

EVALUATION OF LIQUEFACTION POTENTIAL IN SEATTLE, WASHINGTON

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ABSTRACT

Seattle has experienced significant damage related to liquefaction during historical earthquakes and may be subjected to even greater damage in the future due to increased development of the area and the potential occurrence of a subduction-zone earthquake. The methodologies and results of two research studies that evaluated the liquefaction hazard to Seattle are discussed and compiled in a single map indicating the local liquefaction-hazard potential. Delineation of the liquefaction hazard in the area will benefit land-use planning, future building development, and planning for disaster response.

All liquefaction evaluations were based upon existing data from more than 350 boring logs and an empirical procedure that relates Standard Penetration Test *N*-values to threshold ground accelerations needed to initiate liquefaction. A computerized database was developed to facilitate storage and retrieval of the boring data for subsequent analyses. The liquefaction potential was evaluated using two procedures. One procedure grouped similar geologic units and assigned relative rankings to the liquefaction potential based upon the percentage of Standard Penetration Test *N*-values that fell below a threshold *N*-value needed to resist liquefaction resulting from a 0.30*g* (where *g* is equal to 9.8 meters per second) ground acceleration. The second procedure assigned relative liquefaction-potential rankings based upon the computed thickness of material in individual borings that would liquefy for ground accelerations of 0.15*g* and 0.30*g*. A review of all major geologic units within the study area for liquefaction potential, using both criteria and assigned hazard ratings of high, moderate, low, or very low, showed that fills and Holocene alluvial deposits at the mouth of the Duwamish River have a high liquefaction potential; however, Holocene alluvium and beach deposits elsewhere were given a moderate liquefaction rating. Pleistocene

alluvial sedimentary deposits were given a low liquefaction rating, and glacially consolidated Pleistocene sedimentary deposits were assigned a very low potential.

INTRODUCTION

Earthquake-induced liquefaction and related ground failures have caused substantial casualties and major property losses in various parts of the world. Property losses in excess of \$800 million have been attributed to liquefaction-related ground failures that occurred during the 1964 Niigata, Japan, earthquake (Keefer, 1983). Also, property losses related to liquefaction-induced ground failures were estimated to have exceeded \$200 million in the March 27, 1964, Alaska earthquake (Keefer, 1983). During the Alaska earthquake, soil liquefaction induced lateral spreads that compressed or buckled more than 250 bridges, disrupting railroad and vehicular traffic. Liquefaction also generated subaqueous landslides that destroyed sections of the waterfronts of Valdez, Seward, and Whittier.

Earthquake-induced ground failures during the 1949 and 1965 Puget Sound, Washington, earthquakes resulted in substantial damage to buildings, bridges, highways, railroads, water distribution systems, and marine facilities. Property damage from the 1949 and 1965 events totaled \$25 million and \$12 million, respectively. Grant (1986) estimated that 25–50 percent of the total damage from these earthquakes may be attributed to earthquake-induced ground failures such as liquefaction. Whereas this amount of damage may seem relatively minor when compared with other major earthquakes, the damage is consistent with the relatively low levels of ground acceleration (typically less than 0.10*g*, where *g* is equal to 9.8 m/s) that were recorded in Seattle during these events. Should Puget Sound experience a major Cascadia subduction-zone earthquake of magnitude 8.0 or greater, as postulated by Heaton and Kanamori (1984), damage from earthquake-induced ground failures could easily be an order of magnitude higher than the damage experienced in past events.

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This report summarizes the results of two research studies (Grant, 1990; Perkins, 1991) that evaluated the liquefaction-hazard potential in Seattle (fig. 215). Grant (1990) was a U.S. Geological Survey (USGS) sponsored study in which a computer database of existing borings in Seattle was developed and used to evaluate liquefaction potential based

on empirical procedures by Seed and Idriss (1971), Seed and others (1983), and Seed and others (1984). Liquefaction-hazard categories were differentiated on the basis of comparing SPT (Standard Penetration Test) N values for a geologic unit with the N values required to resist liquefaction during an earthquake with a 0.30g ground acceleration. Perkins

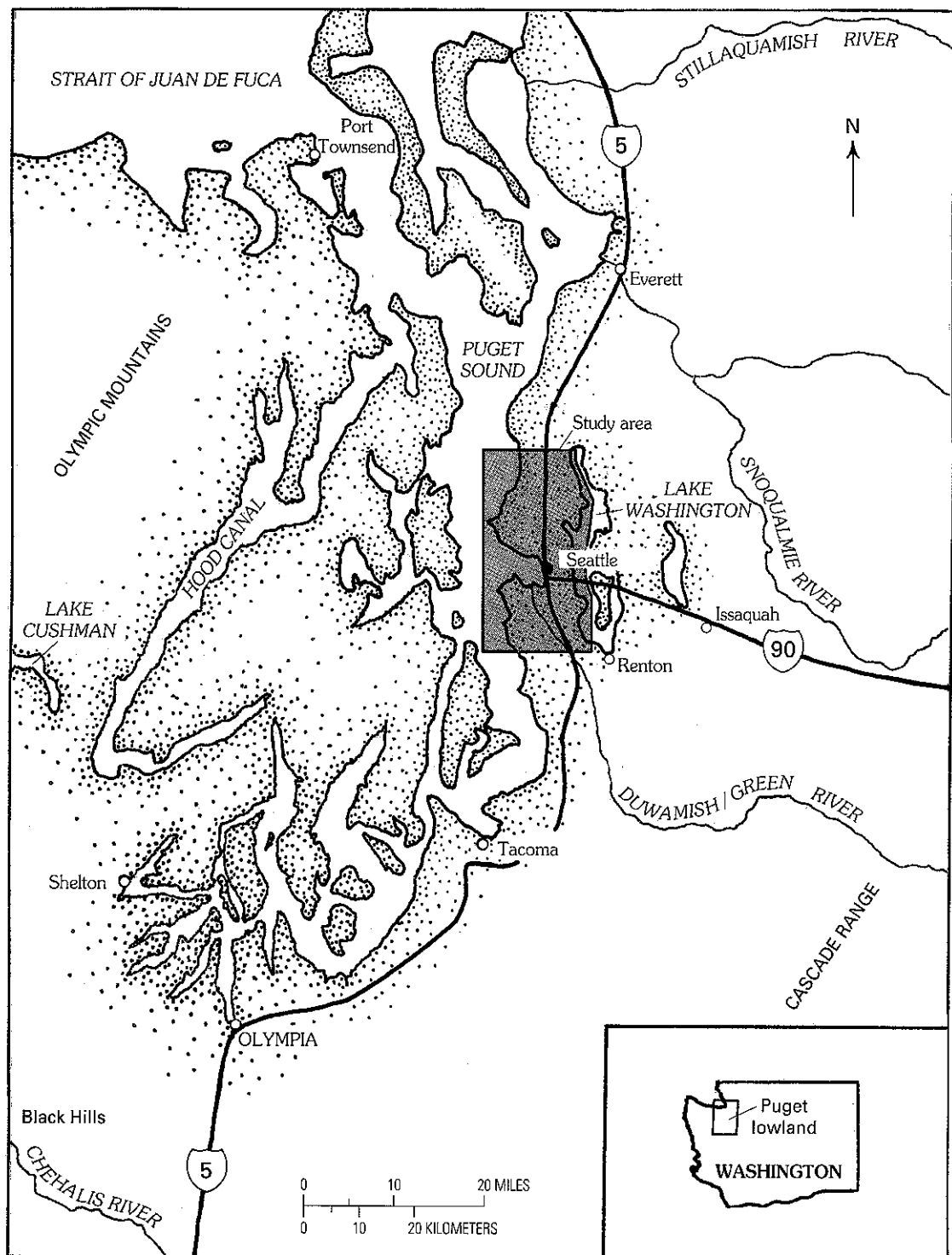


Figure 215. The Puget lowland and location of the study area, Washington.

(1991) used the same database but established liquefaction-hazard categories based upon the cumulative thickness of material liquefying from either 0.15g or 0.30g ground accelerations. The results from both of these studies were combined, resulting in the single liquefaction-hazard map contained on plate 9.

Identifying areas where liquefaction may potentially occur within Seattle provides a tool to aid government agencies in land-use planning, building development, and planning for disaster response. The liquefaction-potential map developed as a result of this study may be used by engineers, city officials, and planners to assess the need for changes in zoning ordinances or building codes to mitigate the hazard. For example, building codes could be modified to require site-specific liquefaction assessments and appropriate foundation designs for structures located in high-risk areas. The map also could be used by engineering departments within various governmental agencies and the insurance industry to estimate the damage potential to existing buildings during a future earthquake, information that may be used to prioritize structures for seismic retrofitting. Finally, the liquefaction-hazard map could be used by emergency-response planners to anticipate areas within the city that may sustain high damage and casualties or where the infrastructure (for example, roads, bridges and water-supply lines) may be particularly vulnerable, affecting emergency response efforts.

ACKNOWLEDGMENTS

The authors gratefully acknowledge the support and sponsorship of this project by the USGS under the Earthquake Hazards Reduction Program. The support provided by Ms. Deborah Walsh, the contracting officer with the USGS, is greatly appreciated. We also wish to acknowledge the efforts of Ms. Joanmarie Gorans-Eggert of Shannon & Wilson, Inc., in creating the computerized database for subsurface boring data and the help of Mr. Steven Bartlett, a research assistant at Brigham Young University, in providing the computer software to evaluate the thickness of liquefied sediments. Finally, we appreciate the valuable technical review of this paper by Dr. Steve Obermeier of the USGS and Dr. Steve Palmer of the Washington State Department of Natural Resources.

THE LIQUEFACTION PHENOMENON

Liquefaction is a phenomenon in which saturated, cohesionless soils are temporarily transformed into a liquid state, most commonly as a result of earthquake-induced ground shaking. Liquefaction occurs as a result of the buildup of excess pore-water pressures during the shaking. When the pore-water pressure exceeds the grain-to-grain (effective) contact pressure of the soil, the soil particles lose contact

with each other and the soil essentially behaves as a liquid. Pore-water pressure in a liquefied soil may become so great as to result in small geysers from which water is ejected, leaving sedimentary features commonly termed sand boils.

The development of liquefaction is controlled by a number of complex and interrelated factors. These factors can be generally related to the following parameters: (1) the strength of the underlying soil deposit, (2) the location of the water table, and (3) the severity of earthquake ground shaking. Historically, clean sands and silty sands are the materials most susceptible to liquefaction. In addition, the liquefaction resistance of any particular soil is affected by its density, fabric, prior earthquake history, and in-place stress conditions. The depth of the water table is a controlling factor because a soil must be saturated or located below the water table for liquefaction to occur. Within any given soil deposit, liquefaction is more likely to occur where the water table is shallow as opposed to conditions where the water table is depressed. Finally, development of liquefaction is dependent upon the magnitude of the earthquake stresses induced in the soil deposit and also the duration of ground shaking. The stresses and duration of ground shaking are, in turn, affected by the size and location of the earthquake, the travel path of the earthquake motions to the site, and any local amplification of ground motions that may occur within the soil column.

Depending on site-specific factors, liquefied soil can cause various types of ground failures that may damage overlying structures. A common result is loss of bearing capacity for shallow foundations located over the liquefied soil. Other adverse effects include flow failures, buoyant rise of buried structures, ground settlement, failure of retaining walls due to an increase of lateral pressures, and lateral spreading.

HISTORICAL LIQUEFACTION IN SEATTLE

Historical records provide valuable information for assessing earthquake-induced liquefaction potential. Specifically, areas that have experienced liquefaction during past earthquakes may liquefy again during a future event. For example, reconnaissance reports from the 1989 Loma Prieta earthquake (U.S. Geological Survey, 1989) indicate that many of the areas that experienced liquefaction and unusually severe ground shaking during the 1906 San Francisco earthquake experienced similar damage patterns in the recent event. In both of these earthquakes, locations of uncontrolled, random fills significantly correlated with severely damaged areas. Therefore, in our study of the liquefaction potential of soils in Seattle, we first reviewed accounts of historical liquefaction in the area and then correlated these accounts with historical maps that show tideland

reclamation along shoreline areas. This information would likely identify the soil conditions having the highest relative liquefaction potential.

Table 34 provides accounts of liquefaction occurrences in Seattle during the 1949 and 1965 Puget Sound earthquakes. The April 13, 1949, event was located about 63 km south-southwest of Seattle, near Olympia, and had a surface-wave magnitude (M_s) of 7.1 (Weaver and Baker, 1988). The body-wave magnitude (m_b) 6.5 earthquake of April 29, 1965, occurred about 23 km south of Seattle (Weaver and Baker, 1988). As indicated in table 34, liquefaction during these earthquakes typically resulted in differential settlement of buildings, lateral movement of bulkheads, and cracking of basement walls.

Because uncontrolled fills have been particularly susceptible to earthquake-induced liquefaction during historical time, old topographic maps of Seattle were reviewed to delineate fill areas. The changes to Seattle shorelines and their associated fills are shown in figure 216. Additionally, the figure shows the locations of sites that liquefied during the 1949 and 1965 earthquakes, the majority of which coincide with shoreline areas that were filled during the early development of Seattle. Experience from earthquakes in other regions also suggests that areas of uncontrolled fill may be particularly vulnerable to liquefaction during a future event.

A final factor that is needed in understanding the historical accounts of liquefaction in Seattle is the level of ground shaking, or peak ground acceleration, that occurred locally during the 1949 and 1965 earthquakes. Fortunately, both events were locally recorded on strong-ground-motion accelerographs (Shannon & Wilson, Inc., and Agabian Associates, 1980), which were located close to areas that experienced liquefaction (fig. 216). These accelerographs recorded peak ground accelerations of about 0.10g during both the 1949 and 1965 earthquakes (California Institute of Technology, 1976; Shannon & Wilson, Inc., and Agabian Associates, 1980). Based upon seismicity studies conducted by Shannon & Wilson, Inc. (1980) for various sites within the Puget Sound region, it is estimated that this level of ground acceleration may have a 20- to 40-year recurrence interval.

GEOLOGIC FRAMEWORK

Because liquefaction resistance can generally be correlated with the age of a geologic deposit, understanding the origins and ages of the different geologic units in the study area helps establish a framework for categorizing potentially liquefiable soils. In the following discussion, primary emphasis is given to the glacial deposits in the region.

REGIONAL GEOLOGY

Seattle is located in the Puget lowland, which is a slightly arcuate, convex-eastward basin lying between the Cascade Range on the east and the Olympic Mountains on the west (fig. 215). The basin is open on the north to the Georgia depression and the Strait of Juan de Fuca, the latter connecting Puget Sound with the Pacific Ocean. Beneath the Puget lowland, nonlithified Quaternary sedimentary deposits of varying thickness generally unconformably overlie Tertiary bedrock. These sedimentary deposits are both glacial and nonglacial in origin.

The incursion of Pleistocene continental ice into the basin is well documented (Willis, 1898; Bretz, 1913; Mackin, 1941; Crandell and others, 1958; Armstrong and others, 1965; Crandell, 1965; Easterbrook and others, 1967; Mullineaux, 1970; Crandell and Miller, 1974; Blunt and others, 1987). Ice originating in the coast and insular mountains of western British Columbia, Canada, coalesced in the Georgia depression and moved south across the 49th parallel to the south end of the Puget lowland, about 80 km south of Seattle. At least four major advances and several partial advances have been identified. Although highly complex, each advance left a sequence of lacustrine, advance outwash, glaciomarine drift, till, and recessional outwash deposits. The nonglacial intervals generated combinations of fine-grained deposits of fluvial and lacustrine origin and also some organic deposits, except along the basin margins, where coarse fluvial deposits and mudflows predominate. The trend of the existing ridges, valleys, and deep inlets of Puget Sound is north-south, with the valleys being scoured to great depths. The thickness of the total unconsolidated basin fill varies from trace amounts in scattered locations throughout the lowland to over 1,100 m in the central part of the basin near downtown Seattle (Hall and Othberg, 1974; Yount and others, 1985) (fig. 217).

The bedrock underlying the Puget lowland is not well understood because of the thick and pervasive mantle of Pleistocene deposits. The sparse knowledge of the configuration of the bedrock surface has been interpreted from geophysical data, a few deep borings, projection of surface exposures along the basin flanks, and several bedrock ridges that partly cross the basin along northwest trends. The rocks are mostly folded, faulted, and deeply eroded Tertiary marine and estuarine sedimentary materials; volcanic materials consisting of basalt, andesite, and volcanoclastic rocks; and terrigenous deposits such as sandstone, shale, and conglomerate, including extensive interbedded coal seams that lie along the Cascade Range flank east and south of Seattle. Gravity- and magnetic-survey data show high differentials in the bedrock elevations, a result most likely related to major faulting. In fact, some of the steepest gravity gradients in the United States have been measured in Seattle (Danes and others, 1965; Rogers, 1970). One of these steep gravity gradients coincides with the Olympic-Wallowa lineament, a

Table 34. Descriptions of liquefaction occurrences in Seattle during the 1949 and 1965 Puget Sound earthquakes.

[Location numbers correspond to those used in figure 216]

Location no.	Location	Quotation and (or) comment	Source
Earthquake of April 13, 1949			
1	Near south end of Green Lake (NE¼ sec. 22, T. 25 N., R. 4 E., Seattle North 7½ quadrangle)	A 60-foot [18 m] section of the south shore of Green Lake dropped four feet [1.2 m] into the lake during the earthquake Wednesday. Cracks three or four inches [8-10 cm] wide also appeared on the bicycle path a short distance from the slide. The ground was broken up and parts of the path appeared to have dropped from four to six inches [10-15 cm] into the lake.	Seattle Post-Intelligencer (1949)
2	Pier 66	The 1949 earthquake resulted in displacement of the transit shed in a seaward direction. The column displacement amount to a maximum of about 9 inches [23 cm] in a lesser displacement at the north end of the north portion.	Olsen (1978)
3	Pier 36 area; exact location not determined	A concrete wall around a tank farm adjacent to the Duwamish waterway indicated considerable earth movement. One east-west wall about 100 feet [30 m] long and 12 feet [4 m] high reveals three vertical construction joints opened 1-5/8, 2, 1-3/4 inches [4.1, 5.1, and 4.4 cm], or a total of 6 inches [15.2 cm] during or since the quake. The joint filler in a north-south wall was squeezed out a maximum of 3 inches [7.6 cm] at the bottom of one joint. The wall nearest to the Duwamish waterway and parallel to it has settled 2 inches [5.1 cm] below adjacent walls and is out of plumb. Lateral and vertical movement of ground is evident.	U.S. Army Corps of Engineers (1949)
4	Pier 30	At the front of the building which faces on Alaskan Way severe cracking of brickwork appears. The cracks are arranged in such a way as to indicate a downward movement of the building relative to the front wall or the corner pier * * *. In this instance the bearing and nonbearing piers moved differentially.	U.S. Army Corps of Engineers (1949)
5	Harbor Island (Fisher Flouring Mills)	Inside the old office, the floor is badly out of level, in one place bulged up 7 inches [18 cm] above the adjacent floor of the new building.	U.S. Army Corps of Engineers (1949)
6	W. Spokane St.	The Northern Pacific Railway Company's bridge (single bascule) crossing Duwamish Waterway near West Spokane Street was reported to have permanently shifted from 4 to 7 inches [10-18 cm]. The bridge was open at the time of the earthquake. Repairs consisted of realigning the tracks * * *. The city double bascule bridges Nos. 1 and 2, crossing Duwamish Waterway at Spokane Street, were closed at the time of the quake. The main piers shifted, reducing the horizontal distance between the piers several inches and jamming the forward edges * * *.	U.S. Army Corps of Engineers (1949)
7	Spokane St.	The Bethlehem Steel Seattle Mill Depot No. 92, Spokane Street, Seattle, was observed by a competent engineer during the quake. The building is a single story concrete [frame] with brick curtain walls on a concrete footing with a concrete floor resting on filled ground. The rear of the building is 9 feet [2.7 m] from the top of a 1 on 2 slope 7½ feet [2.3 m] above adjacent tide land. After the perceptible ground motion of the quake had ceased, cracks started developing in the	U.S. Army Corps of Engineers (1949)

Table 34. Descriptions of liquefaction occurrences in Seattle during the 1949 and 1965 Puget Sound earthquakes—Continued.

Location no.	Location	Quotation and (or) comment	Source
8	177 S. W. Massachusetts (Albers Brothers Elevators)	20-foot [6 m] bay next to the tide land. A triangular section of the brick wall moved vertically downward from 1-13/16 inches to 1-3/4 inches [3.0-4.4 cm]. The ground at the rear of the building pulled away from the wall 2½ inches [6.4 cm] and settled vertically 3 inches and 4 inches [7.6 cm and 10.2 cm] below the marks left on the wall by the soil surface * * *. This failure resulted from downward and lateral movement of the ground after perceptible motion of the ground had ceased, indicating an incipient slide * * *. Later examination all around the building revealed that the end bay on the opposite side had moved in a similar manner but a large door in the bay eliminated the possibility of as clear evidence developing. Several other bays in the building developed cracks which indicate less severe settlement elsewhere. The end wall, facing the tide land reveals cracks in the concrete footings under each bay.	U.S. Army Corps of Engineers (1949)
9	2600 26th Ave. S. W. (KJR radio tower)	Examination of ground around the Albers Brothers Elevators shows no evidence of settlement except that a number of sand boils developed from 5 feet to 15 feet [1.5-4.6 m] away from the elevators on the northeast side. The ground around a large fuel tank has settled differentially from zero to ½ inch [1.3 cm] as evidenced by the soil contact mark.	U.S. Army Corps of Engineers (1949)
10	2613 Marine Ave. S. W.	The KJR Radio transmitting tower suffered structural damage but the ground apparently settled around the footing 1 inch [2.5 cm] below the former level. Cracking of the soil and sand boils occur in the area between the tower and the Duwamish waterway.	U.S. Army Corps of Engineers (1949)
11	1st Ave. S. and S. Lander (Sears)	After the 1949 earthquake the professor and some students came out and took samples of the water in our basement. It was fresh water and we are only about 100 feet from the bay (Mrs. J. Woodhouse, written commun., 1965).	University of Washington, unpub data (1965)
12	Green Lake (NE¼ sec. 7, T. 25 N., R. 4 E., Seattle North 7½ quadrangle)	* * * geysers of water and mud which spurted from the ground reportedly as high as 3 feet [0.9 m], which flowed continuously for as long as 24 hours, and which filled basements * * * Earthquake of April 29, 1965 Green Lake, sloshing back and forth like soup in a shallow bowl, buckled blacktop paving around Aqua Theater and opened large fissures in the ground on the south and west banks of the lake. Water pressure from the lake put two waves in West Green Lake Way between Aqua Theater and Lower Woodland Park. The concrete-block junior crew house at the Aqua Theater was damaged, possibly beyond repair. A wall buckled. [Probable slight slumping along shoreline of Green Lake at or near the same location at the southwest end of the lake as that reported for the April 13, 1949, earthquake. These, like the 1949 ground failures, were probably liquefaction induced].	Seattle Times (1965)

- 13 University of Washington
 A fissure opened in the practice field at the University. Underground pressure from the shock sent sand spurting in a 100-foot-long [30 m] zigzag stretch on the lower football field. Behind the men's pool, areas of the ground dropped as much as a foot [30 cm]. Dirt floor sections in the Hec Edmondson Pavilion also sank slightly.
 North of Union Bay, broad fill over alluvial and lacustrine sediments subsided and exhibited scattered ground cracks and sand mounds.
 The building has suffered damage throughout the years from settlement and cracking due to this settlement of the building. Presently the building appears to have suffered additional damage due to the earthquake in that the front wall has bulged at the northwest corner.
 Hake and Cloud (1967)
 Mullineux and others (1967)
- 14 2422 Elliott Ave.
 (Perfection Smokery)
 Break in underground supply mains.
 MacPherson (1965)
- 15 Pier 66
 Fire protection * * * supply main break.
 MacPherson (1965)
- 16 Piers 64 and 65
 Whole front footing for the building has settled from a few inches to eight to ten inches [20-25 cm]. The beams are generally cracked and sheared. Apparently this has been going on for many years.
 MacPherson (1965)
- 17 410 5th Ave. S. (Apex
 Automotive and Garage)
 The building is supported on posts and blocks. The building has settled very badly throughout the years. This settling has been exaggerated during the present earthquake. The floor has been shored up in different places during the recent years, but all the old existing posts are leaning in various directions and the floor is very badly out of level. The area underneath the building is very damp. During the winter months, the area has standing water * * *. During the present earthquake a considerable amount of water came in. This apparently was from consolidation of the earth below which forced the water in this lower stratum up into the area. This water was also followed by the characteristic fine sifting which now covers most of the underneath area * * *. In past years, the lower floor * * * was used as part of the shop * * *. At the present time it is not possible to walk upright.
 MacPherson (1965)
- 18 1730 1st Ave. S. (Queen
 City Sheet Metal Shop)
 The building is located on dredged fill material and is believed to be on piling * * *. Approximately 8" [20 cm] maximum downward settlement of footings * * *. An upward movement of some footings of 1" to 2" [2.5-5 cm] * * *. The basement floor slabs on grade were severely cracked and displaced due to the action of footing settlement combined with upward pressure of ground water against the bottom of the slabs.
 MacPherson (1965)
- 19 2228 1st Ave. S.
 (Millwork Supply)
 Port of Seattle Engineering Department letter describes ground settlement along bulkhead lines at Piers 5, 20, 28, and 42; broken underground fire mains at Piers 20, 42, and 66.
 MacPherson (1965)
- 20 Various piers
 Breaks in underground piping.
 MacPherson (1965)
- 21 Pier 25
 The low-lying filled areas along the Duwamish River and its mouth settled and were the locations of considerable building damage * * *. A number of bridges were closed temporarily due to slight damage * * *. The 14th Avenue South drawbridge across the Duwamish River had some pier damage * * *. Both of the Southwest Spokane Street bridges were jammed shut when the shock threw them out of line * * *. East-bound lanes of a drawbridge across the Duwamish Waterway were closed * * * because of a drop in the road level * * *. Pier 20 at the East Waterway Terminal settled.
 Hake and Cloud (1967)
- 22 Duwamish River

Table 34. Descriptions of liquefaction occurrences in Seattle during the 1949 and 1965 Puget Sound earthquakes—Continued.

Location no.	Location	Quotation and (or) comment	Source
23	Harbor Island Piers 15 and 16 and Fisher Flouring Mills	<p>"The Seattle Water Department had one break in a 12-inch [30 cm] main in the Harbor Island area * * *. Harbor Island, at the mouth of the Duwamish River, was a special high-damage location * * *. The Fisher Flouring Mills had extensive damage * * *. Underground piping around the plant * * * broke * * *. Piers 15 and 16 on Harbor Island shifted toward the water by about 1 foot [30 cm] due to the soil losing much or all of its strength, or partially liquefying and pushing the dock toward the water. An exception was the northern extension of the pier which was under construction and did not yet have its soil backfill.</p>	Algermissen and others (1965)
24	Pier 5	<p>Pier 5, where construction projects were underway, was hardest hit. The bulkhead and the fill behind it settled, the fill dropping 6 inches to 2 feet [15-61 cm] for a width of 25 to 40 feet [7-12 m]. The bulkhead was reported to be 6 to 8 inches [15-20 cm] out of line. Several Port piers suffered similar damage.</p> <p>Subsidence of the material along the west side of the pier. The area at the north end wall is exposed; the sheet pile wall has displaced downward from the reinforced concrete dock a distance of 8 inches [20 cm]. The soil in this area for a 20-foot [6 m] width has subsided. The ground has displaced to approximately 1½ feet [0.5 m] below the level of existing dock. This subsidence decreases to approximately 8 inches [20 cm] at the southerly end of pier.</p>	Hake and Cloud (1967)
25	Pier 6	<p>Similar problems to * * * Pier 5. There is subsidence of 6" to 12" [15-30 cm] at the land face of the pier.</p>	MacPherson (1965)
26	2613 Marine Ave. S.W.	<p>Damage to "basement floor and fresh water came in." (Mrs. J. Woodhouse, written commun., 1965)</p>	MacPherson (1965) University of Washington, unpub. data (1965)

major west-northwest-trending structural zone that cuts through the Cascade Range and Columbia Plateau to the east. South of this lineament, bedrock is exposed in scattered outcrops in southeast Seattle, at Alki Point, and in a series of prominent ridges to the east, collectively called the Newcastle Hills, that trend eastward to the Cascade Range.

LOCAL GEOLOGY

Seattle consists of several north-south-trending elongated ridges and drift uplands. The hills and uplands are separated by large Pleistocene glacial troughs and outwash channels that are now occupied by tidal waters and large lakes or alluviated by streams that occupied the troughs about 13,500 years ago following the retreat of the last glaciation. Major troughs lie beneath the main body of Puget Sound, the Duwamish River valley, and Lake Washington. The symbols used to identify the geologic units in the following discussion correspond to those used on plate 9 and those used by Waldron and others (1962), who originally mapped the surficial geology of the area.

BEDROCK

A broad band of Tertiary sedimentary and volcanic-intrusive rocks forms the Newcastle Hills promontory between Renton and Issaquah, east of Seattle. This west-northwest-plunging promontory crosses the southern part of the city, with bedrock exposed or subcropping in the southern part of Beacon Hill, locally in the southern Duwamish River valley, and at Alki Point in West Seattle. These rocks are folded into northwest to west-northwest-trending anticlines and synclines that are broken by northeast-trending left-lateral faults (Weaver, 1937; Mullineaux, 1970). The faults were most recently active in early to middle Tertiary time (Gower and others, 1985).

Two Tertiary bedrock units lie beneath and sporadically crop out within the study area. Waldron and others (1962) identified the oldest unit as middle Eocene sedimentary conglomerate, sandstone, siltstone, and shale chiefly with volcanic clasts (Td). This older unit crops out along the southern edge of the study area, along the sides and below the Duwamish River valley. The younger bedrock unit (Tb) contains Oligocene marine to estuarine sandstone and shale with subordinate amounts of conglomerate, mostly of volcanic origin. The unit is part of the Blakely Formation (Weaver, 1937; Waldron, 1962; and Livingston, 1971). This unit crops out across the southern part of Seattle and at Alki Point.

Immediately north of this band of Tertiary rocks, the bedrock surface drops abruptly to depths of more than 1,100 m below sea level in a horizontal distance of less than 1.6 km, as shown in figure 217. Present data suggest that the

bedrock surface rises from its lowest point in downtown Seattle gradually to the northeast to about 400 m below sea level at the north end of Lake Washington (Hall and Othberg, 1974; Yount and others, 1985).

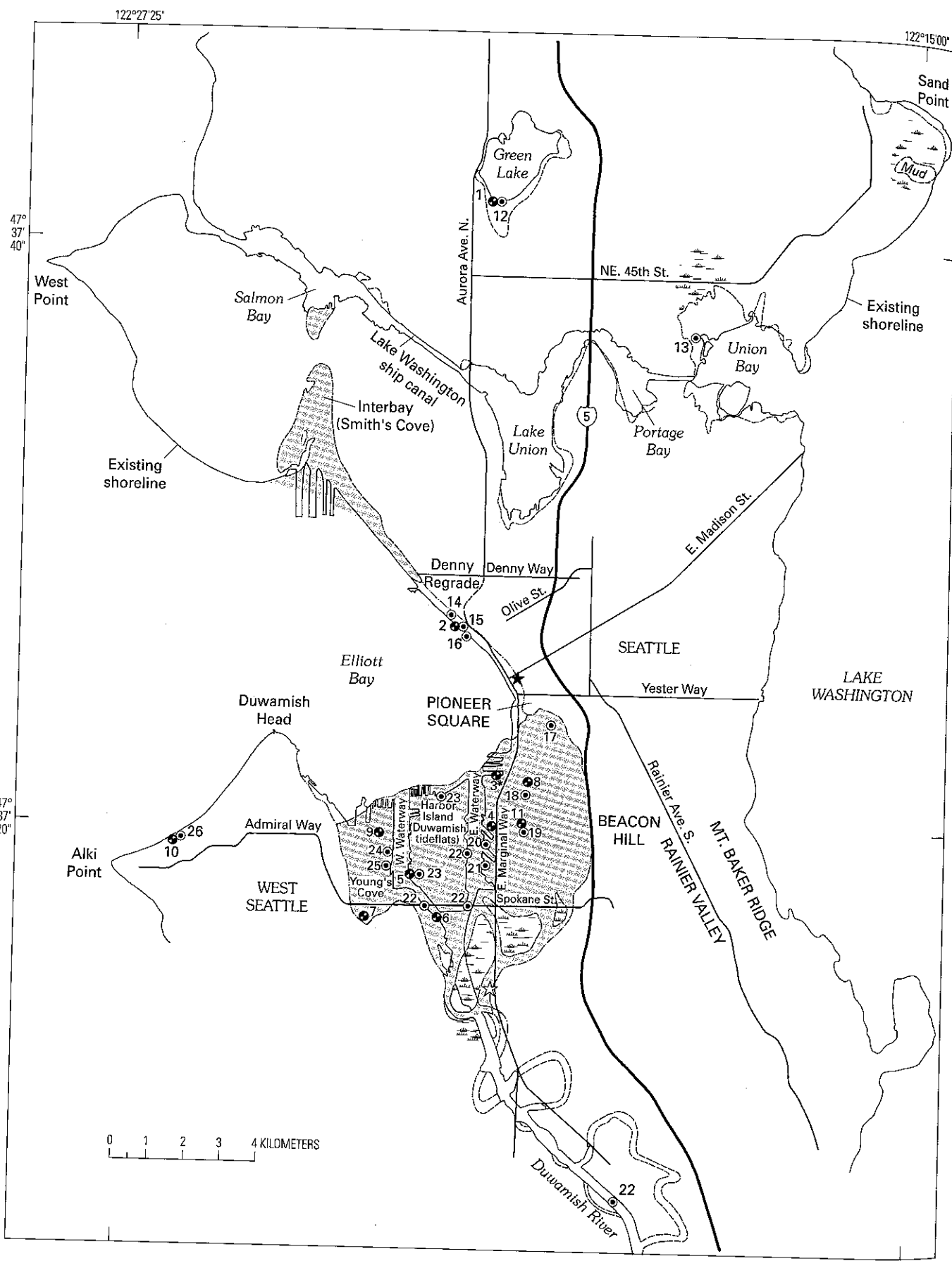
PLEISTOCENE DEPOSITS

Nonlithified, glacially overridden sedimentary deposits generally lie unconformably above the Tertiary bedrock (pl. 9). This sedimentary material is both glacial and nonglacial in origin. The youngest of these sedimentary deposits, the lower parts of the Vashon Drift, were deposited as the Vashon ice lobe advanced southward during the Vashon stade of the Frasier glaciation, about 15,000 yr B.P. (Mullineaux and others, 1965). At its greatest extent, the lobe advanced to a position about 80 km south of Seattle (Booth, 1987). Covering Seattle with an estimated 900-m-thick layer of ice, the weight of the glacier greatly overconsolidated the underlying sediment to various degrees, including the lower parts of the Vashon Drift. In the Seattle area, these very dense sedimentary materials underlie most upland areas, including major hills and ridges.

As the Vashon ice lobe retreated northward, it left behind recessional outwash deposits (Qys, Qyg) largely consisting of mixtures of gravel and sand. The outwash was generally confined to the major glacial troughs but was also irregularly distributed on the drift uplands. Recessional deposits also include coarse-grained outwash deltas, kame terraces and other ice-contact deposits along certain ridge flanks, local fine sand and silt deposited in ephemeral ice-marginal lakes, and sands and gravels in local outwash channels. Locally, recessional deposits attain thicknesses of 30 m or more in major outwash deltas. The younger sands (Qys) are fine to medium grained and generally less than 3 m thick in the upland channels. The younger gravels (Qyg) are composed of sand and pebble-size gravel and are also about 30 m thick.

HOLOCENE DEPOSITS

Holocene deposits in the Seattle area include alluvium (Qa) in the Duwamish River, Rainier, and Interbay valleys; beach and adjacent marine deposits (Qb) along shorelines; colluvial and landslide deposits (Ql); and peat (Qp) and lacustrine deposits (Qsc) in upland depressions and along low-lying lakes. Alluvial deposits (Qa) consist of fine sand, silty fine sand, fine sandy silt, and nonplastic silt, with local pockets or stringers of organic materials. Owing to shifting of depositional channels, individual beds of uniform grain size are rarely laterally continuous over large areas, and interfingering of different soil units is common. Typically, the Holocene deposits consist of very loose to loose granular soils within about 10 m of the ground surface. Subsurface



EXPLANATION

ACCELEROGRAPH STATIONS AND LOCATIONS

- ★ USGS accelerometer station 2170 (discontinued) that recorded the April 13, 1949 earthquake; 4735 E. Marginal Way (≈ 47.55 N.; 122.34 W.)
- ☆ USGS accelerometer station 2102 (existing) that recorded the April 29, 1965 earthquake; 909 1st Ave. (≈ 47.604 N.; 122.335 W.)

SITES OF HISTORICAL LIQUEFACTION

- 1949 earthquake
- ⊙ 1965 earthquake
- * Approximate location of liquefaction site

HISTORICAL SHORELINES

- U.S. Coast Survey (1879)
- U.S. Geological Survey (1897)



Tidal flat

Marsh or swamp

Figure 216 (facing page and above). Sites of historical liquefaction in Seattle. The numbers correspond to the location numbers used in table 34 for descriptions of liquefaction. Base map from Waldron and others (1962).

explorations for the West Seattle Bridge indicated that the alluvium near the mouth of the Duwamish River generally extends to a depth of about 55 m and locally extends to depths of 75 m.

Lacustrine deposits (Qsc) and very soft to soft peat (Qp) also occur in numerous closed depressions on the surface of the drift uplands and along the shorelines of lakes. Lacustrine sedimentary deposits are composed of silt, clay, and fine sand and are usually less than 3 m thick. The peat ranges from fibrous to peaty silt (muck). Both the upland and lowland peats contain a pervasive 2.5- to 5.0-cm-thick layer of ash related to the eruption of Mount Mazama (Crater Lake, Oreg.) 6,800 years ago (Wilcox and Power, 1964; Curran, 1965).

Colluvium is the veneer of loose to medium-dense soil that drapes the sides and toes of slopes throughout the city and its environs. The deposits consist of mixtures of the materials composing the slopes and, hence, the grain size of these colluvial deposits can range from fine-grained clay and silt to boulder-size clasts. Processes forming colluvium range from very slow creep (the imperceptible movement of only fractions of a centimeter per year) to catastrophic landslides. The areal extent of landslide deposits (Ql) is relatively small; these deposits lie near the base of steep hills, ridges, and uplands. Slide material is especially common at or below the contact of the Esperance Sand and Lawton Clay Members of the Vashon Drift (Tubbs, 1974). On steep slopes (greater than 40°), the colluvial veneer is generally very thin (1 m or less), whereas near the toe of the hillside, where slope angles are 10°-20°, thicknesses of colluvium generally range from 5 to 10 m.

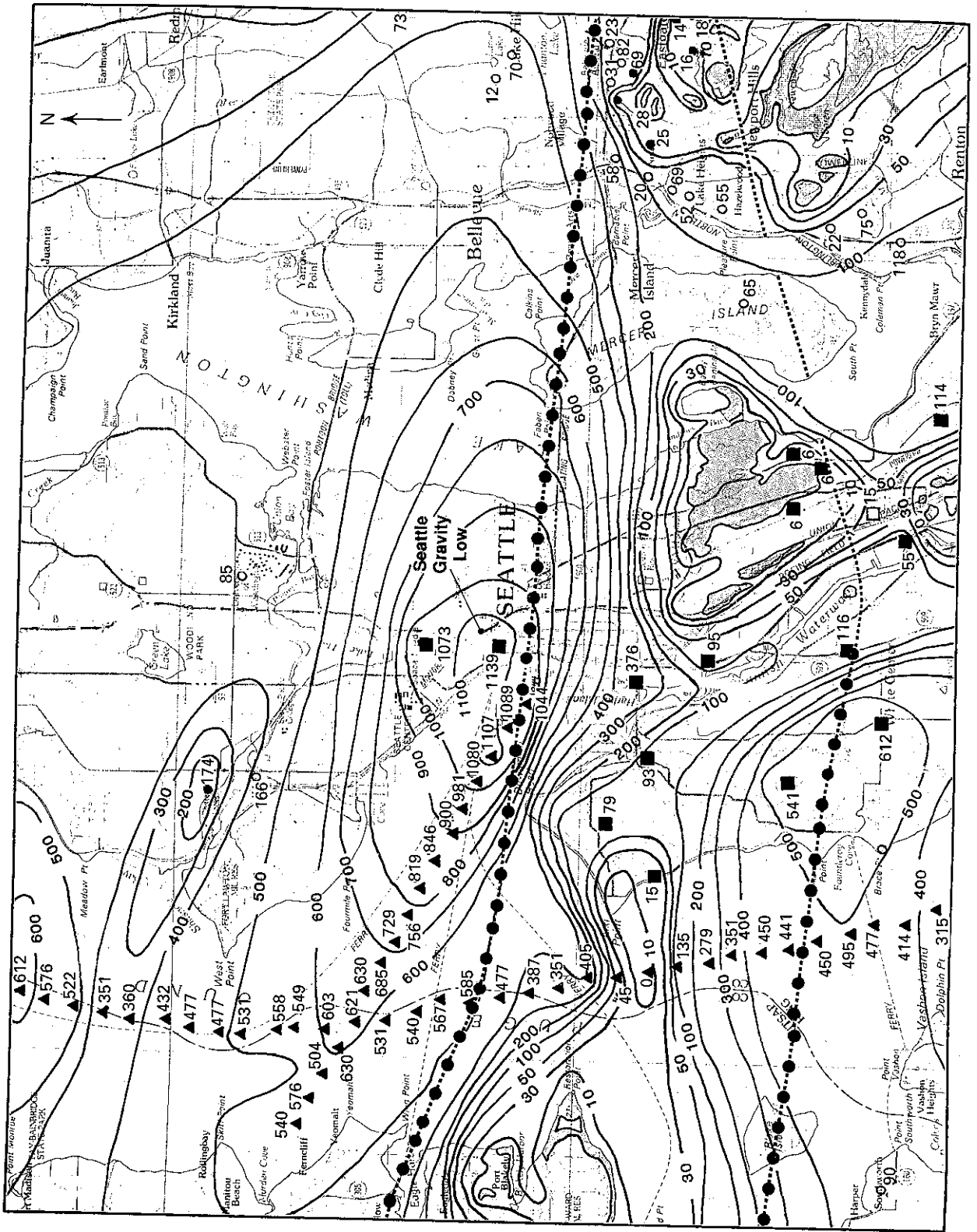
FILLS AND LAND MODIFICATIONS

Major fills (f) and drainage modifications (m) in the city have resulted from engineering projects during the first two decades of the 20th century (Phelps, 1978). Areas of major tideland reclamation are shown in figure 216. In the early 1900's, shallow tidal areas of the Duwamish River delta were filled with material that was largely sluiced from adjacent drift uplands to improve the usability of the seaport and obtain an area for industrial development. As a result of these operations, the mouth of the Duwamish River was extended about 0.80 km northwest to its present location. Also during this period, the sinuous, meandering course of the lower Duwamish River was straightened and deepened to what is now the Duwamish Waterway. Additionally, Harbor Island was built of hydraulic fill placed on tidelands at the river mouth when the East and West Waterways were dredged. A tidal marsh in the Pioneer Square area was filled with soil and with organic debris from nearby lumber mills. Glacial soils from the Jackson and Dearborn Regrades were sluiced via flumes and pipes to the Duwamish tidal flats south of the Pioneer Square area, where it accumulated to depths up to 12 m in the period between 1909 and 1910. The water-laden soil was washed into a series of diked ponds so that the fine particles could settle out of the slurry (Phelps, 1978). The tidal marsh at Smith's Cove (Interbay) was filled, as was the delta of Longfellow Creek (Young's Cove) in West Seattle. These projects provided extensive areas for seaport facilities and industrial expansion.

Logs of geotechnical borings show that these fills are highly variable in composition, ranging from sand to silt to clay and often containing sawdust, bricks, logs, wood fragments, cinders, and other debris. Yount (1983) reported that later fills are generally of better quality (more compact material consisting of medium to coarse sand) than the older fills. The fills are typically 3-5 m thick but can be as much as 10 m thick.

Regrading of the downtown Seattle hillsides was accomplished in two major phases between 1903 and 1928 to facilitate expansion and ease access within the central business district. This included removing Denny Hill entirely, which resulted in an excavation that was locally in excess of 30 m and covered a 62-city-block area (Sale, 1976; Morse, 1989). Glacial soils removed from Denny Hill were either sluiced or dumped by barge into shallow areas of Elliott Bay.

Between 1911 and 1916, the Lake Washington Ship Canal was constructed, linking Lake Washington to Puget Sound. This construction resulted in lowering of the surface of Lake Washington a nominal 3 m to the level of Lake Union. Additionally, the canal construction, which also includes a set of locks, resulted in raising the water surface in Salmon Bay to the level of Lake Union. The lowering of the water surface in Lake Washington eliminated the Black River, which drained from the south end of Lake




- EXPLANATION
- 100— Contours showing depth to bedrock; in meters
 -  Area of bedrock outcrop
 - Major structural or geophysical lineament
 - ▲432 Marine seismic reflection profile
 - 174 Water well
 - 95 Geotechnical boring; depth to rock
 - 15 Exploration not reaching rock

Figure 217 (facing page and above). Map showing depth to bedrock in the Seattle area. From Yount and others (1985).

Washington to the Duwamish River at Tukwila. Also as part of the canal construction, the Cedar River was diverted into Lake Washington (Chrastowski, 1983).

The lowering of Lake Washington left a gently sloping terrace underlain by loose sedimentary materials around the lake's periphery, part of which has been retained as park land and part of which has been privately developed (Galster, 1989). The northern portion of Union Bay was filled subsequent to the lowering of Lake Washington. Originally, the site was a landfill that was later capped with fill material. A peat bog to the north of N.E. 45th Street was partially removed for peat and then filled with granular materials.

During construction of the Sand Point Naval Air Station, the site was extensively graded. Glacial soils from the central portion of the site were excavated and used to fill a small embayment on the north side and at Mud Lake.

Smaller areas of fill, as shown in figure 216, are located at the south end and northwest corner of Green Lake, the south and west sides of Lake Union, and the southern portion of Salmon Bay. Smaller fills were also placed in oxbow features along the Duwamish and Green Rivers.

GROUND WATER

Most of the normally consolidated soil units in the Seattle area lie in alluvial valleys or along lakes and bays where ground-water levels are relatively high. Static water levels recorded or estimated from borings in various areas of the city are summarized in table 35. As shown in the table, the average depth to ground water generally ranges from 0.6 to 3 m except in the upland outwash gravels. Although available boring data for the upland gravels suggest the absence of near-surface ground water, the near-surface presence of water cannot be precluded because of the paucity of data. High ground-water levels are likely where this gravel lies adjacent to lakes or ponds in the upland areas. Additionally, perched ground-water conditions may locally occur.

REGIONAL SEISMICITY

Examining the historical record concerning seismicity, earthquake source mechanisms, and postulated levels of peak ground motion for the Puget Sound region is essential to defining the liquefaction hazard of the Seattle area because the strength of the earthquake and the duration of the ground shaking directly affect the development of liquefaction.

HISTORICAL SEISMICITY

Seattle is located in a moderately active tectonic province that has been subjected to earthquakes of low to moderate strength and occasionally to strong shocks during the 160-year historical record of the Pacific Northwest. The largest historical earthquakes in the region are believed to be associated with deep-seated plate-tectonic activity (U.S. Geological Survey, 1975). Major mapped faults in the region (within 88 km of Seattle) have not been active in the Holocene and, consequently, none are known to be associated with historical seismicity. The nearest faults known to be active are small faults on the Olympic Peninsula, about 65 km west of Seattle.

The more significant historical earthquakes (those of Modified Mercalli intensity VI or greater) that have occurred in the Seattle area are listed in table 36. Of the 18 events listed, 5 had intensities of VII or greater. The largest of these were the April 13, 1949, M_s 7.1 intensity VIII shock and the April 29, 1965, m_b 6.5 intensity VII–VIII event. These earthquakes, which were respectively centered 63 km and 23 km from Seattle, caused considerable property damage in the city.

Other large historical earthquakes that have affected Seattle include one in the North Cascades of Washington and two in western British Columbia. The North Cascades earthquake of December 15, 1872, appears to have been one of the largest in the Pacific Northwest, as it was felt over an area of about 1,295,000 km². It has been estimated that this major shock had a magnitude of about 7 and an MMI of VIII. Although the epicenter of this event is uncertain owing to the sparse population of the area at that time, it apparently occurred somewhere in the northern Cascade Range.

In Canada, major earthquakes occurred on Vancouver Island on June 23, 1946, and in the Queen Charlotte Islands on August 21, 1949 (Coffman and Hake, 1973). The Vancouver Island event had a magnitude of 7.3 and a maximum intensity of VIII. Although the magnitude 8.1 Queen Charlotte Islands earthquake was felt over an area of more than 5,180,000 km², damage was minor because of the sparse population in the epicentral area.

Table 35. Ground-water depths in various parts of the Seattle metropolitan region.

[All values in feet, as originally measured or estimated from borings in study database. Values in parentheses are metric equivalents]

Area	Average depth	Maximum depth	Minimum depth	Standard deviation
Duwamish River valley (includes filled tideflats).	6.5 (2.0)	21.5 (6.6)	0.0 (0.0)	4.4 (1.3)
Alki Beach	10.5 (3.2)	25.0 (7.6)	6.0 (1.8)	3.7 (1.1)
Rainier Valley	8.7 (2.7)	26.0 (7.9)	0.0 (0.0)	8.3 (2.5)
Interbay (includes Lake Washington Ship Canal, unit Qys ¹).	6.4 (2.0)	13.0 (4.0)	0.0 (0.0)	4.3 (1.3)
West Point	15.0 (4.6)	30.0 (9.1)	9.0 (2.7)	6.9 (2.1)
Union Bay	4.0 (1.2)	6.5 (2.0)	2.0 (0.6)	1.3 (0.4)
Shilshole Bay	1.7 (0.5)	4.0 (1.2)	0.0 (0.0)	1.7 (0.5)

¹Glacial recessional outwash deposit; upper part of the Vashon Drift. See plate 9 for description.**Table 36.** Historical earthquakes in the Puget Sound region that exhibited a Modified Mercalli intensity (MMI) of VI or greater and occurred within 60 km of Seattle.[Leaders (--), not measured. PST, Pacific Standard Time; M_s , surface-wave magnitude; m_b , body-wave magnitude]

Source ¹	Year	Date (mo/day)	Time, PST (hr:min)	Lat ² N. (°)	Long ² W. (°)	Magnitude	Maximum MMI	Depth (km)	Epicentral distance (km) and direction from Seattle
A,C	1880	08/22	13:25	48	122	--	VI	--	51 NNE.
A,C	1880	12/12	20:40	47.5	122.5	--	VI	--	16 SW.
A	1928	02/02	04:52	47.8	121.7	--	VI	--	53 NE.
A	1931	12/31	07:25	47.5	123.0	--	VI	--	51 WSW.
A	1932	08/06	14:16	47.7	122.3	--	VI	--	11 N.
A	1939	11/12	23:46	47.4	122.6	³ 5.75	VII	--	31 SW.
B	1945	04/29	12:16	47.4	121.7	--	VII	--	53 ESE.
A	1946	02/14	19:18	47.3	122.9	³ 5.75	VII	--	55 SW.
B,C,E	1949	04/13	11:56	47.1	122.7	7.1 (M_s)	VIII	71	63 SSW.
A	1950	04/14	03:04	48.0	122.5	--	VI	--	47 NNW.
A	1954	05/15	05:02	47.4	122.3	--	VI	--	23 S.
B	1955	03/25	22:56	48.05	122.03	--	VI	--	55 NNE.
B	1960	04/10	22:48	47.57	122.25	--	VI	--	8 SE.
A	1963	01/24	13:43	47.4	122.1	--	VI	--	27 SE.
B,E	1965	04/29	07:29	47.4	122.3	6.5 (m_b)	VII-VIII	60	23 S.
B,C	1965	10/23	08:28	47.5	122.4	³ 4.8	VI	--	13 SSW.
B	1975	04/22	15:04	47.08	122.65	4.0 (m_b)	VI	47	64 SSW.
B	1976	09/08	00:21	47.38	123.08	4.6 (m_b)	VI	48	61 WSW.

¹The following sources were used in compiling the earthquake data:

A. Coffman and von Hake (1973).

B. U.S. Coast and Geodetic Survey (1928-1968).

C. U.S. Geological Survey (1975).

D. Stover and others (1978).

E. Weaver and Baker (1988).

²The range of uncertainty for epicentral locations may be taken as $\pm 0.5^\circ$ for earthquakes prior to 1960 and $\pm 0.2^\circ$ for those after 1960.³Type of magnitude was not provided in the references prior to 1975.

EARTHQUAKE SOURCE MECHANISMS

Earthquake source mechanisms, which have been correlated with the observed historical seismicity, include shallow crustal events and deep subcrustal events. Maximum magnitudes of about 6.0 and 7.5 have been postulated

for these two source zones, respectively (Rasmussen and others, 1974; U.S. Geological Survey, 1975). The deeper events are believed to be associated with faulting or release of extensional stresses in the subducted slab of the Pacific plate beneath the Puget lowland area (Taber and Smith, 1985; Weaver and Baker, 1988). The two major earthquakes in the

region, the 1949 and 1965 events, both had focal depths in excess of 40 km, which is consistent with the deep-source-mechanism hypothesis. The majority of historical events, however, occurred at relatively shallow depths of about 24 km or less, which is consistent with the shallow-earthquake-mechanism hypothesis.

A third source mechanism, which is currently being debated within the scientific community, is the possible occurrence of a major earthquake on the Cascadia subduction zone off the coast of the Pacific Northwest (Heaton and Kanamori, 1984). Presently, the Cascadia subduction zone is quiet, with only scattered and diffuse seismicity, and no large subduction-zone earthquakes have occurred during historical times. However, Atwater (1987) has introduced geologic information that would suggest the possible occurrence of several subduction-zone events during the past 2,000 years.

POSTULATED GROUND MOTIONS

Estimates of seismic peak ground acceleration for the Seattle area have been postulated from regional studies conducted by the USGS and from local microzonational studies conducted by other researchers. Information on the ground acceleration of the area is an essential parameter in conducting a liquefaction-hazard evaluation.

The USGS has performed several regional studies on seismic hazards in the Pacific Northwest (Algermissen and others, 1982; Algermissen, 1988a, b). Figure 218 presents a regional, probabilistic evaluation of peak ground accelerations that could occur on rock within the Pacific Northwest. The accelerations shown in this figure have a 10 percent probability of being exceeded in a 50-year period, which corresponds to a 475-year seismic return period. Figure 219 compares the seismic exposure of Seattle to other areas of the United States. This figure presents ground accelerations on rock that have a 10 percent chance of being exceeded during the indicated time intervals. Both figures indicate that Seattle may be subjected to a ground acceleration of 0.30g an average of every 475 years.

Whereas the ground-motion estimates presented in figures 218 and 219 are based upon conventional earthquake source mechanisms (shallow and deep), recent work by Algermissen (1988a) suggests that ground accelerations in Seattle from a large subduction-zone earthquake occurring off the coast of Washington would not vary appreciably from the 475-year accelerations estimated from the conventional earthquake sources. However, the duration of ground shaking for a subduction-zone earthquake may be several times greater than that associated with more conventional earthquake source mechanisms. Increased duration of ground shaking would tend to increase the areal extent of liquefaction.

On a site-specific basis, Langston and Lee (1983) and Ihnen and Hadley (1987) performed ray-tracing studies to

investigate the local variations in ground response in the Puget Sound region. Whereas Langston and Lee (1983) specifically evaluated amplification of ground motion in the Duwamish River valley, Ihnen and Hadley (1987) developed a seismic hazard map for the entire Puget Sound region that included considerations for ground-motion amplification due to soil type and wave-focusing effects. Results from both of these studies indicated that ground motions along the Duwamish River could be 50–100 percent greater than adjacent elevated areas. Both studies, however, indicated that the computed values of ground motion were highly dependent upon the focal mechanism and location of the generating earthquake. Because of the speculative nature and high degree of sensitivity associated with the results of the local microzonational studies, the results from these local studies have not been widely accepted or used for seismic design within the local engineering community.

STUDY METHODOLOGY

TECHNIQUE

Methods for evaluating liquefaction potential on a regional basis range from empirical techniques relating general liquefaction susceptibility to underlying geologic

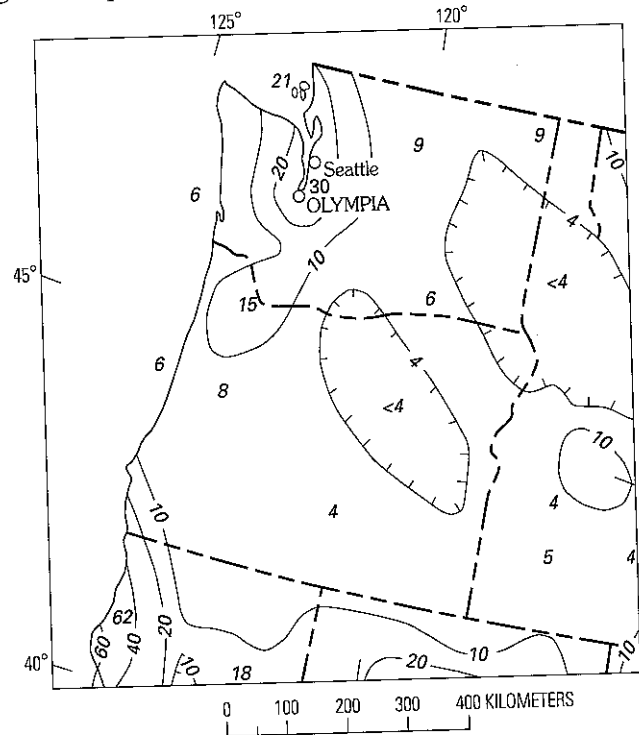


Figure 218. Earthquake peak ground accelerations for the Pacific Northwest. Numbers are peak accelerations in percent of g on rock with a 10 percent chance of exceedance within 50 years (475-year return interval). Contours hachured to indicate closed low. From Algermissen (1988a).

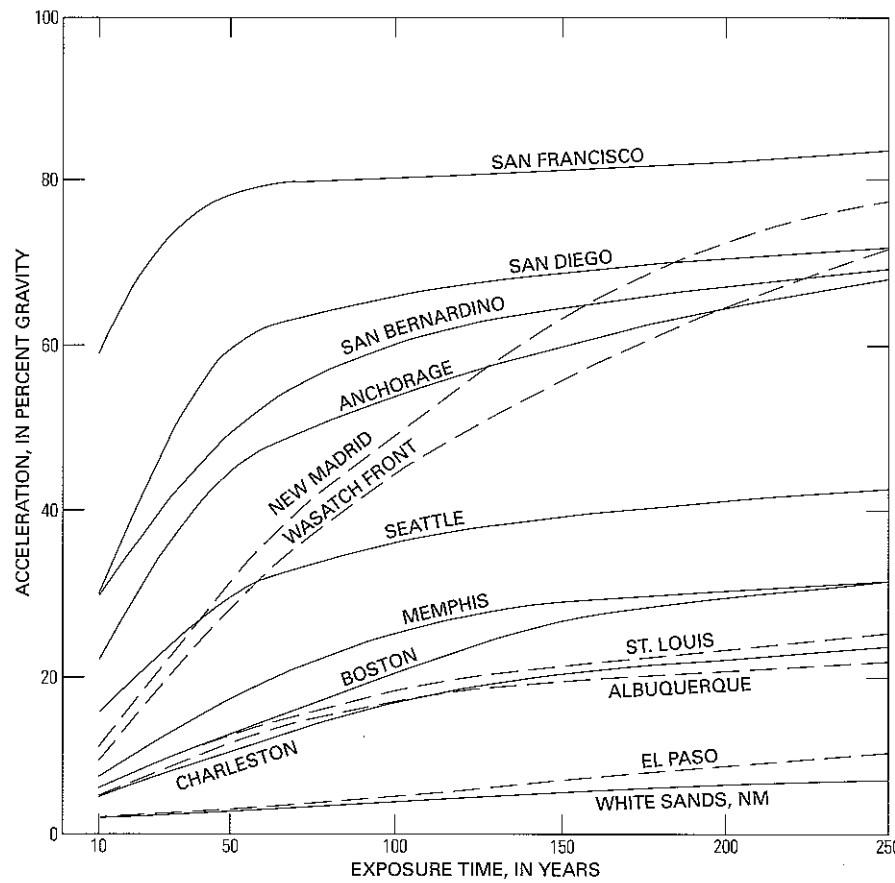


Figure 219. Seismic exposure map indicating peak accelerations on rock in various locations in the United States and corresponding to a 10 percent probability of occurrence for the indicated exposure times. From Algermissen (1988b).

conditions (Youd and Perkins, 1978), to more elaborate probabilistic, analytical evaluations (Power and others, 1986). These techniques have been applied to sites in southern California (Lee, 1977; Youd and others, 1979; Power and others, 1982; Tinsley and others, 1985; and Power and others, 1986), northern California (Youd and others, 1975; Blair and Spangle, 1979; Davis and others, 1982; Youd, 1982; Kavazanjian and others, 1985; Youd and Perkins, 1987; and Power and others, 1988), and other locations in the Western United States (Anderson and Keaton, 1982, 1986; Moriwaki and Idriss, 1987) and Eastern United States (Budhu and others, 1987; Hadj-hamou and Elton, 1989; Elton and Hadj-hamou, 1990).

Our liquefaction study of Seattle used empirical relations developed by Seed and Idriss (1971), Seed and others (1983), and Seed and others (1984) to establish the liquefaction potential of the various geologic units in the area. The procedures were used because of their acceptance and wide use in engineering practice. Furthermore, the use of these procedures permits a better conceptual understanding of the liquefaction phenomenon and the interrelation of the various parameters, such as subsurface geology and SPT N-values, that affect the occurrence of liquefaction.

The first step required the development of a database containing ground-water levels and SPT N-values for the various geologic units within the study area. This

information was obtained from more than 350 borings in Seattle. The SPT N-values of the soils within the various geologic units were then compared with the threshold SPT N-values needed to resist liquefaction. The relative liquefaction hazard of the particular geologic unit was then assessed on the basis of the percentage of SPT values falling below the threshold SPT N-values. Additionally, the liquefaction hazard was assessed on the basis of the computed cumulative thickness of potentially liquefiable soil within the borings.

DATABASE

Because liquefaction susceptibility is affected by the geologic origin, depth, relative density, and gradation of the soil and the depth of the water table, a computerized database was developed to facilitate the storage and retrieval of subsurface data for subsequent use in the liquefaction evaluation (Grant, 1990). The database, which includes the logs of more than 350 borings, allows sorting of the data corresponding to various parameters including geographic location, drilling method, geology, and individual SPT N-value. Data recorded for each boring include UTM (Universal Transverse Mercator) coordinates, location description, date drilled, drilling method, surface elevation of the boring, static ground-water depth, and SPT N-value as a function of

depth. Each SPT sample in a boring was assigned a code corresponding to the geologic unit of the material as well as a separate code describing the composition of the materials within the sample. The data corresponding to a particular geologic unit or material were subsequently retrieved to determine liquefaction susceptibility. By including individual SPT data from each boring, we were able to statistically account for variability of the SPT values within individual borings or within an entire geologic unit. This assessment is discussed subsequently in the evaluation-criteria section.

PEAK GROUND ACCELERATION

A key parameter in the liquefaction evaluation is the selection of a peak ground acceleration value for use in the numerical computations of the liquefaction potential. The following factors were considered in selecting the peak ground acceleration for the liquefaction study:

- Scenario earthquake or probabilistic assessment
- Criteria for probabilistic determination
- Uniform risk or site-specific studies

The first factor considered was whether to base the evaluation upon a scenario earthquake, such as a repeat of the 1949 or 1965 Puget Sound earthquakes, or to conduct the evaluation based upon a probabilistic risk assessment of the study area. One advantage of selecting a scenario earthquake is that other studies (U.S. Geological Survey, 1975; Langston and Lee, 1983; Ihnen and Hadley, 1987) have been conducted using such scenario events. Additionally, the results from a scenario-earthquake evaluation may be compared with historical earthquake damage in the area. The disadvantage to a scenario-earthquake study is that the earthquake sources in the area are not constrained to well-defined, known faults with surface rupture. Hence, it is quite probable that future earthquakes could occur at any location within the Puget Sound region and not at the epicenters of past events. Accordingly, it was decided to conduct the liquefaction evaluation based upon a probabilistic assessment of the earthquake hazard in the study area.

In selecting a probabilistic approach, it is necessary to establish the criteria for defining the design earthquake. In this regard, the design-earthquake ground acceleration was selected to correspond with motions having a 10-percent probability of being exceeded within 50 years. This approximately corresponds to a 475-year return interval. The criterion was selected to be consistent with local building practice in Seattle, which is based upon the Uniform Building Code (International Congress of Building Officials, 1991). Thus, the 475-year return interval provides consistency between the liquefaction-hazard map and nationally recognized standards for earthquake design of buildings.

The third factor considered in the liquefaction evaluation was whether to assume that the seismic risk or ground-shaking potential was uniform throughout the entire study area or whether the level of peak ground acceleration

should be varied throughout the study area, considering amplification from topographic effects or subsurface soil conditions. Clearly, one would expect variations in ground accelerations throughout the study area for any given earthquake. These variations could be attributed to differences in subsurface geology or geometric attenuation of energy from the earthquake source. In fact, studies have been conducted to evaluate the local influence of such effects (Langston and Lee, 1983; Ihnen and Hadley, 1987). However, one limitation of microzonational studies of local ground motion is that the subsurface conditions throughout the study area are not perfectly known. Furthermore, the results of local studies of ground-motion effects in the Seattle area (Langston and Lee, 1983; Ihnen and Hadley, 1987) have shown that the calculated results were highly dependent upon the focal mechanism and location of the generating earthquake. Thus, it would appear that whereas techniques are available for computing ground motions on a microzonational level, these computed ground motions may be highly speculative and their application may be limited, considering the unconstrained location of future earthquakes in the Puget Sound region.

To avoid introducing additional uncertainties that are associated with the calculation of site-specific earthquake ground motions, it was decided to base the liquefaction evaluation upon a single level of ground acceleration. This would imply a uniform seismic risk throughout the entire study area. Although in reality ground motions may vary throughout the study area, there are several reasons that support selection of a single value of ground-surface acceleration. First, the fact that earthquakes in the Puget Sound region are not constrained to well-known structural features indicates that future earthquakes will likely occur at random in the region. This factor is consistent with the assumption of a uniform seismic risk. A second factor supporting the selection of a single ground-acceleration value is that the study area is predominantly underlain by similar soil conditions. Specifically, about 80 percent of the study area is underlain by glacially consolidated sedimentary deposits that would be categorized as "stiff soils," or "S₂" soils using the Uniform Building Code (International Conference of Building Officials, 1991) soil classification scheme. The remaining 20 percent of the study area is underlain by alluvial soils that may have a somewhat greater potential for ground-motion amplification. These alluvial soils, however, generally do not include the thick sequences of clay that have characteristically resulted in large ground-motion amplifications in other areas, such as the San Francisco Bay region, during prior earthquakes. Thus, based upon the random location of future earthquakes in the Puget Sound region and the predominance of a single soil type underlying the study area, it was concluded that it is reasonable to use a single ground-acceleration value to represent the seismic risk in the liquefaction evaluation.

Based upon a 10-percent probability of exceedance during a 50-year interval, it was decided to use a peak ground

acceleration of 0.30g for the liquefaction-hazard evaluation. This acceleration corresponds to the bedrock acceleration that is indicated in figures 218 and 219. Additionally, it is consistent with the seismic-hazard map recently developed by the USGS (Building Seismic Safety Council, 1991) for sites in the United States that are underlain by "stiff soils," or "S₂" soils as defined in the Uniform Building Code (International Conference of Building Officials, 1991). Thus, this acceleration would likely apply to at least 80 percent of the study area. Furthermore, it is assumed in the liquefaction analysis that this acceleration would correspond to an earthquake having a magnitude of about 7¹/₂. This level of acceleration was used in the liquefaction studies of both Grant (1990) and Perkins (1991). In addition, Perkins (1991) used a peak ground acceleration of 0.15g to evaluate the effects of liquefaction from a smaller earthquake that may have a higher probability of occurrence.

EVALUATION CRITERIA

The final and, perhaps, most important factor in the liquefaction study was the selection of criteria for assigning the relative hazard ranking to the various geologic units in the study area. Selection of an appropriate hazard ranking scheme is complicated by the fact that no one criterion has been consistently used in prior liquefaction studies. Consequently, any liquefaction evaluation criteria used in a mapping study may appear arbitrary and require adjustments to reconcile the predicted performance with past observations of liquefaction. For example, the liquefaction study of San Mateo County, Calif. (Youd and Perkins, 1987), includes an adjustment factor of 10 to reconcile the study results with damage resulting from the 1906 San Francisco earthquake. Consequently, rating criteria developed for other geographic locations may not necessarily be applicable to the Pacific Northwest.

Two criteria were selected to assess the relative hazard rankings of the local geologic units: threshold and thickness. Threshold is based upon the relative percentage of SPT N-values in a geologic unit that would signify liquefaction during the 0.30g earthquake. Thickness differentiates the liquefaction hazard on the basis of the computed thickness of a geologic unit that may liquefy during a 0.15g and a 0.30g earthquake. Both the threshold and thickness criteria were selected to provide a reasonable segregation of the liquefaction hazard of the different geologic units in the area.

THRESHOLD CRITERION

The threshold liquefaction criterion (Grant, 1990) is based on evaluating the liquefaction resistance of a geologic unit, defined by the SPT N-values for the unit, as compared with a minimum SPT N-value needed to resist liquefaction for a 0.30g peak ground acceleration.

Minimum SPT N-values needed to resist liquefaction, with appropriate adjustments for fines content, were determined from the following equation based on empirical correlations by Seed and others (1984):

$$N_{uncorr} = \frac{(N_1)_{60}}{C_N} = \frac{0.65(A_{max})\sigma}{g\sigma' r_m C_N} r_d \quad (1)$$

where

- N_{uncorr} is the uncorrected SPT value,
- $(N_1)_{60}$ are the corrected SPT values adjusted for fines content (Seed and others, 1984),
- C_N is the correction factor for overburden pressure (Seed and others, 1984),
- A_{max} is the peak ground acceleration (0.30g),
- σ is the total overburden pressure,
- σ' is the effective overburden pressure,
- r_d is the reduction factor for depth (Seed and others, 1984),
- r_m is the factor for earthquake magnitude (Seed and others, 1984), and
- g is the gravitational acceleration

The liquefaction evaluations primarily concentrated on the materials within 13 m of the ground surface because historical accounts of substantial damage from liquefaction have been concentrated within this depth range. The uncorrected SPT N-values characterizing a particular geologic unit were compared with the minimum SPT N-values to resist liquefaction for each 1.5-m depth interval of that unit. This incremental evaluation would account for potential variability of the N-values with depth within the geologic unit.

The following rating scheme was used to differentiate the hazard potential of the soils in the study area:

Percent of N-values below the 0.30g threshold criteria	Hazard rating
>50	High
25-50	Moderate
10-25	Low
<10	Very low

The percentage cutoff levels in the above tabulation were selected in an attempt to provide reasonable segregation of the data. Whether other cutoff values may be used, too stringent criteria could result in all of the soils falling in the high-hazard rating, and too lax criteria could result in all soils having a very low liquefaction potential. Thus, it is more important to develop rating criteria that segregate the data than it is to use criteria from other locations that may not adequately describe the relative local hazard.

THICKNESS CRITERION

The liquefaction potential of the various geologic units in the area was also evaluated using the thickness criterion (Perkins, 1991), which is based on not only a threshold acceleration but also a minimum thickness of liquefiable material. The total amount (cumulative thickness) of potentially liquefiable soil in each boring was computed using equation 1 and the peak ground accelerations of 0.15g and 0.30g. The calculations were completed for borings that were typically less than 16 m deep. Liquefaction was defined to be significant at locations where a minimum of 3 m of soil (cumulative thickness) would liquefy in the 0.30g earthquake and a minimum of 0.3 m soil would liquefy in the 0.15g earthquake. Although these thickness values are somewhat arbitrary, when combined with the 0.30g and 0.15g acceleration levels this criterion provides a basis for segregating the performance of the underlying geologic units under conditions of a large earthquake and a more common, but smaller event.

The following classification scheme was selected to differentiate the hazard potential of the soils in the study area using the thickness criterion:

Percent of borings with computed liquefaction ¹	Hazard rating
>50	High
25-50	Moderate
<25	Low

¹3 meters liquefaction—0.30g; 0.3 meters liquefaction—0.15g.

SPT BIAS

One potential concern in the liquefaction evaluation was that the drilling method may have a significant effect on the SPT N-values obtained in the borings. Whereas rotary techniques have been recommended as a standard procedure in liquefaction evaluations (Seed and others, 1984), the vast majority of borings drilled in the Puget Sound region were advanced using hollow-stem auger drill rigs.

To evaluate the potential effect of drilling procedures upon the resulting SPT N-values, a comparative study was made of N-values at sites at the mouth of the Duwamish River, where there is a high concentration of both hollow-stem auger and rotary borings in a relatively confined area. The results from this study, which are presented in figure 220, indicate that the N-values obtained in the hollow-stem auger borings are about 6-7 blows per foot less than the N-values from the rotary borings. Additionally, the data presented in figure 220 indicate that the mean N-values from the hollow-stem auger borings reasonably approximate the lower quartile N-values from the rotary borings.

On the basis of these results, it is concluded that whenever feasible, the rotary-boring data set should be used in the liquefaction evaluations. Additionally, it was assumed that the mean N-values from the hollow-stem auger data would reasonably approximate the lower quartile N-values if all data were obtained using rotary techniques. This assumption is an integral part of our evaluation because of the lack of coverage of rotary borings within some of the geologic units.

STUDY RESULTS

As previously indicated, two separate but parallel studies (Grant, 1990; Perkins, 1991) were conducted to delineate the liquefaction hazard of the soils in Seattle. Although different criteria were used in these studies, the results of both research efforts were quite similar. Because of this similarity, a single liquefaction-hazard map (pl. 9) has been developed representing both research efforts and using the previously described methods and database. Both background studies focused upon ranking the relative liquefaction hazard of the major geologic units in the study area because it was assumed that units having the same general depositional characteristics should also have the same liquefaction resistance, provided all other factors are equal, such as the ground-water depth and assumed level of earthquake ground shaking. Three geologic groupings were evaluated for liquefaction resistance: fills, Holocene deposits, and Pleistocene deposits. The three groups were primarily differentiated by age because it was assumed that the youngest deposits would likely have the highest liquefaction potential and the oldest deposits would have the least potential. The areal extent of these geologic units and the assigned hazard rankings are indicated on plate 9.

FILLS

DUWAMISH TIDEFLATS

The liquefaction resistance of the fill and underlying alluvial soils in the Duwamish River tideflats was evaluated because this area represents the largest uncontrolled fill in Seattle (fig. 216). The area is bounded on the north by Elliott Bay, on the east by Beacon Hill, on the west by West Seattle, and on the south by Orcas Street. The ground surface is at about 3 m elevation (city of Seattle datum), and the ground-water table is typically present at depths ranging between 0.6 and 4 m below the ground surface. The subsurface geology typically consists of 3-5 m of fill materials, chiefly sands, underlain by alluvial deposits that are also predominantly sand. The fill material within the tideflat area has largely been deposited using hydraulic techniques. It is estimated that the fill consists of about 70 percent clean sand, 10 percent silty sand, and the remainder

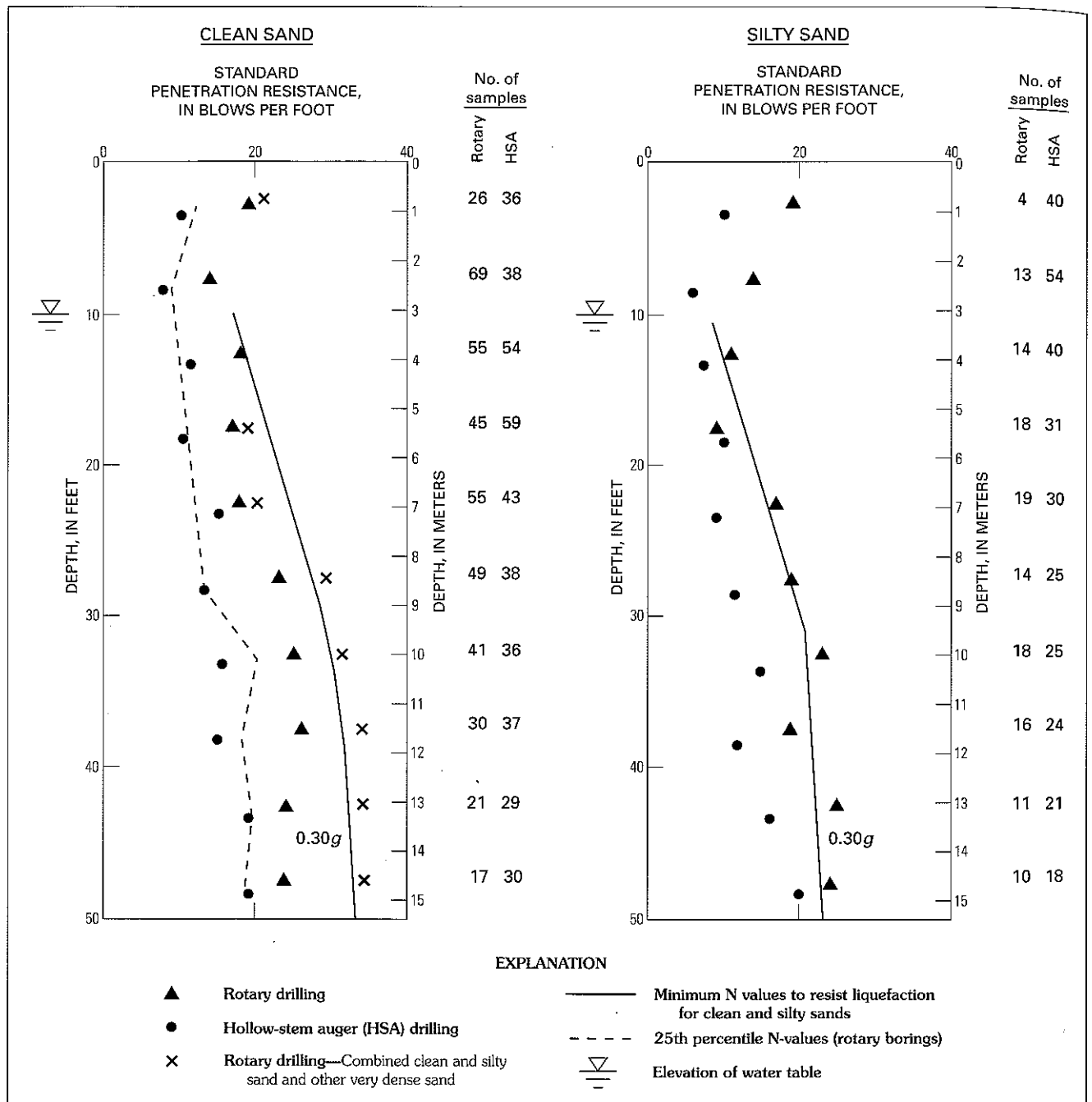


Figure 220. Liquefaction evaluation of the Duwamish River tideflats fill using the threshold criterion. The drilling data represent mean SPT values.

sandy silt and clayey silt. Based on the logs in the study area, the underlying alluvial materials are composed of about 50 percent clean sand and 20 percent silty sand. The remaining materials range from sandy silt to clayey silt.

The threshold criterion was applied to the fills and alluvial soils in the tideflats to evaluate their liquefaction susceptibility. The minimum SPT N-values needed to resist liquefaction from the 0.30g threshold earthquake are indicated, together with the SPT values for the underlying soils, in figure 220. Although the data in figure 220 have

been segregated into both clean sand and silty sand units, the most significant characterization of the data is the composite plot of rotary boring data that includes not only clean and silty sands but also SPT N-values that were excluded from the other plots because the SPT N-values exceeded 40 blows/foot. These high SPT N-values were initially excluded from the data set because of a belief that any N-value of 40 blows/foot or greater may be the result of driving the sampler on a rock. However, a more detailed review of the logs indicated that very few rocks are present in the underlying soils in the

tidelflats area and that excluding N-values above 40 blows/foot would bias the data set. Thus, the composite data set represents the most accurate data set for evaluating the liquefaction resistance of the underlying soils.

Using the threshold criterion and the composite data, figure 220 indicates that the mean (50 percentile) SPT N-values fall below the threshold level in the zone within 10 m of the ground surface. This condition corresponds to a high hazard rating. Because 25–50 percent of the composite SPT N-values of soils below a depth of 10 m fall below the threshold level, it is concluded that the underlying soils would have a moderate liquefaction rating. The high liquefaction-potential rating given to the topmost 10 m of soil in the tidelflats is consistent with the site-specific studies for the West Seattle Freeway Bridge replacement (Shannon & Wilson, 1980), which similarly showed depths of liquefaction in this area to be on the order of 6–9 m for an earthquake with a ground acceleration of about 0.30g.

Data supporting the thickness criterion evaluation of the liquefaction susceptibility of the tidelflats fill are presented in figure 221A. The curves in the figure represent the percentage of borings in the Duwamish tidelflats data set that would experience liquefaction over an interval ranging from 0 to 8 m as a result of earthquake ground shaking with peak accelerations of 0.15g and 0.30g. The cumulative thickness of liquefaction computed for each boring does not necessarily represent a continuous zone of liquefaction. The intent of these evaluations is to further quantify the liquefaction potential and qualitatively indicate the areal extent where liquefaction may occur to a significant degree.

On the basis of the data presented in figure 221A and the thickness criterion previously discussed (Perkins, 1991), we conclude that the fill soils along the Duwamish River tidelflats have a high liquefaction potential. This conclusion is based on the observation that a cumulative thickness of 3 m of liquefaction would occur in about 65 percent of the borings for a 0.30g earthquake. Similarly, it is observed that a cumulative thickness of 0.3 m of liquefaction would occur in 68 percent of the borings for a 0.15g earthquake. A high liquefaction-hazard rating would apply to the tidelflats fill soils because the computed cumulative thickness of potentially liquefiable soils would exceed the minimum thickness criterion in over 50 percent of the borings.

We conclude that the liquefaction potential of the soils in the filled Duwamish River tidelflats is high on the basis of the criteria used in both methods of evaluation. This conclusion is in reasonable agreement with the historical record (fig. 216), which indicates that instances of reported liquefaction primarily occurred in the tidelflats area. Furthermore, the high hazard rating of this area is consistent with the findings of the site-specific liquefaction study for the West Seattle Freeway Bridge replacement (Shannon & Wilson, 1980). Whereas the fill in the tidelflats has been assigned a high liquefaction-hazard rating (pl. 9), areas on the tidelflats within about 60 m of open bodies of water

would have an even higher liquefaction potential and the potential for lateral spreading, based on the historical performance of the area.

INTERBAY

A liquefaction evaluation was performed for the fill soils found in the Interbay area, which is bounded by Salmon Bay on the north, Elliott Bay on the south, Queen Anne Hill on the east, and Magnolia on the west. This location was also identified for special study because it contains a significant amount of uncontrolled fill that was deposited during the early 1900's. Ground-surface elevations in this area typically range between 3 and 6 m (city of Seattle datum), and ground-water levels commonly are about 3 m below the ground surface. Soils in the Interbay area can include as much as 6–9 m of fill soils overlying alluvial deposits. The fill soils may have a variable composition including clean sand, silty sand, garbage, and construction debris or rubble. The underlying native soils range from clean sand to clayey silt.

Data relevant to the threshold evaluation of the liquefaction hazard of the Interbay fill are presented in figure 222. Conclusions drawn from the figure may be compromised somewhat because the data set is relatively small and consists exclusively of hollow-stem auger borings. Nevertheless, because the mean (50 percentile) SPT N-values for the Interbay fill soils fall below the threshold criterion for liquefaction corresponding to a 0.30g earthquake, it is concluded that the Interbay fills have a high liquefaction-hazard rating. Because the mean SPT N-values from the hollow-stem auger data set are typically 10–15 blows per foot below the threshold criterion, the high hazard ranking would not be changed if the hollow-stem auger data were increased by 6–7 blows per foot to provide equivalency with rotary borings (see fig. 220).

Data supporting the thickness evaluation of the liquefaction susceptibility of the Interbay fills are presented in figure 221B. About 65 percent of the borings were calculated to have 3 m and 68 percent to have 0.3 m of sediment that may liquefy during 0.30g and 0.15g events, respectively. Using the thickness criterion previously discussed, the soils in the Interbay area have high liquefaction potential.

Based on the application of both criteria, the Interbay area is judged to have a high liquefaction potential. This liquefaction rating, however, may be somewhat conservative when compared with the high hazard rating also given to the Duwamish River tidelflats fill because historical liquefaction has not been reported at Interbay whereas numerous locations of liquefaction have been reported along the tidelflats. Although this would not preclude liquefaction in the Interbay area, it does demonstrate a higher hazard potential for the Duwamish River tidelflats. Nevertheless, considering the potential variability of soil conditions in the Interbay area, the Interbay fills were assigned a high liquefaction-hazard rating (see pl. 9).

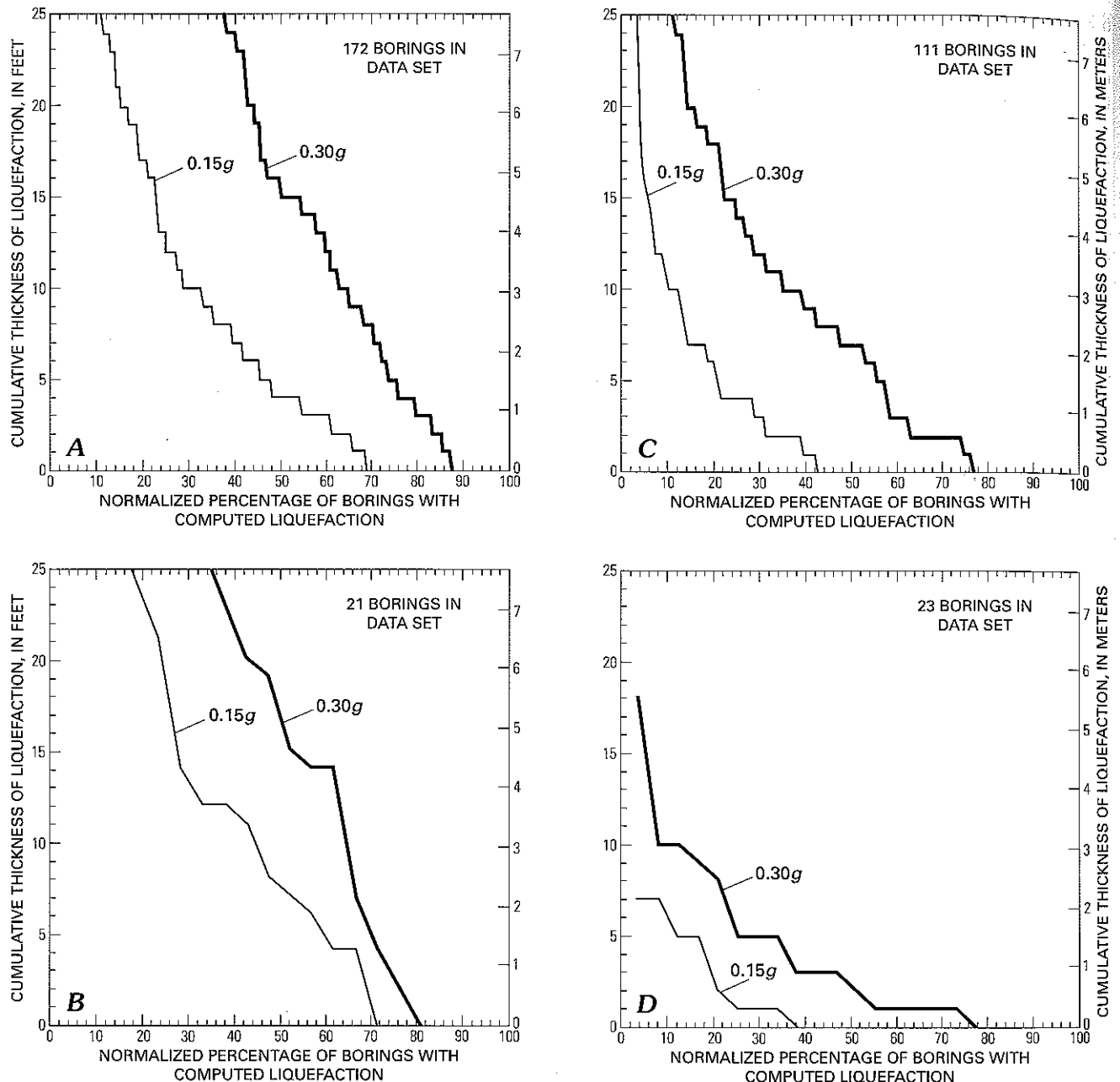


Figure 221. Liquefaction evaluation of soils in the Seattle area using the thickness criterion. Each curve represents the cumulative thickness of liquefaction for the indicated seismic ground acceleration. A, Duwamish River tideflats fill; B, Interbay fill; C, Duwamish River valley Holocene alluvium; D, Alki Point Holocene beach deposits.

OTHER FILLS

Other fills have been mapped by Waldron and others (1962) throughout the Seattle area. Although boring information was sparse or not available for these fills, it was judged prudent to conservatively represent these materials as having a high potential for liquefaction, considering their variable composition and density. This high hazard rating is partly substantiated by the performance of fills at the University of Washington athletic fields and at the south end of Green Lake (fig. 216), which experienced liquefaction

during the 1965 Puget Sound earthquake. Therefore, all significant fills mapped by Waldron and others (1962) have been designated as having a high liquefaction-hazard rating (see pl. 9).

HOLOCENE DEPOSITS

ALLUVIUM

The most significant deposit of Holocene alluvium within Seattle consists of flood-plain material (hereafter

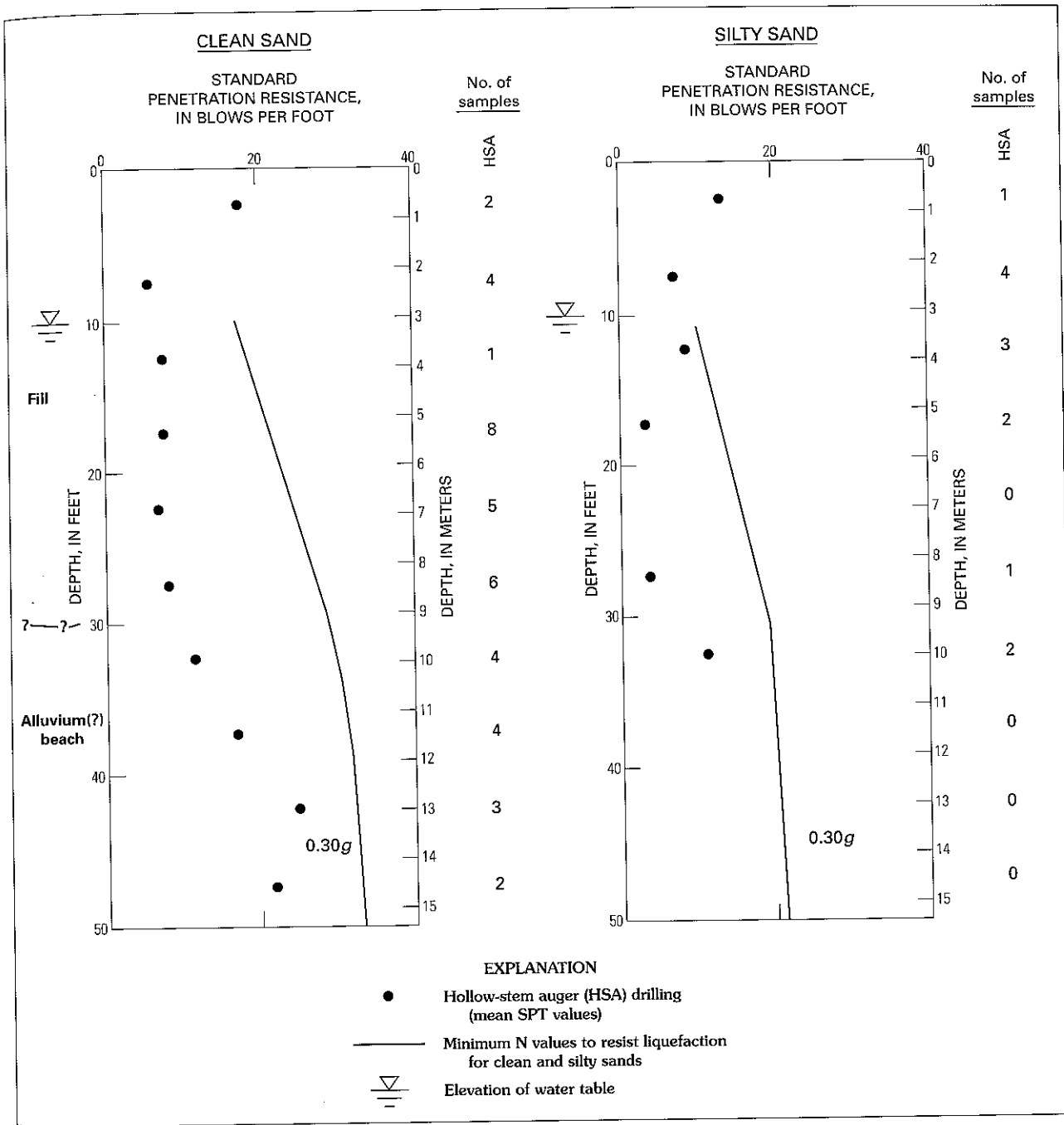


Figure 222. Liquefaction evaluation of the low-lying Interbay area fills using the threshold criterion.

informally referred to as the Duwamish alluvium for simplicity) in the Duwamish River valley. The valley typically extends several hundred to several thousand meters on either side of the Duwamish River. This zone is characterized by a relatively flat lying area with ground surface elevations ranging between 3 and 6 m (city of Seattle datum) and ground-water levels 0.6–3 m below the ground surface. Typically, soils within the area consist of shallow fill overlying alluvial deposits that may contain about 60 percent clean to silty sand and about 40 percent sandy to clayey silts. The

alluvial material in the upper parts of the Duwamish alluvium contains a somewhat larger percentage of silt compared with the alluvial materials underlying the Duwamish tideflats.

Data relevant to the threshold evaluation of the liquefaction hazard of the Duwamish alluvium are presented in figure 223. The N-values in this plot have been segregated based upon drilling technique as well as the material encountered within the sampling depth. As indicated in the figure, there is a relatively small percentage of rotary borings

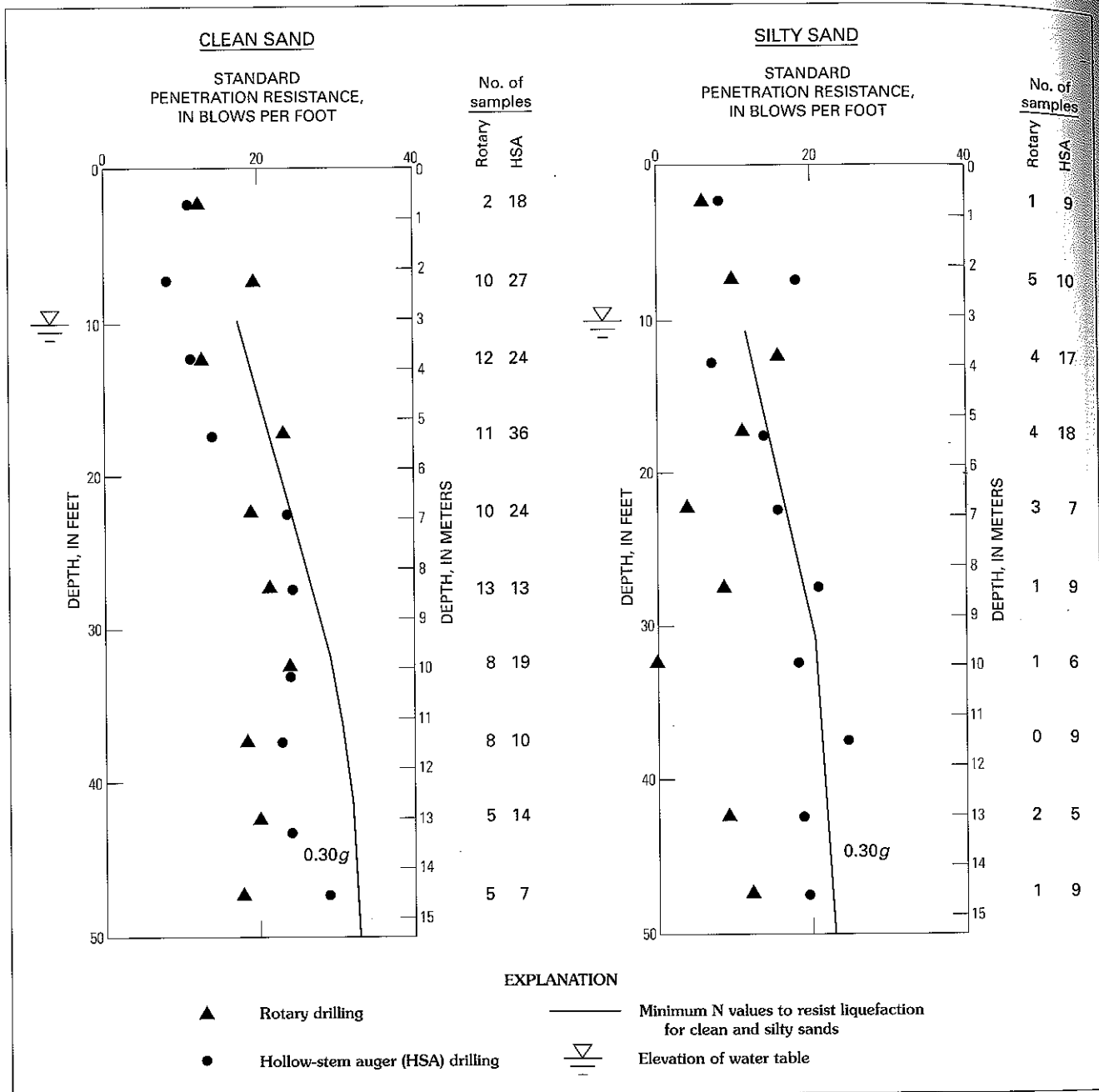


Figure 223. Liquefaction evaluation of Holocene alluvium in the Duwamish River valley using the threshold criterion. The drilling data represent mean SPT values.

in the data set, with most of the information obtained from hollow-stem auger drilling borings. Additionally, the smaller set of rotary-boring SPT data is somewhat suspect because the consistency that existed between the larger data set of rotary and hollow-stem auger borings in the Duwamish River tideflats (fig. 220) is absent in the data from the Duwamish alluvium. Furthermore, figure 223 shows the mean SPT data from the rotary borings to be erratic whereas the mean SPT data from the hollow-stem auger borings are less variable and similar to the data shown for the tideflats fill (fig. 220).

Due to these inconsistencies, it was decided to evaluate the liquefaction potential of the Duwamish alluvium based upon the hollow-stem auger data set. Because the SPT N-values shown in figure 223 suggest that between 25 and 50 percent of the data would fall below the 0.30g threshold criterion, it was concluded that the Duwamish alluvium has a moderate liquefaction potential. This conclusion is based on the assumption that the mean SPT N-values from the hollow-stem auger borings correspond to the 25-percentile values of the equivalent rotary data and

that there would be a differential of about 6 blows/foot between the 25- and 50-percentile values (see fig. 220).

Data supporting the thickness criterion evaluation of the liquefaction susceptibility of the Duwamish alluvium are presented in figure 221C. As shown in this figure, about 40 percent of the borings in the area were calculated as having at least 3 m of sediment that may liquefy during a 0.30g earthquake. Also, about 38 percent of the borings were calculated as having at least 0.3 m of sediment that may liquefy during a 0.15g event. Thus, we conclude from the thickness criterion evaluation that the soils in the upper part of the Duwamish alluvium have a moderate liquefaction rating.

The moderate liquefaction rating of the Duwamish alluvium appears to be reasonably consistent with the rating given to the materials within the Duwamish tideflats area because the materials in the upper parts of the Duwamish alluvium appear to have higher SPT values, on the average, compared with the materials in the tideflats area. This conclusion is based on reviewing the SPT N-values from hollow-stem auger borings advanced in both areas where the SPT N-values in the upper part of the Duwamish alluvium are about 7 or 8 blows/foot higher than the values obtained at the mouth of the Duwamish River. Similarly, this liquefaction rating appears to be consistent with the historical performance of the area during the 1949 and 1965 earthquakes, which indicated relatively few instances of liquefaction in the upper Duwamish portion of the study area.

Whereas alluvial materials are present in areas other than along the Duwamish River, the boring coverage that exists in these areas is too sparse to support a significant evaluation of liquefaction resistance. Therefore, based on the Duwamish River valley data, all Holocene alluvial deposits in the study area were classified as having a moderate liquefaction-potential rating (see pl. 9).

BEACH DEPOSITS

Beach deposits within the Seattle area are primarily found along Puget Sound at West Point, west of Magnolia, and by Alki Point in West Seattle. These deposits form relatively local zones where there may be either residential development or municipal treatment plant facilities such as at West Point. These zones typically have ground-surface elevations ranging between 3 and 9 m (city of Seattle datum) and ground-water levels typically on the order of 3 m or more beneath the ground surface. The beach deposits are predominantly clean, fine sands.

Data relevant to the threshold evaluation of the liquefaction hazard of these Holocene beach deposits are presented in figure 224. Whereas there are relatively few borings in the data set, the data show that the mean SPT N-values from the rotary borings are consistently higher than the hollow-stem auger data, similar to the trend observed in the larger data set of the Duwamish tideflats area (fig. 220).

Furthermore, the mean SPT N-values from both the hollow-stem auger and rotary data show consistency (absence of erratic N-values) at various depths below the ground surface. Thus, it was concluded that the rotary boring data presented in figure 224 are applicable for the liquefaction evaluation. Because the SPT N-values of the rotary borings in the figure indicate that between 25 and 50 percent of the data fall below the 0.30g threshold criterion (mean SPT N-values are above the threshold), it was concluded that the Holocene beach deposits have a moderate liquefaction potential.

A similar conclusion was derived using the thickness criterion and the data presented in figure 221D for the Alki Point area. As shown in this figure, about 35 percent of borings were found to have at least 0.3 m of sedimentary materials that may liquefy during a 0.15g event. This percentage corresponds to a moderate liquefaction rating. However, only 13 percent of borings were calculated to have at least 3 m of potentially liquefiable soil during a 0.30g earthquake, which would correspond to a low rating. In light of this variance, a moderate liquefaction-hazard rating was conservatively assigned to the Holocene beach deposits.

The moderate liquefaction-susceptibility rating of the beach deposits is reasonable when considering that only two instances of liquefaction of beach deposits were reported during the 1949 and 1965 Puget Sound earthquakes. Both observations occurred at the same residence in the Alki Point area. Liquefaction was not reported at West Point following either event. However, excavations conducted for the West Point treatment plant have encountered materials that would suggest ancient liquefaction (paleoliquefaction). Thus, on the basis of these limited and scattered observations, we conclude that the moderate liquefaction-hazard rating conservatively but accurately represents the relative hazard of the beach deposits. This rating is consistent when compared with the high hazard rating given to the Duwamish tideflats, where numerous instances of liquefaction occurred during prior historical earthquakes.

OTHER SEDIMENTARY DEPOSITS

Other Holocene sedimentary deposits mapped by Waldron and others (1962), such as their lacustrine sediments (Qsc) and peat (Qp) units, underlie small, isolated parts of the study area. Unfortunately, there is relatively little information in the database to characterize the liquefaction susceptibility of these units. Considering that these units may be largely composed of cohesive sediments, it is believed the liquefaction potential for these soils is relatively low. However, because the composition of these units is largely unsubstantiated by the information contained in the database, these materials were conservatively assigned a moderate liquefaction-potential rating, as shown on plate 9.

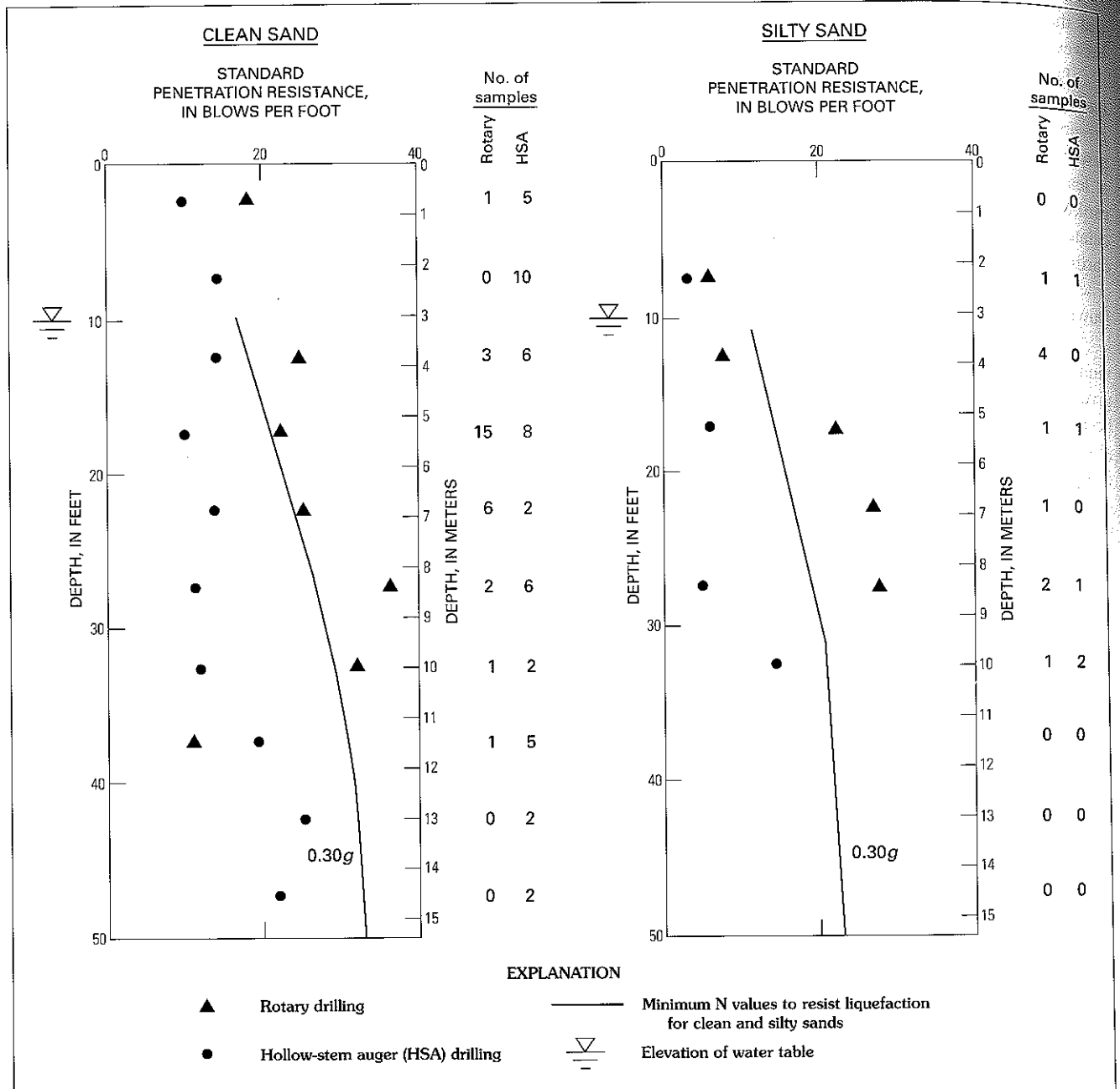


Figure 224. Liquefaction evaluation of Holocene beach deposits found in all borings in the database using the threshold criterion. The drilling data represent mean SPT values.

PLEISTOCENE DEPOSITS

NORMALLY CONSOLIDATED ALLUVIUM

Normally consolidated Pleistocene alluvial deposits typically are found at higher elevations (above 6 m) in scattered locations throughout the study area. Significant deposits of Pleistocene alluvium exist in West Seattle and have been described by Waldron and others (1962) as deposits of sand or gravel. These materials may typically include up to

about 70 percent of clean to silty sand, with the remaining materials consisting of silt or gravel. Because these materials are recessional outwash deposits, they have not been glacially consolidated. Ground-water levels within these deposits may be quite variable, considering that perched water tables exist at higher elevations in the Seattle area. Shallow ground-water conditions would be anticipated within these deposits in areas adjacent to creeks or lakes.

Data relevant to the threshold evaluation of the liquefaction hazard of the Pleistocene alluvial deposits are

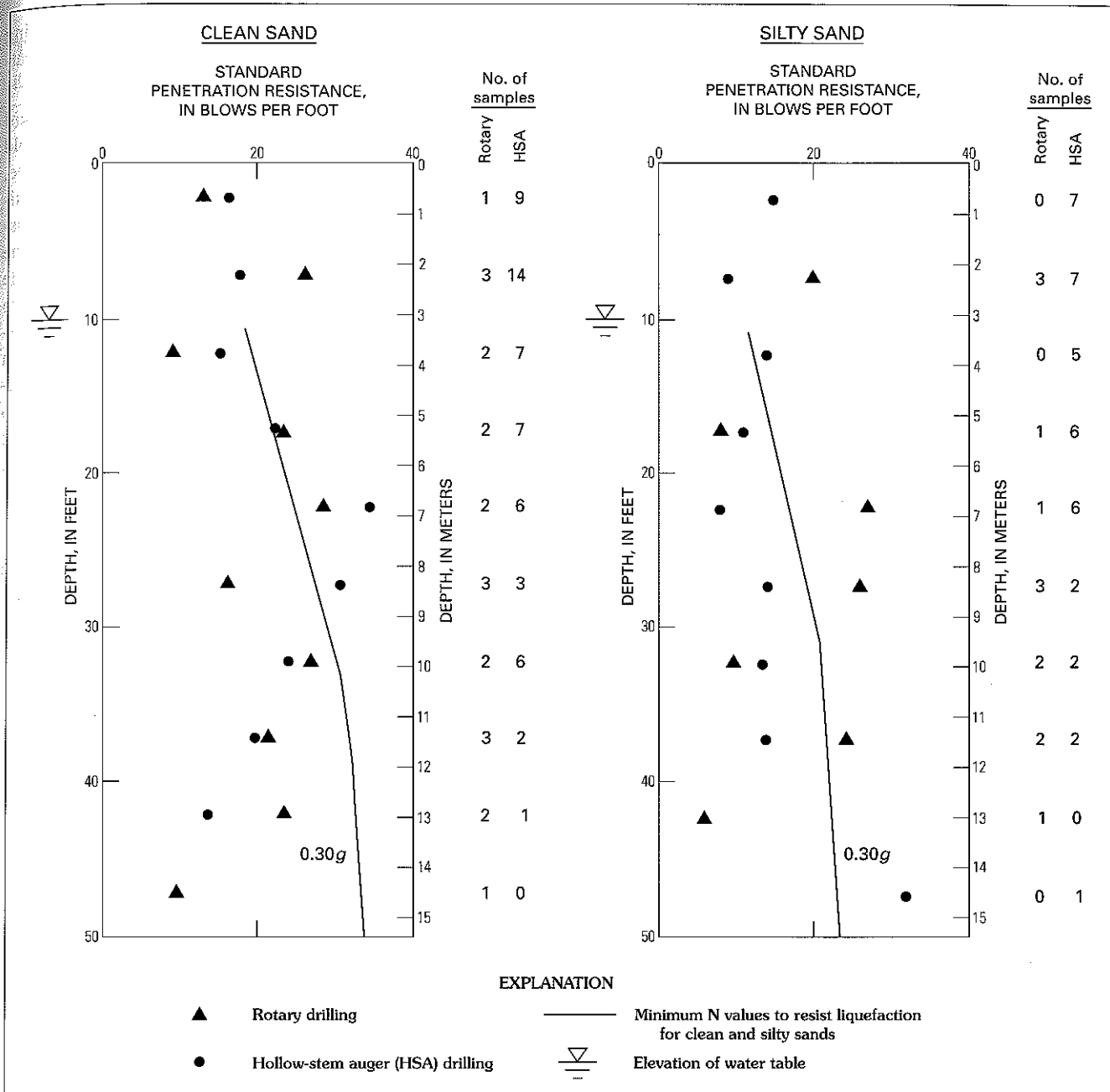


Figure 225. Liquefaction evaluation using the threshold criterion for Pleistocene alluvium found in borings where ground surface elevations exceed 6 m in the Seattle area. The drilling data represent mean SPT values.

presented in figure 225. Because of the size of the data set and some of the inconsistencies noted in the rotary boring data (such as erratic N-values and SPT resistance values lower than the hollow-stem auger data), it was decided to base the liquefaction evaluation on the hollow-stem auger data. The data in figure 225 generally show that the mean SPT values from the hollow-stem auger installations, which are assumed to be equivalent to the lower quartile values from rotary borings, are typically near the minimum SPT

required to resist liquefaction during a 0.30g earthquake. Thus, these high SPT values, combined with the anticipated relatively low ground-water levels, lead to the conclusion that the deposits will be best categorized as having a low liquefaction-potential rating. This rating also is consistent with the observed historical performance of these materials during the 1949 and 1965 earthquakes, as no instances of liquefaction were reported within Pleistocene alluvial deposits.

GLACIALLY CONSOLIDATED SEDIMENTARY DEPOSITS

The majority of the soils within the study area are composed of glacially consolidated sedimentary materials ranging from till to sand and gravel. These materials are found throughout the city, typically at higher elevations, with variable perched ground-water tables.

SPT data obtained from these glacially consolidated sedimentary deposits are summarized in table 37. All the SPT data were obtained using hollow-stem auger drilling techniques. The data in the table have been differentiated on the basis of undetermined-origin material, till, and glaciolacustrine deposits. High blow counts were obtained in practically all cases, and the 10-percentile values are greater than the minimum SPT values required to resist liquefaction during a 0.30g earthquake. As such, we conclude that these materials would have a very low liquefaction-potential rating and that the cohesive soils within this grouping, such as glacial till, are not susceptible to liquefaction.

DISCUSSION

The liquefaction-hazard ratings that were developed for each of the generalized geologic units in the study area were based upon the data presented in figures 220–225 as applied to the threshold and thickness ranking schemes. To provide a means of evaluating the internal and external consistency of the results of the liquefaction study, the rankings of each of the individual geologic units have been summarized in table 38 along with information on their liquefaction performance and the relative liquefaction ranking that would be assigned to the deposits using the liquefaction classification system of

Youd and Perkins (1978). Agreement between these rating schemes increases the confidence of the findings of the Seattle liquefaction study.

Several trends are apparent in the liquefaction hazard rankings shown in table 38. First, although the threshold and thickness criteria are not necessarily mutually inclusive, the hazard rankings developed from both criteria are identical except for the low rating for the Holocene beach deposits corresponding to a 0.30g earthquake. Additionally, the assigned relative hazard rankings are in agreement with the liquefaction performance of the soils. Specifically, (1) areas assigned a high hazard rating frequently had numerous instances of reported liquefaction, (2) areas assigned a moderate rating had minor, scattered occurrences of liquefaction, and (3) areas assigned a low hazard rating had no reported liquefaction. The final external consistency check is the comparison of the assigned hazard ratings and those that would have been assigned using the ranking scheme of Youd and Perkins (1978). As shown in the table, the assigned liquefaction rankings are identical to those that would be determined from the Youd and Perkins (1978) classificational scheme with the exception of fill soils that Youd and Perkins (1978) have ranked as having a very high hazard. On the basis of these favorable internal and external comparisons, we conclude that the liquefaction potential map presented in plate 9 provides reasonable and realistic seismic hazard rankings.

SUMMARY AND CONCLUSIONS

Seattle is located in a tectonic and geologic environment that is conducive to the development of liquefaction during

Table 37. Summary of Standard Penetration Test (SPT) data for glacially consolidated sedimentary deposits in the Seattle area.

[(-), no data available. n, number of samples; N, average standard penetration resistance (blows/foot); 10, 10th percentile of SPT N-values]

Depth range in feet (meters)	Soil source								
	Undetermined origin			Till			Lacustrine		
	n	N	10	n	N	10	n	N	10
0–5 (0–1.5)	1	40	--	--	--	--	--	--	--
5–10 (1.5–3.0)	2	100	--	3	103	--	--	--	--
10–15 (3.0–4.6)	8	100	34	5	120	82	1	68	--
15–20 (4.6–6.1)	15	91	50	6	170	150	2	104	--
20–25 (6.1–7.6)	20	71	32	6	115	33	5	109	38
25–30 (7.6–9.1)	16	101	38	13	118	50	13	113	36
30–35 (9.1–10.7)	27	92	28	7	98	42	7	91	34
35–40 (10.7–12.2)	26	107	39	8	133	100	5	57	31
40–45 (12.2–13.7)	31	135	46	3	120	39	6	102	31
45–50 (13.7–15.2)	32	124	39	2	125	--	4	60	32

Table 38. Comparison of the liquefaction ratings of geologic deposits in the Seattle area.[(-), not available. *g*, acceleration due to gravity (approximately 9.8 m/sec²)]

Deposit	Liquefaction potential					Relative ranking ¹
	Threshold criterion	Thickness criterion		Assigned rating	Historical liquefaction	
	(0.30g)	(0.15g)	(0.30g)			
FILLS						
Duwamish River	High	High	High	High	Numerous	Very high
Interbay	High	High	High	High	None	--
Other	--	--	--	High	Occasional	--
HOLOCENE ALLUVIUM						
Mouth of Duwamish River	High	High	High	High	Numerous	High
Flood plain	Moderate	Moderate	Moderate	Moderate	Occasional	Moderate
HOLOCENE BEACH DEPOSITS	Moderate	Moderate	Low	Moderate	Occasional	Moderate
PLEISTOCENE ALLUVIUM	Low	--	--	Low	None	Low
PLEISTOCENE GLACIAL DEPOSITS	Very low	--	--	Very low	None	Very low

¹Based on the classification system of Youd and Perkins (1978).

relatively strong earthquakes. Instances of liquefaction have been reported during the two largest historical earthquakes in the region, the April 13, 1949, magnitude 7.1 Olympia earthquake and the April 29, 1965, magnitude 6.5 Seattle-Tacoma earthquake. Although only moderate damage occurred in Seattle as a result of these earthquakes, the damage level is consistent with the low level of acceleration recorded locally (approximately 0.10g) during both these events. This historical damage, however, may not accurately represent the potential hazard in Seattle because the area may likely experience an earthquake with a ground acceleration of 0.30g (Algermissen, 1988b; International Conference of Building Officials, 1991). The liquefaction-hazard potential of the area may be even greater during a subduction-zone earthquake because the duration of such an earthquake may be several times greater than any historical event experienced in the Puget Sound region.

The methodology used for evaluating and mapping the local liquefaction hazard assumed that the entire study area would be subjected to a uniform peak ground acceleration of either 0.15g or 0.30g. The 0.30g level of acceleration has a 10-percent chance of being exceeded in 50 years (475-year return period), and this acceleration is consistent with either bedrock (Algermissen, 1988b) or stiff soil deposits (Building Seismic Safety Council, 1991). The 0.30g acceleration is also consistent with local practice for the seismic design of buildings (International Conference of Building Officials, 1991). This level of acceleration was coupled with the empirical liquefaction procedures of Seed and Idriss (1971), Seed and others (1983), and Seed and others (1984) to determine the liquefaction resistance of the various geologic units

in the area. A generalized liquefaction-hazard rating for each geologic unit was evaluated using threshold performance and thickness criteria. The threshold criterion ranked the relative liquefaction potential on the basis of the percentage of SPT N-values falling below the minimum N-value required to resist liquefaction during the 0.30g event. A high liquefaction potential was assigned to units with a mean (50 percentile) SPT N-value below the threshold value. A low liquefaction-potential rating was given to materials with lower quartile N-values than the 0.30g threshold level. Intermediate values were ranked as having a moderate liquefaction potential.

The second evaluation method was based on the thickness, or vertical extent, of soils that would potentially liquefy in 0.15g and 0.30g earthquakes. A high hazard rating was given to geologic units in which the thickness of the liquefied layer was predicted to exceed 3 m during a 0.30g earthquake and 0.3 m during a 0.15g earthquake in at least 50 percent of the borings. A low rating was assigned to units in which these thicknesses would develop in less than 25 percent of the borings. Intermediate values were ranked as having a moderate liquefaction potential.

On the basis of the methodologies and criteria described, a liquefaction-hazard map was developed for the Seattle area (pl. 9). We conclude that fill soils and underlying alluvial deposits, particularly in the Duwamish River tideflats and the Interbay area, have a high potential for liquefaction during a 0.30g earthquake. Deposits having a moderate liquefaction potential at this seismic level include Holocene alluvium, beach deposits, and other sedimentary materials. The most significant Holocene alluvial deposits

occur along the Duwamish River valley. Pleistocene alluvial deposits, which have not been glacially consolidated, were given a low liquefaction-potential rating. Pleistocene glacially consolidated sedimentary deposits received a very low liquefaction-potential rating.

The results from these studies are intended to provide a regional assessment of liquefaction potential and should not be considered as a substitute for site-specific studies for individual buildings or other structures. Because conditions vary locally, site-specific geotechnical investigations are required to accurately assess liquefaction potential at any given location.

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