. Mass-wasting processes occur regularly along the more active portions of delta fronts.

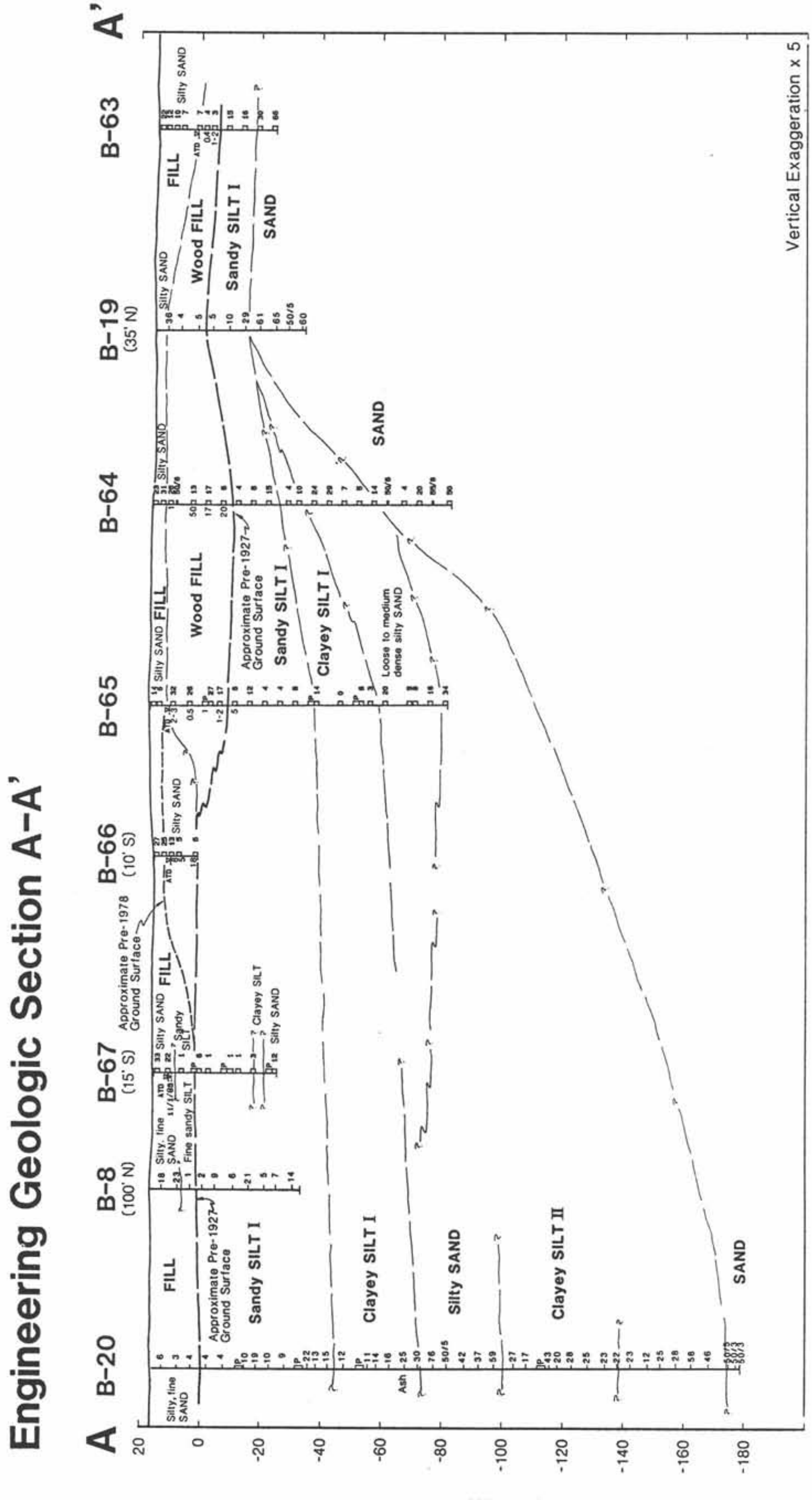
Dynamic forces, such as storms and earthquake-induced strong ground motion, can trigger mass-wasting events which are typically larger in scale than static condition events. Engineering analysis indicates that some of these delta soils would liquify under design earthquake loading. The seismicity of the Pacific Northwest is briefly discussed in the following section as it pertains to the potential for dynamically induced mass wasting.

Seismicity

Seismicity in the greater Puget Sound region is largely controlled by the complex interaction of two major crustal plates: the continental North American plate and the oceanic Juan De Fuca plate offshore to the west

Table 1. Descriptions and physical properties of engineering geologic units

Unit	Description	Thickness (ft)	Penetration resistance (tsf)	Pore pressure (tsf)	Sleeve friction (tsf)	Engineering properties
Fill	Loose to medium dense, gray, very silty, fine to very fine sandy SILt	0-15	--	--	--	Low strength due to inhomogeneity
Marine Silt	Very soft, gray to black, very fine sandy SILT with scattered wood fragments	0-15	--	--	--	Very low strength, large settlements expected
Sandy Silt I	Very soft to stiff, gray, very fine sandy SILT with thin beds to clayey silt and very silty, fine sand	10-55	5-55	0-2	0.1-0.5	Predominantly nonplastic, susceptible to consolidation due to sandy beds which allow drainage
Clayey Silt I	Very soft to stiff, gray slightly clayey to clayey, fine sandy SILT	5-55	5-15	2-7	0.05-0.2	Cohesive soil with relatively slow consolidation rate, plasticity index of 9, low to moderate undrained shear strength
Silty Sand	Dense to very dense, gray, silty to very silty SAND	5-43	10-65	2-5	0.2-0.5	High density for good to moderate pile support, small expected settlement, possible difficult driving
Sandy Silt II	Very stiff, gray, slightly clayey, very fine sandy SILT	10-90	45-195	0-2	0.5-2.0	Moderate pile support, small settlement
Clayey Silt II	Stiff to very stiff, gray, slightly clayey, fine sandy SILT	5-70	10-25	5-14	0.1-0.3	Can provide moderate pile friction support but very little end bearing support
Sand	Very dense or hard, gray, silty, gravelly SAND with interbeds of slightly gravelly, very sandy silt	5-20	70-7,200	--	--	Relatively incompressible, high end bearing pile support
Silty Clay	Medium stiff to very stiff, gray, interbedded clay SILT and silty CLAY	15-60	10-15	8-15	0.1-0.3	Low pile capacity


Engineering Geologic Section A–A'
A
A'
B-20
B-8 (100' N)
B-67 (15' S)
B-66 (10' S)
B-65
B-64
B-19 (35' N)
B-63
FILL
Wood FILL
Sandy SILT I
Clayey SILT I
Silty SAND
Clayey SILT II
SAND
Silty SAND
Silty, fine SAND
Fine sandy SILT
Approximate Pre-1927 Ground Surface
Approximate Pre-1978 Ground Surface
Loose to medium dense silty SAND
Ash
Vertical Exaggeration x 5
Elevation in Feet
20
0
-20
-40
-60
-80
-100
-120
-140
-160
-180

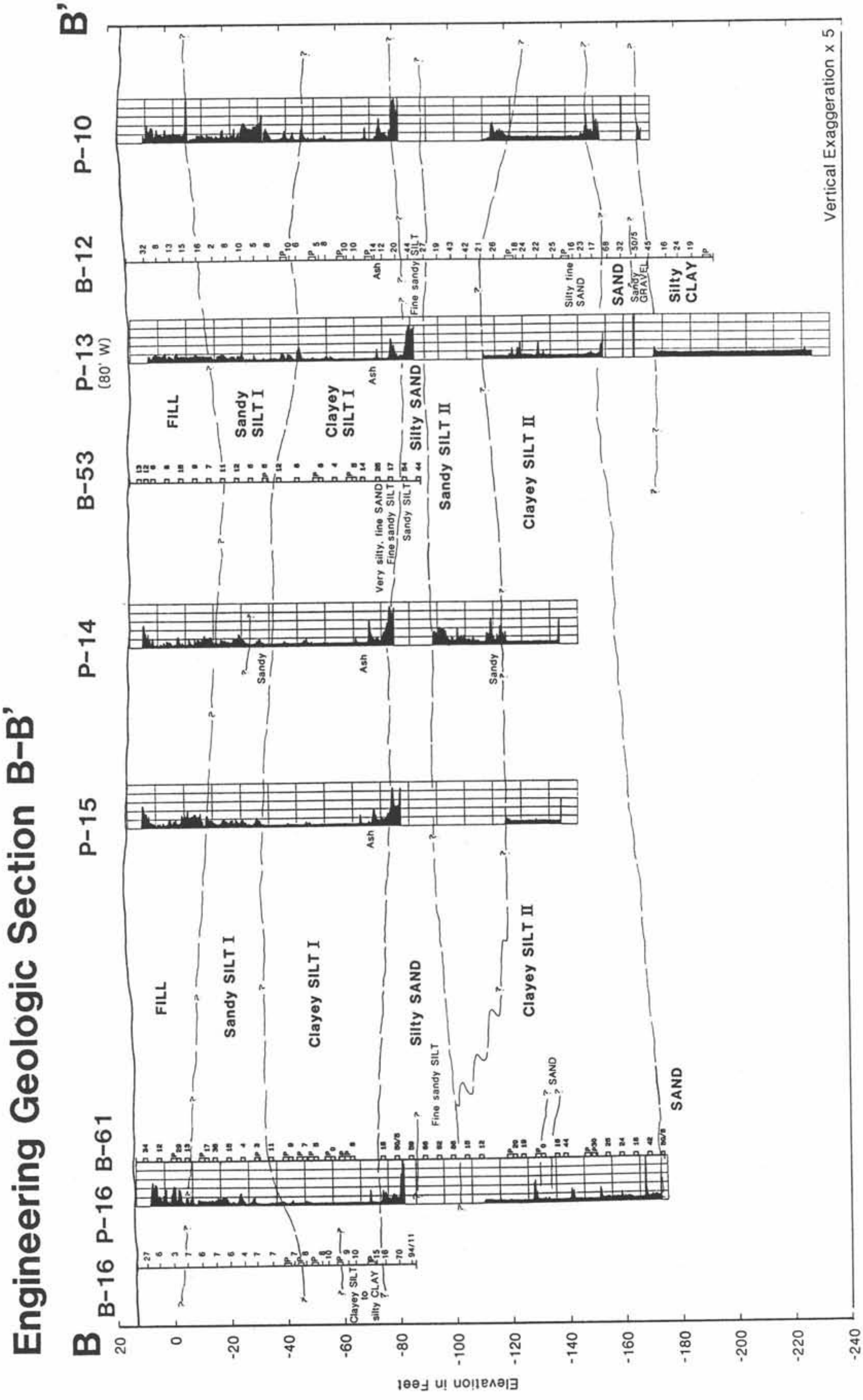


Figure 3. Generalized engineering geologic sections representative of deltaic stratigraphy near Port Gardner. Section A-A′ shows off- to onshore conditions. Section B-B′ is a north-south section about 100 ft offshore.

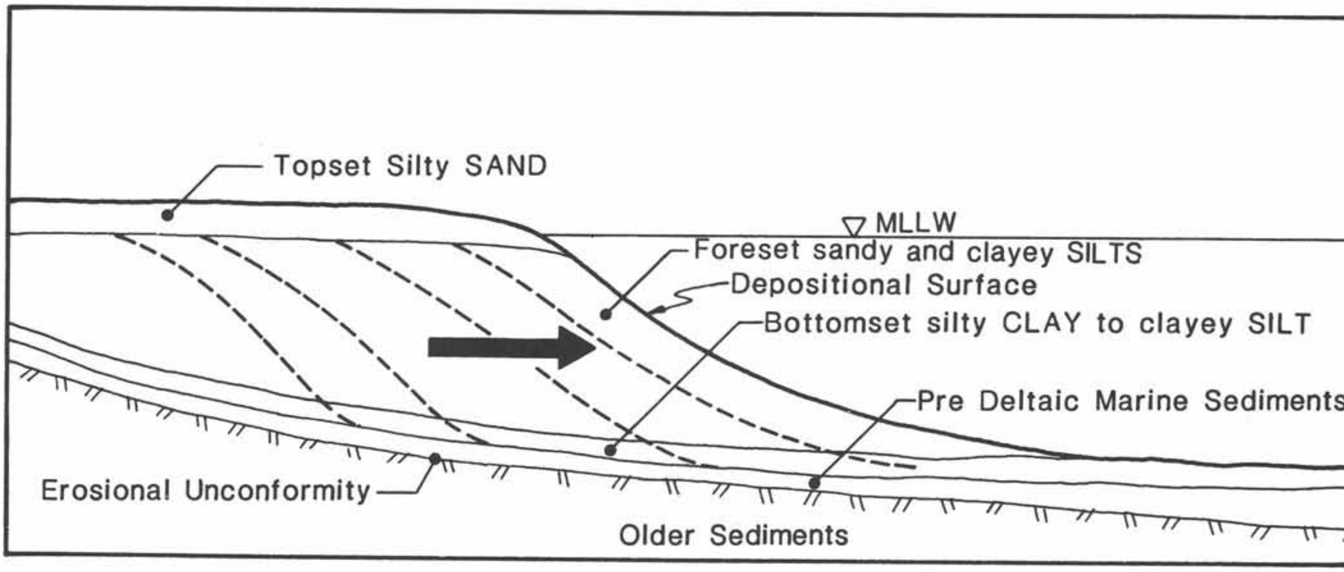


Figure 4. Diagrammatic representation of the internal structure of a prograding delta.

(Heaton and Kanamori, 1984). Good evidence for prehistoric earthquakes in the Puget Lowland is lacking. Atwater (1987) has presented rather convincing evidence in support of large prehistoric events, of probable seismic origin, along the coast of Washington. The mechanism for these large events may have included large-scale thrust faulting. The possible effects of such events can only be postulated, but strong ground motion would be expected in the lowland (Ihnen and Hadley, 1985).

Historical seismic events in the Puget Sound region appear to result from two mechanisms. East-west compression from the interaction of the two crustal plates is postulated to be responsible for shallow (5 to 15 km) and comparatively small (typically less than magnitude 6) earthquakes. Deeper (30 to 50 km) and frequently larger (Richter magnitude 7.3) earthquakes are postulated to be related to subducted oceanic lithosphere (Juan de Fuca plate).

Most historical Puget Sound earthquakes have been concentrated in a north-trending belt about 100 km wide centered on the lowland. Within this belt, seismic activity increases from north to south, reaching a maximum in the area from Seattle to Olympia. Historical documentation of earthquakes exists for the past 140 yr. The record is fairly complete for large magnitude events (Modified Mercalli Intensity of VII or greater, Table 2) and is incomplete for smaller earthquakes.

The Snohomish river-mouth delta experienced an undetermined level of ground motion from the Olympia, 1949, and Seattle-Tacoma, 1965, earthquakes. Major damage was not reported at the delta from either event. The only significant port damage reported in the Puget Sound during those earthquakes involved movement of a bulkhead on Harbor Island and some ground failure at the Port of Olympia.

Historical Evidence

Bathymetric survey data for the Snohomish delta (U.S. Coast and Geodetic Survey and U.S. Army Corps of Engineers) that spans the period from 1886 to present were evaluated in regard to historical changes in delta morphology. Survey data (soundings to the nearest foot) are available for every year since 1928. Surveys prior to 1928 were done at least once every 10 yr for the entire delta.

With the use of overlays of bathymetric data for subsequent years, an evaluation of changes in the morphology of the delta surface was possible. This evaluation did not identify any changes or inconsistencies in the survey data for the past 100 yr that could be attributed to mass wasting, and it indicated with a high degree of certainty that major mass-wasting events have not occurred. This suggests that the delta has not experienced historical ground motion of sufficient acceleration to induce major failures near the port.

Sedimentation rates greatly influence submarine mass wasting. The bathymetric survey data suggested that sedimentation rates in the portion of the delta near the port were low. A more accurate determination of sedimentation rates near the port was obtained using lead-210 isotopic age-dating techniques.

Numerous 3-m sediment cores were obtained by piston sampler at locations on the delta and in deeper water beyond the delta near the port. Incremental portions of the core were analyzed for stable lead and copper, and a determination of the excess lead-210 activity was made. Age estimates were obtained, based on a half-life

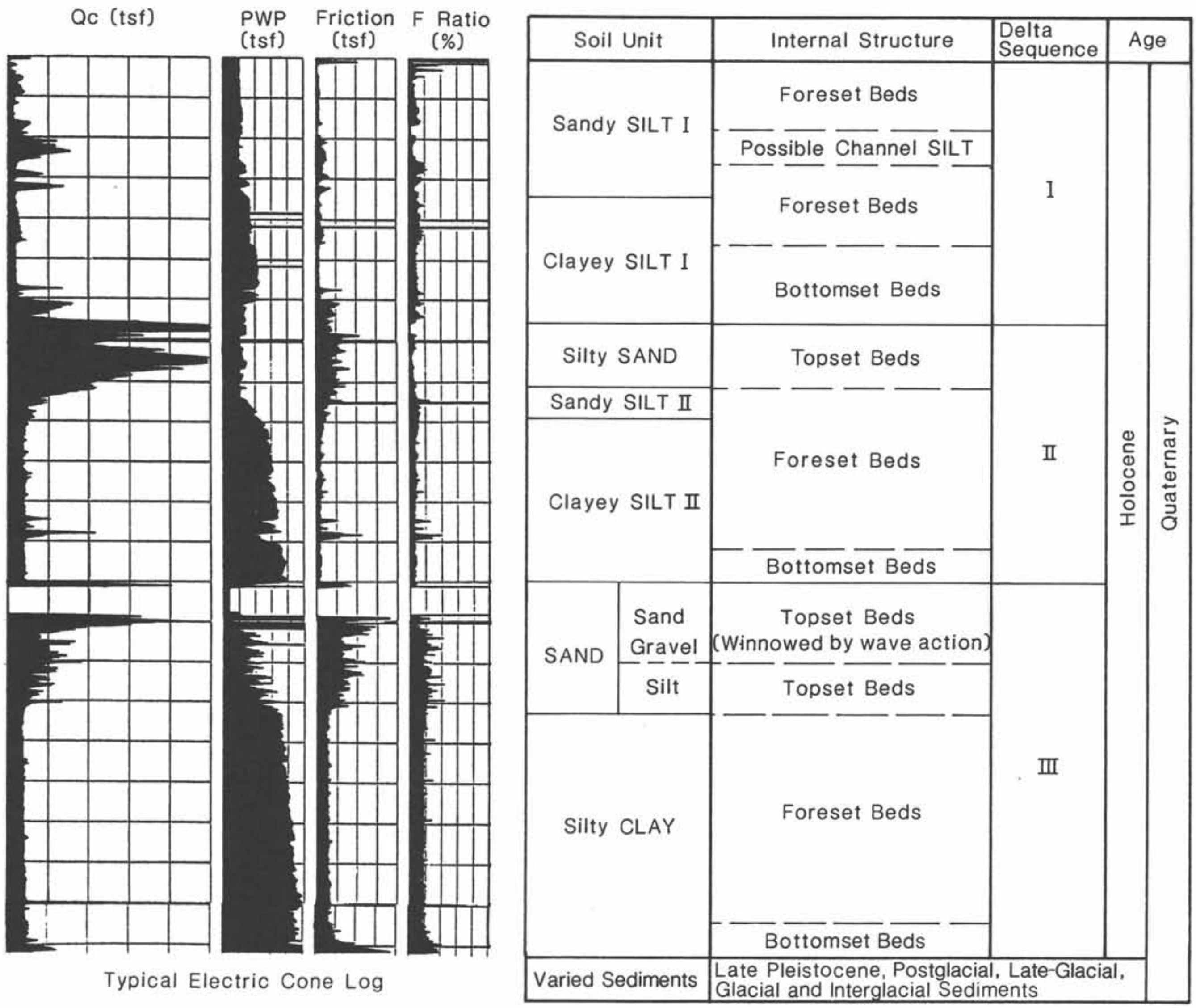


Figure 5. Three deltaic sequences, as suggested from electric cone log plots, supported by soil samples from geotechnical borings, for a portion of the delta near Port Gardner.

of lead-210 of 22.3 yr, by calculating from a plot of log base 10 (excess lead-210 activity) versus depth. Previous work in the Puget Sound indicates that lead and copper concentrations began to increase above background in about 1890. Sedimentation rates derived from the cores ranged from 0.06 to 0.1 in./yr. These rates are lower than those of other areas of the Puget Sound, but they compare favorably with estimated sedimentation rate minimums determined using tephrochronologic techniques.

Late Holocene Evidence

Geophysical surveys made along the delta included side-scan sonar, seismic-reflection surveys, and high resolution subbottom profiling (NORTEC, 1985). These data were obtained for several purposes, one of which was identifying features suggestive of prehistoric mass wasting. The area of the delta covered by surveys is shown on Figure 6.

Anomalous bathymetric contours obtained from high resolution subbottom profiling, interpreted in geomorphologic analyses as suggestive of prehistoric mass wasting, were identified at one location on the delta front about 1/2 mi south of the port (Figure 6). The geomorphic expression of this anomalous feature predates the late 1800s as shown on the 1886 bathymetric survey chart. Geophysical survey lines and scans of the anomalous geomorphic expression in the delta front did not reveal any seismic signatures indicative or suggestive of mass wasting.

Table 2. Earthquake history for major events in the Puget Sound region

Earthquake	Date	Modified Mercalli Intensity (at epicenter)	Richter scale magnitude
North Cascades	12/15/1872	VIII +	7.5
Puget Sound	12/12/1880	VI - VII	5.8
North Olympia	11/13/1939	VII	5.7
Pickering Passage	2/15/1946	VII	5.7
Strait of Georgia	6/23/1946	VIII +	7.3
Olympia	4/13/1949	VIII	7.1
Seattle - Tacoma	4/29/1965	VII	6.5

There was, however, evidence suggestive of small-scale, localized slope failures (Figure 6). These failures occurred on the relatively steep delta front or shorelines, a situation which is expected and typical for these environments and sediment types. Analysis of historical bathymetric survey data, or lack thereof, did not allow bracketing ages for these slumps. Historical ground acceleration can not be ruled out as a possible triggering mechanism for the failures.

Early Holocene Evidence

One additional indicator of the stability of the delta is a stratigraphic marker bed. Near the port, numerous geotechnical borings commonly encountered this thin bed of distinctively white, coarse silt at an elevation of -70 to -72 ft. This laterally extensive silt bed was consistently observed a few feet above the basal contact of the "Clayey SILT I" unit (Figures 3 and 5). As noted, this bed is probably tephra from the eruption of Mount Mazama. The consistent elevation and lateral continuity of the tephra indicate long-term stability with respect to differential vertical deformation related to large-scale mass wasting.

CONCLUSION

The deltaic stratigraphy near Port Gardner is basically simple and is internally consistent in comparison to other prograding deltas. The stratigraphic sequence reflects a combination of late-glacial and postglacial geologic processes that included shifting of the locus of delta deposition with time during relative, and possibly episodic, late-glacial to Holocene sea-level rise.

Limited direct and indirect evidence obtained from the delta supports an interpretation of both short- and long-term stability with respect to large-scale mass-wasting processes. The port is located on a portion of the delta where current sedimentation rates are relatively low and estimates of average minimum rates for the past several thousand years are also relatively low. These low rates substantially reduce the probability and frequency of mass-wasting events under static conditions. Substantial changes to sedimentation rates in the future are not anticipated. Historically, mass wasting has not significantly changed the gross morphology of the delta near the port, as indicated by the bathymetric and geophysical surveys. Data from borings and cone penetrometer probes indicate the delta complex near the port has been stable for the past several thousand years, and possibly throughout the Holocene.

The general conclusions derived from this evaluation raise some rather perplexing questions. This region is seismically active, and some researchers suggest that it is also influenced by great earthquakes. The stratigraphic record of the delta shows little to support the occurrence of major events that could be attributed to seismicity. Are there serendipitous reasons why this delta has not been subject to accelerations resulting in large-scale failures in the latter part of the Holocene? Or has the delta, in fact, experienced accelerations in excess of those estimated to be required to cause major deformation? If the delta has experienced such large accelerations, do we simply not recognize the events in the record? Is our record too short, or are data too limited? Are estimates of accelerations required to cause major failure too conservative? Do seismic events in the region not generate accelerations sufficient to cause major failures in the deltas? Finally, are the general conditions observed in this work representative of the entire delta or other major river-mouth deltas in the Puget Sound? We look forward to the answers to these and other questions during the next decades of engineering geology in Washington State.

ACKNOWLEDGMENTS

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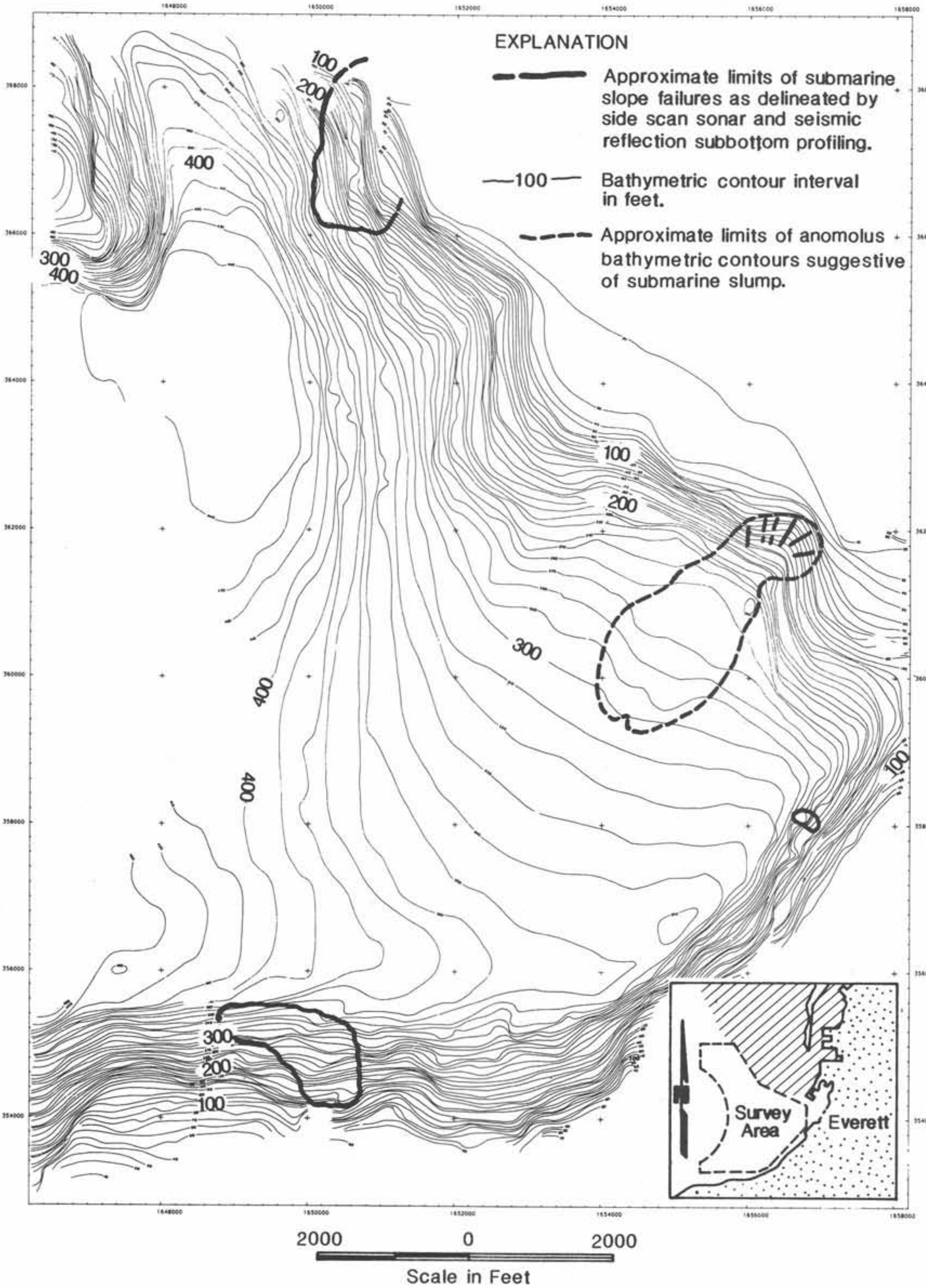

Figure 6. Bathymetric contour map prepared from high resolution subbottom profiling for a portion of the Snohomish river-mouth delta (NORTEC, 1985). Identification of slope failures from seismic reflection data by NORTEC. Geomorphic interpretation of submarine failure is by the authors.

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Mouth of the Snohomish River; aerial view to the west.
Photograph by R. W. Galster, February 1971.

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Ediz Hook—A Case History of Coastal Erosion and Mitigation

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INTRODUCTION

Ediz Hook is a Holocene spit about 3-1/2 mi long that varies in width from 90 ft at its narrowest part to about 900 ft at the widest part of the distal bulb. It projects eastward subparallel to the south shore of the Strait of Juan de Fuca and provides a natural breakwater for Port Angeles Harbor (Figures 1 and 2). It rises about 15 ft above mean lower low water (MLLW). The hook is composed entirely of sand, gravel, and cobbles derived by dominant eastward littoral transport from two sources: (1) sea cliffs developed in glacial drift between the hook and the Elwha River, and (2) the detrital load of the Elwha River, which enters the Strait of Juan de Fuca 4-1/2 mi to the west (U.S. Army Corps of Engineers, 1971, 1976; Galster and Ekman, 1977). Tidal range at Ediz Hook is from +11.0 ft to -3.5 ft MLLW.

The distal bulb of the hook is occupied by the Port Angeles U.S. Coast Guard Station. The lagoonal root is occupied by a major paper manufacturing facility owned by Crown Zellerbach Corp. The harbor shoreline of the hook includes facilities for the Puget Sound Pilots Association, public boat launching, and a log dump; there is an extensive log storage area in the harbor adjacent to the hook.

Port Angeles Harbor is the only harbor of refuge for large vessels along the south shore of the strait. It is a major shipping port for lumber and raw logs. Ediz Hook also was the proposed site for a major crude oil transfer terminal.

GEOLOGIC DEVELOPMENT

The formation of Ediz Hook is closely linked with the Elwha River and dates back to early Holocene time. Early during the deglaciation, when sea level was 60 to 120 ft lower than at present, short spits formed on the eastern margin of the asymmetric Elwha delta. Fossil spits are still preserved at 8 and 11 fathoms (Figures 2, 3A, and 3B). As sea level rose, responding to the worldwide melting of continental ice sheets, the Elwha aggraded in response to a rising base level, and sea cliffs in glacial drift retreated southward by erosion, furnishing a large volume of detritus to the prevailing eastward littoral transport system (U.S. Army Corps of Engineers, 1971; Galster and Ekman, 1977).

Development of the present configuration began when the sea had essentially reached its present stand about 5,000 yr ago (Downing, 1983) when the sea cliffs west of the present spit lay 3,000 to 5,000 ft north of their present position (Figure 3C). The subaerial spit apparently began at a point where the east-trending sea cliff gave way to a southeast-trending shoreline. As the spit progressively grew eastward and the sea cliffs retreated, the western neck and root of the spit migrated southward into the Port Angeles harbor area by a process of overtopping and breaching during heavy storms followed by healing during quieter periods. This process of migration of the older sections of the spit resulted in a truncated platform (shelf) at about 5 fathoms where the base of the original spit once lay (Figures 3D and 3E). The supply of littoral detritus was apparently continually adequate to heal any storm breaching of the neck and permit a progressive eastward extension of the spit at the rate of about 5 ft/yr from a furnished annual detrital volume estimated at 340,000 cy. Fifteen percent of the detrital volume is estimated to have originated as bedload from the Elwha River, the remainder was from sea cliff erosion (U.S. Army Corps of Engineers, 1971).

HISTORIC CHANGES

Changes in the annual volume of detritus supplied to the littoral system were mainly the direct result of two activities in an effort to industrialize Port Angeles. In 1911 Elwha Dam was constructed 4.8 mi upstream from the mouth of the Elwha River. The lake behind the dam serves as a sediment trap for an estimated 50,000 cy of sand, gravel, and cobble bed load material formerly annually contributed to the littoral system. In 1930 an industrial waterline was buried along the toe of 3.3 mi of eroding sea cliff west of the root of Ediz Hook. A 2,400-ft section of the pipeline was initially protected by construction of a wooden bulkhead. Between 1958 and 1961 protective works in the form of steel piling and rock riprap along 6,800 ft of the pipeline were com-

Figure 1. Ediz Hook. View west, with Port Angeles Harbor at the left and Strait of Juan de Fuca at the right. The Elwha River enters the strait at Angeles Point, the first promontory beyond the root of the hook. Photo by Seattle District, U.S. Army Corps of Engineers.

pleted. Because of the reduced rate of sea-cliff retreat, the supply of littoral detritus contributed by sea-cliff erosion was reduced from 270,000 cy/yr in 1911, to 95,000 cy/yr in 1930, and to 40,000 cy/yr in 1961. This represented an 89 percent reduction from the estimated total available material prior to construction of Elwha Dam and the pipeline (U.S. Army Corps of Engineers, 1971).

During this 50-yr period of reduction in littoral detritus, the distal bulb of Ediz Hook grew, at decreasing rates, a total of 352 ft, apportioned as follows:

Period	Growth Rate	Quantity added
1870 - 1917	4.2 ft/yr	39,000 cy/yr
1917 - 1948	3.7 ft/yr	34,000 cy/yr
1948 - 1970	1.8 ft/yr	17,000 cy/yr

In addition to the above 56 percent reduction in bulb growth, a corresponding steepening of beach profile and increase in erosion of littoral detritus from the foreshore west of the distal bulb on the strait side of the hook was experienced. On the basis of bathymetry changes between 1940 and 1970, engineers calculated that an estimated 82,000 cy/yr was eroded from the foreshore area west of the root, 26,000 cy/yr was eroded from the root and neck, and 39,000 cy/yr was accreted to the distal bulb. This provided clear evidence that the bulb was being extended at the expense of the western portion of the hook (U.S. Army Corps of Engineers, 1971).

By the mid-1930s the loss of beach nourishment was manifest in damage to facilities on Ediz Hook. Coordinated corrective effort was made difficult because ownership of the spit was divided among the U.S. Government (Coast Guard), City of Port Angeles, and Crown Zellerbach Corp. Between 1937 and 1970 each of these groups was instrumental in constructing varied piecemeal protective works, including timber and pile

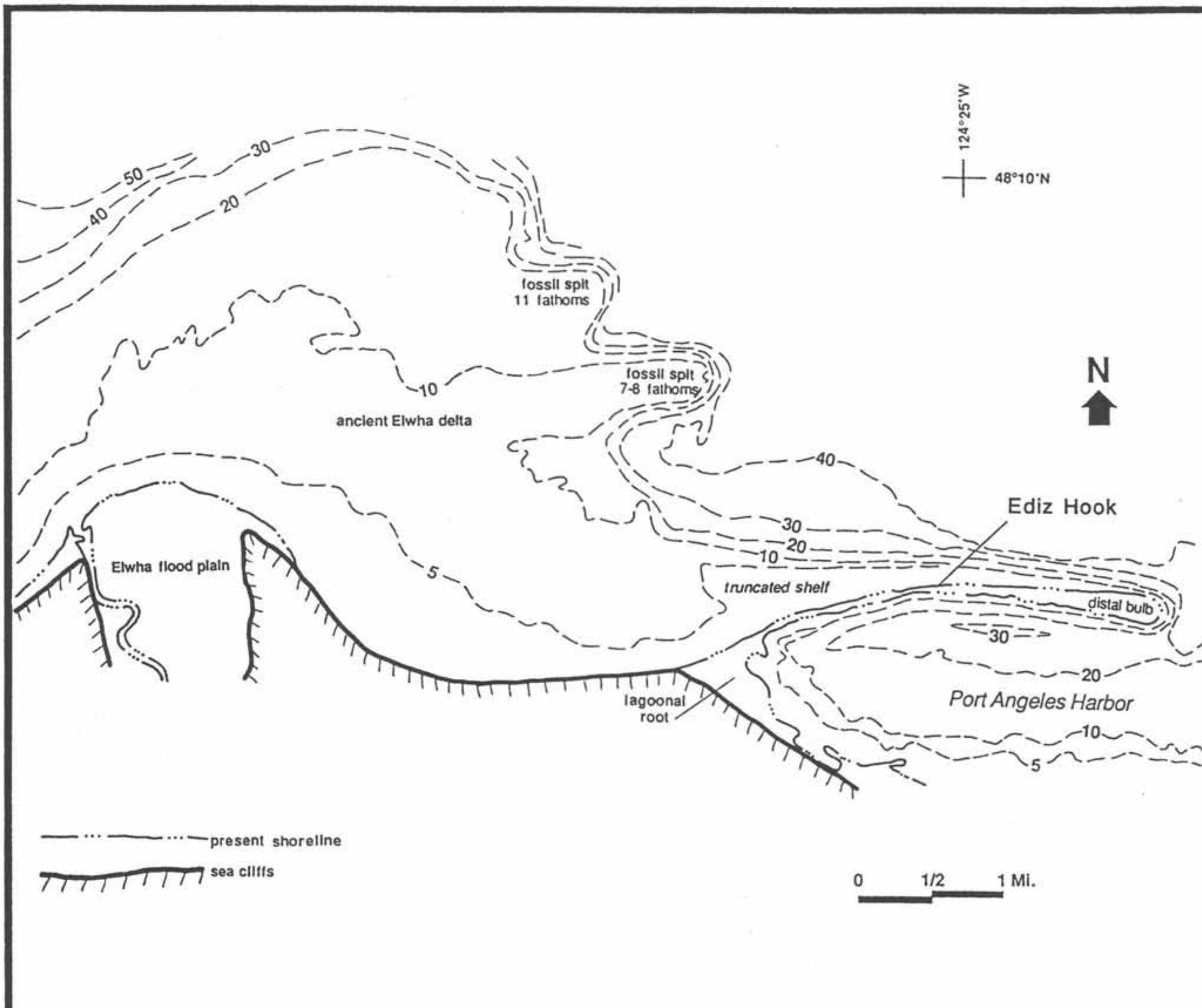

Figure 2. Subaerial and submarine landforms at Ediz Hook as outlined by present bathymetry (in fathoms).

bulkheads, rock riprap, timber and timber and rock groins, steel pile walls faced with riprap, and log crib bulkheads. Total expenditures were on the order of $900,000. The continued lowering of the beach profile owing to lack of littoral supply damaged or destroyed many of these protective structures.

EROSION CONTROL PROJECT

In 1970, Congress, in response to appeals by local government, asked the U.S. Army Corps of Engineers to study the causes of erosion at Ediz Hook and recommend appropriate measures for its control if economically justifiable. The resulting studies are documented in two reports (U.S. Army Corps of Engineers, 1971, 1976), which considered a number of alternate plans, as well as the plan presently in place. The benefit-cost ratio for the project varied from 17 to 15, one of the greatest benefit-cost ratios in Corps' history. A substantial part of project approval was based on the projection of ultimate permanent breaching of the neck of the spit, the ephemeral formation of Ediz "island" (Figure 3F), and the destruction of the spit as a breakwater for Port Angeles Harbor.

The studies included a Beach Feed Evaluation Test to assist in determining positions of beneficial beach nourishment placement (U.S. Congress, 1973). The test included placement of: (1) 8,600 cy of select gravel and cobbles at two locations, on the root and western neck of the spit, (2) 1,000 cy of 10- to 500-lb quarry rock as an experimental rock blanket, and (3) 7,300 cy of pit-run sand and gravel; both the quarry rock and pit-run

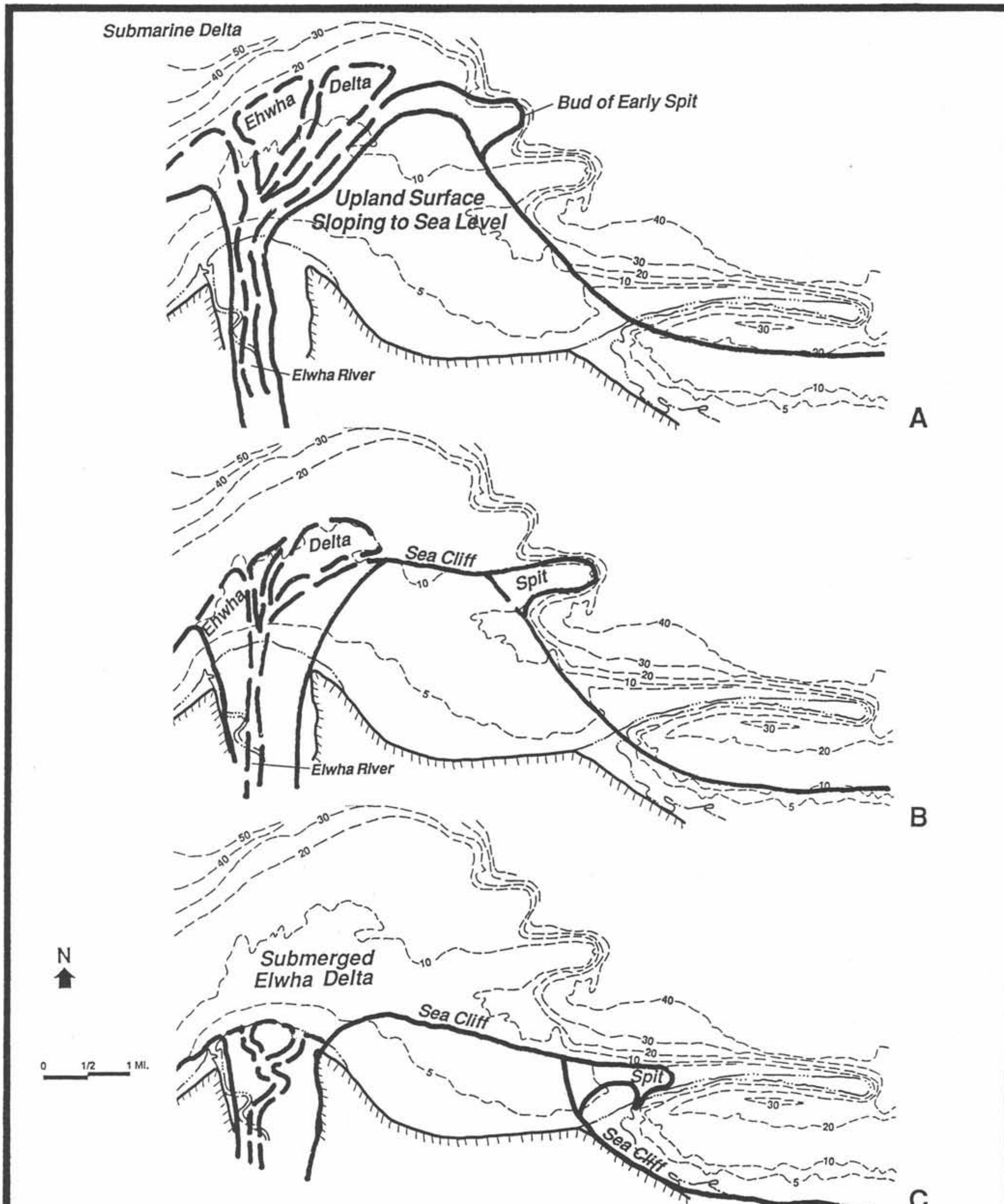

Figure 3. Development of Ediz Hook. A. Early spit formation about 9,000 yr ago when sea level was about 120 ft below present stand. B. Formation of a new spit about 7,000 yr ago, with sea level about 60 ft below present stand. C. Beginning of present formation spit about 5,000 yr ago, when sea level was nearing its present stand.

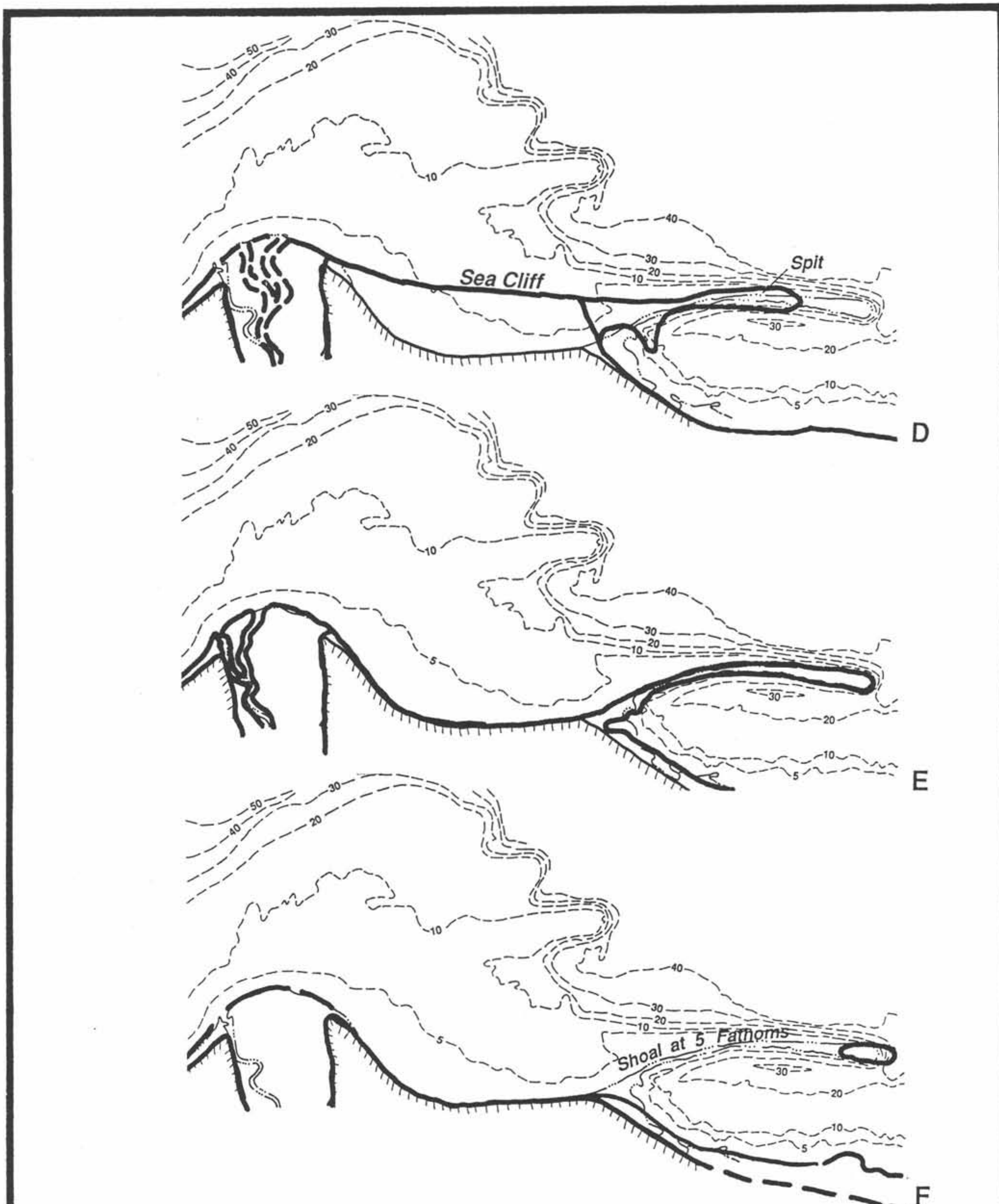

Figure 3. Development of Ediz Hook, continued. D. Growth of the spit from 1,000 to 2,000 yr ago accompanied by sea cliff retreat. E. Present configuration of the hook. F. Probable future configuration resulting from continued lack of beach nourishment.

material were placed on the eastern limit of the neck. Tests were conducted during the spring and summer of 1975. They showed that 38 to 74 percent of the select gravel and cobbles was retained in the downdrift littoral zone. The lower percentage was experienced near the root of the hook, where 62 percent of the material moved offshore. The higher percentage was retained to the east along the hook, where only 26 percent moved to the nearshore area. Stockpile erosion rates were high at first but tapered to typical erosion rates after about 3 months. The experimental rock blanket was not decimated and exhibited a tendency to improve accretion of finer grained littoral material (U.S. Army Corps of Engineers, 1976). As a result of these studies, the select gravel/cobble nourishment was chosen as the most cost-effective nourishment program, in lieu of pit-run or quarry spalls (rock blanket).

The erosion control project (Figure 4) consisted of two basic elements: (1) redesign and reconstruction of existing revetment sections on the strait side together with expansion to a continuous revetment from the west end of the Crown Zellerbach plant on the root of the hook to the western part of the distal bulb, and (2) placement of 100,000 cy of select, high caliber (cobbles and coarse gravel) beach nourishment at five stockpile locations along the strait side of the neck and western portion of the distal bulb (U.S. Army Corps of Engineers, 1976; Galster and Ekman, 1977). A rock blanket was also provided along 3,100 ft of the distal bulb to aid in the entrapment of littoral material (Figure 4). Thus, the concept was to rebuild a revetment with a weighted toe covered by a dynamically stable beach nourished periodically by coarse gravel and cobbles. The project was accomplished in 1977-1978 at a cost of $5.6 million.

Specifications for materials used in the work were as follows:

Rock quality

Specific gravity: 2.65 minimum

Absorption: not greater than 3 percent

Accelerated expansion: not more than 15 percent breakdown

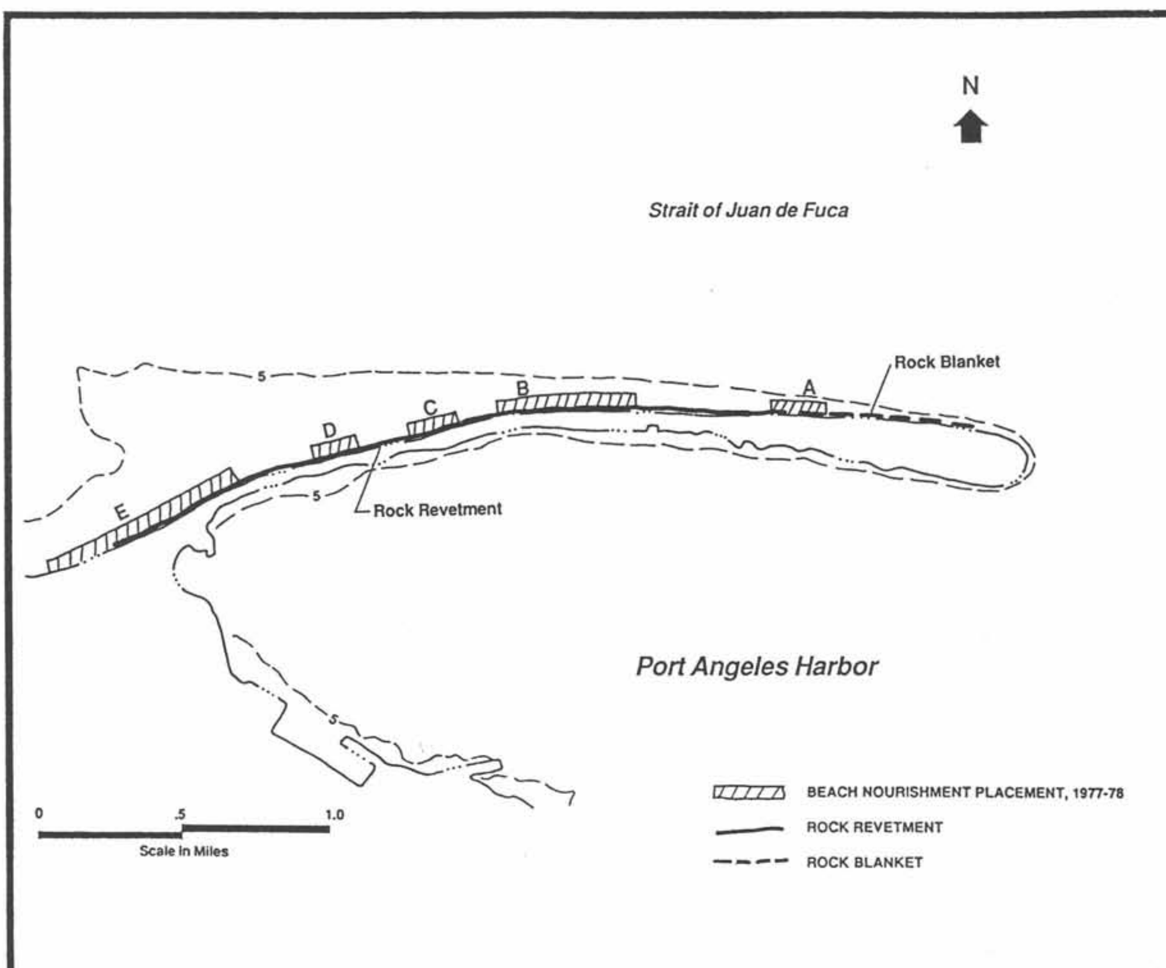

Figure 4. Principal features of the Ediz Hook Beach Erosion Control Project, 1977-1978 (5-fathom contour shown).

Soundness ($MgSO_4$): not greater than 5 percent loss

Freezing and thawing: not more than 10 percent loss

Gradation

Armor rock: 3,000-8,000 lb with 75 percent greater than 5,000 lb

Class C rock: 25-600 lb, with 50 percent greater than 200 lb

Rock blanket: 25-1,000 lb with 50 percent greater than 500 lb

Beach nourishment: gravel and cobble material from 1 to 12 in. with 50 percent larger than 3 in. and not more than 5 percent passing the 1-in. screen.

The rock work required reworking of about 90,000 tons of rock already in revetments on the hook. Rock judged as below specification in quality, size, or shape was wasted immediately offshore at low tide. Approximately 120,000 tons of new armor rock were obtained by select quarrying at Mats Mats Bay near Port Ludlow, and 45,000 cy of finer rock (class C rock) was provided by the same source for the filter sections of the revetment. The completed revetment contained 199,000 tons of armor and 206,000 tons of smaller rock.

Beach nourishment was obtained by processing material from a designated source on the glacial drift upland west of the Elwha River valley. The source origin is a combination of outwash from alpine glaciation in the Olympic Mountains and outwash from the Juan de Fuca lobe of the continental ice sheet. The source included both Olympic and Coast mountains rock types. Pre-construction exploration indicated a reserve in excess of 220,000 cy of select gravel and cobbles comprising 48 to 70 percent of the total material (U.S. Army Corps of Engineers, 1976; Galster and Ekman, 1977). Later pit development beyond the investigated sections revealed a much higher percentage of finer material.

The beach nourishment was placed following completion of successive downdrift (eastward) revetment sections. Armor stone was removed from the revetment crest at each nourishment location to provide access to the beach for trucks and dozers to distribute the material within specified limits in either direction (Figure 4); placement was completed as shown on Table 1.

Long-term project costs included planned replacement of beach nourishment on the order of 100,000 cy at 5-yr intervals.

Table 1. Beach nourishment

Area	Length (ft)	Date Placed	Quantity (cy)
		Original Placement	
A	1,000	Nov 1977/Jun 1978	15,000
B	2,400	Jul-Aug 1978	36,000
C	700	Aug 1978	10,400
D	600	Aug-Sept 1978	9,000
E	3,000	Sept 1978	45,000
		Rehabilitation Placement	
B/C	2,500	Aug 1985	14,000
E	2,400	Aug 1985	16,000

PROJECT EFFECTS AND PROBLEMS

Project Monitoring

Project monitoring included direct beach observations, aerial photograph analysis, hydrographic surveys, and, in 1983, a side-scan sonar and bottom sampling mission. Ten cy of tracer cobbles were placed at two locations on the beach following placement of beach nourishment at the west ends of areas E and B (Figure 4). Tracer cobbles in Area E were a granite and basalt combination from a pit 2 mi downstream from Chief Joseph Dam on the Columbia River. Tracer cobbles in Area B were Belt Supergroup quartzite and argillite from a pit north of Kalispell, Montana. Both combinations were believed to be sufficiently distinct from the Olympic-Coast mountains natural beach detritus and beach nourishment to be easily recognizable during beach observation. In practice, the Area E tracers were not easily differentiated from native rock types. The Area B tracer cobbles provided valuable data on movement of beach nourishment along the spit. Rates of movement varied from 0 to 25 ft/day; the average was 8.8 ft/day for the first 2-1/2 yr. Observations ceased when the first tracer cobbles from Area B reached the distal end of the spit.

Beach Nourishment Areas

Area A

By March 1979 a substantial portion the 15,000 cy of nourishment placed in Area A had moved rapidly downdrift (eastward) to partly cover and mix with the rock blanket material. By May 1981 the visible nourishment berm had been reduced to a length of about 300 ft, and by August 1983 no nourishment berm was discernible.

Area B

This second largest nourishment area (36,000 cy) had the greatest longevity. By March 1979 a secondary nourishment berm, 800 ft long, had developed beginning about 300 ft east of the eastern end of the original placement. The Area C berm had merged with that of Area B, and by May 1981, the combined berm was 5,000 ft long and the east end was 1,100 ft east of its original position. By August 1983 the eastern end of the berm was in about the same position, although the western end had migrated 2,000 ft east and the combined nourishment berm had considerably diminished in elevation.

Area C

By March 1979, this small (10,000 cy) nourishment berm had merged with that of Area B and lost its separate identity. The western end of the placement had migrated about 100 ft downdrift (east) during the same period. By May 1980 the west end had migrated nearly 400 ft from its original placement, and it moved an additional 100 ft by May 1981. By August 1983 the nourishment berm was barely discernible.

Area D

As might be expected, this smallest (9,000 cy) of the nourishment placements was the least successful, partly because the contribution from Area E updrift did not move downdrift as rapidly as expected. By March 1979 the eastern limit had migrated about 1,000 ft east, while the western limit moved only 100 ft. By May 1980 the eastern end had only moved an additional 100 ft downdrift, while the eastern end had been overtaken by nourishment from Area E. By June 1982, nourishment from Area D was no longer discernible. The Area D nourishment berm never merged with that of Area C, thus leaving more than 300 ft of unprotected revetment which suffered some derangement and transport of armor stone during winter storms. On the basis of the results from the Beach Feed Evaluation Test, at least one-fourth of the nourishment volume in areas A through D is assumed to have moved to the foreshore area.

Area E

This largest (45,000 cy) of the nourishment areas was expected to contribute considerable detritus to downdrift areas as well as to protect the revetment adjacent to the Crown Zellerbach plant. However, this segment of the revetment, completed during the fall of 1977, was subject to a season of winter storms and suffered some derangement before placement of beach nourishment the following year. By March 1979 the entire nourishment placement had migrated about 500 ft eastward, and by May 1980 the eastern end had merged with the nourishment material in Area D, while the western end of the nourishment berm had migrated about 1,000 ft eastward from its original position. By June 1982 the entire berm had been reduced, and the western end had migrated a total of 2,100 ft downdrift from its original position. By August 1983 its position was unchanged, but there was further reduction in the berm size. By January 1984 the western end of the berm was 600 ft east of the original eastern end of placement, a total migration of 3,600 ft that was characterized by a major reduction in volume. On the basis of data developed during the Beach Feed Evaluation Test, an estimated two-thirds of the nourishment volume from Area E is assumed to have moved into the foreshore area.

Foreshore Conditions

Monitoring of the foreshore area was accomplished by hydrographic surveys and, in October 1983, a side-scan sonar survey coupled with grab sampling of bottom materials was completed. In general, the hydrographic surveys showed considerable restoration of the nearshore beach profile between MLLW and -10 ft, seaward of which little change had taken place.

Interpretation of data from the side-scan sonar and sampling mission (Figure 5) indicates considerable nourishment deposition in the foreshore areas of the neck and distal bulb nearly to depths of 5 fathoms. Very little is noted offshore of the root. However, perhaps the most disturbing revelations of the sonar survey were several areas along the outer neck and distal bulb which are suggestive of numerous submarine slope failures. Some of the failures cut areas interpreted as reworked beach nourishment. The sonar indicated that the floors of these "slide scars" lay 9 to 25 ft below the adjacent foreshore elevations. While such features are hardly surprising on such steep (4.3H to 1V) submarine slopes dropping to 50 fathoms, their positions were previously unknown.

REHABILITATION

By the winter of 1984, a year past the anticipated rehabilitation by beach nourishment, the mid-tide shoreline was impinging directly on the revetment toe over much of the length of Ediz Hook, and the only substantial berm of beach nourishment remaining was an approximately 700-ft-long section of Area B. The remainder of the nourishment material had been redistributed along the littoral zone, and some had moved on to the foreshore. Much of the rock blanket along a portion of the distal bulb had been reworked and incorporated into beach nourishment in this area. The major problem was the continued distribution of debris, together with littoral material, onto the U.S. Coast Guard aircraft runway during heavy storm periods, a question the original construction did not address (U.S. Army Corps of Engineers, 1984).

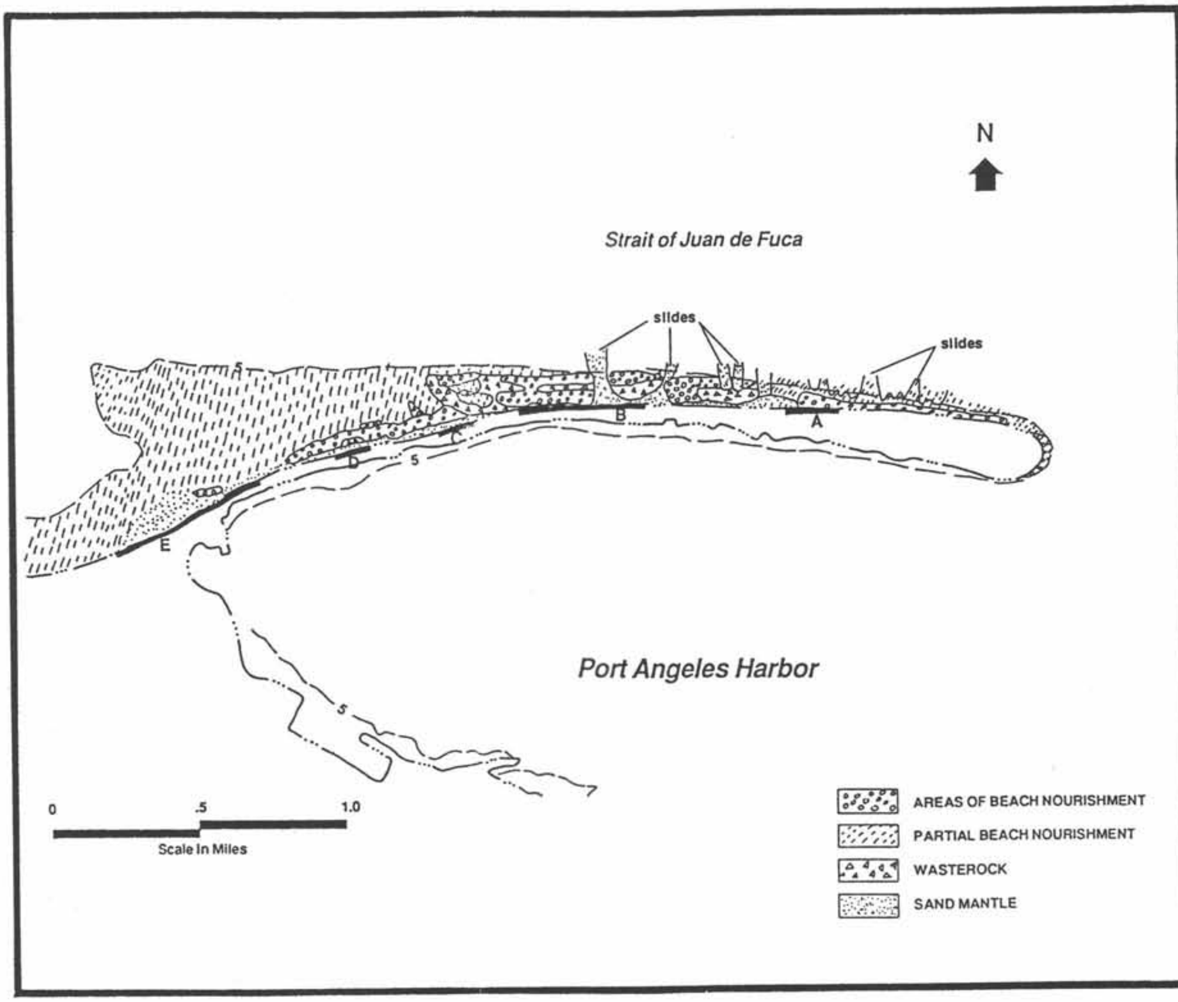

Figure 5. Ediz Hook. Project condition in October 1983 from interpretation of side-scan sonar and grab sample data to approximately the 5-fathom contour.

The 1985 rehabilitation and maintenance program included placement of 30,000 cy of beach nourishment in two areas, the B/C area and Area E (Table 1), and rekeying of about 500 armor stones that had become deranged from the revetment as a result of insufficient beach nourishment (Figure 6). In addition, the revetment was extended eastward 2,100 ft over the former rock blanket area to mitigate the problem of storm debris on the Coast Guard runway. Design of the new revetment was similar to that of the original revetment. Rock was obtained from the Haller quarry south of Sequim and from Mats Mats quarry near Port Ludlow. About 30,000 tons of armor rock and 6,000 tons of bedding rock were used.

CONCLUSIONS

The concept of a rock revetment the integrity of which is maintained by a minimum of high caliber beach nourishment appears to have had satisfactory results at Ediz Hook. However, because of placement of minimal quantities of beach nourishment, during both initial project phases (1977-78) and first maintenance rehabilitation (1984), there is little or no conservatism built into the nourishment program. Thus the 5-yr scheduled maintenance program requires strict adherence or damage to the revetment will likely result. Deposition of a substantially larger quantity of relatively inexpensive beach nourishment would lengthen the periods between required maintenance. Ultimately portions of the beach nourishment reaching the distal end of the hook could be artificially recycled back to the root, reducing the amount of new material required from upland sources.

The removal of the two dams on the Elwha River has been a subject of discussion in recent years. Such action would have a positive effect on the nourishment of Ediz Hook, but would probably not eliminate the need for artificial nourishment. Even the positive effects from removal of the industrial water line and its protective works from the beach west of the hook, allowing the bluffs to erode and further restore natural nourishment, would not eliminate the requirement for some artificial nourishment. Man has now taken over a geologically

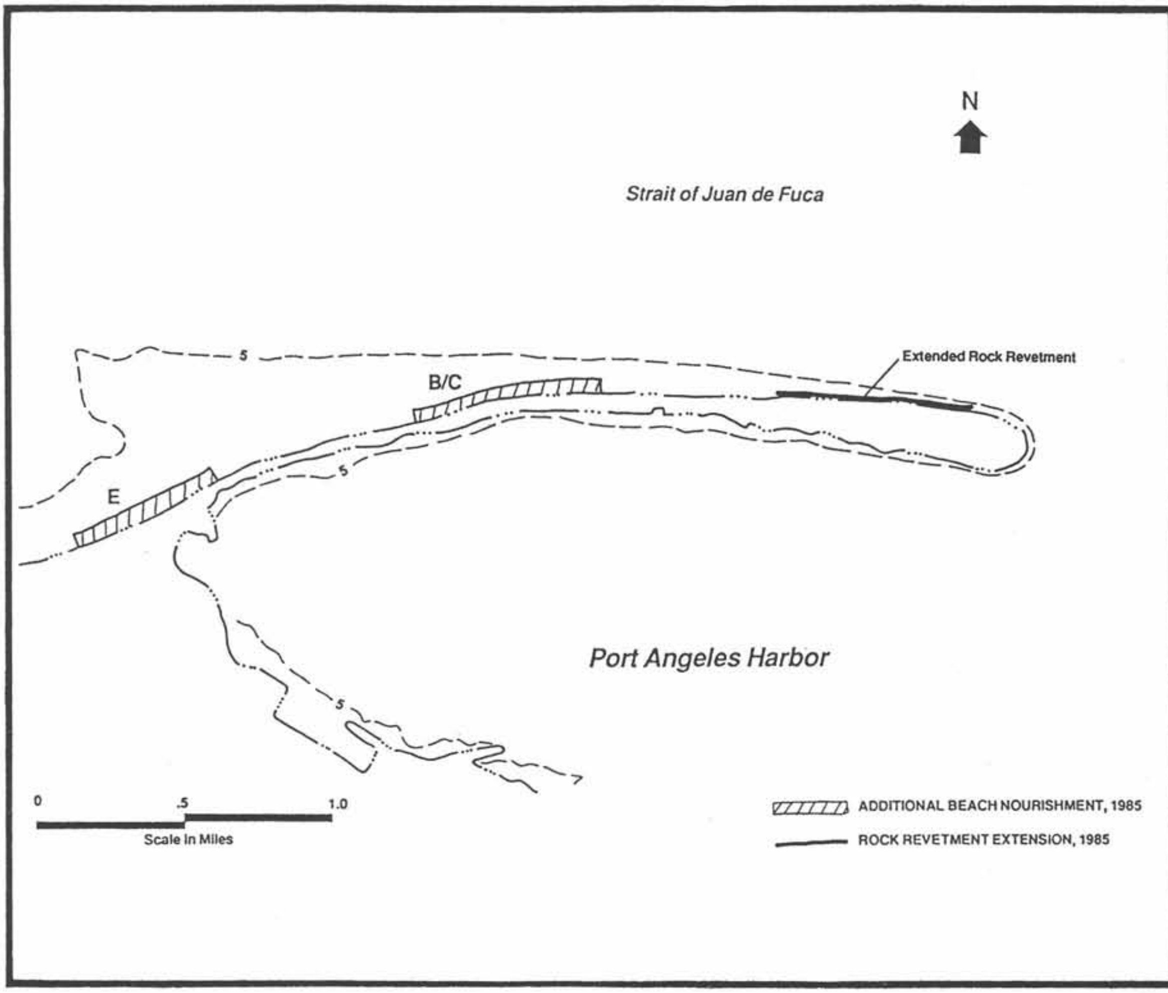

Figure 6. Ediz Hook. Project rehabilitation in 1985 (5-fathom contour shown).

ephemeral landform that was in the process of change for several thousand years. We now wish to keep its position stable so that it will continue to protect the harbor and so its surface may be utilized for commercial purposes. No longer can we permit it to migrate landward by occasional breeching and overtopping. Therefore, we are required to periodically maintain the feature in perpetuity so long as a harbor at Port Angeles is an economic entity.

ACKNOWLEDGMENTS

In addition to the referenced reports, I wish to acknowledge the work of J. McBain, M. Satter, and R. D. Eckerlin of the Seattle District, U.S. Army Corps of Engineers, for their assistance in monitoring of the Ediz Hook project during my involvement with it. I also wish to express my appreciation to A. D. Schuldt of the Corps' Engineering Division, whose discussion over the years have been most valued, and to the late J. M. Nelson, whose early geological investigations lent much to my understanding.

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Engineering Geology in Washington, Volume II
Washington Division of Geology and Earth Resources Bulletin 78

Geologic Aspects of Navigation Canals of Western Washington

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INTRODUCTION

The abundance of deep waterways and harbors in the protected inland waters of western Washington and the easy access to them by way of the Strait of Juan de Fuca have made the necessity for canal construction a rare occurrence. Although a number of shallow harbors and estuaries have required and continue to require dredging in order to permit use of these waterways by large vessels (and in some cases, small craft), only three true canals exist, all constructed during the early part of the 20th century. An ambitious plan for a canal connecting the southern end of Puget Sound with Grays Harbor, with additional connections between Grays Harbor and Willapa Bay and between Shoalwater Bay and the Columbia River, has not yet come to fruition. Thus the Lake Washington Ship Canal, Port Townsend Canal, and Swinomish Channel represent the only true navigation canals in the state. The construction of all three canals predates the advent of modern engineering geology and geological considerations were somewhat peripheral to their siting design and construction. However, each has different geological conditions associated with it, making interesting contrasts in early 20th century construction.

Unless otherwise stated, elevations through this discussion refer to the level of mean lower low water (MLLW) rather than mean sea level (MSL). The difference between the two elevations varies from location to location.

LAKE WASHINGTON SHIP CANAL

General

Probably no single project in Washington has had such a profound influence on a major urban area as the Lake Washington Ship Canal (LWSC) in Seattle (Figure 1). Constructed during the second decade of the 20th century, the canal is taken for granted by much of the local population. Yet its influence on the development around lakes Washington and Union and the drainage alteration resulting from its construction changed the face of the region remarkably. Were such a project to be proposed now, it would likely be doomed to failure through the process of environmental impact statements, public hearings, and court actions. As it was, the project had a 57-yr history of investigation and false starts, from 1854 until construction actually began in 1911 (U.S. Army Corps of Engineers, 1969).

Historical Review

Prior to construction of the canal, the level of Lake Washington varied between 29 and 33 ft, with a nominal elevation of 30 ft. The lake basin drained via the Black River at the south end of the lake to the Duwamish River at Tukwila and thence into Elliott Bay (Figure 2). The Cedar River joined the Black River about 1/2 mi south of the lake in the center of what is now Renton.

Lake Union, having a nominal elevation of 20 ft, collected local drainage from the surrounding drumloidal hills. Drainage from the lake was conducted by a small creek and as shallow ground water northwest to Salmon Bay, which was little more than a tidal lagoon. Although the idea of joining Lake Washington with Puget Sound was proposed in 1854 by Thomas Mercer, no direct action was taken until Harvey Pike began digging (by hand) a channel across The Portage (between Union Bay and Portage Bay) in 1869. However, Pike soon tired, presumably because of difficult digging in the septum of till that separates the lakes. Finishing the task was left to the Lake Washington Improvement Co., which resumed the work in 1884 to complete a 16-ft-wide log chute across The Portage (Figure 3). The chute had a wooden control works at the Union Bay end (Figure 4). The purpose of these improvements was to move logs from the Lake Washington basin to about 20 saw mills then located on Salmon Bay. The works also included a lock for passage of small vessels between Portage Bay and Union Bay (U.S. Army Corps of Engineers, unpublished data). The company also dug a log transit channel between Lake Union and Salmon Bay; a wooden spillway dam was built at the Lake Union outlet (Figure 5) (U.S. Army Corps of Engineers, 1969). The dam controlled the level of Lake Union at about elevation 21 ft. (The dam failed in March 1914 during LWSC project construction, allowing Lake Union to drop to elevation 12 ft. It was replaced by a temporary

Figure 1. Lake Washington Ship Canal. View SSE from Shilshole Bay to Lake Union and Lake Washington. Part of the Shilshole Bay breakwater and boat basin is at the left foreground, the Magnolia district and Discovery Park/Fort Lawton are in the right foreground. Elliott Bay is on the right. Photo by Seattle District, U.S. Army Corps of Engineers, January 18, 1980.

wooden structure which controlled the level of Lake Union until Chittenden Lock and Dam was completed.) This is the configuration shown on the 1909 edition of the U.S. Geological Survey Seattle quadrangle map.

As the Federal Government became interested in the canal project during the first decade of this century, five possible routes were under consideration (Figure 2). In addition to the final selected route, two alternate routes into Lake Union from Puget Sound were considered: one from Smith Cove through the Interbay area to Salmon Bay and thence into Lake Union (A, Figure 2), and a second along "Mercer's Farm Line" between Queen Anne Hill and Denny Hill, through what is now the Seattle Center, to the south end of Lake Union (B, Figure 2). At that time, the regrading of Seattle's central hills was in progress, and the latter route appeared to be a viable alternative. Two other alternate routes avoided Lake Union. The shorter was a straight east-west route (south route) between Elliott Bay and Lake Washington just south of the present Dearborn Street cut between Beacon Hill and First Hill (C, Figure 2). Its construction would have entailed deep cuts through Beacon Hill and Mt. Baker Ridge. The longer route considered was by way of the Duwamish and Black rivers to Lake Washington at Renton (D, Figure 2). The length of the Duwamish-Black River route and the likelihood of some rock excavation effectively eliminated that route. The "south" route through Beacon Hill and Mt. Baker Ridge was eliminated partly due to the recognition of potential major landslide problems that would be associated with the required deep cuts. In fact, the 1903 report indicated that it was hard to find areas in Seattle over 1 to 2 mi where sliding on clay was not in evidence (U.S. Congress, 1903). The final route was selected as the route of least resistance in terms of excavation quantities and cost (U.S. Congress, 1903).

During early project formulation, a second lock to be located at The Portage was considered so that the level of Lake Washington might be maintained. Arguments for lowering the lake included reduction of flooding in the Duwamish valley by the Cedar River diversion (U.S. Congress, 1908).

Project Description

The canal project is 8 mi long (Figure 6) and consists of a 5,500-ft tidal approach channel from Shilshole Bay to Chittenden Lock and Dam, the main feature of the project. The dam is a 55-ft-high concrete gravity structure; its gated spillway crest is at an elevation of 30 ft. The large double lock chamber is 825 ft long and 80 ft wide. The small lock chamber is 150 ft long and 30 ft wide. Both chambers are contained by monolithic concrete gravity walls having maximum heights of about 50 ft. Both locks are controlled by miter gates.

Additional elements included deepening of Salmon Bay and excavation of the Fremont cut between Salmon Bay and Lake Union and the Montlake (Portage) cut between Portage Bay and Union Bay on Lake Washington, north of the site of the log chute. The tidal approach channel extends to about elevation -34 ft, and the canal

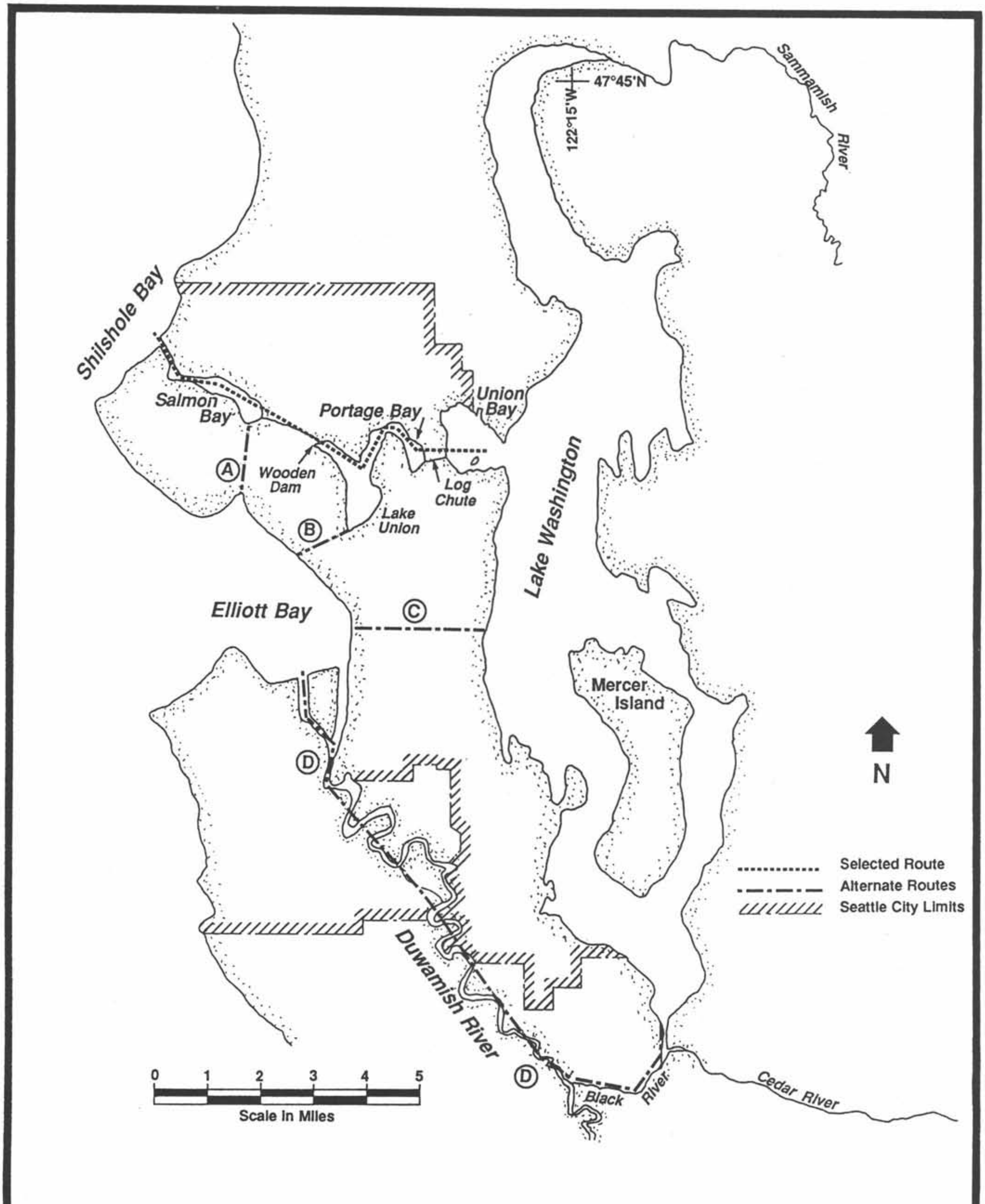

Figure 2. Drainages configuration in the Seattle area prior to construction of the Lake Washington Ship Canal showing alternate canal routes considered.

Figure 3. Lake Washington Ship Canal. Portage log canal in about 1911; view to the west showing steep cut in Vashon till and an early edition of the Montlake Bridge. Photo from U.S. Army Corps of Engineers collection.

behind the lock and dam extends to about -10 ft. The level of the somewhat expansive reservoir, including Lake Union and Lake Washington, normally fluctuates annually between elevation 20 and 22 ft.

Geology and Construction

Chittenden Lock and Dam

The monolithic concrete gravity structures that comprise both the spillway dam and the lock walls are founded on glacially overconsolidated clay, probably the Lawton Clay Member of the Vashon Drift (U.S. Army Corps of Engineers, 1982b). The dam foundation lies at about elevation -25 ft. The large lock walls are founded at about elevation -34 ft (Figure 7), and the south wall of the small lock about elevation -15 ft. The contact between the Lawton Clay and the overlying Esperance Sand Member (Vashon Drift) lies at about elevation -5 ft on both abutments, with a transition zone of interbedded sands and silts above that elevation. Standard penetration tests in the silty clay of the Lawton unit range from 44 to 78 blows/ft with the average about 60 blows/ft. Test values for the overlying silts and sands of the transition zone and Esperance unit average slightly lower but have a smaller range. Core borings drilled in 1980 through the concrete dam and into the foundation revealed a tight foundation contact. The Esperance is present at elevation 40 ft on the south shore and at elevations 18 to 20 ft on the north shore; about 10 ft of fill overlie it in the low area of the project

Figure 4. Lake Washington Ship Canal. Wooden control works at Union Bay (east) end of Portage log canal, 1911. In addition to the log chute head gate, the structure included a wooden lock (right) for transit of small craft between Lake Union and Lake Washington. View to the northeast across Union Bay to what is now the Laurelhurst district. Photo from U.S. Army Corps of Engineers collection.

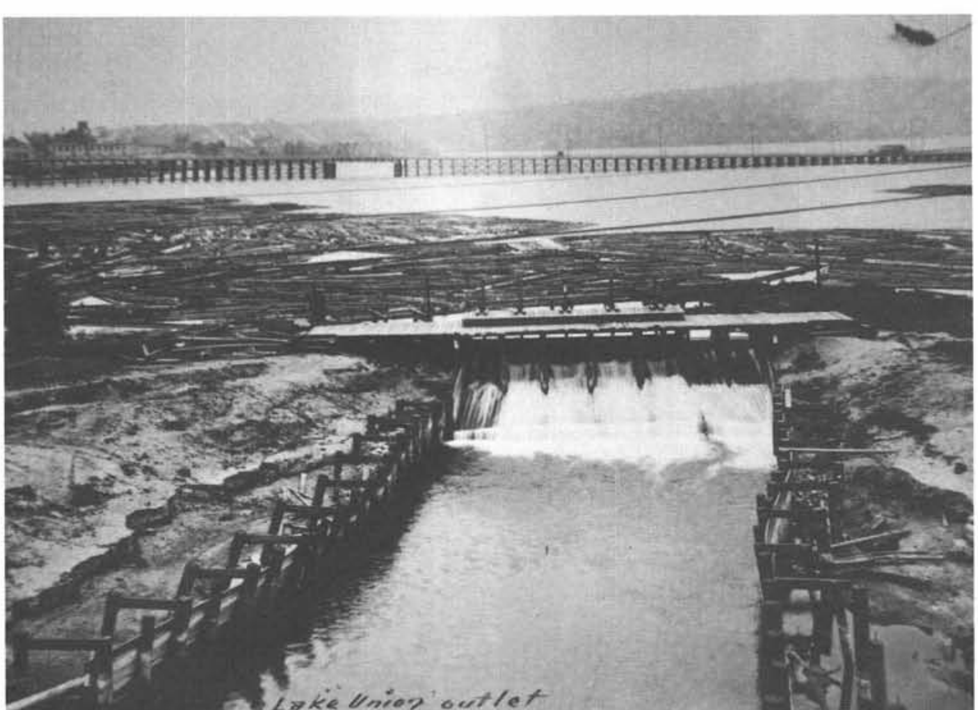

Figure 5. Lake Washington Ship Canal. Original wooden control structure at Lake Union outlet just east of the present Fremont Bridge as seen from the west. The structure failed in March 1914 and was replaced by a second wooden structure pending project completion. Photo from U.S. Army Corps of Engineers collection.

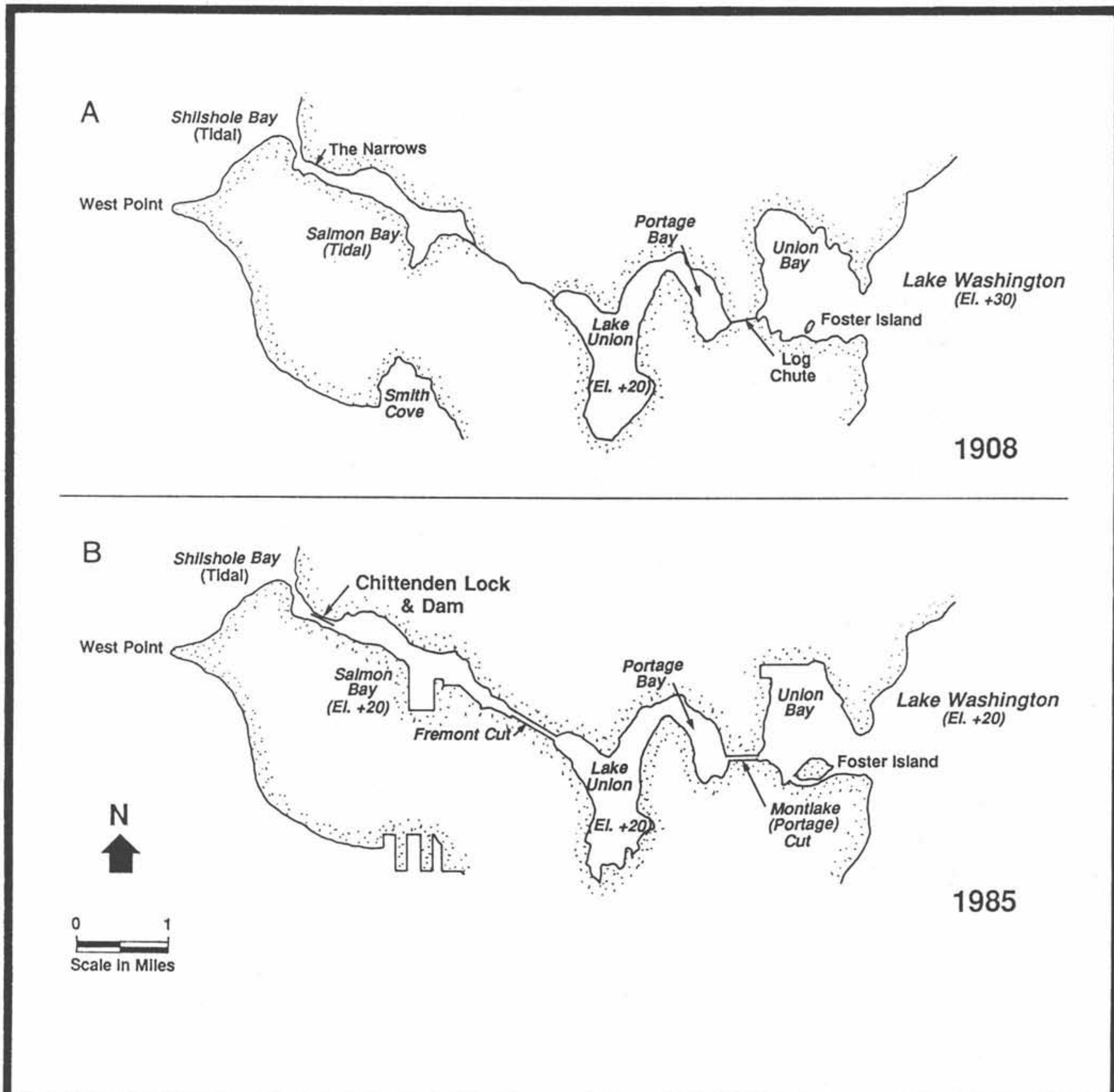

Figure 6. Features of the Lake Washington Ship Canal prior to construction in 1908 (above) and in 1985.

facilities (Figure 7). The backfill of excavated material lies against the north wall of the large lock for the full height of the wall. The high water table combined with the generally unconsolidated mixture and varied permeability of the backfill permit a leakage path around the north side of the large lock wall. On the south bank, the fish ladder facility is also founded on the Lawton Clay at about elevation -15 ft, although the upper part of the south wall abuts into the Esperance Sand (U.S. Army Corps of Engineers, 1982b).

Prior to construction, the topography at the Shilshole-Salmon Bay narrows was characterized by a steep slope on the south bank dropping sharply to a channel bottom at about elevation -3 ft. The north bank, though somewhat more gentle, steepened downstream where the low-water channel swung against it. The channel at zero tide was about 150 to 180 ft wide, narrowing to less than 20 ft immediately upstream. The siting of the locks was kept tight against the north bank and actually cut into the north bank to permit construction of the temporary

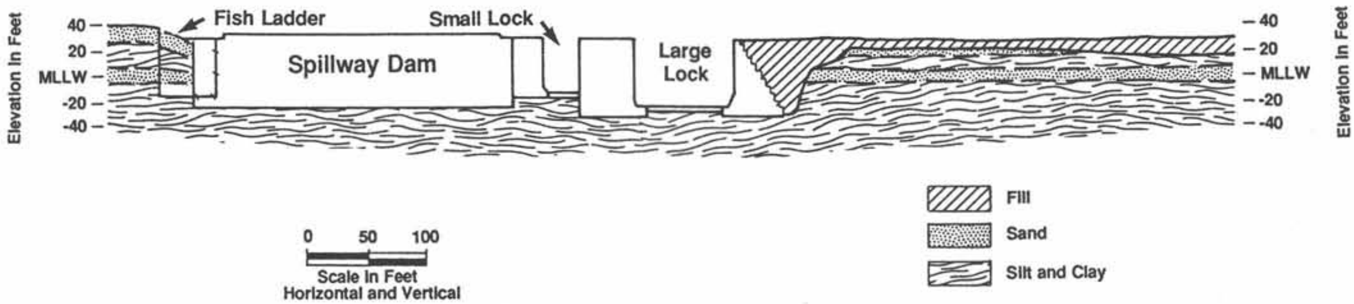

Figure 7. Geologic section through Chittenden Dam and Locks on the Lake Washington Ship Canal. View northwest (downstream) toward Shilshole Bay.

tidal channel around the cofferdam. Soft bay mud and marine sand extended to about elevation -6 ft over much of the cofferdam route but reached about -20 ft downstream and -30 ft upstream. The double-row wood plank pile cofferdam was driven several feet into the underlying "hard clay" to obtain an appropriate seal. Cuts in the clay were made nearly vertical. Attempts to do so in the sand and silt of the transition zone were moderately successful. The sand was again an important consideration on the south bank in 1975 during reconstruction of the fish ladder. Soldier piles and tieback anchors were installed to support the south wall of this excavation. The wall was closely monitored, as was the magnitude of vibration in adjacent apartment buildings during driving of the piles.

Construction in the lock site began with suction dredging of a temporary channel along the south shore of The Narrows late in 1911. The dredged material, mostly soft mud, was deposited along the adjacent Ballard waterfront. The cofferdam consisted of a double row of timber lagged piles with earth fill between and was completed by summer 1912, with dewatering in August. By October, excavation for the locks was well under way (Figure 8), although the first concrete was not placed until March 15, 1913 (U.S. Army Corps of Engineers, unpublished data). Upon completion of the

Figure 8. Excavation for large lock chamber at Chittenden Dam and Locks, Lake Washington Ship Canal. View SSE toward Salmon Bay. Here, the Lawton Clay and the overlying sand and silt are well exposed in the north wall of excavation. Photo by U.S. Army Corps of Engineers, October 1912.

lock concrete monoliths in early 1916, the tidal waters were diverted through the large lock, upstream and downstream double row, pile, timber, and fill cofferdams were built across the temporary channel, and by the end of March 1916, the excavation area for the spillway dam was dewatered. The dam was completed in less than 4 months; on July 12, 1916, the lock gates were closed for about 2 weeks, and the water level of Salmon Bay allowed to rise to the level of Lake Union (*Seattle Times*, July 3, 12, 16, 1916).

Fremont Cut

Prior to the cutting of a log transit channel between Lake Union and Salmon Bay and the construction of wooden control works at the Lake Union outlet in the mid-1880s, the maximum ground elevation of the isthmus was about 30 to 35 ft, only 10 to 15 ft higher than the normal level of Lake Union. A small stream flowed from Lake Union to Salmon Bay. A portion of the area was swampy, and ground water presumably also moved northwestward. The low area was underlain by postglacial alluvium/colluvium in the form of mud, sand and soft clay, which was underlain by hard clay or hard pan, presumably Vashon Till in the eastern portion and possibly Lawton Clay in the western portion. Borings drilled in 1901 and 1902, after the log canal had been in use for some time, showed the clay till at elevations varying between 2 and 8 ft east of 3rd Avenue N.W. near the midpoint of the cut and at elevations of -5 to -10 ft west of this area. The till surface dropped to about -12 ft near Salmon Bay. However, there were local departures from these depths where the soft material extended to lower elevations and the hard till was at higher elevations.

Construction of the ship canal through this area was done during 1911 and 1912 largely by hydraulic sluicing and steam shovel (Figure 9). Much of the material was disposed of on adjacent lands as a pumped slurry. The predominantly soft material in the cut was found to be highly susceptible to erosion, requiring a temporary pile and timber revetment for control. This was replaced several years later by the present concrete revetment, which extends several feet below canal water level and rests on piling backed by brush mats (fascines). Although several minor failures have occurred through the fascine and pile section over the years (largely as a result of excavation for placement of buried utilities), the basically unlined channel has served well.

Montlake (Portage) Cut

The 0.4-mi-wide isthmus that separated Portage Bay on Lake Union and Union Bay on Lake Washington consisted of sandy, gravelly Vashon Till and associated intra-till fluvial sediments. The isthmus was a ridge that

Figure 9. Excavation of Fremont cut, Lake Washington Ship Canal, in 1912. View to the northwest showing sluicing and pumped slurry activities for excavation and material transport. Photo from U.S. Army Corps of Engineers collection.

reached an elevation of about 80 ft. The construction of a log chute through the isthmus in the mid-1880s and the wooden control structure at the Union Bay outlet resulted in a narrow, near-vertical cut through the till which withstood serious erosion for a quarter of a century (Figures 3 and 4). The log chute was cut a short distance south of the present Montlake cut and exited into Union Bay southeast of the present Seattle Yacht Club; it was backfilled following completion of the cut. Borings drilled in 1911 showed that the till surface beneath the Portage Bay section of the canal varied from MLLW to -10 ft; a local rise to elevation 18 ft exists on the south side near the present University Bridge. On the Union Bay (Lake Washington) side, the till surface dropped off sharply to well below MLLW, and the channel into Lake Washington was largely dredged in mud and soft clay, an ooze which was later found to extend to considerable depths. The single exception to this was just north of Foster Island, where a narrow rib of till rose to elevation 25 ft, a mere 5 ft below the nominal level of the lake.

Construction of the Montlake cut took place between 1912 and 1914. Excavation was mainly by steam shovel, and material was removed by muck cars on rails (Figure 10). Concern for bank erosion resulted in construction of concrete walls on both banks. The south bank concrete wall, completed in 1914, was founded at elevations -12 to -15 ft, slightly below the channel bottom. The north bank concrete revetment was constructed after the canal was operational and was founded on a spread footing at elevation 19 ft. Current erosion gradually undermined its foundation, and in 1947 steel sheet piling was driven on the canal side of the wall to about elevation 2 ft, about 12 ft above the canal bottom. In February 1982, a 245-ft-long section of the wall and sheet pile structure failed because of continued erosion on the channel side of the sheet pile structure. The wall was rebuilt with sheet piling extending to elevation -10 ft. The canal floor and the north submarine channel slope were reinforced with riprap to prevent further erosion (U.S. Army Corps of Engineers, 1982a).

The final cut to change the outlet of Lake Washington from the Black River to Montlake was made in August 1916, permanently rerouting the drainage of the Cedar and Sammamish basins through the Lake Washington Ship Canal. At 2:00 p.m. on August 25, the cofferdam on the Portage Bay (west) end of the cut was breached (Figure 11), and the water from Lake Union filled the cut in about an hour (*Seattle Times*, August 26, 1916). Three days later, gates at the temporary wooden control works on the Union Bay (east) end were opened. The level of Lake Washington gradually dropped in 4 to 5 weeks, exposing a wave cut terrace around much of its periphery. On the Seattle

Figure 10. Excavation of Portage (Montlake) cut. View to the east on August 12, 1912. Photo from U.S. Army Corps of Engineers collection.

Figure 11. Breaching of cofferdam at west end of Portage (Montlake) cut, Lake Washington Ship Canal, August 25, 1916. View southwest across Portage Bay to Capitol Hill. Photo from U.S. Army Corps of Engineers collection.

side, far-sighted Seattle planners acquired the newly exposed (state-owned) land, which has since become Lake Washington Boulevard Park (U.S. Army Corps of Engineers, 1969). Total excavation for the project was about 4 million cy, and total concrete was about 230,000 cy. Cost of the project was about $5 million (*Seattle Times*, July 1, 1917).

Dredging

The approach channel between Shilshole Bay and Chittenden Locks required dredging by both hydraulic suction and dipper dredges. The softer muds and marine sand, which characterized much of the approach channel to about elevation -23 ft, were largely removed by hydraulic dredge, while removal of the till or hard clay required the dipper dredge. Final dredging was not completed until early 1917. The remainder of the dredging, including the deepening to Salmon Bay and its expansion, together with deepening in northern Lake Union, Portage Bay, and Union Bay, was largely in soft mud and sand and was presumably accomplished by hydraulic dredging and nearby land disposal. Thus, many areas along the canal are underlain by fine dredge spoils, providing numerous local foundation problems for the variety of commercial structures that line the canal.

Earthquake Effects

Chittenden Locks and Dam survived Modified Mercalli Intensity (MM) VIII during the magnitude 7.1 earthquake of 1949 and MM intensity VII during the magnitude 6.5 earthquake of 1965 without damage. Minor cracking of ancillary buildings founded on fill was experienced. No damage to canal walls was reported (U.S. Army Corps of Engineers, unpublished data).

Analyses completed in 1982 indicated that the locks and dam would survive a magnitude 7.5 subcrustal earthquake located directly beneath the structure without significant damage (U.S. Army Corps of Engineers, 1982b).

PORT TOWNSEND CANAL

The existing canal connecting the heads of Port Townsend and Oak bays provides a calmer passage for small tows and small craft and avoids the often turbulent water at the entrance to Admiralty Inlet. The canal is 4,800 ft long and has a controlling width of 75 ft and a controlling depth to elevation -15 ft.

The original configuration of the canal area (Figure 12) was a narrow tidal inlet off Port Townsend Bay, constricted on the north by sandstone bedrock walls (the

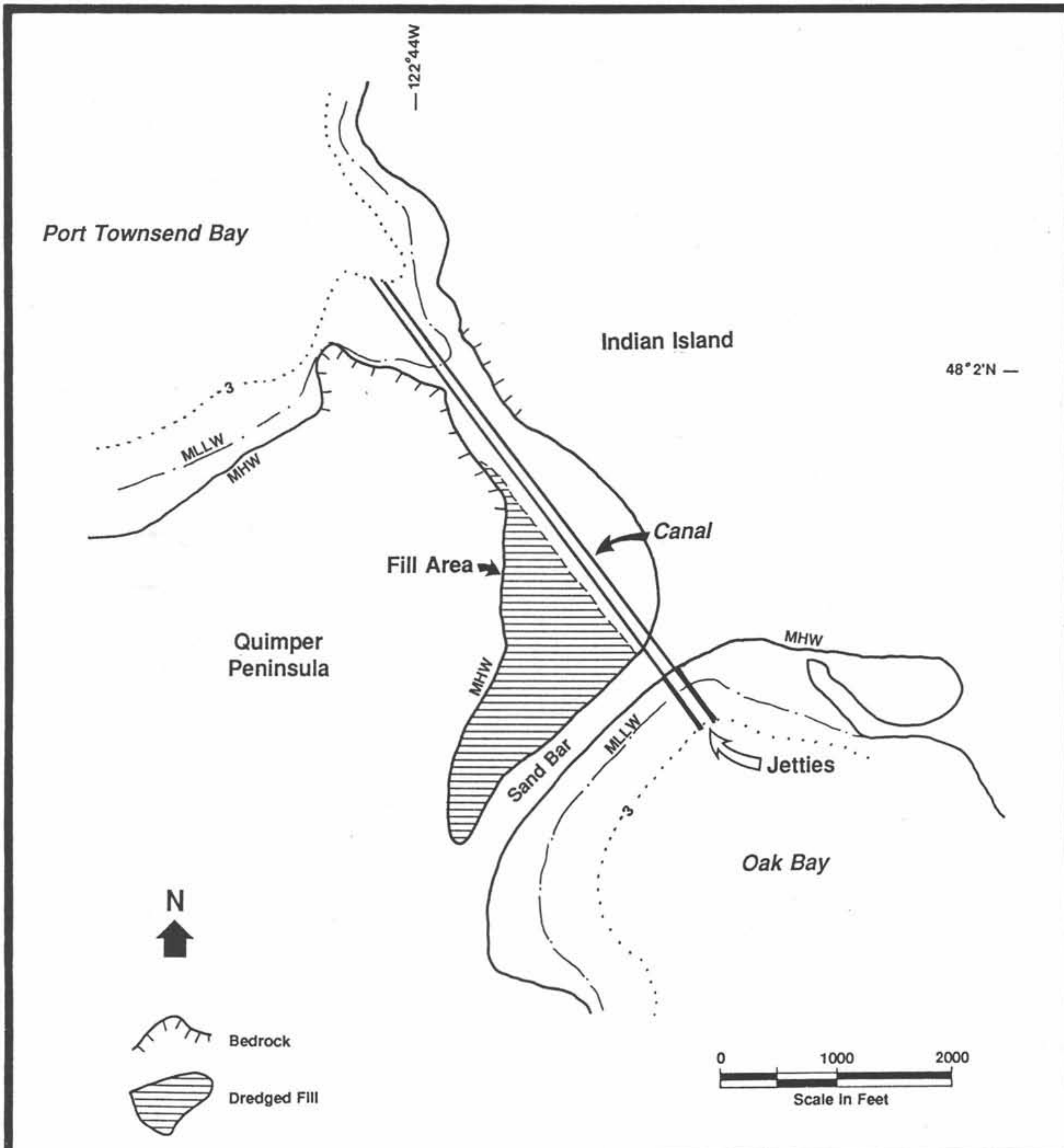

Figure 12. Port Townsend Canal area showing shoreline configuration and 3-fathom contour prior to construction and former tidal area filled with dredged material. MLLW, mean lower low water; MHW, mean high water.

site of the present highway bridge) and a 450-ft-wide sand and gravel barrier beach rising to 3 ft above high water at the southern (Oak Bay) end (U. S. Congress, 1912). The barrier beach was all that separated the two bays at high tide.

The earliest survey for a canal at this location was made in 1889 by R. Kendrick, who was then the superintendent of Hadlock Mills. The early survey mentioned the "sandstone formations" on both sides of the proposed canal and the "igneous and volcanic origin of the [detritus in the] sand and gravel bar" (U.S. Congress, 1890). At that time the economic future for the Port Townsend-Hadlock area was bright indeed, with a lumber mill, alcohol plant, and steel mill in the Hadlock area and mills and a port of entry at Port Townsend. The canal would ease water access between Port Townsend Bay and ports on Puget Sound.

The project was authorized by Congress in 1912. Construction began in 1913, and the project was completed in 1915. The canal required dredging of more than 265,000 cy of sand, gravel, and silt and removal of a sandstone ledge near the northern end by blasting (4,500 cy of rock excavation). Charges were set by a diver from the Government snag boat *Swinomish* (U.S. Army Corps of Engineers, 1915). Two jetties were constructed to elevation +14 ft at the south end of the project to control shoaling. The east jetty is 600 ft long and is of rock-reinforced pile and whale construction with a brush core. The west jetty is a 550-ft-long rubblemound structure constructed on the ruins of the original jetty which was presumably of pile and whale with brush core construction. Both jetties were founded on soft sand and bay mud and have suffered considerable settlement. An 80-ft-long low area in the east jetty was discovered in 1959 and repaired in 1961. A 2,100-ft-long pile and timber bulkhead extending north across the tidal area from the root of the west jetty was part of the original construction behind which much of the dredged material was placed to elevation 14 ft.

The tidal range in Oak Bay is about 2 ft greater than that in Port Townsend Bay; the greatest elevation differential is 1 ft. Shoaling within the canal has been a frequent problem since initial dredging. Maintenance dredging is required at about 10-yr intervals. In 1961, 1974, and 1984, dredges removed 8,500 cy, 20,000 cy, and 21,500 cy of material, respectively (U.S. Army Corps of Engineers, unpublished data). At its north end the canal is frequently encroached upon by a coarse sand and gravel shoal from the west. The canal was completed a scant 2 yr before economic tragedy hit the area, closing down the mills. Plans to widen the canal to 100 ft and deepen it to -25 ft were proposed in 1918 (U.S. Congress, 1918) but, with the economic downturn, were never carried out. Small craft continue to use the canal, however.

SWINOMISH CHANNEL

Swinomish Channel, originally a shallow tidal channel along the boundary between the eastern edge of Fidalgo Island and the distal end of the Skagit delta, connects Skagit Bay (Saratoga Passage) on the south with Padilla Bay on the north (Figure 13). It is probably the oldest navigation project in the state, and it has the longest history of construction, having been started in 1893 but not completed until 1937. The natural channel or slough had depths as shallow as 1 ft over substantial portions. Although the distal portions of the Skagit delta were (and still are) characterized by numerous active and abandoned distributaries, all influenced by tidal waters, Swinomish was the only one with through-going tidal action.

The purpose of the channel is to provide tow and small craft navigation in protected waters (behind Whidbey and Fidalgo islands) and eliminate the transit through Deception Pass which for many vessels requires slack or appropriate tides. Alternative routes were considered between Similk Bay on the south and Fidalgo or Padilla bays on the north (A and B, Figure 13) (U.S. Congress, 1914). Either alternate route would have provided a shorter canal length but greater excavation in glacial drift and bedrock.

The delta adjacent to most of the Swinomish slough is underlain by 4 to 8 ft of silty peat and humus, which in turn is underlain by loose, fine deltaic sand. Farmers on land adjacent to the tidal sloughs used the peat-humus for construction of dikes to keep tidal waters out of their fields. West of Swinomish, the eastern portion of Fidalgo Island, which is a bedrock upland mantled by glacial drift, rises from tidewater. The southern end of the slough passes around and through a rock promontory; the western channel, bounded by bedrock, is known as Hole in the Wall. Several rock islands protrude through the deltaic sand and silt of Skagit Bay. Others may have been drowned by the westward encroachment of the Skagit delta.

The channel is 11 mi long and has a project width of 100 ft and a project depth of 12 ft. It consists of three segments: (1) The southern 2.0-mi traverses the shallow portion of Skagit Bay between parallel dikes; (2) a narrow 0.7 mi segment passes through Hole in the Wall; and (3) the main 8.3-mi segment crosses the delta. The third segment includes a 2.7-mi channel dredged to water of navigable depth in Padilla Bay. Original dredging in 1893-1894 deepened only the southern two-thirds of the channel to elevation -4 ft, and dredging of the entire channel length to -12 ft was not completed until 1937. Controlling shoaling in the loose, silty sand underlying Skagit Bay required parallel dikes across this reach. The north dike consists of a double row of wood piles connected by plank whales and flanked by a rubble mound structure to elevation 8 ft resting on a fascine mattress below MLLW. The south dike is an unzoned rubblemound structure also resting on a fascine mattress. These dikes were apparently long-term projects; construction started in 1894 and was not completed until 1935. A 6,000-ft-long dike or breakwater was constructed between McGlinn Island and Goat Island in 1937-1938 to reduce shoaling caused by flood sediments from the Skagit North Fork distributary. As with the other dikes, it was a rubblemound structure founded on fascines on the soft deltaic sediments. The structure, along with the south dike, was damaged by storms in 1940 and repaired. Between 1941 and 1944 the breakwater had settled 3.5 ft over its entire length, and in 1946 it was rebuilt to elevation 15 ft. It was again rebuilt in 1965, owing to additional settlement. The south dike west of Goat Island was rebuilt in 1973. Much of the rock used for original construction and

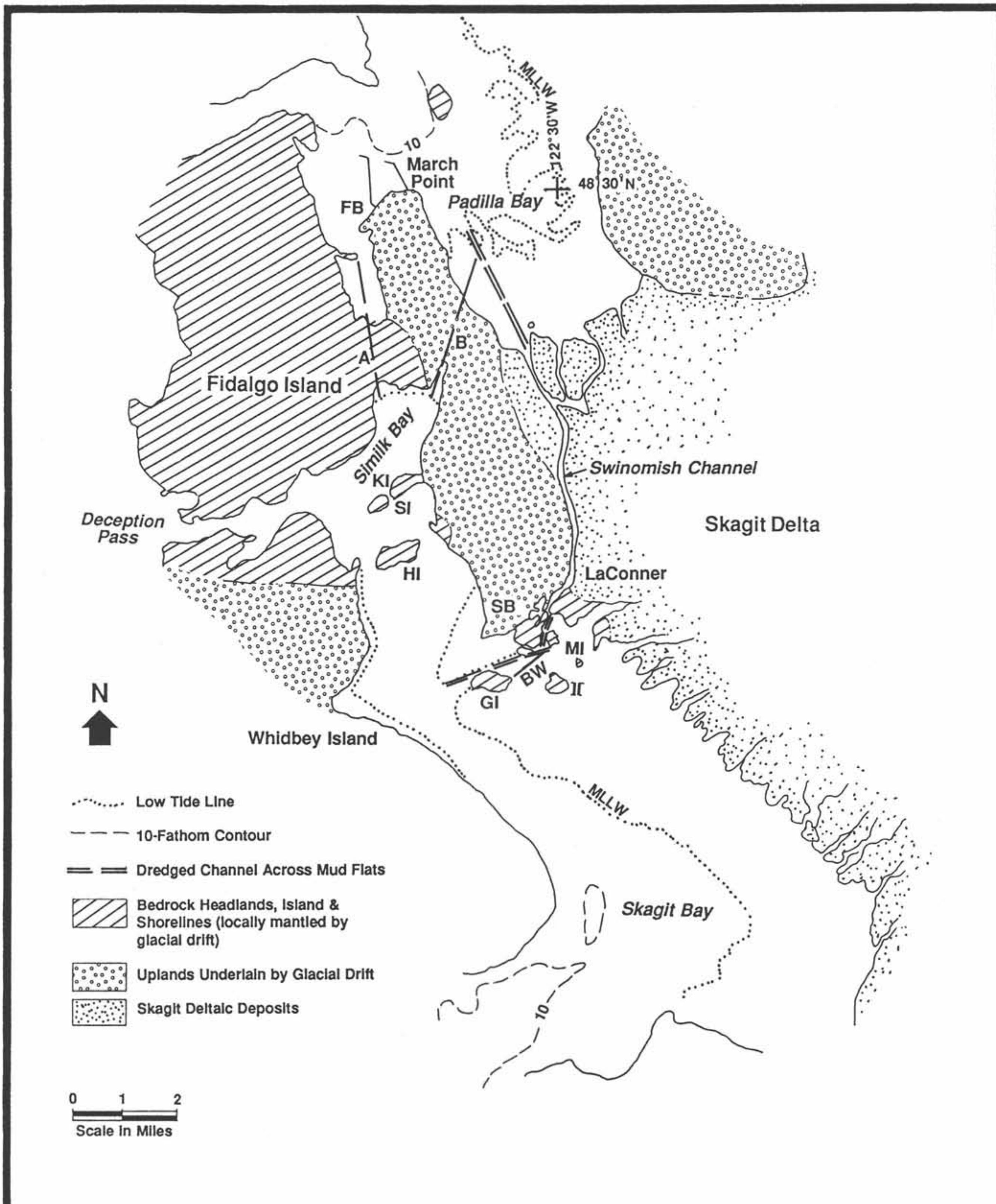

Figure 13. Generalized geology of the Swinomish Channel area. BW, breakwater; FB, Fidalgo Bay; GI, Goat Island; HI, Hope Island; II, Ika Island; KI, Kiket Island; MI, McGlinn Island; SB, Shelter Bay; SI, Skagit Island. A and B, alternate routes considered.

rebuilding of the dikes was obtained from a quarry developed on Goat Island.

Dredging of the main 8.3-mi channel has not been without problems. Increased wave action from boat wakes and increased tidal current velocities in the improved channel have resulted in considerable erosion of the loose sand from unprotected channel banks. Many of the sod dikes that protect crops from seawater, were undermined or threatened. Collapse features, caused by withdrawal of sand, formed behind the dikes. Even though the dikes were 100 ft or more from the original bank, additional set-back was required. Many of the narrow sod dikes were widened and raised by the disposal of hydraulically dredged material. Erosion and channel deepening along the La Conner waterfront has resulted in settlement of sidewalks and exposure of piles.

Sand waves formed by strong tidal currents have been identified in the central part of the channel. These have amplitudes of 2 to 8 ft with the steep side facing north (U.S. Army Corps of Engineers, 1981).

The combination of bank erosion and channel shoaling from Skagit River sediments has required from 45,000 to 140,000 cy of maintenance dredging every year or two. Reducing the maintenance depths has reduced both the shoaling rate and the amount of required maintenance dredging in recent years. This dredging has required land disposal, which has locally caused transient contamination of nearby shallow water wells. Land disposal on productive agricultural lands, no longer acceptable to local government, has been partly replaced by open-water disposal or disposal in tidal areas. A moderate amount of bank protection has been constructed. Channel straightening through the Hole in the Wall section in 1965 required rock excavation. The waste rock was placed along sections of the channel for riprap protection.

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The 1980 Eruptions of Mount St. Helens: Engineering Geology

Robert L. Schuster, Chapter Editor

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Engineering Geologic Effects of the 1980 Eruptions of Mount St. Helens

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INTRODUCTION

The May 18, 1980, eruption of Mount St. Helens (pre-eruption view, Figure 1), a 2,950-m volcano in the Cascade Range of southwestern Washington, began with the collapse of the bulging northern sector of the cone of the volcano. The cone had been weakened by nearly 2 months of earthquakes, hydrothermal activity, and magmatic intrusion. This collapse immediately followed a magnitude 5 earthquake and resulted in a 2.3-cu-km rockslide which degenerated into an enormous, hot debris avalanche (Figure 2) that swept some 24 km down the valley of the North Fork Toutle River (Figure 3). After collapse of the north flank of the volcano, the main eruption began with a directed blast that devastated the area north of the mountain to a distance of more than 25 km from the pre-eruption summit. These events in turn triggered a 9-hr dacitic eruption that drove a column of ash more than 20 km high (Figure 4) and produced pumiceous ash flows on the volcano's north flank (Christiansen and Peterson, 1981). Ash fall-out was recorded more than 1,500 km to the east.

For the next several hours, the debris avalanche was followed by floods of water derived from melting glaciers and snow. The inundation remobilized parts of the already-saturated debris avalanche to form large debris flows/mudflows. These flows (which I will call mudflows for simplicity) continued downstream for 95 km beyond the distal margin of the debris avalanche, modifying a total of more than 120 km of river channel, including the main Toutle River and stretches of the Cowlitz and Columbia rivers (Figure 3) (Janda and others, 1981).

In addition to the flows derived from the debris avalanche, mudflows formed in other valleys from mixtures of catastrophically ejected coarse lithic debris, lapilli, ash, and water and entrapped air and from debris-laden snow and ice on the volcano. The largest of these mudflows coursed the entire 45-km length of the South Fork Toutle River into the main Toutle River where it joined the mudflow from the North Fork Toutle River (Figure 3). Smaller mudflows occurred in almost all drainages from the volcano, with significant flows along the Lewis River and its tributaries (Figure 3).

In spite of the fact that the volume of ejecta from the May 18 eruption was relatively small compared to amounts from several other well-known eruptions (Figure 5), destruction from the eruption and associated processes was significant. More than 60 people were killed, a number that would have been much larger but for pre-eruption evacuation of a high percentage of the area's residents, employees, and tourists. All vegetation on about 600 sq km of public and private forest land was destroyed or badly damaged (U.S. Department of Agriculture, 1981). All fish and wildlife in the immediate blast zone were killed, as were most fish in streams draining the area.

Total economic losses in the State of Washington due to the May 18 eruption and the succeeding lesser eruptions of May 25 and June 12 have been estimated at $970 million (MacCready, 1982). The largest losses consisted of forest damage, about $450 million. Cleanup costs amounted to $363 million. Property valued at $103 million was destroyed or damaged. Agricultural losses were about $40 million. The vast majority of these losses, including all timber losses, over two-thirds of all cleanup costs, and well over one half of property losses, occurred in the immediate vicinity of the volcano.

The May 18 eruption and associated processes resulted in flooding, burial, and erosion of highways and railways; destruction of bridges and buildings; partial filling of a major reservoir; and disruption of water-supply, sewage-disposal, and flood-control systems in southwestern Washington (Schuster, 1981, 1983). In addition, ash deposited over eastern Washington, northern Idaho, and western Montana hampered transportation and municipal operations. Ash from subsequent smaller eruptions on May 25 and June 12 had a similar, but lesser, effect on southwestern Washington and the Portland, Oregon, area.

The State of Washington, the U.S. Forest Service, the Weyerhaeuser Company (timber production), and Cowlitz County sustained considerable damage and

Figure 1. View of Mount St. Helens and Spirit Lake from the north before the 1980 eruptive activity. Photograph by W. C. Guy, U.S. Forest Service.

Figure 2. Debris avalanche in the upper valley of the North Fork Toutle River. View east from near the distal margin of the debris avalanche toward the devastated cone of Mount St. Helens, which is partially obscured by clouds in the left background. Photograph by Austin Post, U.S. Geological Survey, May 1980.

destruction of transportation facilities due to the eruptions. The movement of ocean-going shipping on the Columbia River between the port of Portland, Oregon, and the Pacific Ocean was severely disrupted by sediment deposited in the Columbia River from the mudflow and flood in the Cowlitz River. Previous flood-control measures in the valleys of the Toutle and Cowlitz rivers, under the jurisdiction of the U.S. Army Corps of Engineers, were disrupted by sediment deposited in these rivers. In addition, significant damages occurred to water-supply systems in municipalities along these rivers.

Two problems related to the Toutle River have continued to plague authorities for years after the eruptions:

(1) The erosion-prone volcanic and landslide materials to the north and west of the mountain still have the potential for contributing large amounts of sediment to the Toutle River and possibly to the Cowlitz and Columbia rivers.

(2) The damming by the debris avalanche of the main stem and tributaries of the North Fork Toutle River, which was the direct outlet from Spirit Lake prior to 1980, poses a flood danger to downstream areas (Schuster, 1985; Meyer et al., 1986). In their original natural state, these landslide dams and their impoundments had the potential for failure and catastrophic downstream flooding (Jennings et al., 1981).

This paper discusses the effects on engineered works and operations in both the public and private sectors resulting from the following processes related to the 1980 eruptions of Mount St. Helens: (1) the directed blast, (2) the debris avalanche, (3) mudflows, (4) sedimentation in major rivers, and (5) ash fall. It also deals with continuing problems such as erosion and sedimentation and the possibilities of flooding due to failure of landslide dams. A short section at the end of the report presents judgments by Hoblitt et al. (1987) on future hazards from Mount St. Helens.

DIRECTED BLAST

The lateral blast due to the May 18 eruption began about 8:32 a.m. PDT (Pacific Daylight Time). It con-

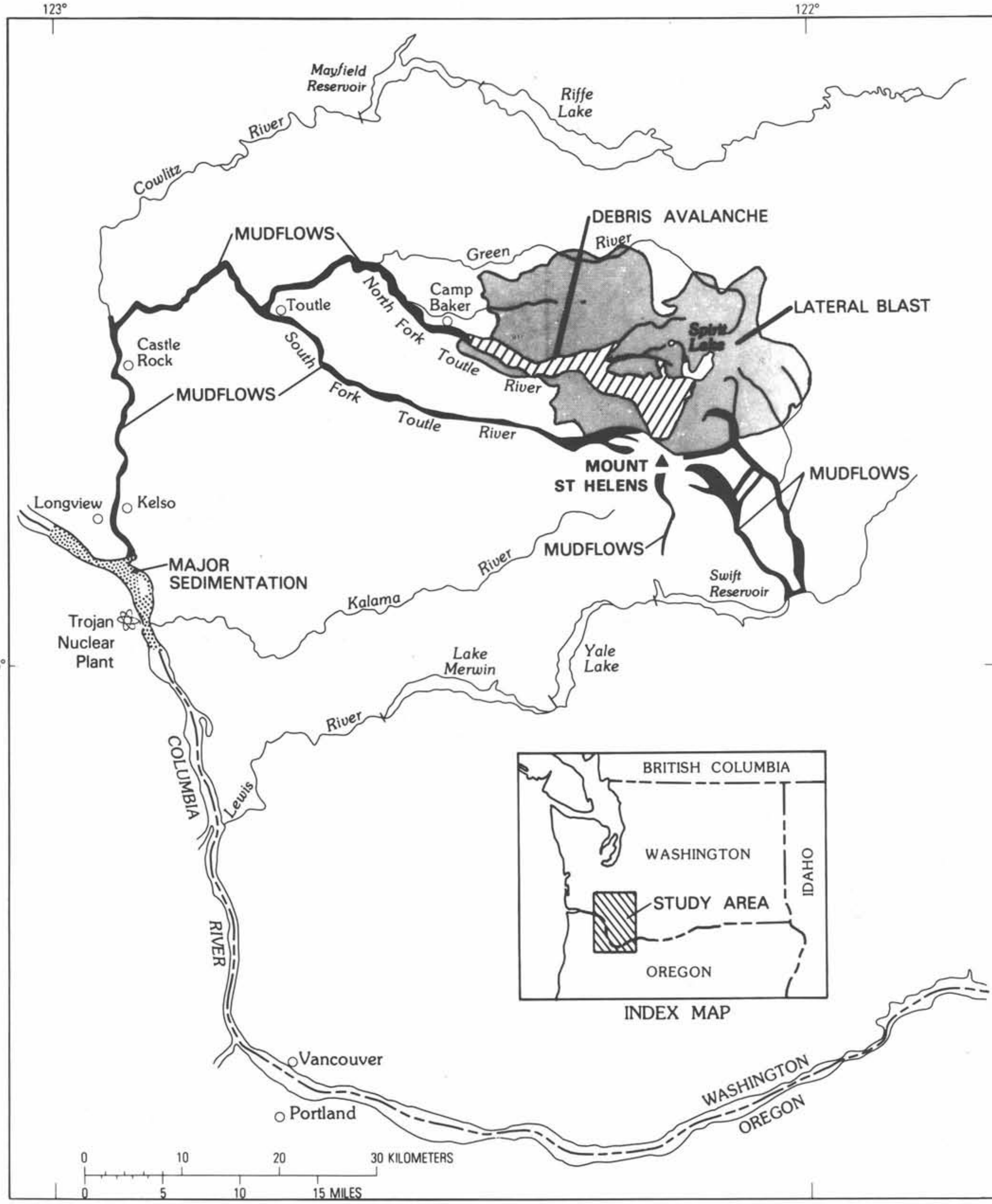

Figure 3. Location map of the area of southwestern Washington directly affected by the May 18 eruption.

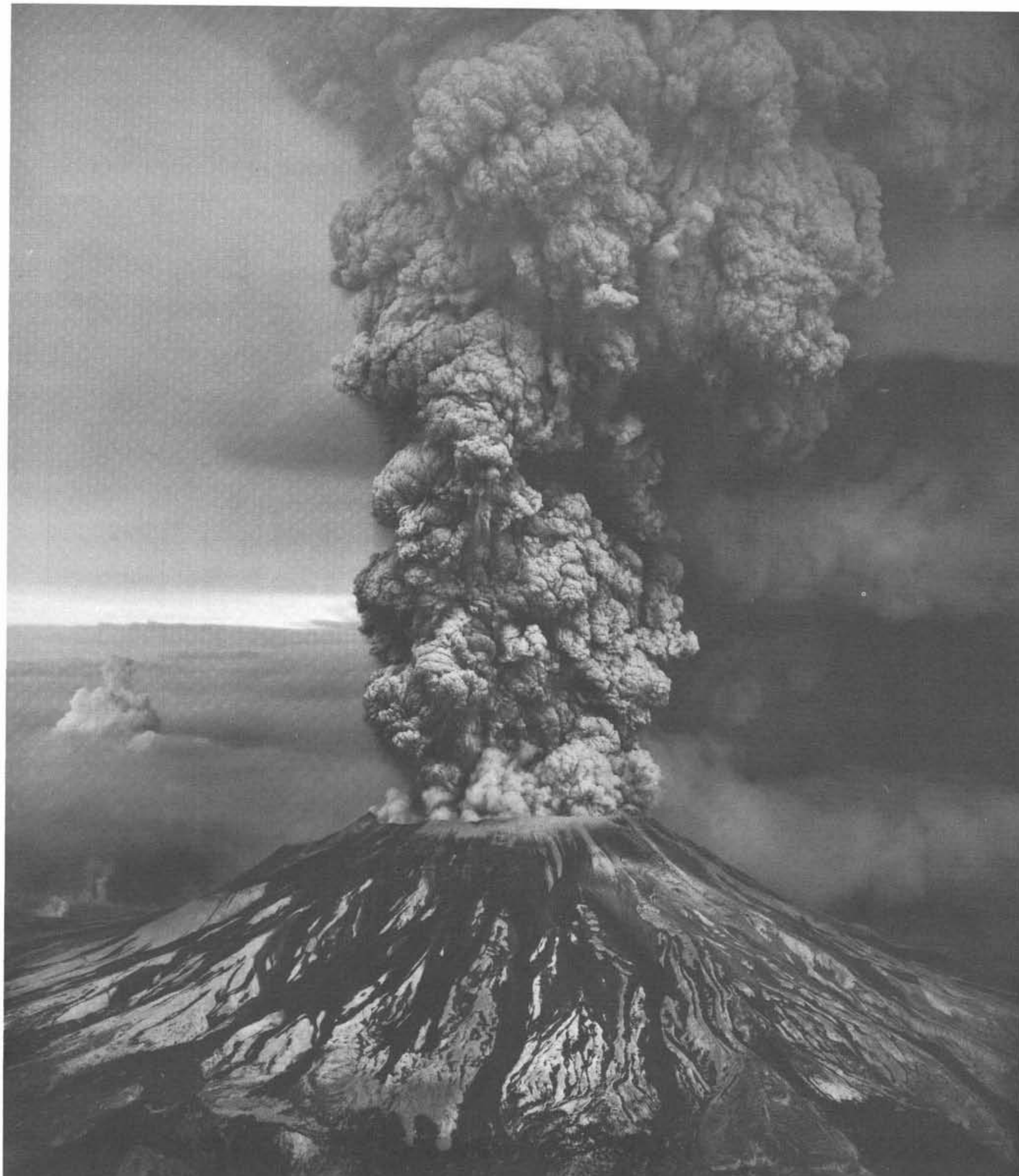

Figure 4. Catastrophic eruption of Mount St. Helens, May 18, 1980. After the first few minutes of powerful, laterally directed explosions, the direction changed, producing the vertical column shown here, which rose quickly to heights of at least 25 km (Peterson, 1986). Photograph by R. M. Krimmel, U.S. Geological Survey.

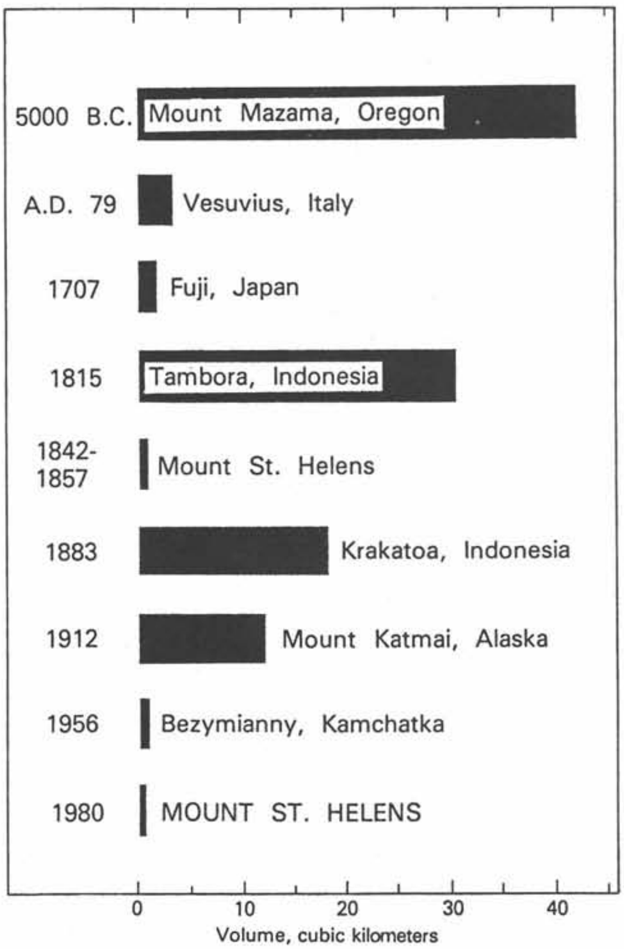

Figure 5. Comparison of the volumes of ejecta erupted by selected volcanoes during historic eruptions. The volume of ejecta from the May 18 eruption of Mount St. Helens (less than 0.5 cu km, not including avalanche or mudflow deposits) is relatively small in comparison with amounts from several earlier eruptions. From Foxworthy and Hill (1982).

sisted of a shock wave, thermal effects, and massive amounts of hot airborne pyroclastic debris moving at more than 100 m/sec. Hoblitt et al. (1981) have called this explosion a "directed blast", following Gorshkov's (1963) definition of the term.

The blast affected a 600-sq-km area that extends generally northward from the mountain through an arc of approximately 120° from northwest to east-northeast (Figure 3). Destruction was essentially total within a radial distance of 13 km from the crater of Mount St. Helens; all vegetation and most of the soil were removed from slopes that face the volcano (Figure 6). Within this zone, temperatures of the pyroclastic cloud constituting the blast exceeded 350° ± 50°C (Moore and Sisson, 1981). Because this zone was principally a recreation and timber-producing area, the engineered works destroyed consisted primarily of roads and bridges, recreational buildings, campgrounds, and timber-harvesting facilities.

Beyond the zone of total devastation, in an area extending to about 18 km east-northeast and 24 km north-northwest of the crater, old-growth timber was blown down and lesser vegetation was killed by the blast, buried by airborne debris, or both. In this zone, falling trees caused extensive damage to recreational and logging roads and operations. Surrounding this blowdown zone was a halo as much as 2 to 3 km wide within which the thermal effect of the blast killed the vegetation; damage from the blast itself to engineered structures within this zone was minimal.

DEBRIS AVALANCHE

Within about 10 min after the eruption, the debris avalanche had progressed as far as 8 km north and 22 km west of the mountain; it had an average thickness of 45 m, a maximum thickness of 195 m, and an accumulated volume of 2.8 cu km (Voight et al., 1983). About 60 sq km of the North Fork Toutle River valley (Figures 2 and 3) was covered by hummocky-surfaced, poorly sorted debris ranging in size from clay to blocks whose individual volumes were as great as several thousand cubic meters. The level of Spirit Lake, a 5-km-long lake north of the mountain (Figure 1), was catastrophically raised 60 m as water was displaced by avalanche material.

No major structures such as dams and reservoirs, power plants, or large buildings were in the area devastated by the debris avalanche. However, civil works were affected as follows:

(1) Public and private buildings and attendant facilities on the shore of Spirit Lake were obliterated. Nearly all of these properties were related to recreation on the lake and in the surrounding area. Most buildings destroyed were recreational cabins.

(2) State Highway 504 was buried from timberline on Mount St. Helens to the terminus of the debris avalanche in the valley of the North Fork Toutle River, a roadway length of some 32 km. Two major bridges were destroyed on this segment of the highway. In addition, many kilometers of private logging roads and five private and two U.S. Forest Service bridges were destroyed.

Since its occurrence, the Mount St. Helens debris avalanche has posed a continuing major threat to valleys downstream in two ways: (1) potential for damaging sedimentation and flooding due to erosion of the surface of the debris avalanche; and (2) the possibility of failure of the debris-avalanche blockages of the main stem of the North Fork Toutle River and its tributaries, which could result in catastrophic flooding downstream.

Figure 6. Stump (about 1 1/2 m high) of fir tree located 8 km north of the volcano. Large trees were wrenched from their stumps during the passage of the dense, high-velocity blast cloud. Note that the lateral blast has stripped nearly all vegetation from the slopes surrounding Spirit Lake. Volcano in background is Mount Adams. Photograph by Lyn Topinka, U.S. Geological Survey.

Potential for Sedimentation and Flooding due to Erosion

Until the surface of the debris avalanche has been covered by vegetation, it will be extremely susceptible to erosion. If the volume of the eroded material that is washed downstream is large enough, it could further reduce the size of the channels of the Toutle and Cowlitz rivers, resulting in increased likelihood of flooding of low-lying parts of the river valleys. As a stop-gap measure to temporarily reduce the volume of sediment moving downstream, the Corps of Engineers, soon after the eruption, constructed a pervious embankment dam 1,300 m long and 12 m high on the North Fork Toutle River 2 km downvalley from the terminus of the debris avalanche. This dam served as a debris-retention structure; water passed through it, but sediment was deposited in the reservoir. By October 1981, about 7 million cu m of sediment had been excavated from this reservoir, and 1.5 million cu m had been removed from a similar, but smaller structure on the South Fork Toutle River. Because of a lack of continuing maintenance funds, the retention structure on the North Fork failed due to floods in 1982, and the smaller structure was removed in late summer of 1982 to allow for fish passage (Meyer and Janda, 1986). Sediment since then has been allowed to move downstream freely. Meyer and Janda (1986) noted that the amount of sediment derived annually from the debris avalanche by channel erosion and stream-side landslides is 20 to 30 million cu m. To prevent this sediment from moving downstream into the lower Toutle River and the Cowlitz and Columbia rivers, a much larger sediment-retention structure (Figure 7) is currently under construction by the Corps of Engineers on the North Fork Toutle River 11 km downstream from the terminus of the debris avalanche. This earth and rockfill check dam, with a height of 54 m, a crest length of approximately 550 m, and a projected reservoir volume of 197 million cu m, is scheduled for completion in September 1989.

Potential for Failure of Debris-Avalanche Dams

The debris avalanche formed several new lakes by damming the main stem and tributaries of the North

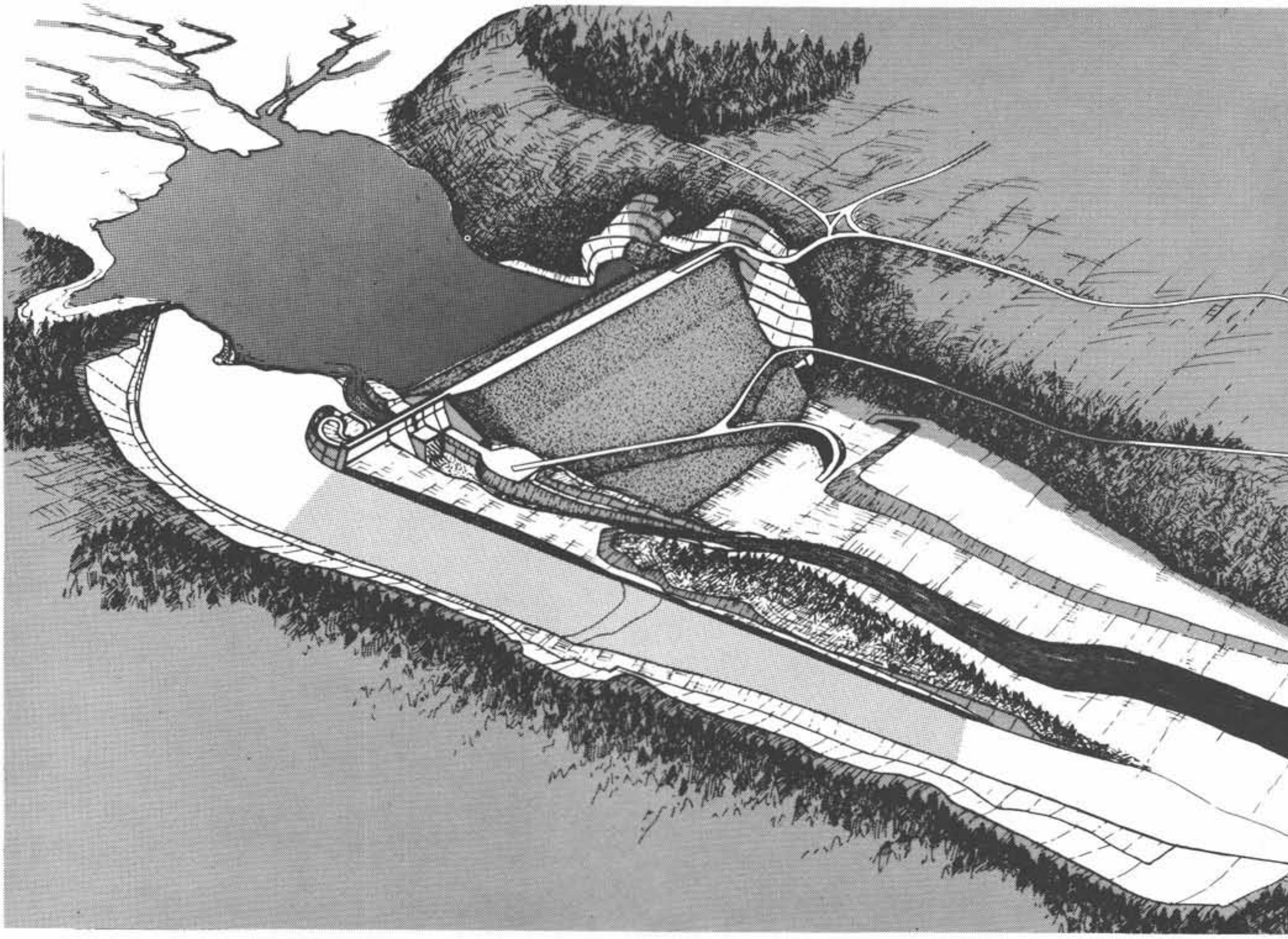

Figure 7. Diagram of sediment-retention structure currently under construction on the North Fork Toutle River. Courtesy of U.S. Army Corps of Engineers, Portland District.

Fork Toutle River. Soon after the debris avalanche occurred, it was recognized that any of these natural dams could fail, and that rapid failure of the larger ones would cause severe flooding downstream (Jennings et al., 1981). The four most important of these lakes (Elk Rock, Spirit, Coldwater, and Castle; Figure 8) and their landslide dams will be discussed here.

Elk Rock Lake (Figure 9), the smallest of the four, filled first. By August 1980, it was 9 m deep and had a volume of 300,000 cu m (Jennings et al., 1981). The new lake was recognized as a flood hazard. On August 27, the landslide dam failed after a rainstorm, discharging a peak flow of 450 cu m/sec. The ensuing minor flood resulted in damage to equipment used for channel maintenance, but it caused no injuries or deaths. However, in a small way this failure provided a warning of what might happen if the much larger Spirit, Coldwater, or Castle lakes should be allowed to breach their respective landslide dams. Thus, monitoring and control measures were established on each of these three dams and their impoundments.

Spirit Lake (Figure 10) is the largest of three remaining impoundments. Soon after the May 18 eruption, Youd et al. (1981) recognized that eventual overtopping

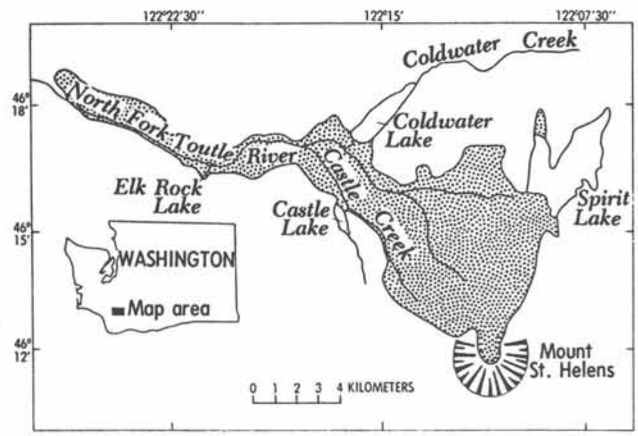

Figure 8. Area covered by debris avalanche (stippled) showing locations of Elk Rock, Castle, Coldwater, and Spirit lakes.

Figure 9. Elk Rock Lake (middle foreground) as it appeared in the summer of 1980 shortly before breaching its natural dam and draining. View is to the west down the Mount St. Helens debris avalanche. Copyrighted photograph by Jan Fardell, The Daily News, Longview, WA, published with permission.

of the blockage by the rising lake could cause erosion that would lead to breaching. By August 1982, about 330 million cu m of water had been impounded by the debris-avalanche dam. Lake level at that time was at an elevation of 1,055 m. At about the same time, a government task force concluded that the dam could not impound water above elevation 1,060 m without danger of breaching (Sager and Chambers, 1986). Using a mathematical dam-break model, Swift and Kresch (1983) arrived at a hypothetical flood/mudflow of 75,000 cu m/sec if the Spirit Lake blockage were to breach. This catastrophic flooding would inundate

Figure 10. View from the north of the truncated cone of Mount St. Helens a few days after the May 18 eruption. Spirit Lake (middle and right foreground) is largely covered by floating logs that were stripped from the surrounding slopes by the lateral blast. The approximate crest of the debris-avalanche dam impounding Spirit Lake is shown by the arrows at the right edge of the photograph. Photograph by R. M. Krimmel, U.S. Geological Survey.

towns downstream along the Toutle and Cowlitz rivers to depths of as much as 20 m. In addition, using a one-dimensional sediment-transport computer model to study the effects on the Columbia River of an outburst flood from Spirit Lake, Sikonia (1985) estimated that such a flood would dump 13 m of sediment into the Columbia at the mouth of the Cowlitz River, thus temporarily blocking the Columbia and impounding its waters above that point.

As a result of these findings, in November 1982 the Corps of Engineers installed a large temporary pumping facility to maintain lake level at 1,055 m. This 16-pump system, with a maximum capacity of 5 cu m/sec, pumped water from the lake through a 1,112-m-long, 1.5-m-diameter buried pipe across the crest of the blockage to a stilling basin emptying into the North Fork Toutle River.

As a permanent solution, it was decided to maintain a constant lake level at 1,048 m by means of a 2,590-m-long, 3.4-m-diameter gravity-flow tunnel through Tertiary volcanic rocks composing the ridge immediately west of Spirit Lake (Sager and Chambers, 1986). Construction of the tunnel commenced in the summer of 1984 and was completed in early April 1985. Reduction of lake level to 1,048 m resulted in a "safe" lake volume of 258 million cu m.

Coldwater Lake (Figure 11), if it had been allowed to overtop naturally, would have developed an estimated maximum volume of 128 million cu m by late 1981. To prevent catastrophic breaching of the debris dam, the Corps of Engineers during the summer of 1981 constructed a permanent spillway for the lake through the bedrock right abutment. This spillway has stabilized the volume of Coldwater Lake at 83 million cu m and apparently has insured the stability of the natural dam.

Figure 11. Coldwater Lake and its debris-avalanche dam, June 1983. Note spillway (white arrows) excavated through the bedrock right abutment, and incised North Fork Toutle River (foreground) which flows from right to left through the debris avalanche.

Similarly, the volume of Castle Lake (Figure 12) has been maintained at about 24 million cu m by a spillway excavated through its natural dam. Thus, there is little possibility of Castle Lake failing by overtopping. However, significant seepage occurs through the Castle Lake blockage at current lake levels, and analytical studies have suggested that the Castle Lake blockage may be only marginally stable in regard to the possibility of piping (Meyer et al., 1986). Using a dam-break computer model, Laenen and Orzol (1987) have estimated that failure of this natural dam would cause a flood with a peak magnitude of 42,500 cu m/sec immediately downstream. According to this scenario, the peak flood would attenuate to 27,000 cu m/sec at Toutle (on the Toutle River 50 km downstream), 9,000 cu m/sec at Castle Rock (on the Cowlitz River 80 km downstream), and 3,000 cu m/sec at Longview, where the Cowlitz River enters the Columbia River (Figure 3); the communities of Toutle, Castle Rock, Kelso, and Longview would experience extreme to moderate flooding. For these reasons, periodic monitoring of the seeps through the Castle Lake debris dam is continuing.

MUDFLOWS

The term mudflow is commonly applied to flows of earth consisting of material wet enough to flow rapidly and containing at least 50 percent sand-, silt-, and clay-size particles (Varnes, 1978). Debris flows are distinguished from mudflows in that they contain higher percentages of coarse particles. The flows originating from Mount St. Helens from the May 18 eruption were both debris flows and mudflows and, considering their volcanic origin, can be called lahars (Mullineaux and Crandell, 1962). However, because most of these flows had the characteristics of mudflows rather than debris flows (Janda et al., 1981), and inasmuch as this paper is concerned more with effects of geologic processes than with genesis of these processes, I will use the term mudflow throughout this discussion as a means of simplification.

Figure 12. Castle Lake and its debris-avalanche dam (1983 photograph). Water exits the lake through spillway at left end of dam. The debris avalanche descended the valley of the North Fork Toutle River from left to right (east to west).

The largest and most destructive mudflows associated with the May 1980 eruptions were in the valleys of the North and South Fork Toutle rivers, the main Toutle River, and the Cowlitz River downstream from its confluence with the Toutle (Figure 3). However, mudflows along tributaries of the Lewis River on the east, southeast, and south sides of the mountain were large enough to destroy forest roads and bridges and to flow into Swift Reservoir. Although these mudflows resulting from the 1980 eruptions were devastating in their impacts on valleys and on structures therein, they were smaller and less extensive than some of the earlier Holocene mudflows from Mount St. Helens (Janda et al., 1981).

Toutle River Mudflows

Because mudflows in the Toutle River valley were formed primarily by remobilization of material in the debris avalanche, they affected the valley only downstream from the terminus of that landslide. Conversely, the mudflow in the valley of the South Fork Toutle River (Figure 13) originated on the slopes of Mount St. Helens and impacted the entire length of the valley.

The Toutle River mudflows and giant log jams they were carrying destroyed or badly damaged about 200 homes and had the following effects on other structures and facilities:

(1) Buried under as much as 2 m of sediment about one-half of the 28 km of Washington State Highway 504 in the valley of the North Fork Toutle River immediately downstream from the terminus of the debris avalanche (Figure 14). Mudflows also buried many kilometers of private logging roads and Cowlitz County roads. In addition, 17 km of private logging railway were destroyed. Most of the buried stretches of highways and roads were restored merely by removing the sediment. The railroad was abandoned, however.

Figure 13. Mudflow on the valley floor of the South Fork Toutle River; view to the east toward steaming Mount St. Helens. The debris-avalanche-covered valley of the North Fork Toutle River is in the left middle background. Mount Adams is in the far background. Photograph by Austin Post, U.S. Geological Survey.

(2) Destroyed or badly damaged 27 bridges (22 highway and 5 railway) in the Toutle River valley (example, Figure 15). Most of these were modern concrete and/or steel structures, the longest being a 160-m steel-girder bridge spanning the Toutle River on Washington State Highway 504 near the town of Toutle (Figure 16).

(3) Partially submerged and heavily damaged three large logging camps (two on the North Fork Toutle River and one on the South Fork). Much equipment was also badly damaged (Figures 17 and 18).

(4) Destroyed or badly damaged water-supply and sewage-disposal systems along the Toutle River that were owned by local governments, companies, and private residents.

Cowlitz River Mudflows

Mudflows from the tributary Toutle River first entered the Cowlitz River, some 70 km downstream from Mount St. Helens, early in the afternoon of May 18, approximately 4 to 5 hr after the beginning of the main eruption. The first to arrive came from the valley of the South Fork Toutle River. Although the mass of sediment and water coming from the Toutle River was diluted considerably by the large volume of water in the Cowlitz, the flow could still be classified as a mudflow while it passed down the Cowlitz (Janda et al., 1981). The mudflows and accompanying floods originally deposited about 30 million cu m of sediment in the Cowlitz River downstream from the mouth of the Toutle River (U.S. Army Corps of Engineers, 1984b). Depth of deposition in the lower Cowlitz River was as much as

Figure 14. Mudflow deposit covering State Highway 504 to a depth of 2 m near the town of Toutle. Geologist is D. R. Crandell, U.S. Geological Survey).

5 m (Figure 19; Lombard et al., 1981); this was nearly enough to fill the original Cowlitz River channel. After the original mudflows and floods subsided, deposition continued in the Cowlitz as new sediment was washed into the river from the volcano, the debris avalanche, and mudflows in the Toutle River valley. Between June 1980 and May 1981 (a year after the main eruption), the Corps of Engineers dredged approximately 45 million cu m of sediment from the Cowlitz and lower Toutle rivers in order to restore the original channel and to remove new sediment (U.S. Army Corps of Engineers, 1984b).

The mudflow, flooding, and accompanying deposition of debris drastically affected operations of water-supply and sewage-disposal systems for cities and towns along the Cowlitz River. For example, 21 km downstream from the point where the Toutle River empties into the Cowlitz, the water-supply system intake and treatment plant for the city of Longview (population 30,000) were completely clogged by sediment and debris and were out of service for several weeks (Edtl, 1980). The first flood and debris reached Longview late on the afternoon of May 18; by late that evening, the flood had passed, but turbidity of water entering the treatment plant increased, at first to 10 JTU (Jackson turbidity units), then to 40, 120, and by the end of the day, to 420 JTU. By 3:00 a.m. on May 19, the Cowlitz River, clogged by mud and debris, had risen 4 m and had swept away the plant's water-supply intake structure shearboom. By 8:00 a.m., the flood had partially receded, but it left behind a 3 1/2-m-thick deposit of mud that clogged the plant's intake, pumps, and screens, and partially filled its sedimentation basins; no permanent damage was done, but it was several weeks before the system was back in full service. On May 19, the raw-water temperature in the Cowlitz, which normally is about 7°C at that time of the year, had risen to 32°C due to the addition of hot debris from the eruption, and turbidity levels were an almost unmeasurable 50,000 to 70,000 JTU (Edtl, 1980).

Two weeks before the eruption, the city of Kelso (Longview's sister city, population 12,000) had installed a new collector system to pump water from gravels permeated by the river (Wilder, 1980). The system was temporarily put out of action by electrical and mechanical problems due to flooding and sedimentation. A longer range problem was caused by sediment clogging the river gravels, which reduced their per-

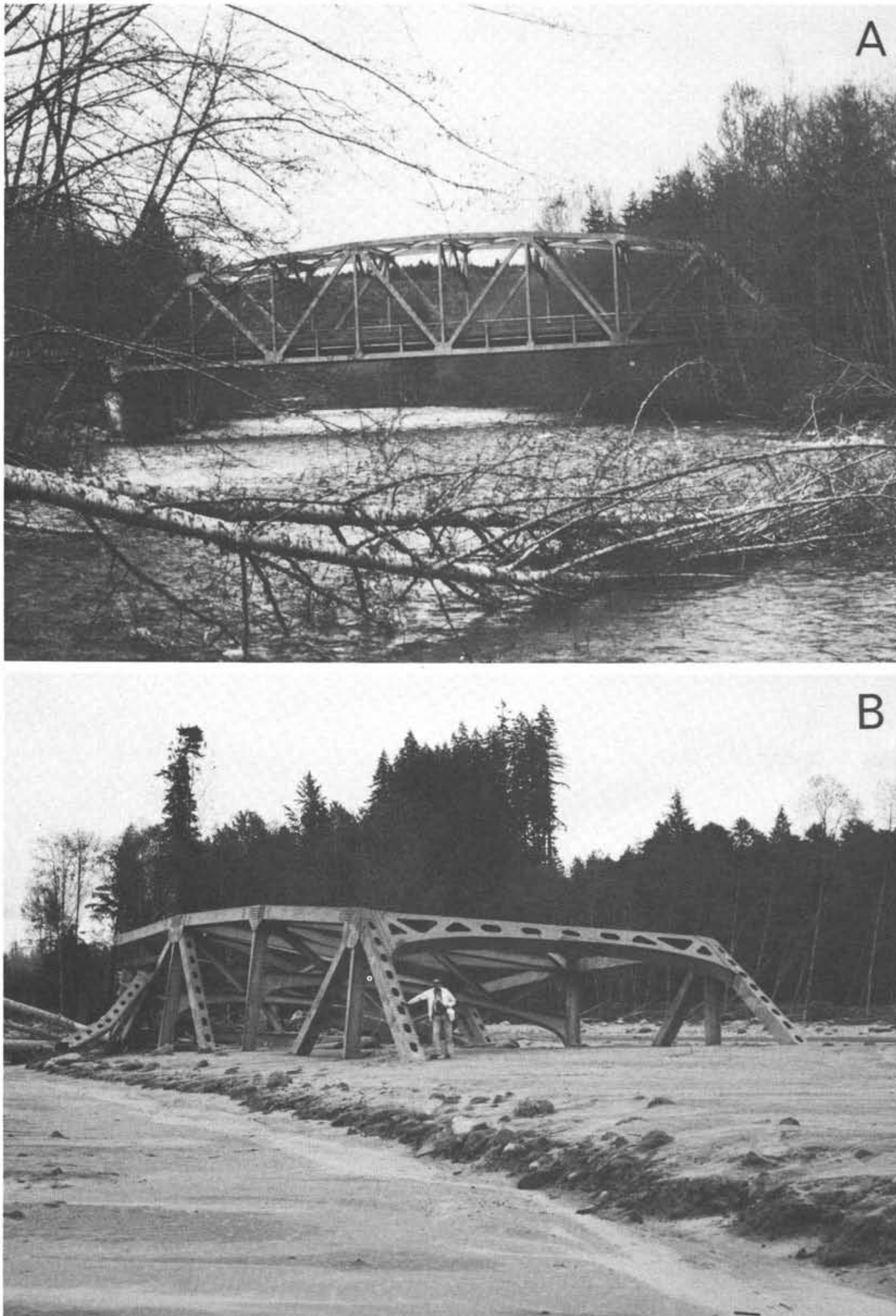

Figure 15. The 75-m-long St. Helens bridge on State Highway 504. A, before the May 18 eruption, with North Fork Toutle River in the foreground; photograph by D. R. Crandell, U.S. Geological Survey. B, after May 18 when it was washed out by the mudflow on the North Fork Toutle River. This steel structure was carried about 1/2 km downstream and was partially buried by the mudflow.

Figure 16. The 160-m steel-girder Coal Creek bridge spanning the Toutle River near the town of Toutle. A, ground-level view before May 18; B, aerial view after the bridge had been destroyed by the May 18 mudflow. Photographs by D. R. Crandell, U.S. Geological Survey.

meability and their potential as a water-supply source. As a direct result, the collector system, which had a rated production of 19,000 cu m/day, was able to produce only 5,700 cu m/day more than a month after the eruption.

The city of Castle Rock (population 2,000) is on the low flood plain of the Cowlitz River, only 3 km downstream from the mouth of the Toutle River. The Castle Rock fairgrounds were covered by river sediment to a depth of as much as 1 to 1-1/2 m. The outfall for the city sewage-treatment plant was washed away, and storm sewers were completely clogged by mud. In addition, the city lost its water intake in the May 18 flood and, until repairs were made, had to truck water from the small town of Vader, about 14 km to the north.

Mudflows on the Lewis River and Its Tributaries

As a result of the May 18 eruption, mudflows occurred in the upper Lewis River and its tributaries on the east, southeast, and south sides of Mount St. Helens (Figure 3) (Janda et al., 1981). These mudflows buried several kilometers of U.S. Forest Service recreational and timber-haul roads and destroyed 16 bridges on these roads. The bridges were concrete, steel, and log-stringer structures ranging in length from 17 to 104 m (Schuster, 1981). The most serious potential hazard from mudflows on the upper Lewis River was to Swift Reservoir and Dam, which lie about 15 km south and southeast of Mount St. Helens (Figure 3). Swift Dam is an earthfill structure that was built for hydroelectric power generation and recreation. It has a crest length of 640 m and a height of 156 m, and it impounds a reser-

Figure 17. Weyerhaueser Company employee bus heavily damaged and partially buried near Camp Baker by the May 18 mudflow on the North Fork Toutle River. Bus was unoccupied when hit by the mudflow.

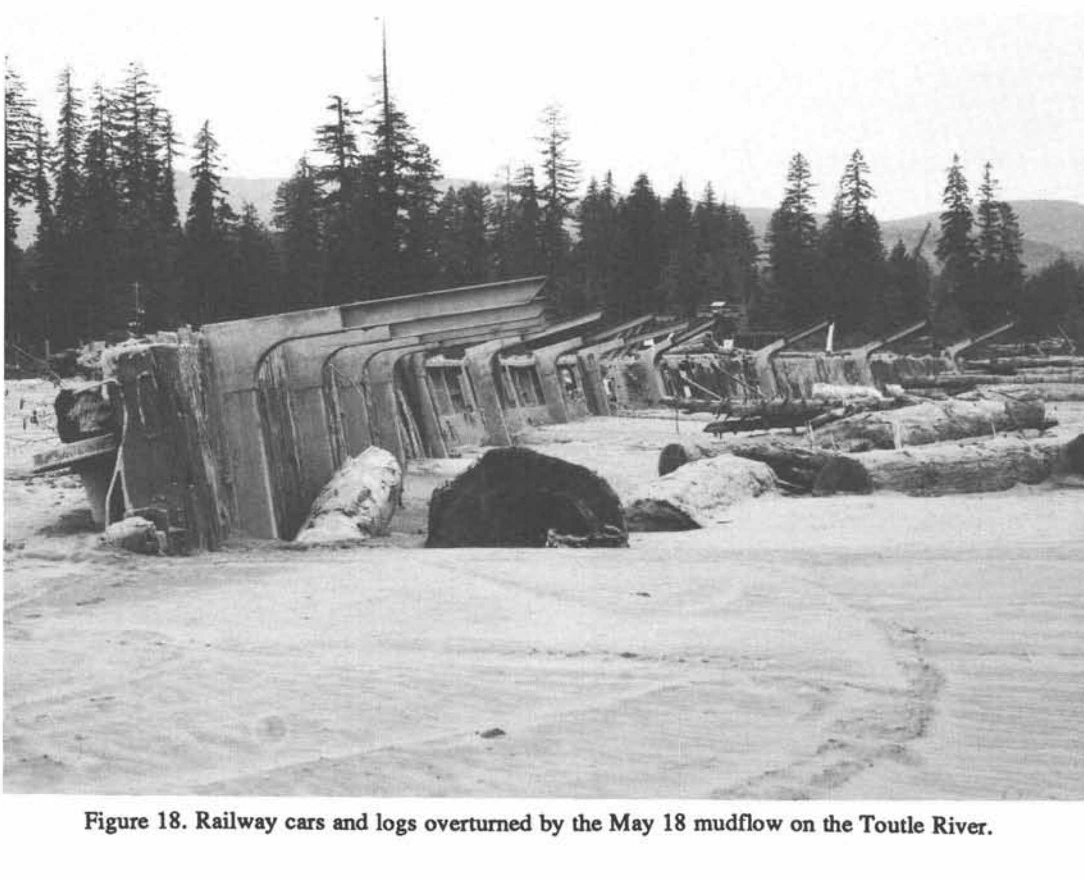

Figure 18. Railway cars and logs overturned by the May 18 mudflow on the Toutle River.

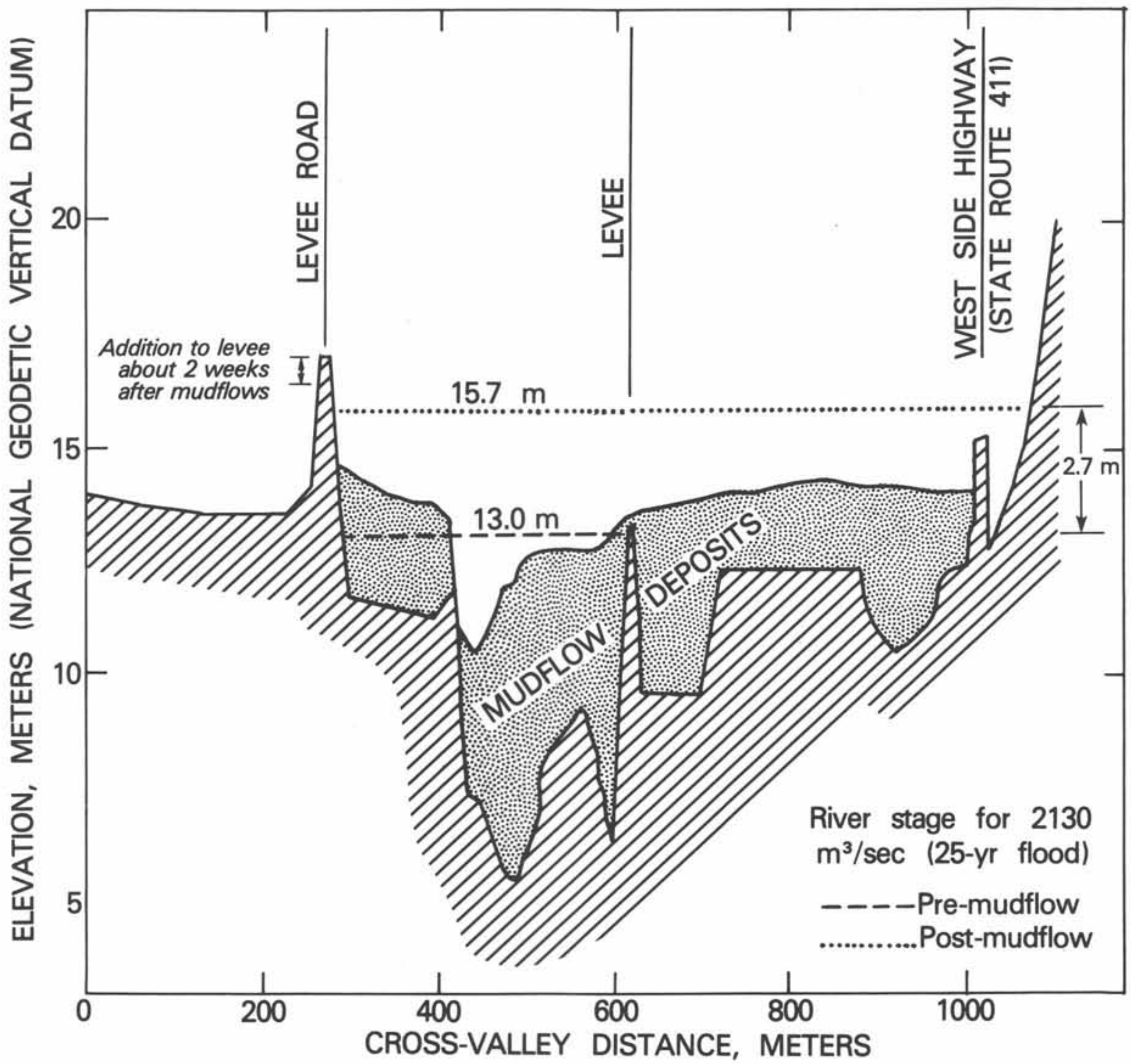

Figure 19. Channel and valley conditions of the Cowlitz River near Castle Rock prior to and after the mudflows of May 18 (Lombard et al.,1981).

voir with a capacity of 920 million cu m. Crandell and Mullineaux (1978) noted that, if Swift Reservoir were at maximum pool, a mudflow of large volume flowing into the reservoir could create a wave that would overtop the dam. Such a wave could cause downstream flooding that would endanger Yale Lake and Lake Merwin reservoirs as well as residents of the lower Lewis River valley, particularly those in the town of Woodland. Crandell and Mullineaux (1978) estimated that the largest single mudflow that might enter Swift Reservoir in the event of an eruption of Mount St. Helens would have a volume of no more than 123 million cu m. On the basis of this estimate and the recognized possibility of a major eruption, the reservoir was drawn down 7 m between March 24 and 28, 1980, to produce the recommended level of extra capacity. The reservoir was still at this level at the time of the May 18 eruption (Pacific Power & Light Company, 1980).

The May 18 eruption sent mudflows down Pine Creek and Muddy River into the Lewis River at the head of Swift Reservoir (Figure 3); inflow of more than 13.5 million cu m of water, sediment, and debris into the reservoir raised the pool level about 3/4 m (Pacific Power & Light Company, 1980). In addition, a water-level recorder indicated that a wave about 1/2 m high occurred at the dam face. The only consequence to the reservoir was a loss of storage capacity due to the sediment; however, because only about 545 million cu m of the original reservoir consisted of usable capacity, and because most of .the sediment flowed into a zone of "dead storage" at the bottom of the reservoir, the resulting loss in power-producing capacity was insignificant.

Realizing that a potential hazard to Swift Reservoir still exists because of the possibility of future eruptions of Mount St. Helens, Pacific Power & Light Company (1980) sponsored a model study to determine the potential of future mudflows on the reservoir. In that study, model "mudflows", consisting of bentonite, barite, and water, were released at different volumes and velocities into a 1:500-scale model of the reservoir. The effects on the reservoir were determined by wave-height measurements. Application of the results of these studies will enable better estimation of the necessary drawdown if future activity of Mount St. Helens indicates a possibility of mudflows entering Swift Reservoir.

Sedimentation in the Columbia River

The Cowlitz River mudflow and floods caused by the May 18 eruption deposited about 34 million cu m of sediment in the Columbia River 100-120 km upstream from where the Columbia enters the Pacific Ocean (Figure 3) (Bechly, 1980). [Note that this estimate of volume is somewhat in disagreement with that of Haeni (1983), who, based on U.S. Army Corps of Engineers bathymetric surveys, has calculated that only 27.2 million cu m of mudflow sediment was deposited in the Columbia River.] This material can be considered to be part of the mudflow system from Mount St. Helens; it consisted mainly of pumiceous sands and fine gravels because most clay- and silt-size materials were distributed farther downstream or were carried into the Pacific Ocean. As illustrated in Figure 20, most of the sediment was deposited upstream from its point of entry into the Columbia from the mouth of the Cowlitz River; this material was carried upstream as much as 12 km by tidally influenced upstream flow in the Columbia (Haeni, 1983).

An immediate effect of this sedimentation was the blocking of the Columbia River to ocean-going freighter traffic to and from the deep-water port of Portland, Oregon. During the next few months after the eruption, the U. S. Army Corps of Engineers removed some 11.1 million cu m of sediment in dredging a shipping channel about 12 m deep and 180 m wide. As of October 1981, the Corps of Engineers had dredged from the Columbia River about 18 million cu m of sediment resulting from the May 18 eruption. The total amount of sediment removed from the Cowlitz, Columbia, and lower Toutle rivers from May 1980 to October 1981 was about 63 million cu m (U.S. Army Corps of Engineers, 1984b).

The Trojan nuclear power plant, with a rated electrical-power production of 1.13 million kw, is probably the most important structure along the segment of the Columbia River affected by sedimentation from the mudflow. This plant, which is located 8 km upstream from the Cowlitz River (Figure 3), is cooled by water pumped from the Columbia River immediately adjacent to the plant. Soundings made a few days after the eruption showed that the river channel in front of the plant, which originally had a depth of as much as 36 m, had been partially filled, resulting in new minimum bottom depths of only 11 m. However, because the intake structure was located at a depth of only 3 m, this posed no serious threat to the cooling system.

As the result of this sediment deposition, the U.S. Nuclear Regulatory Commission asked the Portland General Electric Company (owner of the Trojan plant) what sedimentation effects would be expected at the plant if an eruption similar to the May 18 event were to occur on the southwest flank of Mount St. Helens (Portland General Electric Company, 1980). Debris flows and mudflows from such an eruption could conceivably flow down the Kalama or Lewis rivers (Figure

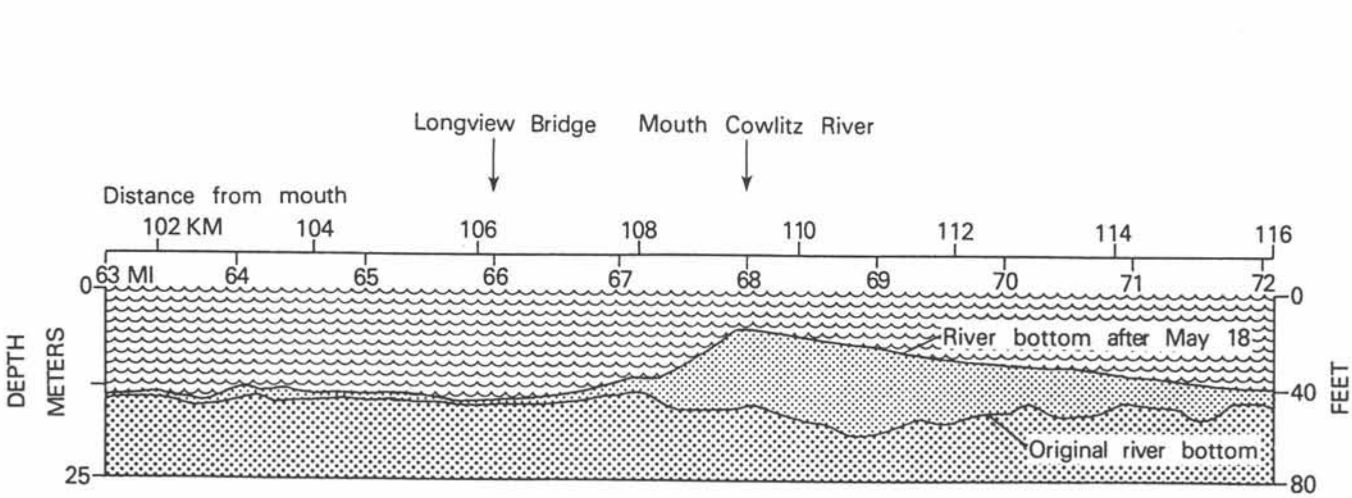

Figure 20. Longitudinal profile of the bottom of the Columbia River where sediment resulting from the May 18 mudflows entered from the Cowlitz River. Note that most of the sediment was deflected upstream. Modified from Bechly (1980).

3) into the Columbia River a few kilometers upstream from the plant, thus potentially endangering the cooling system. In response, Portland General Electric Company noted that studies sponsored by its organization have predicted that maximum thickness of deposition in the Columbia River from such an event would be 3 1/2 m at the mouth of the Kalama River and 7 1/2 m at the mouth of the Lewis River; thicknesses closer to the plant would be less. Thus, anticipated deposition at the cooling-water intake would not be great enough to pose a threat to the system.

ASH FALL, MAY 18 ERUPTION

The May 18 eruption of Mount St. Helens ejected an estimated total mass of 490 million metric tons of tephra (Sarna-Wojcicki et al., 1981), nearly all consisting of ash that fell in a broad band across eastern Washington, northern Idaho, and western Montana (Figure 21). This heavy ash fall, which had an estimated volume of about 0.73 cu km, greatly affected civil works and operations in the areas of deposition. The bulk of the ash fell in Yakima, Grant, Adams, Whitman, and Spokane counties of eastern Washington. Depths of as much as 10 cm were recorded near Ritzville, Adams County, but, as shown in Figure 21, average depths were less. Ash sufficiently deep to require removal by mechanical equipment or by jetting with water occurred as far east as Helena, Montana.

Effects on Sewage-Disposal and Storm-Drain Systems

Almost all municipalities in the area of thick ash accumulation experienced severe problems due to clogging of storm sewers, and several had plugged sanitary sewage-disposal systems. Particularly hard hit were Moses Lake (population 12,000) and Yakima (population 50,000) in eastern Washington.

The Moses Lake storm drain system, which consisted of 380 catch basins, was solidly plugged by the 5 to 7 cm of ash washed from the streets during cleanup. In addition, Moses Lake's entire sewage-disposal plant, consisting of four clarifiers, two trickling filters, and two digesters, was plugged by a jelly-like mixture of sewage and ash. As a result, the system was out of service for 5 weeks.

Yakima, the second largest city in eastern Washington, received about 1 to 2 cm of ash from the May 18

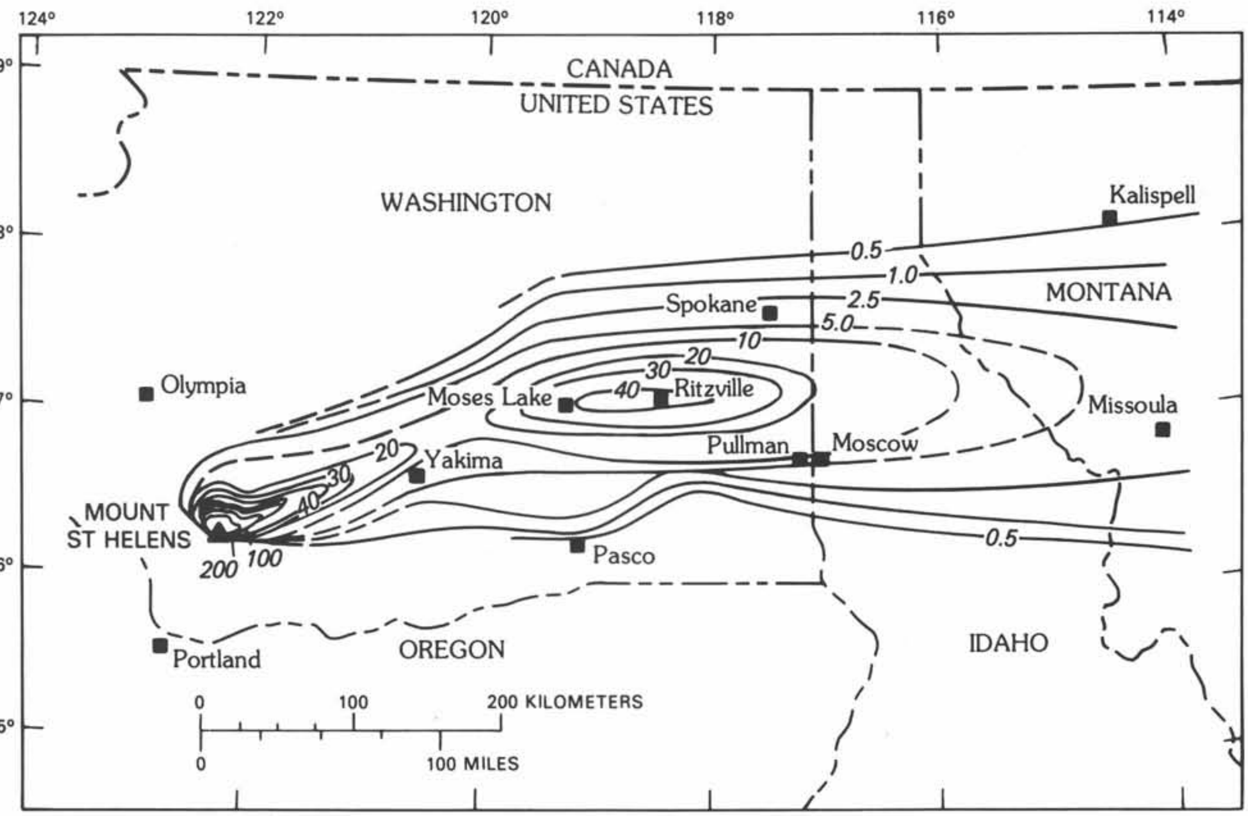

Figure 21. Isopach map of ash erupted from Mount St. Helens on May 18. Lines represent accumulated thicknesses in millimeters. Depths shown are generally a little less than those reported by local officials and noted in the text of this report. The difference probably is due to natural compaction that occurred before the measurements used in this figure were made. After Sarna-Wojcicki et al. (1981).

eruption. From May 21 to 25, the Yakima wastewater-treatment plant was inoperable due to plugging and damage to pumps, filters, and clarifiers; during this period, the system was by-passed, and all sewage was dumped directly into the Yakima River. Direct damage costs to the system were estimated as: treatment plant, $700,000; sanitary collection system, $290,000; and storm-sewer system, $90,000 (White, 1980).

Spokane, the largest city in eastern Washington (population 175,000), received 1/2 cm of ash in spite of being located 400 km east-northeast of Mount St. Helens. Spokane's 1-1/2-yr-old sewage treatment plant did not plug up, even though some 13,600 metric tons of ash had to be flushed from the system. However, all 13,000 catch basins in the city's storm-drain system were filled and had to be cleaned (Yake, 1980). Other smaller cities in eastern Washington and northern Idaho experienced similar problems, but on a lesser scale.

Cities in eastern Washington rely primarily on deep wells for water supply, and water storage commonly is in enclosed reservoirs or tanks. For these reasons, the ash caused only minor problems to municipal water-supply systems.

Effects on Transportation

The ash fall immediately paralyzed both surface and air transportation in much of eastern Washington and parts of northern Idaho. Within a few hours of the eruption, 2,900 km of state highways were closed in eastern Washington due to ash accumulation (Anderson, 1980). Interstate Highway 90, which crosses the state from Seattle to Spokane, was closed for a week. Many thousands of kilometers of municipal streets, county roads, and irrigation-district service roads also were closed, some for a few hours, some for weeks. Ash had to be removed from an estimated 10,000 km of county roads in Adams, Grant, Whitman, and Yakima counties before the road system could be restored to normal traffic (County Road Administration Board, 1980; McLucas, 1980).

Air transportation in the area was temporarily paralyzed by the ash cloud itself; however, it was affected for a much longer period of time by ash accumulation on airport runways, taxiways, and aprons, which curtailed airport operations until it was removed. Table 1 denotes periods of shutdown and loss of scheduled air traffic for commercial airports in eastern Washington due to ash from the May 18 eruption.

Rail transportation in the area fared much better than either automobile or air transport. Except for slowdowns and equipment problems for a couple of days, rail transport was nearly normal, and the railway lines did not present ash-removal problems nearly as serious as those on highway and airport pavements.

Table 1. Days of shutdown and numbers of commercial operations cancelled due to ash accumulation at eastern Washington airports from the May 18 eruption.

Airport	Days of shutdown	Commercial operations cancelled
Spokane International	3	576
Yakima	7	160
Pullman-Moscow	7	117
Tri-Cities (Pasco, Kennewick, and Richland)	0	97
Grant County	15	80[1]

[1] In addition, Japan Air Lines training operations at Grant County Airport were shut down for a total of 28 days, during which 1,386 flights were cancelled.

Ash Removal

Because of the great volume of material involved, removal and disposal of ash from highways, roads, streets, sidewalks, airports, storm-drain systems, and public buildings and lands presented an extremely difficult problem to public works officials in eastern Washington, northern Idaho, and western Montana. Except in areas fairly close to Mount St. Helens, such as Yakima, where the grain size ranged from silt to fine sand (Sarna-Wojcicki et al., 1981), most of the ash deposited in eastern Washington, northern Idaho, and western Montana resembled Portland cement in both fineness and color (Moen and McLucas, 1981). Removal and disposal of the ash was made more difficult by this fineness. In many places, cleanup crews saturated the ash with water to facilitate removal. Heavy equipment, such as truck-mounted snow plows, mechanized street brooms, and vacuum trucks, removed much of the ash, but there was considerable reliance on hand labor provided both by public maintenance forces and by private individuals. In some municipalities the ash was removed from streets by jetting with water. Storm drains were cleaned by combinations of hand mucking and flushing with water.

The Washington Department of Transportation estimated that its maintenance crews removed 540,000 metric tons of ash from state highways after the May and June eruptions; ash removal from median strips, ditches, and gutters on Interstate Highway 90 in eastern Washington continued as late as October 1980. As another example, the East Columbia Basin Irrigation District estimated that its maintenance forces removed some 350,000 cu m of ash from its 2,250 km of roads along irrigation canals in Adams, Grant, and adjacent counties in eastern Washington.

Municipalities in eastern Washington experienced some of the most serious ash-removal problems. For example, at Moses Lake where the average ash depth was about 6 cm, approximately 230,000 cu m was placed in disposal piles (McLanahan, 1980). Ritzville (population 2,000), the county seat of Adams County, was covered by about 8 to 10 cm of ash; 115,000 cu m of ash from the town and nearby county roads was permanently disposed of in an old quarry outside town (Figure 22) (Miller, G., 1980). Farther east, Pullman (population 22,000), Washington, and Moscow (population 16,000), Idaho, each received about 1 cm of ash; each city removed about 15,000 cu m of ash from its street system.

Airports in eastern Washington also experienced significant problems in ash removal. For instance, approximately 45,000 cu m of ash was removed from the Grant County Airport at Moses Lake, which served as a training center for airlines; Japan Air Lines shut down its training operations from this airport for 28 days due to ash. Twenty thousand metric tons of ash were removed from runways, taxiways, and aprons at the Yakima Airport. Some 6,500 cu m of ash were removed from the Spokane International Airport, although the thickness there was only about 1/2 to 1 cm.

Post-May 18 Eruptions

Although the eruptions of May 25 and June 12 were much less catastrophic than that of May 18, they did produce significant ash falls (Sarna-Wojcicki et al., 1981). During these subsequent eruptions, however, the winds were generally from the east instead of from the west as had been the case on May 18. The May 25 event deposited a blanket of ash as much as 1 cm thick in western Washington northwest of Mount St. Helens as far as the Pacific Ocean (Figure 23), causing major transportation and ash-removal problems on the highway system and in several small cities; hardest hit were Centralia (population 11,000) and Chehalis (population 6,000). The City of Centralia removed an estimated 30,000 cu m of ash from streets and had to excavate about 0.3 m of compacted ash from a covered creek which served as the outlet for the downtown storm-sewer system (Schwiesow, 1980).

The June 12 eruption spread a thinner ash cover to the southwest, affecting Vancouver (population 50,000), Washington, and Portland (population 400,000), Oregon, and vicinity (Figure 24). Although Portland received only about 1/4 cm of ash, its 2,400 km of city streets required removal of 5,300 cu m of ash at a cost of $1.2 million (Lang, 1980). In addition, operations of commercial airlines into and out of Portland International Airport were curtailed on both May 25 and June 12 because of poor visibility and possible harmful effects of the ash clouds on in-flight aircraft.

POTENTIAL FOR FUTURE VOLCANIC ACTIVITY AT MOUNT ST. HELENS

Since the major eruptions of 1980, Mount St. Helens has continued to be in a stage of minor volcanic activity, consisting mainly of dome-building (Figure 25). In a recent report to the U.S. Nuclear Regulatory Commission on volcanic hazards in the Pacific Northwest in regard to siting of nuclear power plants, Hoblitt et al.

Figure 22. Temporary spoil pile at Ritzville containing 10,000 cu m of ash from the May 18 eruption. Photograph by G. B. McLucas, Washington Division of Geology and Earth Resources.

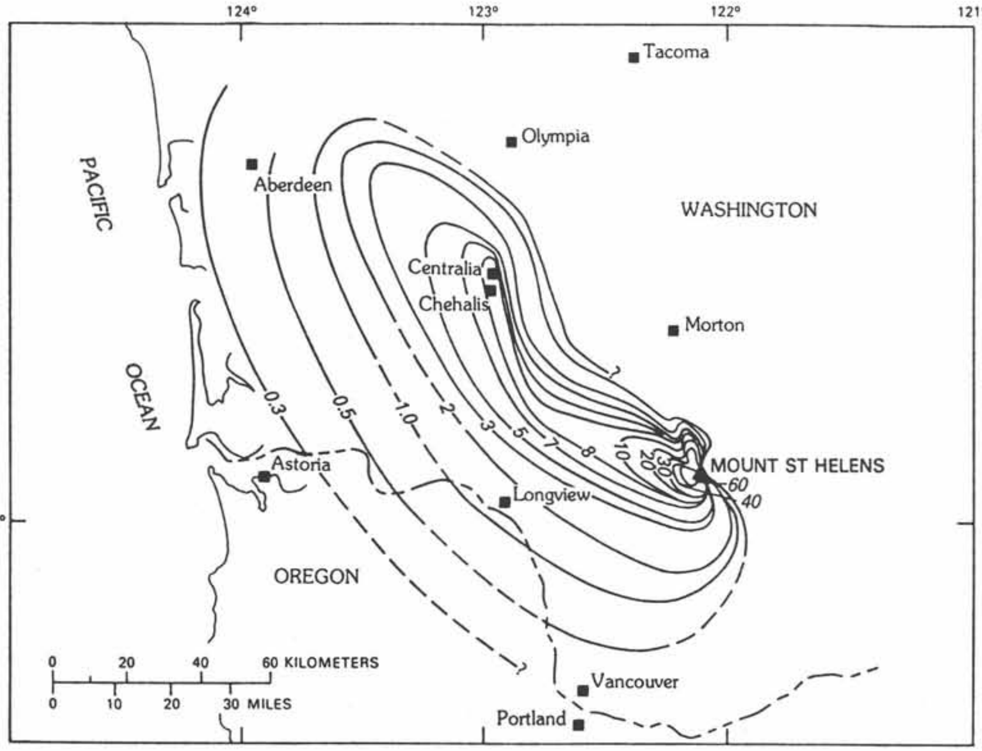

Figure 23. Isopach map of air-fall ash from the eruption of May 25. Ash thickness in millimeters. After Sarna-Wojcicki et al. (1981).

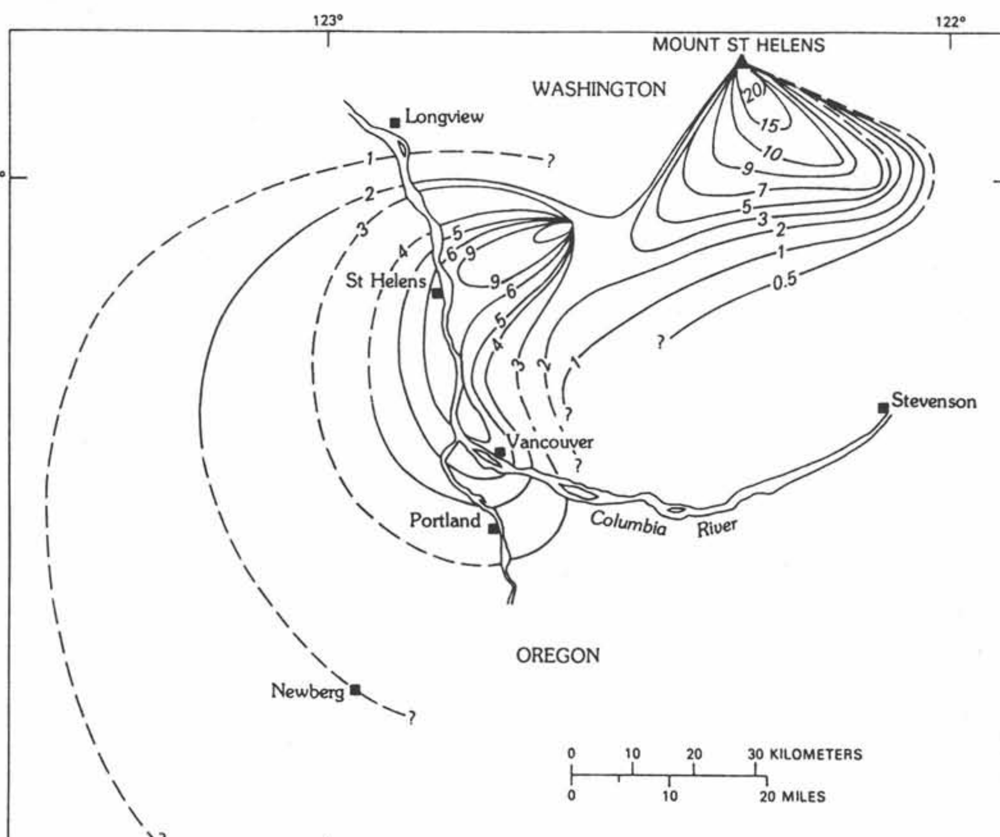

Figure 24. Isopach map of air-fall ash from the June 12 eruption. Ash thickness is in millimeters. After Sarna-Wojcicki et al. (1981).

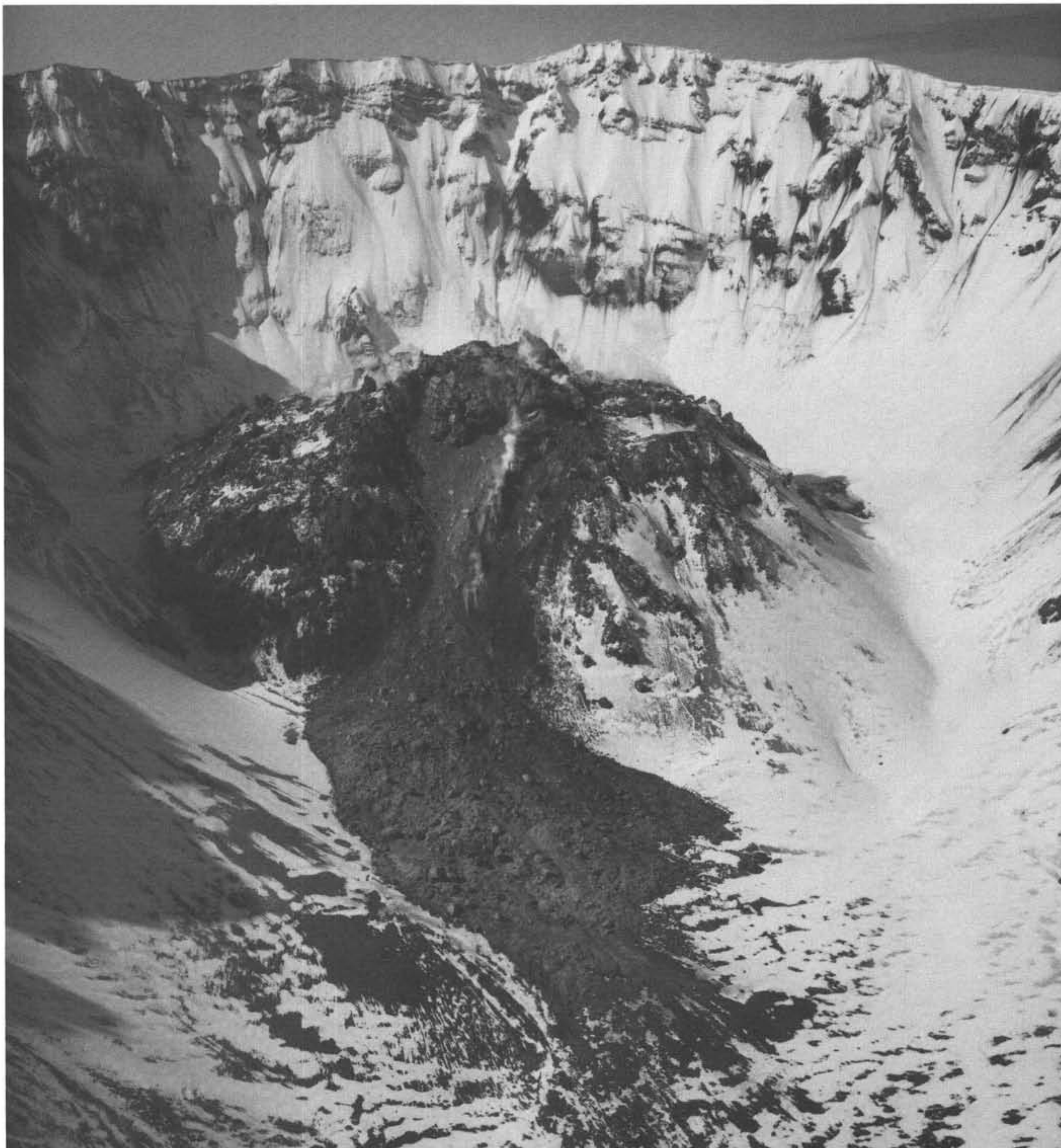

Figure 25. Mount St. Helens crater and dome, May 1986; view from the north. The dome at that time was 250 m high and about 1 km wide and had a volume of about 70 million cu m. Note the new large rock avalanche on the front of the dome. Photograph by Lyn Topinka, U.S. Geological Survey.

(1987, p. 50, 52) noted this stage of activity and discussed future hazards from the volcano:

"During past eruptive periods, such as the Kalama period of 500-350 yr ago (Hoblitt and others, 1980), the mean annual frequency of lahars and pyroclastic flows affecting areas within 10-20 km of the volcano increased greatly over the longer term mean annual frequency, which included repose intervals as well as eruptive periods. Annual probabilities of such events estimated from the Kalama period, which we infer are applicable to the present eruptive period, are more than 1×10^{-1}.

"Voluminous lahars, which occur less often, could affect areas far beyond a distance of 20 km. A recent study of the Toutle River valley (Scott, 1986) concludes that lahars or lahar-runout flows large enough to inundate flood plains 50 km or more from the volcano have an annual probability of at least 1×10^{-2}. Generation of lahars in the Toutle River basin and erosion of deposits of the current eruptive period will continue to aggrade river channels and flood plains farther downstream and will increase floodpeaks in the lower Toutle and Cowlitz River valleys and in the Columbia River near the mouth of the Cowlitz (U.S. Army Corps of Engineers, 1984[a]).

"In the near term (the next 1-10 yr), large explosive eruptions with widespread tephra fall and pyroclastic flows are less likely than continued dome building. The annual probability of a large (>0.1 cu km^{-3}) explosive eruption is about 1.5×10^{-3} for the past 4400 yr and 8×10^{-3} for the past 500 yr. It should be noted that two large tephra-producing eruptions occurred just 2 yr apart, in 1480 and 1482 A.D., during the Kalama eruptive period (Yamaguchi, 1985). Voluminous tephra deposits would result from high eruption columns; tephra falling back onto the volcano from such columns would increase the likelihood of pyroclastic flows and lahars affecting all flanks of the volcano.

"The Kalama and Lewis River valleys could also be affected by eruptions if the present dome continued to grow and eventually filled the crater. Pyroclastic flows from the upper part of the dome could then descend other flanks of the volcano. If such pyroclastic flows were as large as ones of the Swift Creek period, those on the south side could move down the valley of Swift Creek and enter the Swift Creek Reservoir. Pyroclastic flows could also melt snow on and beyond the south flank of the volcano and generate lahars and floods that could reach the reservoir."

SUMMARY AND CONCLUSIONS

People, property, engineered structures, civil works and operations, and transportation in the vicinity of, downstream from, and downwind from Mount St. Helens were severely affected by the 1980 eruptions. The growth of communities, development of forest, agricultural, and recreational land uses, and construction of civil works in areas downvalley and downwind from Mount St. Helens took place almost entirely after 1857, the year of the last eruption of this volcano before the 1980 events. During the following 123 years of inactivity of Mount St. Helens, communities, highways, bridges, hydroelectric dams and reservoirs, railroads, airports, and recreational developments were constructed in the region with little or no thought being given to the effects of possible eruptions on these developments and facilities. This disregard for possible volcanic eruptions in planning new developments has not been unique to the Mount St. Helens region, but has also characterized land use and development in the entire Cascade Range of Washington, Oregon, and northern California, a large area with many currently inactive, but potentially dangerous, volcanoes (Crandell et al., 1979).

This report has described examples of the effects of various kinds of volcanic events on structures and on other civil developments that were vital to the economic and social well-being of an entire region. Many of these events were specifically predicted in volcanic-hazards assessments of Mount St. Helens and other volcanoes in the Cascade Range (for example, Crandell, 1976; Crandell and Mullineaux, 1978; and Miller, C.D., 1980). Now that these recent events have occurred at Mount St. Helens and are within the experience of planners, engineers, and public officials, there is hope that potential hazards from volcanoes will no longer be disregarded but will help to guide future land-use planning in the vicinity of Mount St. Helens and other volcanoes. In future planning, conscientious choices should be made between the acceptance of the possibility of increased costs due to changes in locations or designs of developments and facilities, and the risk of much greater costs (both economic and social) that might result from an eruption. Judgments as to which alternative is preferable will vary from one kind of land use to another, and from one volcano to another; these judgments should be based on social and economic factors as well as on scientific appraisals of the potential hazards at each volcano.

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Geology and Construction of the Spirit Lake Outlet Tunnel, Mount St. Helens, Washington

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INTRODUCTION

Mount St. Helens in the Cascade Range of Washington is one of a series of active volcanoes comprising a part of the "Ring of Fire", a circle of volcanoes and earthquake activity rimming the Pacific Ocean. The mountain is about 45 mi northeast of the Portland-Vancouver metropolitan area (Figure 1). Before the eruption, the mountain stood 9,677 ft high; it was surrounded by state and private forest land and the Gifford Pinchot National Forest. A major feature of the landscape was 1,300-acre Spirit Lake, approximately 4 mi northeast of the mountain.

The eruption of Mount St. Helens began in the spring of 1980. Since then, the movement of millions of tons of sediment has created a serious threat of area flooding in the Toutle, Cowlitz, and Columbia rivers and of navigation disruption in the Columbia River.

On the morning of May 18, 1980, the north slope of Mount St. Helens collapsed following a magnitude 5.0 earthquake. This rock slide precipitated (almost simultaneously) the most catastrophic volcanic eruption in the continental United States in recorded history. A massive debris avalanche ensued, transporting an estimated 0.6 cu mi of debris into the upper North Toutle River drainage basin (Figure 1). This debris avalanche completely filled and dammed Spirit Lake, blocking the natural outlet to the North Fork Toutle River with a deposit several hundred feet thick. This filling caused the lake to rise 200 ft, to elevation 3,400 ft. At that elevation the lake volume was 126,000 acre-ft, 34,000 acre-ft less than the pre-eruption volume (160,000 acre-ft). By the summer of 1982, the lake had risen almost 60 ft higher, increasing the volume of water held back by the debris dam to nearly 275,000 acre-ft.

As an interim solution, a temporary pumping plant on a barge and a pipe line were installed in November 1982. The barge held 20 large-capacity pumps that pumped a total average flow of 180 cfs through a 5-ft-diameter pipe to a discharge point approximately 3,450 ft from the lake. That facility was able to maintain the lake level at about elevation 3,460 ft until April 1985, when the completed tunnel went into service.

The debris dam consists mainly of silt- to boulder-size material that was deposited by the debris avalanche. Overlying this material is a relatively thin deposit (generally 10 to 15 ft thick) of rock fragments called the blast deposit. This term refers to fragments ejected from the mountain. Overlying this debris avalanche material is a layer of ash and pumice from pyroclastic flows and airfall ("ash-cloud") deposition that occurred during the May 18 and subsequent eruptions. This layer ranges in thickness from a few inches to many tens of feet where the pyroclastic flows were channeled into depressions on the debris-avalanche-deposit surface. Minor mudflows from the mountain since the main eruption have added to the material forming the Spirit Lake blockage. The deposits making up this blockage are, by nature, extremely erodible and constitute a very large sediment source.

In 1982, a governmental task force was formed to evaluate the hazard posed by the blockage of Spirit Lake. The group determined that because of the composition of the debris-avalanche deposit, subsidence potential, and active erosion, the debris dam could not safely pond water above elevation 3,475 ft. If no preventive measures were taken and the area received average precipitation, it was estimated that Spirit Lake would reach elevation 3,475 ft by March 1983. The task force concluded that if Spirit Lake were allowed to reach that level, the debris dam could be breached, probably by piping through the uppermost ash (pyroclastic and airfall) material. This failure could cause catastrophic flooding and widespread damage in the Cowlitz River valley and could interrupt Columbia River navigation.

As a result of these findings, the President of the United States directed the Federal Emergency Management Agency (FEMA) to design and construct an immediate, interim solution to mitigate the flood threat and to study alternatives for a permanent solution. FEMA, in turn, charged the U.S. Army Corps of Engineers with these tasks, which led eventually to the design and construction of the Spirit Lake outlet tunnel.

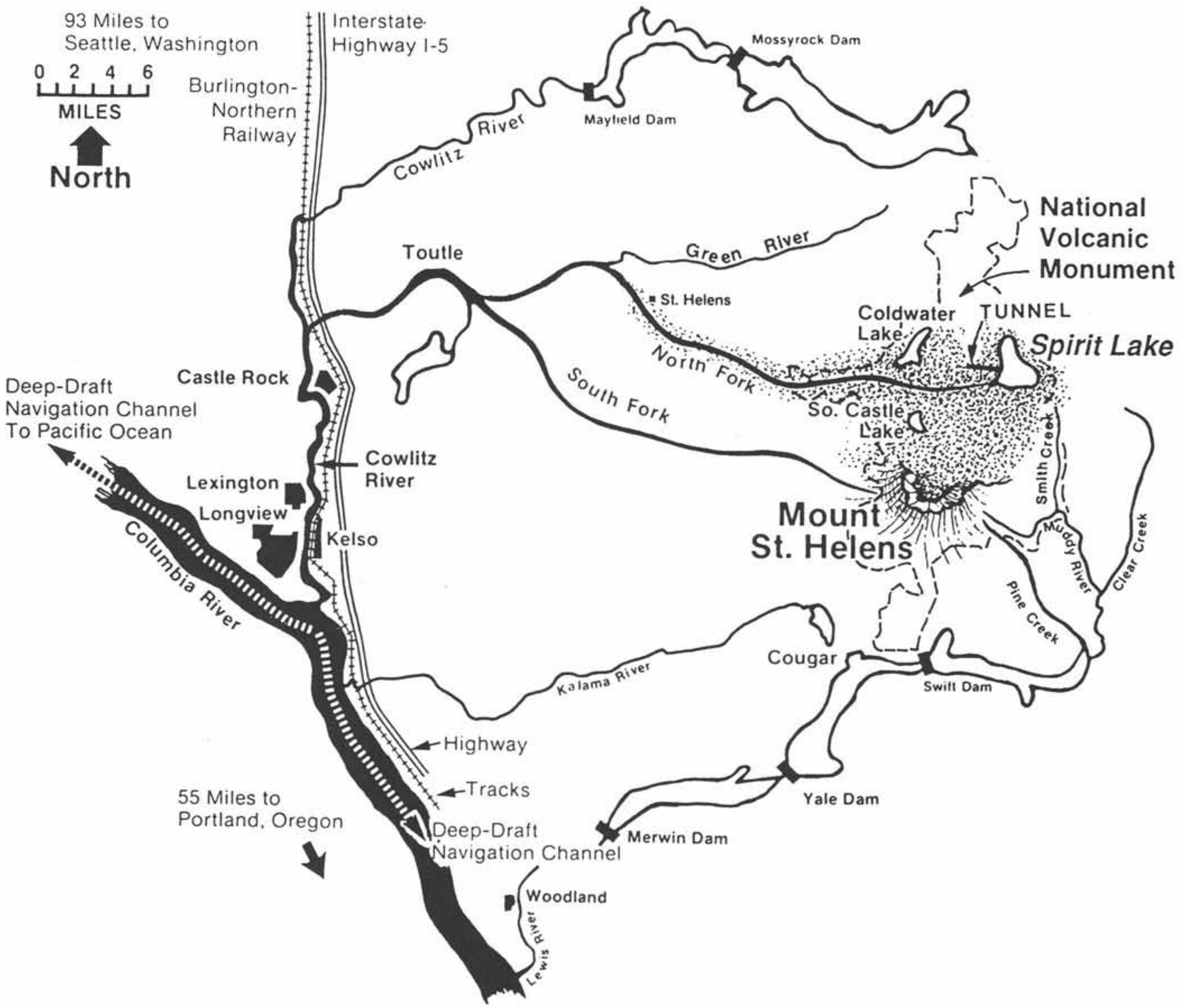


Figure 1. Location of Mount St. Helens and Spirit Lake.

GEOLOGY

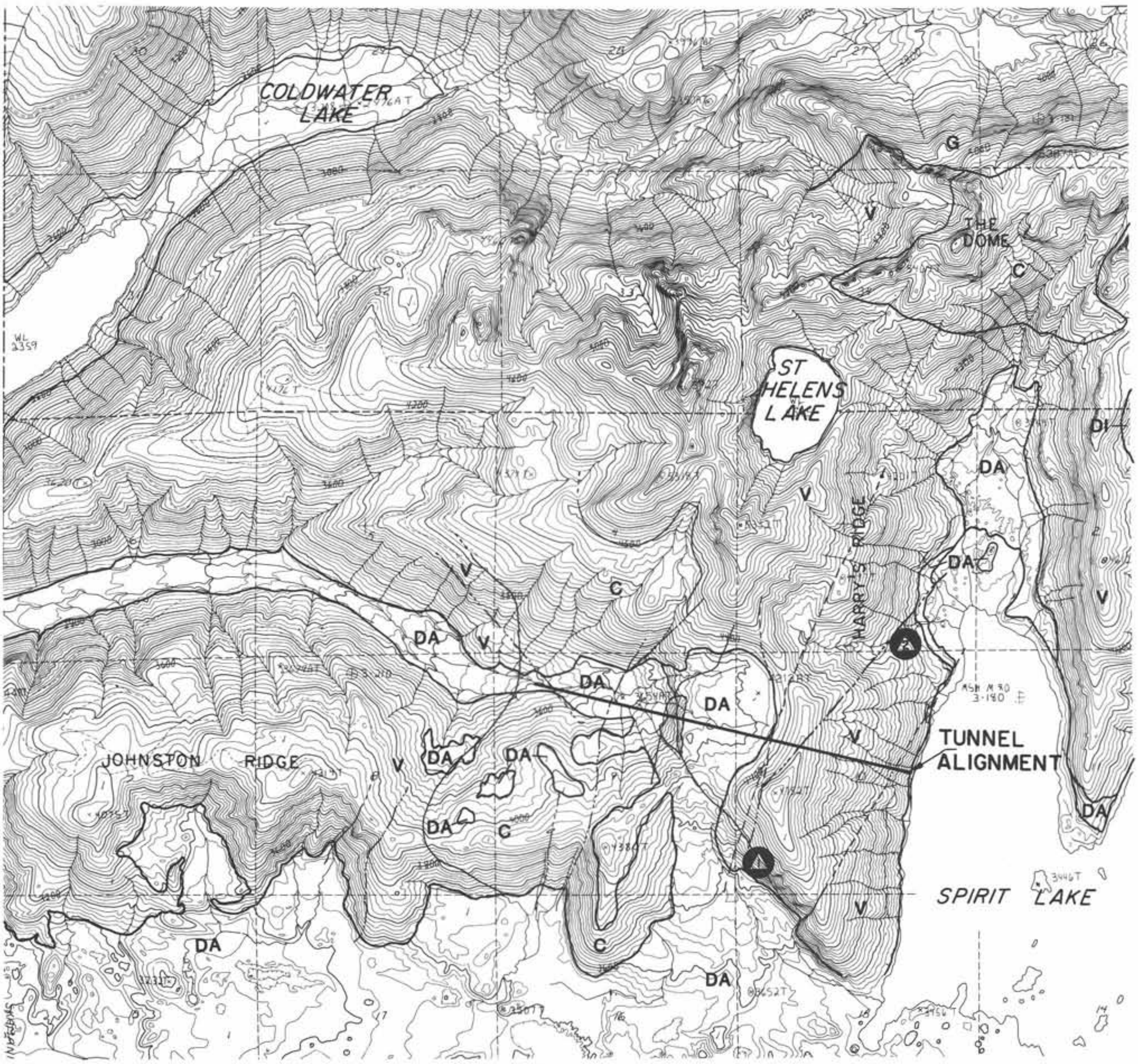
Overburden Materials

Because of glacial scour, a minimum of weathered rock is found in the study area. Glacial moraine, slope wash, and related deposits are found locally in the valleys and in some places on the hillsides. In the past, the region has been overlain, at least locally, by volcanic debris and pumice from previous eruptions of Mount St. Helens. The 1980 eruption deposited avalanche, pyroclastic-flow, and blast material, as well as a layer of volcanic ash. The overburden materials range from silt and very fine grained ash to boulders, and the deposits vary in thickness from 0 to 80 ft along the tunnel alignment.

Bedrock Stratigraphy

The tunnel was excavated in bedrock composed of layered Tertiary tuffaceous and volcanic flow rocks. The generalized geologic map of the area is shown in Figure 2 (Evarts and Ashley, 1984). Rock types present along the tunnel alignment (Figure 2) include tuffs (fine-grained tuffs, lapilli tuffs, tuff breccias, welded tuffs), basalt or andesite flow rocks, and flow breccias. Intrusive dikes of composition similar to that of the flow rocks are also present, mainly toward the downstream (west) end of the tunnel alignment.

The rock is moderately to highly fractured. Many fractures are tight, but some are open or contain clay fillings. The rocks toward the east end of the alignment



Legend

Contacts Between Rock Units
- Known
- Inferred

Faults
- Mapped Trace
- Inferred Trace

Symbols
- 38 Dip and Strike of Rock Unit
- Fault Dip and Direction of Dip

1000 0 1000 2000 3000
SCALE IN FEET

Geologic Units

DA DEBRIS-AVALANCHE DEPOSIT, Unconsolidated debris deposits from 18 May 1980 Mount St. Helens eruption.

V VOLCANIC ROCKS, Predominantly basaltic andesite or andesitic flow breccia units, including localized discontinuous units of epiclastic and pyroclastic origin.

C CLASTIC ROCKS, Predominantly pyroclastic and epiclastic rock including ash-flow and air-fall tuffs, laharic breccias, mudstones, siltstones, sandstones, conglomerates, breccias of unknown origin and local thin lava flows.

DI DIORITE AND QUARTZ DIORITE, Occur as irregular but generally sill-like bodies.

G GRANITE, GRANODIORITE, QUARTZ MONZONITE, AND QUARTZ DIORITE, Occur as Spirit Lake pluton.

Figure 2. Generalized geologic map of the Mount St. Helens tunnel area. Triangles indicate approximate locations of faults that were projected to intersect the tunnel alignment. After Evarts and Ashley, 1984.

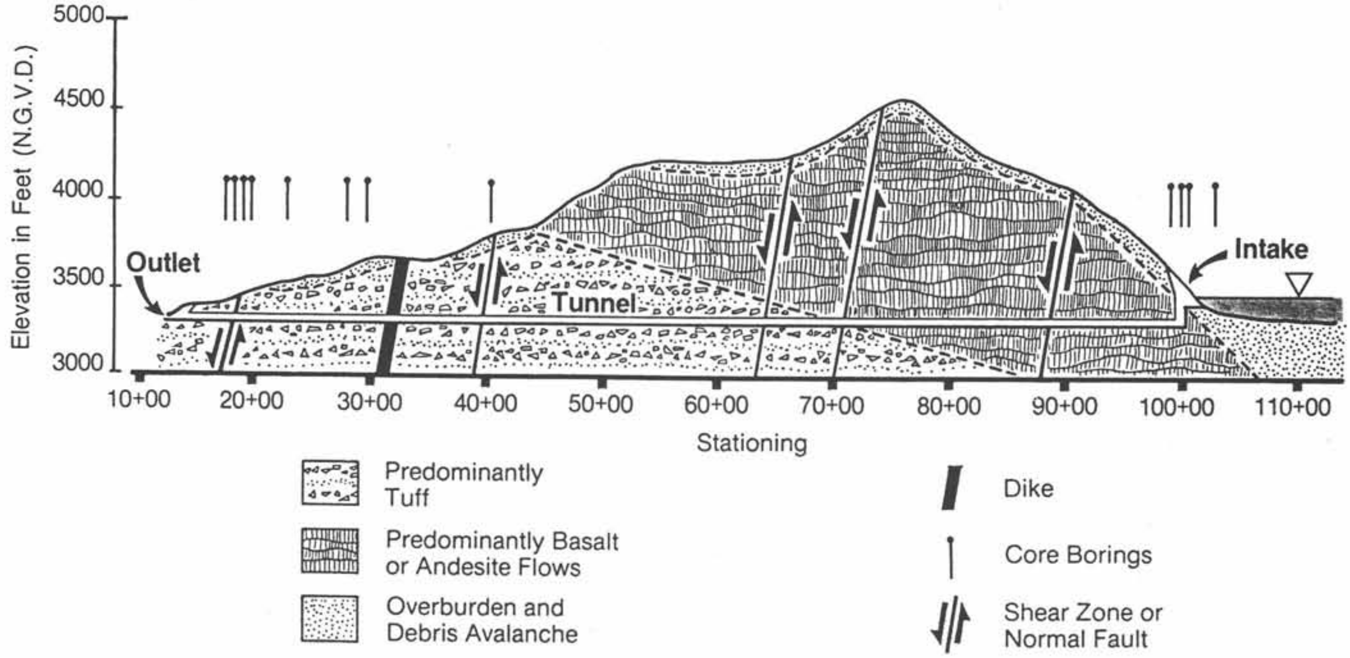


Figure 3. Geologic profile of the Spirit Lake Tunnel; this section represents the general geology and major geologic features encountered.

have undergone chemical alteration and have been brecciated and rehealed with secondary mineralization. As a result, the tuffaceous rocks in this area are now stronger than they originally were and are more similar in appearance to the volcanic flow rock.

The tuffaceous rock is continuous over the downstream 40 percent of the tunnel length. The individual beds of these various rock types vary in thickness from a few tens of feet to more than 200 ft. The tuffs are primarily light green, dense, and fresh and contain angular, multicolored fragments in the lapilli beds. A few layers have a very dark brown to black matrix. A zone of red tuff, ranging from ash to lapilli, is sandwiched in the flow rock toward the east.

The flow-rock layers range from less than 100 to several hundred feet thick. The individual flows commonly have a layer of flow breccia between them. The contacts are gradational and, in some places, hard to detect. The flow breccias are fresh, dense, and red to purple. The flow rock is fine grained, dense, fresh, hard, and for the most part black to blue-black. A few locations along the alignment had porphyritic and/or amygdaloidal zones. No areas having pronounced mineralization were noted.

Most dikes are present in the tuff beds, and most cut these beds at high angles, cross the tunnel alignment at near right angles, and range from 10 to several tens of feet thick.

Bedrock Engineering Properties

The bedrock is dense, with unit weights ranging from about 154 to 178 pcf. Testing of samples for the various rock types indicated unconfined compressive strengths ranging from 5,000 to 15,000 psi for the tuffs and flow breccias and 15,000 to 35,000 psi for the flow rocks. Rock core from subsurface drilling indicated good quality rock, with fracture spacings ranging from a tenth of a foot to several feet. Rock Quality Designation (RQD) values obtained from the drilling ranged from 0 to 100 percent; average RQD was estimated to be in the 70 to 75 percent range (moderately to very blocky and seamy). Permeability of the rock mass is controlled by rock fractures. Because many of the fractures are tight, permeability is generally low; however, some hydraulic pressure tests indicated open fractures (U.S. Army Corps of Engineers, 1984).

The in-place rock mass varied from massive and intact over most of the tunnel to crushed where shear zones crossed the tunnel alignment. Both the tuffaceous rock and the flow rock stood well after tunneling except for certain parts of the shear zones. Some short sections of the tunnel had spalling rock in the more ashy tuffaceous beds with a large amount of cover. In one section, squeezing ground was present along with some spalling and popping rock in a sandwiched tuff bed in the flow rock under Harry's Ridge (Figure 2).

Bedrock Structure

The rock units in the region have been folded and are cut by shear and fault zones. The layered tuffs and flow rocks along the tunnel alignment are on the limb of a regional fold. The beds dip to the east between 30° and 40°, and the strike is normal to the tunnel centerline (with minor local deviations).

Twelve shear zones and several shear planes were encountered along the tunnel. The affected areas vary from a few feet to more than 50 ft across. Most of the shear zones and shear planes are steeply dipping and cross the tunnel nearly normal to the centerline. There are seams of clay gouge in the shear zones, but most of these are thin. The primary condition in the shear zones is fractured rock in various stages of decomposition and some fines of medium to low plasticity.

Rock joints and fractures of various spacings and orientations also exist along the alignment. Those in areas of minimum cover toward the west end have opened due to glacial unloading and have been filled with fines.

SITE INVESTIGATIONS

Geologic exploration at the tunnel site consisted of core drilling and surface mapping. Strength testing of representative rock specimens was also performed. The short time available for exploratory work, the depth of cover, the rough terrain and remote nature of the site, and time of year all severely limited the amount of exploration work that could be accomplished. The work was begun in March and completed in May of 1984. Ten core borings were drilled during this period. Three holes were drilled under contract at the intake, and seven were drilled by the U.S. Army Corps of Engineers for approximately the downstream 30 percent of the tunnel alignment (Figure 3). Two other holes were drilled for this tunnel alignment in 1982 and 1983, when several tunnel routes were being considered. The 12 holes totaled 1,553 linear feet of drilling. Further geologic investigations for the rest of the alignment consisted of surface mapping to define rock units and structure (Figure 3). Unpublished geologic maps made by the U.S. Geological Survey (USGS) during 1981 and 1982 were also used. The geology along the tunnel alignment was mapped during tunnel excavation by Corps geologists.

TUNNEL CONSTRUCTION

Fourteen firms submitted bids for construction of the Spirit Lake tunnel; five firms bid a drill-and-blast option and nine bid the tunnel boring machine (TBM) option. On June 7, 1984, a contract in the amount of $13,469,247 was awarded to a joint venture of Peter Kiewit and S. J. Groves. The contractor selected the TBM option, choosing to use an 11-ft-diameter Robbins Model 119-222 boring machine that previously had been used at the Terror Lake Project in Alaska. Weighing approximately 112 tons, the 55-ft-long TBM has a partially shielded cutterhead with 27 17-in.-diameter disc cutters; a two-gripper advance system; four electric motors, each with a 200-hp capacity; and about 400 ft of trailing gear and platform cars (Figure 4). The Corps contractor proposal accepted substituted smaller (W4 x 13) rib sets than were specified and would use precast concrete sections in lieu of shotcrete for the tunnel invert (Figure 4). The precast invert slabs worked satisfactorily; however, many of the slabs broke or cracked during construction.

The tunnel was driven from the downstream end toward Spirit Lake, concurrently with excavation for the intake at the upstream end. At its maximum depth, the tunnel was mined 1,200 ft below ground. The first 230 ft of tunnel were excavated by drill-and-blast to proceed past a known shear zone before starting machine boring. The TBM started operations on September 28, 1984. On March 5, 1985, the TBM holed through into the completed intake structure transition, and the control gate opened in late April 1985.

In the portion of the tunnel mined by the TBM, rib sets were used in areas determined by the Corps field geologist or the contractor. Behind the TBM, steel fiber-reinforced shotcrete was applied by the wet-mix process, in areas determined by the Corps field geologist, to prevent possible future erosion, plucking, or sloughing. Mining and support were accomplished on a 6-day-week, 3-shift/day schedule. The average advance rate during TBM operation was 65 ft/day. Total ground-water inflow into the tunnel varied from a sustained average of 600 gpm to maximum short-period inflows as high as 1,200 gpm in one fault zone that contained a relatively large amount of trapped water.

A total of 241 rib sets encased in shotcrete was used in nine shear/faulted rock zones. The foot plates of the rib sets were shaped to fit the edges of the precast invert slabs. The rib sets were blocked with timber wedges until shotcrete placement. Where squeezing ground was encountered (as predicted), rib-set spacing was decreased from 4 to 2 ft. In the worst ground, invert W4 x 13 struts were welded in place, and concrete was placed to match the invert finish surface (U.S. Army Corps of Engineers, 1987).

A total of 2,760 cy of shotcrete was placed in the TBM portion of the tunnel. No rock bolts were required. Shotcrete thickness averaged about 2 in. in the areas where no steel sets were installed and about 7 in. where it was necessary to cover the sets.

SUMMARY AND CONCLUSIONS

The Spirit Lake tunnel was mined through volcanic rock that consists primarily of moderately hard tuff and hard basaltic rock. The rock conditions were amenable to use of the TBM used to construct the tunnel, with the

TUNNEL BORING MACHINE

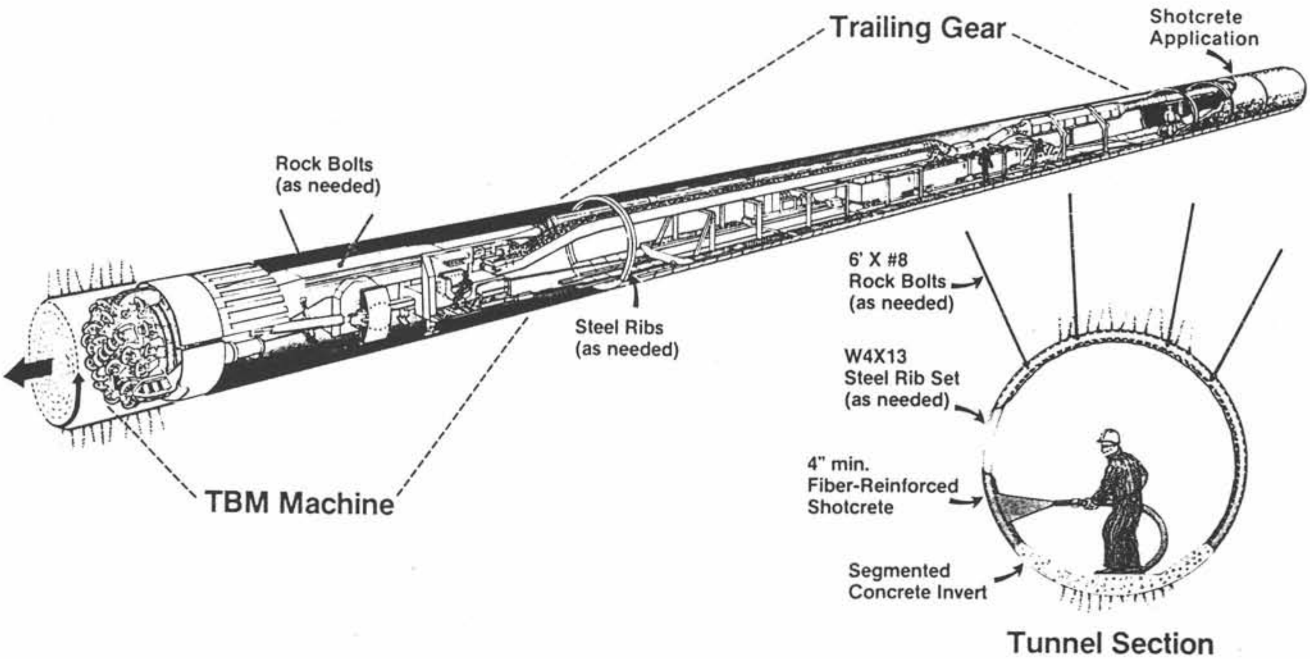


Figure 4. Sketch of the tunnel boring machine (TBM) used at the Spirit Lake tunnel. Courtesy of U.S. Army Corps of Engineers, Portland District.

exception of the segment containing the major fault/shear zones near the center of the tunnel. The geologic conditions predicted for the tunnel were very close to those actually encountered during construction.

The total cost of the Spirit Lake Outlet Project was approximately $29,000,000. This figure included about $12,350,000 for the cost of the interim pumping necessary to maintain the lake level at a safe elevation until the tunnel became operational. It also included approximately $1,100,000 for engineering and design (including $793,000 for exploration) and $1,600,000 for supervision and administration during project construction. The tunnel bid price of $13,469,247 was increased to a final cost of about $13,660,000 as a result of change-order work added during the contract. Considering that the government estimate at bid opening was $12,090,723 and the highest bid was $25,123,900, the final cost of the tunnel appears to have been reasonable (Sager and Chambers, 1986).

As he spoke at the April 27, 1985, Spirit Lake tunnel dedication, Booth Gardner, Governor of the State of Washington, summed up the feelings of those people who had lived below Mount St. Helens for 5 yr, always wondering whether Spirit Lake was going to remain leashed. "The completion of the tunnel, an extremely impressive engineering feat, relieves the threat on the populated areas," he stated, "and allows the people of Cowlitz County to move one step closer to business as usual." Today the Spirit Lake tunnel serves and protects the people who live downstream of Mount St. Helens.

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