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Earthquake Hazards in the Pacific Northwest of the United States

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EVALUATION OF LIQUEFACTION POTENTIAL, SEATTLE, WASHINGTON

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Foreword

This paper is one of a series dealing with earthquake hazards of the Pacific Northwest, primarily in western Oregon and western Washington. This research represents the efforts of U.S. Geological Survey, university, and industry scientists in response to the Survey initiatives under the National Earthquake Hazards Reduction Program. Subject to Director's approval, these papers will appear collectively as U.S. Geological Survey Professional Paper 1560, tentatively titled "Assessing Earthquake Hazards and Reducing Risk in the Pacific Northwest." The U.S. Geological Survey Open-File series will serve as a preprint for the Professional Paper chapters that the editors and authors believe require early release. A single Open-File will also be published that includes only the abstracts of those papers not included in the pre-release. The papers to be included in the Professional Paper are:

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ABSTRACT

This paper presents the results of two research studies (Grant, 1990; and Perkins, 1991) which evaluated the liquefaction hazard potential in Seattle, Washington. Seattle has experienced significant damage related to liquefaction during historic earthquakes, and the City may be subjected to even greater damage in the future, considering the increased development of the area and the potential occurrence of a subduction zone earthquake. Therefore, the purpose of this paper is to discuss the methodologies of both studies and present a singular 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.

The liquefaction hazard evaluation was based upon existing data from over 350 boring logs. A computerized data base was developed to facilitate storage and retrieval of the boring data for subsequent analyses. All liquefaction evaluations were based upon Seed's empirical procedure which relates Standard Penetration Test (SPT) N-values to threshold ground accelerations needed to initiate liquefaction. The liquefaction potential was evaluated using two procedures. The first procedure grouped similar geologic units and assigned relative rankings to the liquefaction potential based upon the percentage of SPT N-values that fell below a threshold N-value needed to resist liquefaction resulting from a 0.30g ground acceleration. The second procedure assigned relative liquefaction potential rankings based upon the computed thicknesses of material in individual borings which would liquefy for ground accelerations of 0.15 and 0.30g. All major geologic units within the study area were reviewed for liquefaction potential using both criteria and assigned one of the following hazard ratings: high, moderate, low or very low. These hazard zones are also delineated on a map of Seattle. The study results showed that Fills and Holocene alluvial deposits at the mouth of the Duwamish river have a high liquefaction potential. Other than at the mouth of the Duwamish, Holocene alluvium and beach deposits were given a moderate liquefaction rating. Pleistocene alluvial sediments were given a low liquefaction rating and glacially-consolidated, Pleistocene sediments 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. For example, 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 Alaskan 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 earthquakes resulted in substantial damage to buildings, bridges, highways, railroads, water distribution systems, and marine facilities. Property damage from the 1949 and 1965 Puget Sound earthquakes totaled $25 million and $12 million, respectively, at the time of each event. Grant (1985) estimated that 25 to 50 percent of the total damage from these earthquakes may be attributed to earthquake-induced ground failures, such as liquefaction. While this amount of damage may seem relatively minor when compared to other major earthquakes in the world, the damage is consistent with the relatively low values of ground acceleration (typically less than 0.10g) that were recorded in Seattle during these events. Should the Puget Sound experience a major 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 the past events.

This paper summarizes the results of two research studies (Grant, 1990; and Perkins, 1991), both of which evaluated the liquefaction hazard potential in Seattle, Washington (see figure 1). The first study (Grant, 1990) evolved as a USGS-sponsored effort in which a computer database of existing borings in Seattle was developed and used to evaluate liquefaction potential based on Seed's empirical procedures (Seed and Idriss, 1971; Seed and others, 1983 and 1984). Liquefaction hazard categories were differentiated on the basis of comparing SPT N-values for a geologic unit with the N-values required to resist liquefaction during an earthquake with a 0.30g ground acceleration. Perkins (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 single liquefaction hazard map which is contained in this report.

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 liquefaction hazard map also could be used by engineering departments within various governmental agencies and the insurance industry to estimate the damage potential to the existing building stock during a future earthquake. The map also 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 (roads, bridges, water supply lines) may be particularly damaged, affecting emergency response efforts.

LIQUEFACTION PHENOMENON

Liquefaction is a phenomena 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 earthquake ground 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 pressures 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, however, can be generally related to: 1) parameters characterizing the strength of the underlying soil deposit, 2) the location of the water table, and 3) parameters defining the severity of earthquake ground shaking. Specifically, materials that historically have been the most susceptible to liquefaction include clean sands and silty sands. In addition, the liquefaction resistance of any particular soil is affected by the density, fabric, prior earthquake history, and in situ stress conditions. The second major parameter affecting liquefaction development is the presence and depth of the water table 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 the duration of ground shaking. The earthquake induced stresses and duration of ground shaking are, in turn, affected by the size and location of the causative earthquake, the travel path of the earthquake motions to the site, and any local amplification of ground motions which may occur within the soil column.

Depending on site-specific factors, liquefied soil can cause various types of ground failures, resulting in the adverse performance of overlying structures. Specifically, the occurrence of liquefaction may result in a loss of bearing capacity for shallow foundations located over the liquefied soil. Other adverse performance characteristics associated with liquefaction 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

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

Table 1 provides citations of accounts where liquefaction occurred in Seattle during the 1949 and 1965 Puget Sound earthquakes. The 1949 earthquake occurred on April 13 and was located about 39 miles south-southwest of Seattle, near Olympia. This earthquake had a magnitude (M_S) of 7.1 (Weaver and Baker, 1988). The magnitude (m_b) 6.5 earthquake of April 29, 1965 occurred about 14 miles south of Seattle (Weaver and Baker, 1988). As indicated in table 1, liquefaction during these earthquakes typically resulted in differential settlement of buildings, lateral movement of bulkheads, and cracking of basement walls.

Since uncontrolled fills have been particularly susceptible to liquefaction from historic earthquakes, old topographic maps of Seattle were reviewed to delineate fill areas. The locations of historic shorelines and the associated fills are shown on figure 2. Additionally, figure 2 shows the locations of sites that have liquified during the 1949 and 1965 earthquakes. As illustrated in this figure, the vast majority of reported localities of liquefaction 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 which is needed to aid in the understanding of the historical accounts of liquefaction in Seattle is knowledge of the level of ground shaking or peak ground acceleration which occurred locally during the 1949 and 1965 earthquakes. Fortunately, both events were locally recorded on strong ground motion accelerographs (Shannon & Wilson-Agbabian Associates (SW-AA), 1980) which were located close to areas that experienced liquefaction. The locations of these recording stations are also shown on figure 2. These accelerographs recorded peak ground accelerations of about 0.10g during both the 1949 and 1965 earthquakes (SW-AA, 1980 and California Institute of Technology (CIT), 1976). Based upon seismicity studies conducted by Shannon & Wilson (S&W) for various sites within the Puget Sound region (SW, 1980), it is estimated that this level of ground acceleration may have a 20- to 40-year recurrence interval.

GEOLOGY

Background information is presented on the geology of the Puget Sound and Seattle areas to provide a framework for understanding the various geologic units that may be susceptible to liquefaction. In the following discussion, primary emphasis is given to the glacial deposits in the region. 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.

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 (coastal range) on the west. The basin is open to 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, non-lithified Quaternary sediments of varying thickness generally unconformably overlie Tertiary bedrock. These sediments are both glacial and non-glacial in origin.

The incursion of Pleistocene continental ice into the basin is well documented (Willis, 1898; Bretz, 1913; Mackin, 1941; Crandell and others, 1958, Crandell, 1965; Armstrong and others, 1965; Easterbrook and others, 1967; Mullineaux, 1970; Crandell and Miller, 1974; and 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 southern end of the Puget Lowland some 80 kilometers (50 miles) 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 non-glacial intervals generated combinations of fine-grained deposits of fluvial and lacustrine origin, and 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 meters (3,700 feet) in the central basin near downtown Seattle (Hall and Othberg, 1974; and Yount and others, 1985) (figure 3).

The details of the bedrock underlying the Puget Lowland are not well-defined because of 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 from several bedrock ridges which partly cross the basin along northwest trends. The rocks are mostly folded, faulted, and deeply eroded Tertiary marine and estuarine sediments; volcanic materials consisting of basalt, andesite, and volcaniclastic rocks; and terrigenous deposits such as sandstone, shale, and conglomerate, including extensive interbedded coal seams which lie along the Cascade flank east and south of Seattle. Gravity and magnetic survey data show high differentials in the bedrock elevations which are 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; and Rogers, 1970). One of these steep gravity gradients coincides with the Olympic-Wallowa lineament, a 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, which trend eastward to the Cascade Range.

LOCAL GEOLOGY

GENERAL

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, large lakes, or have been alluviated by streams that occupied the troughs about 13,500 years ago, following retreat of the latest glaciation. Major troughs lie beneath the main body of Puget Sound, the Duwamish River valley, and Lake Washington. The surficial geologic units in the area have been mapped by Waldron (Waldron and others, 1962). The following discussion and symbols used to identify the geologic units correspond to those mapped by Waldron.

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 Valley and at Alki Point in West Seattle. These rocks are folded into northwest to west-northwest-trending anticlines and synclines which are broken by northeast-trending, left lateral faults (Weaver, 1937; and Mullineaux, 1970). All of these faults were last active in early to mid-Tertiary (Gower and others, 1985).

Two Tertiary bedrock units lie beneath and sporadically outcrop within the study area. Waldron and others (1962) identify the oldest unit as middle Eocene sedimentary conglomerate, sandstone, siltstone, and shale chiefly with volcanic clasts. This older unit outcrops along the southern edge of the study area, along the sides and below the Duwamish River valley. The youngest bedrock unit contains Oligocene marine to estuarine sandstone and shale with subordinate amounts of conglomerate, mostly of volcanic origin. These rocks are part of the Blakely Formation (Weaver, 1937; Waldron, 1962; and Livingston, 1971) or Lincoln Creek Formation (Mullineaux, 1970). This unit outcrops across the southern part of Seattle and at Alki Point.

Immediately north of this band of Tertiary rocks, the bedrock surface drops abruptly to elevations of more than 1,100 meters (3,700 feet) below sea level in a horizontal distance of less than 1.6 km (1 mile), as shown on figure 3. Present data suggests that the bedrock surface rises from its lowest point in downtown Seattle gradually to the northeast and is about 400 meters (1,200 feet) below sea level at the north end of Lake Washington (Hall and Othberg, 1974; and Yount and others, 1985).

PLEISTOCENE DEPOSITS

Non-lithified, glacially overridden sediments generally lie unconformably above the Tertiary bedrock (see figure 3). This sediment is both glacial and non-glacial in origin.

The youngest of these sediments, the lower units of Vashon Drift, were deposited as the Vashon ice lobe advanced southward during the Vashon Stade of the Frasier Glaciation approximately 15,000 years bp (Mullineaux and others, 1965). At its greatest extent, the lobe advanced to a position about 80 kilometers (50 miles) south of Seattle (Booth, 1987). Covering Seattle with an estimated 900-meter- (3000 feet) thick layer of ice, the weight of the glacier greatly over-consolidated the underlying sediment, including the lower units of the Vashon Drift. Consequently, sedimentary units deposited before the advance of the Vashon ice lobe were over-consolidated to various degrees. In the Seattle area, these very dense sediments underlie most upland areas, including major hills and ridges.

As the Vashon ice lobe retreated northward, recessional outwash (Qys and Qyg), largely consisting of mixtures of gravel and sand, was deposited. 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 meters (100 feet) or more in major outwash deltas. The younger sands (Qys) are fine- to medium-grained, and generally less than 3 meters (10 feet) thick in the upland channels. The younger gravels (Qyg) are composed of sand and pebble size gravel and are as thick as 30 meters (100 feet).

HOLOCENE DEPOSITS

Holocene deposits in the Seattle area include alluvium (Qa) in the Duwamish, Rainier, and Interbay valleys; beach and adjacent marine deposits (Qb) along shorelines; colluvial and landslide deposits (Qls); and peat (Qp) and lacustrine deposits (Qsc) in upland depressions and along the low-lying lakes. Alluvial deposits (Qa) consist of fine sand, silty fine sand, fine sandy silt, and non-plastic 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 meters (30 feet) of the ground surface. Subsurface explorations for the West Seattle Bridge indicate that the alluvium near the mouth of the Duwamish River generally extends to a depth of about 55 meters (180 feet) and locally extends to depths of 75 meters (250 feet).

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 sediments are composed of silt, clay, and fine sand, and are usually less than 3 meters (10 feet) thick. The peat ranges from fibrous to peaty silt (muck). Both the upland and lowland peats pervasively contain a 2.5- to 5.0-centimeter (1 to 2 inch) layer of ash related to the eruption of Mount Mazama (Crater Lake, Oregon) 6,800 years ago (Wilcox and Power, 1964; and Curran, 1965).

Colluvium is the veneer of loose to medium dense soil that drapes the sides and toes of slopes throughout the city and environs. The deposits consist of mixtures of the materials comprising the slopes, and hence, the grain size of the 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 an inch per year) to catastrophic landslides. The areal extent of landslide deposits (Qls) 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 degrees), the colluvial veneer is generally very thin (1 meter or less), whereas near the toe of the hillside, where slopes angles are 10 to 20 degrees, thicknesses of colluvium generally range from 5 to 10 meters (15 to 30 feet).

LAND MODIFICATION

Major fills (f) and drainage modifications in the city have resulted from engineering projects accomplished during the first two decades of the 20th Century (Phelps, 1978). Areas of major tideland reclamation are shown on figure 2. In the early 1900's, shallow tidal areas of the Duwamish River delta were filled with material which 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 was extended about 0.80 kilometers (0.5 miles) 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 flats south of the Pioneer Square area, where it accumulated to depths up to 12 meters (40 feet) 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 to 5 meters (10 to 15 feet) thick but can be as much as 10 meters (30 feet) thick.

Regrading of the downtown Seattle hillsides was accomplished in two major phases between 1903 and 1928 to facilitate expansion and ease of access within the central business district. This included excavating an entire hill (Denny Hill) which resulted in an excavation which was locally in excess of 30 meters (100 feet). This excavation covered a 62-city block area (Sale, 1976; and Morse, 1989). Glacial soils removed from Denny Hill were either washed or dumped by barge into shallow water 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 the surface of Lake Washington a nominal 3 meters (10 feet) 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 attendant lowering of the water surface in Lake Washington eliminated the Black River, which drained from the south end of Lake Washington to the Duwamish

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

The lowering of Lake Washington left a gently sloping terrace underlain by loose sediments around the lake's periphery, part of which has been retained as parkland 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 which 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 Mud Lake.

Smaller areas of fill, as shown on figure 2, are located at the southern end and northwest corner of Green Lake, the south and west sides of Lake Union, and a portion of Salmon Bay. Smaller fills were also placed in ox-bow features along the Duwamish and Green Rivers.

GROUNDWATER

Most of the normally consolidated soil units in the Seattle area lie in alluvial valleys or along lakes and bays where groundwater levels are relatively high. Static water levels recorded or estimated from borings in various areas of the city are summarized on table 2. As shown in table 2, the average depth to groundwater generally ranges from 0.6 to 3 meters (2 to 10 feet), except in the upland outwash gravels. Although available boring data for the upland gravels suggest the absence of near-surface groundwater, the near-surface presence of water can not be precluded, based on the paucity of data. High groundwater levels are likely where this gravelly unit lies adjacent to lakes or ponds in the upland areas. Additionally, perched groundwater conditions may locally occur.

SEISMICITY

The following provides background information on the historical seismicity, earthquake source mechanisms, and postulated levels of peak ground motion for the Seattle area. An understanding of the seismicity of the area is essential to defining the liquefaction hazard of the 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 brief 160-year historic record in the Pacific Northwest. The largest historic earthquakes in the region are believed to be associated with deep-seated, plate tectonic activity (USGS, 1975). Major mapped faults within the region (55 miles 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 40 miles west of Seattle.

The more significant historic earthquakes (those of Modified Mercalli Intensity VI or greater) that have occurred in the Seattle region are listed in table 3. Of the 18 events listed, 5 have intensities of VII or greater. The largest of these were the April 13, 1949, magnitude (M_s) 7.1, intensity VIII shock, and the April 29, 1965, magnitude (m_b) 6.5, intensity VII-VIII event. These earthquakes, which were respectively centered 63 and 23 kilometers (39 and 14 miles) from Seattle, caused considerable property damage in the city.

Other large historic earthquakes that have affected Seattle include one in the North Cascades of Washington and two in western British Columbia, Canada. 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 approximately 1,295,000 square kilometers (500,000 square miles). It has been estimated that this major shock had a magnitude near 7 and a maximum intensity of VIII. Although the epicentral location of this event is uncertain, owing to the sparse population of the area at that time, it apparently occurred somewhere in the northern Cascades.

In Canada, major earthquakes occurred on Vancouver Island on June 23, 1946, and in the Queen Charlotte Islands on August 21, 1949 (Coffman and Von 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 square kilometers (2,000,000 square miles), damage was minor owing to the sparse population in the epicentral area.

EARTHQUAKE SOURCE MECHANISMS

Earthquake source mechanisms, which have been correlated with the observed historic 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; USGS, 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 kilometers (25 miles) which is consistent with the deep source mechanism hypothesis. The majority of historic events, however, occur at relatively shallow depths of about 24 kilometers (15 miles) 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 earthquakes have occurred in this zone during historic times (160 years). 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 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 and 1988b). Figure 4, which is based on Algermissen and others (1982), presents a regional, probabilistic evaluation of peak ground accelerations that could occur on rock within the Pacific Northwest. The accelerations shown on this figure have a 10 percent probability of being exceeded in a 50-year period, which corresponds to a 475-year return period. Figure 5, which is based on Algermissen (1988b), compares the seismic exposure of Seattle to other cities in the United States. This figure presents ground accelerations on rock which have a 10 percent chance of being exceeded during the indicated time intervals. Both figures 4 and 5 indicate that Seattle may be subjected to a ground acceleration of 0.30g on the average of every 475 years.

While the ground motion estimates presented on figures 4 and 5 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 more local, site specific basis, Langston and Lee (1983) and Ihnen and Hadley (1987) have performed ray tracing studies to investigate the local variations in ground response in the Puget Sound area. 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 greater Puget Sound area that included considerations for ground motion amplification from soil type and wave focusing effects. Results from both of these studies indicated that ground motions along the Duwamish could be 50 to 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. Considering 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 conditions (Youd and Perkins, 1978), to more elaborate, probabilistic, analytical evaluations (Power and others, 1986). These various 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; Youd, 1982; Davis and others, 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 and 1986; and Moriwaki and Idriss, 1987), and the eastern United States (Budhu and others, 1987; Hadj-hamou and Elton, 1989; and Elton and Hadj-hamou, 1990).

Our liquefaction study of Seattle used empirical relations developed by Seed and his colleagues (Seed and Idriss, 1971; Seed and others, 1983 and 1984) to establish the liquefaction potential of the various geologic units in the area. Seed's procedures were used because of their acceptance and wide use in engineering practice. Furthermore, the use of Seed's 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 application of the Seed procedure first required the development of a data base containing groundwater levels and SPT N-values for the various geologic units within the study area. This information was obtained from over 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, liquefaction hazard was also assessed on the basis of the computed cumulative thickness of potentially liquefiable soil within the borings. Details of the data base, peak ground motions, and the evaluation criteria are subsequently discussed.

DATABASE

Because liquefaction susceptibility is affected by the depth, relative density, and gradation of the soil; depth of the water table; and geologic origin of the soil, a computerized data base was developed to facilitate the storage and retrieval of subsurface data for subsequent use in the liquefaction evaluation (Grant, 1990). The data base, which includes the logs of over 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 in the data base includes Universal Transverse Mercator (UTM) coordinates, location description, date drilled, drilling method, surface elevation of the boring, static groundwater depth, and SPT N-values as a function of depth. Each SPT sample in a boring was assigned a code corresponding to the geologic unit of material as well as a separate code describing the composition of the materials within the sample. The data was subsequently retrieved, corresponding to a particular geologic unit or material, to determine its liquefaction susceptibility. By creating the data base to include 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 in the liquefaction evaluation 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 (USGS, 1975; Langston and Lee, 1983; and Ihnen and Hadley, 1987) similarly have been conducted for scenario events. Additionally, the results from a scenario earthquake evaluation may be compared with historic earthquake damage in the area. However, 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 area 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 next necessary to establish the criteria for defining the design earthquake. In this regard, the design earthquake ground acceleration was selected to correspond to motions having a 10 percent probability of being exceeded within 50 years. This approximately corresponds to a 475-year return interval. This criteria was selected to be consistent with local building practice in Seattle which is based upon the Uniform Building Code (International Congress of Building Officials (ICBO), 1991). Thus, this 475-year return interval provides consistency between the liquefaction hazard map and nationally recognized standards for the 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 while 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 in the liquefaction analysis that are associated with the calculation of site-specific earthquake ground motions, it was decided to base the liquefaction evaluation upon a singular 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 singular value of ground surface acceleration. First, the fact that the earthquakes in the Puget Sound area are not constrained to well known structural features indicates that future earthquakes will likely occur at random in the area. This factor is consistent with the assumption of a uniform seismic risk. A second factor supporting the selection of a singular value of ground acceleration throughout the study area is the fact that the study area is predominantly underlain by similar soil conditions. Specifically, approximately 80 percent of the study area is underlain by glacially consolidated sediments that would be categorized as "stiff soils" or an "S_2" soil using the Uniform Building Code (ICBO, 1991) soil classificational 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 thick sequences of clay that have characteristically resulted in large ground motion amplifications that have occurred 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 area and the predominance of a singular soil type underlying the study area, it was concluded that it is reasonable to use a singular value of ground acceleration to represent the seismic risk in the liquefaction evaluation.

Based upon the above criteria (10 percent probability of exceedance during a 50-year interval), it was elected to use a peak ground acceleration of 0.30g for the liquefaction hazard evaluation. This ground acceleration corresponds to the bedrock acceleration that is indicated in Figures 4 and 5. Additionally, this acceleration is consistent with the seismic hazard map recently developed by the U.S. Geological Survey (Building Seismic Safety Council (BSSC), 1991) for sites in the United States that are underlain by "stiff soils" or "S_2" soils as defined in the Uniform Building Code (ICBO, 1991). Thus, this level of acceleration would likely apply to at least 80 percent of the study area. Furthermore, it is assumed in the liquefaction analysis that this level of acceleration would correspond to an earthquake having a magnitude of about 7.5, which is consistent with the largest earthquake magnitude that could likely occur in the Puget Sound region (USGS, 1975). This level of acceleration was used in the liquefaction studies of both Grant (1990) and Perkins (1991). In addition, Perkins 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, the 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 criteria 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 (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" criteria and "Thickness" criteria. The threshold criteria is based upon the relative percentage of SPT N-values in a geologic unit that would signify liquefaction during the 0.30g earthquake. The thickness criteria differentiates the liquefaction hazard on the basis of the computed thickness of a geologic unit that may liquify 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. Details of both criteria are discussed below.

THRESHOLD CRITERIA

The threshold liquefaction criteria (Grant, 1990) is based on evaluating the liquefaction resistance of a geologic unit, as 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 acceleration. Minimum SPT N-values needed to resist liquefaction, with appropriate adjustments for fines content, were determined from the following equation based on Seed's empirical correlations (Seed and others, 1984):

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

where:

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

The liquefaction evaluations primarily concentrated on the materials within 13 meters (40 feet) 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 5-foot 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:

THRESHOLD CRITERIA

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. While other cutoff values may be used, too stringent a criteria could result in all of the soils falling in the high hazard rating, whereas too lax a criteria could result in all soils having a very low liquefaction potential. Thus, it is more important to develop a rating criteria that segregates the data, than it is to use criteria from other locations that may not adequately describe the relative local hazard.

THICKNESS CRITERIA

The liquefaction potential of the various geologic units in the area was also evaluated using the "thickness" criteria (Perkins, 1991) which is based not only on 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 0.15 and 0.30g. The calculations were completed for borings that were typically less than 16 meters (50 feet) deep. Liquefaction was defined to be significant at locations where a minimum of 3 meters (10 feet) of soil (cumulative thickness) would liquefy in the 0.30g earthquake and a minimum of 0.3 meters (1-foot) 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 criteria 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 criteria was selected to differentiate the hazard potential of the soils in the study area using the "thickness" criteria:

THICKNESS CRITERIA

Percent of Borings With Computed Liquefaction*	Hazard Rating
> 50	High
25-50	Moderate
< 25	Low

*3 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. While 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 area have been 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 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 6, indicate that the N-values obtained in the hollow-stem auger borings are about 6 to 7 blows per foot less than the N-values from the rotary borings. Additionally, the data presented in figure 6 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 the above, 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 and 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 singular liquefaction hazard map (Plate 1) has been developed representing both research efforts. This map has been developed using the previously described methods and data base. Both 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 providing that all other factors are equal, such as the groundwater 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 considered 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 1. The following discusses the detailed evaluations of the liquefaction potential of each of these units.

FILLS

DUWAMISH TIDEFLATS

A study was conducted to evaluate the liquefaction resistance of the fill and underlying alluvial soils in the Duwamish tideflats area because this area represents the largest uncontrolled fill in Seattle (see figure 2). This area is bounded on the north by Elliott Bay, on the east by Beacon Hill, on the west by West Seattle, and the south by Orcas Street. The ground surface within the area is about elevation 3 meters (10 feet) (City of Seattle datum), and the groundwater table is typically present at depths ranging between 0.6 and 4 meters (2 and 11 feet) below the ground surface. Subsurface conditions typically consist of 3 to 5 meters (10 to 15 feet) of fill materials, chiefly sands, that are underlain by alluvial deposits that are also predominantly sands. The fill material within the tideflat area has largely been placed by hydraulic techniques. It is estimated that the fill consists of approximately 70 percent clean sand, approximately 10 percent silty sand, and the remainder sandy silt and clayey silt. Based upon the logs in the study area, the underlying alluvial materials are composed of approximately 50 percent clean sand and 20 percent silty sand. The remaining materials range from sandy silt to clayey silt.

The "threshold" criteria was applied to the fills and alluvial soils in the Duwamish Tideflats area 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 6. Although the data in figure 6 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 have been 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 it was felt that any N-value of 40 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 Tideflats area and that excluding N-values above 40 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 criteria and the composite data, figure 6 indicates that the mean (50 percentile) SPT N-values fall below the threshold level in the zone within 10 meters (30 feet) of the ground surface. This condition corresponds to a "high" hazard rating. Since 25 to 50 percent of the composite SPT N-values of soils below a depth of 10 meters (30 feet) 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 surficial 10 meters (30 feet) of soil in the Duwamish tideflats 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 to 9 meters (20 to 30 feet) for an earthquake with a ground acceleration of about 0.30g.

Data supporting the "thickness" criteria evaluation of the liquefaction susceptibility of the Duwamish Tideflats fill soils are presented in figure 7. The curves in figure 7 represent the percentage of borings in the Duwamish tideflats data set that would experience liquefaction over an interval ranging from 0 to 8 meters (0 to 25 feet) as a result of earthquake ground shaking with peak accelerations of 0.15 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 7 and the thickness criteria previously discussed (Perkins, 1991), it is concluded that the fill soils along the Duwamish Tideflats have a high liquefaction potential. This conclusion is based on the observation that a cumulative thickness of 3 meters (10 feet) of liquefaction would occur in about 65 percent of the borings for the 0.30g earthquake. Similarly, it is observed that a cumulative thickness of 0.3 meters (1-foot) of liquefaction would occur in 68 percent of the borings for the 0.15g earthquake. A high liquefaction hazard rating would apply to the Duwamish Tideflats fill soils because the computed cumulative thickness of potentially liquefiable soils would exceed the minimum thickness criteria in over 50 percent of the borings. Hence, based upon the "thickness" criteria, the Duwamish Tideflats Fills have a high liquefaction rating.

We conclude that the liquefaction potential of the soils in the filled Duwamish tideflats is high, on the basis of the criteria used in both methods of evaluation. This conclusion is in reasonable agreement both with historic performance in the study area (see figure 2), which indicates that instances of reported liquefaction primarily occurred in the Duwamish tideflats 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). While the fill in the tideflats has been assigned a high liquefaction hazard rating (see plate 1), areas on the tideflats within about 60 meters (200 feet) of open bodies of water would have an even higher liquefaction hazard, on the basis of historic performance of the area and the potential for the development of lateral spreading.

INTERBAY

A liquefaction evaluation was also 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 as it contains a significant amount of uncontrolled fill that was placed during the early 1900's. Ground surface elevations in this area typically range between 3 to 6 meters (10 and 20 feet) (City of Seattle datum) and groundwater levels commonly are at about 3 meters (10 feet) below the ground surface. Soils in the Interbay area can include as much as 6 to 9 meters (20 to 30 feet) 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 8. Conclusions from the data set presented in figure 8 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 criteria 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 to 15 blows per foot below the threshold criteria, the conclusion on the high hazard ranking would not be changed if the hollow-stem auger data were increased by 6 to 7 blows per foot to provide equivalency with rotary borings (see figure 6).

Data supporting the "thickness" evaluation of the liquefaction susceptibility of the Interbay Fills are presented on figure 9. As shown in this figure, approximately 65 and 68 percent of the borings were calculated to have 3 and 0.3 meters (10 and 1 feet) of sediment which may liquefy during the 0.30g and 0.15g events, respectively. Using the thickness criteria previously discussed, the soils in the Interbay area would 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 to the high hazard rating also given to the Duwamish Tideflats fill. This rating may be conservative because historic liquefaction has not been reported at Interbay whereas numerous locations of liquefaction have been reported along the Duwamish Tideflats. While this would not preclude liquefaction in the Interbay area, it does demonstrate a higher hazard potential for the Duwamish Tideflats. Nevertheless, considering the potential variability of soil conditions in the Interbay area, the Interbay Fills were assigned a high liquefaction hazard rating (see Plate 1).

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 the variable composition and density of these materials. This high hazard rating is partly substantiated by the performance of the fills at the University of Washington athletic fields and at the south end of Green Lake (figure 2) that 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 Plate 1).

HOLOCENE DEPOSITS

ALLUVIUM

The most significant deposit of Holocene alluvium within Seattle consists of flood plain material within the Duwamish River Valley. This area typically extends several hundred to several thousand feet on either side of the Duwamish River. This zone is characterized by a relatively flat-lying area with ground surface elevations ranging between 3 to 6 meters (10 and 20 feet) (City of Seattle datum) and groundwater levels typically encountered at depths about 0.6 to 3 meters (2 to 11 feet) below the ground surface. Typically, soils within the area consist of shallow fill overlying alluvial deposits that may contain approximately 60 percent clean to silty sand and approximately 40 percent sandy to clayey silts. The alluvium in the upper portions of the Duwamish contains a somewhat larger portion 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 on figure 10. 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 on this figure, there is a relatively small percentage of rotary borings comprising 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 Tideflats (figure 6) is absent in the data of the Duwamish alluvium (figure 10). Furthermore, figure 10 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 Duwamish Tideflats fill (figure 6).

Because of these inconsistencies, it was decided to evaluate the liquefaction potential of the Duwamish alluvium based upon the hollow-stem auger data set. Since the SPT N-value data shown on figure 10 would suggest that between 25 and 50 percent of the data would fall below the 0.30g threshold criteria, 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 figure 6).

Data supporting the "thickness" criteria evaluation of the liquefaction susceptibility of the Duwamish alluvium are presented in figure 11. As shown on this figure, approximately 40 percent of the borings in this area were calculated to have at least 3 meters (10 feet) of sediment which may liquefy during the 0.30g earthquake. Also, about 38 percent of the borings were calculated to have at least 0.3 meter (1 foot) of sediment which may liquefy during the 0.15g event. Thus, it is concluded from the thickness criteria evaluation that the soils in the upper Duwamish have a moderate liquefaction rating.

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

While alluvial materials are present in areas other than along the Duwamish, relatively sparse boring coverage exists in these areas to support a significant evaluation of liquefaction resistance. Therefore, based on the Duwamish data, all Holocene alluvial deposits in the study area were classified as having a moderate liquefaction potential rating (see plate 1).

BEACH DEPOSITS

Beach deposits within the Seattle area are primarily found along Puget Sound at West Point, which is west of

Magnolia, and along the Sound by Alki Point in West Seattle. These beach deposits comprise relatively local zones along the Sound where there may be either residential development or municipal treatment plant facilities, such as the location at West Point. These zones typically have ground surface elevations ranging between 3 and 9 meters (10 and 30 feet) (City of Seattle datum) where the groundwater levels are typically on the order of 3 meters (10 feet) or greater beneath the existing ground surface. The beach deposits are predominantly composed of clean, fine sands.

Data relevant to the "threshold" evaluation of the liquefaction hazard of the Holocene Beach deposits are presented in figure 12. While there are relatively few borings comprising the data set in figure 12, 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 trends observed in the larger data set of the Duwamish Tideflats area (figure 6). Furthermore, the mean SPT N-values from both the hollow-stem 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 12 are applicable for the liquefaction evaluation. Because the SPT N-values of the rotary borings shown on figure 12 indicated that between 25 and 50 percent of the data fall below the 0.30g threshold criteria (mean SPT N-values are above the threshold), it was concluded that the Holocene Beach deposits have a moderate liquefaction potential.

A similar conclusion on the hazard rating was derived using the "thickness" criteria and the data presented in figure 13 for the Alki beach area. As shown in this figure, approximately 35 percent of borings were found to have at least 0.3 meter (1-foot) of sediment which may liquefy during the 0.15g event. This percentage corresponds to a moderate liquefaction rating. However, only 13 percent of borings was calculated to have at least 3 meters (10 feet) of potentially liquefiable soil from the 0.30g earthquake, which would correspond to a low rating. In light of this variance, however, a moderate liquefaction hazard rating was conservatively assigned to the Holocene beach deposits.

It is concluded that the moderate liquefaction susceptibility rating of the beach deposits is reasonable when considering that only two instances of liquefaction were reported for locations of Beach deposits during the 1949 and 1965 Puget Sound earthquakes. These observations occurred at the same residence in the Alki 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, it is concluded that the moderate liquefaction hazard rating conservatively represents the relative hazard of the beach deposits. This moderate hazard rating is consistent when compared to the high hazard rating given to the Duwamish Tideflats where numerous instances of liquefaction occurred during prior historic earthquakes.

OTHER SEDIMENTS

Other Holocene sediments mapped by Waldron and others (1962), such as lacustrine sediments and peat deposits, comprise relatively small isolated portions of the study area. Unfortunately, there is relatively little information in the data base to characterize the liquefaction susceptibility of these units. Considering that these units may be largely comprised of cohesive sediments, it is considered that 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 data base, these materials were conservatively assigned a moderate liquefaction potential rating as shown on plate 1.

PLEISTOCENE DEPOSITS

NORMALLY CONSOLIDATED ALLUVIUM

Normally consolidated Pleistocene alluvial deposits typically exist at higher elevations (elevations above 6 meters (20 feet)) in scattered locations throughout the study area. Significant deposits of Pleistocene alluvium exists 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. Groundwater levels within these deposits may be quite variable, considering that perched water tables exist at higher elevations in the Seattle area. Shallow groundwater 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 presented in figure 14. Because of the size of the data set and some of the inconsistencies noted in the rotary boring data (i.e., 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 14 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 the 0.30g earthquake. Thus, these high SPT values, combined with the anticipated relatively low groundwater levels within this deposit, lead to the conclusion that these materials will be best categorized as having a low liquefaction potential rating. This low hazard rating also is consistent with the observed historic performance of these materials during the 1949 and 1965 earthquakes, as no instances of liquefaction were reported within these materials.

GLACIALLY CONSOLIDATED SEDIMENTS

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

SPT data obtained from these glacially consolidated sediments is summarized in table 4. Typically, all of the SPT data contained in table 4 were obtained using hollow-stem auger drilling techniques. The data in the table have been differentiated on the basis of material composition including undetermined material origin, till, and glaciolacustrine deposits. In essentially all cases, high blow counts were obtained within the glacially consolidated sediments, and the 10 percentile values are greater than the minimum SPT values required to resist liquefaction during the 0.30g earthquake. As such, it is concluded that these materials would have a very low liquefaction potential rating. Furthermore, 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 was based upon the data presented in figures 6 through 14 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 has been summarized on table 5 along with information on the liquefaction performance of the deposits and the relative liquefaction ranking that would be assigned to these materials using the liquefaction classificational 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 that are reported in table 5. 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 hazard rating that was computed for the Holocene Beach Deposits corresponding to the 0.30g earthquake. Additionally, the assigned relative hazard rankings are in agreement with the liquefaction performance of the site 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 which would have been assigned using the ranking scheme of Youd and Perkins (1978). As shown on table 5, 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 which Youd and Perkins (1978) have ranked as having a very high hazard. On the basis of these favorable internal and external comparisons, it is concluded that the Liquefaction Potential Map presented in plate 1 provides reasonable and realistic seismic rankings.

SUMMARY AND CONCLUSIONS

Seattle is located in a tectonic and geologic environment that is conducive to the development of liquefaction during relative strong earthquakes. Instances of liquefaction have been reported during the two largest historic 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 moderate damage occurred in Seattle as a result of these earthquakes, this damage level is consistent with the low level of acceleration recorded locally (approximately 0.10g) during these historic events. This historical damage, however, may not accurately represent the potential hazard in Seattle as the area may likely experience an earthquake with a ground acceleration of 0.30g (Algermissen, 1988b; ICBO, 1991).

The liquefaction hazard potential of the area may be even greater considering the potential occurrence of a subduction zone earthquake because the duration of such an earthquake may be several times greater than any historic event experienced in the Puget Sound area.

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 return period), and this acceleration is consistent with either bedrock (Algermissen, 1988b) or stiff soil deposits (BSSC, 1991). The 0.30g acceleration is also consistent with local practice for the seismic design of buildings (ICBO, 1991). This level of acceleration was coupled with the empirical liquefaction procedures of Seed (Seed and Idriss, 1971; Seed and others, 1983 and 1984) to determine the liquefaction resistance of the various geologic units in the area. The generalized liquefaction hazard rating for each geologic unit was evaluated using two criteria: a threshold performance criteria and a thickness criteria. The threshold criteria 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 where the mean (50 percentile) SPT N-value fell below the threshold value. A low liquefaction potential rating was given to materials where the lower quartile N-values fell below the 0.30g threshold level. Intermediate values were ranked as having a moderate liquefaction potential.

The second evaluation method was developed based upon the thickness or vertical extent of soils that would potentially liquefy in a 0.15g and a 0.30g earthquake. A high hazard rating was given to geologic units in which the thickness of the liquefied layer would be predicted to exceed 3 meters (10 feet) for a 0.30g earthquake and 0.3 meters (1-foot) for 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. Using both criteria, a singular liquefaction hazard map was developed for the Seattle area (plate 1).

On the basis of the above methodologies and criteria, it was concluded that fill soils and underlying alluvial deposits, particularly in the Duwamish tideflats area and the Interbay area, have a high potential for liquefaction during the 0.30g earthquake. Deposits having a moderate liquefaction potential during this earthquake included Holocene alluvium, beach deposits, and other sediments. The most significant Holocene alluvial deposits occur along the Duwamish River Valley. Pleistocene alluvial deposits, which have not been glacially overconsolidated, were given a low liquefaction potential rating. Pleistocene glacially consolidated sediments 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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TABLE 1. SITES OF HISTORICAL LIQUEFACTION

Location Number[1]	Site Address	Remarks	Reference
		<u>Earthquake of April 12, 1949</u>	
1	Seattle (near south end of Green Lake), NE 1/4 sec. 22, T. 25 N., R. 4 E., Seattle North 7 1/2 quad.	"A 60-foot section of the south shore of Green Lake dropped four feet into the lake during the earthquake Wednesday. Cracks three or four inches 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 into the lake."	Seattle Post-Intelligencer 4/14/49
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 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 long and 12 feet high reveals three vertical construction joints opened 1-5/8, 2, 1-3/4 inches, or a total of 6 inches during or since the quake. The joint filler in a north-south wall was squeezed out a maximum of 3 inches at the bottom of one joint. The wall nearest to the Duwamish waterway and parallel to it has settled 2 inches 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 newbuilding. This may be actual..."	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 to 18 cm]. The bridge was open at the time of theearthquake. 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 thequake. The main piers shifted, reducing thehorizontal distance between the piers severalinches and jamming the forward edges of ..."	U.S. Army Corps of Engineers, 1949

TABLE 1. SITES OF HISTORICAL LIQUEFACTION (con't)

Location Number[1]	Site Address	Remarks	Reference
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 posts 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 7-1/2 feet [2.3 m] above adjacent tide land. After the perceptible ground motion of the quake had ceased, cracks started developing in the 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 to 4.4 cm]. The ground at the rear of the building pulled away from the wall 2-1/2 inches [6 m] and settled vertically 3 inches and 4 inches [7.6 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 possi- bility 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
8	177 S.W. Massachusetts (Albers Brothers Elevators)	"Examination of ground around the Albers Brothers Elevators show no evidence of settle- ment except that a number of sand boils developed from 5 feet to 15 feet [1.5 to 4.6 m] away from the elevators on the northeast side. The ground around a large fuel tank has settled differentially from zero to 1/2 inch [1.3 cm] as evidenced by the soil contact mark."	U.S. Army Corps of Engineers, 1949
9	2600 26th Ave. S.W. (KJR Radio Tower)	"The KJR Radio Transmitting tower suffered structural damage but the ground apparently settled around the footings 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
10	2613 Marine Ave. S.W.	"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 From Hopper, 1981
11	Sears - 1st Ave. S. & S. Lander	"...geyers of water and mud which spurted from the ground reportedly as high as 3 feet, which flowed continuously for as long as 24 hours, and which filled basements...."	Edwards, 1950

TABLE 1. SITES OF HISTORICAL LIQUEFACTION (con't)

Location Number[1]	Site Address	Remarks	Reference
		Earthquake of April 29, 1965	
12	Seattle (Green Lake), NE 1/4 sec. 7 T. 25 N., R. 4 E., Seattle North 7 1/2 quad.	"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, possible beyond repair. A wall buckled." (Seattle Times, 4/30/65)./ 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, 4/30/65
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] zig-zag 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."	Von Hake & Cloud, 1967
		"North of Union Bay, broad fill over alluvial and lacustrine sediments subsided and exhibited scattered ground cracks and sand mounds."	Mullineaux, et al., 1967
14	2422 Elliott Ave.(Perfection Smokery)	"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."	MacPherson, 1965 From Hopper, 1981
15	Pier 66	"Break in underground supply mains."	MacPherson, 1965
16	Piers 64 and 65	"Fire protection...supply main break."	MacPherson, 1965
17	410 5th Ave. S. (Apex Automotive and Garage)	"Whole front footing for the building has settled from a few inches to eight to ten inches [20 to 25 cm]. The beams are generally cracked and sheared. Apparently this has been going on for many years."	MacPherson, 1965

TABLE 1. SITES OF HISTORICAL LIQUEFACTION (con't)

Location Number[1]	Site Address	Remarks	Reference
18	1730 1st Ave. S. (Queen City Sheet Metal Shop)	"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 earthbelow which forced the water in this lower stratum up into the area. This water was alsofollowed by the characteristic fine siltingwhich now covers most of the underneatharea....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
19	2228 First Ave. S. (Millwork Supply)	"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 to 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
20	Various Piers	Port of Seattle Engineering Department letter describes ground settlement along bulkhead lines at Piers 5, 20, 28, and 42; broken underground firemains at Piers 20, 42, and 66.	MacPherson, 1965
21	Pier 25	"Breaks in underground piping."	MacPherson, 1965
22	Duwamish River	"The low-lying filled areas along the Duwamish River and its mouth settled and were the locations of considerable building damages....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 SpokaneStreet 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."	Von Hake & Cloud, 1967
23	Harbor Island Piers 15 and 16 and Fisher Flouring Mills	"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."	Algermissen and others, 1965

TABLE 1. SITES OF HISTORICAL LIQUEFACTION (con't)

Location Number[1]	Site Address	Remarks	Reference
24	Pier 5	"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 for a width of 25 to 40 feet. The bulkhead was reported to be 6 to 8 inches out of line. Several Port piers suffered similar damage." (von Hake and Cloud, 1967). "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 at distance of 8 inches. The soil in this area for 20-foot width has subsided. The ground has displaced to approximately 1-1/2 feet below the level of existing dock. This sub- sidence decreases to approximately 8 inches at the southerly end of pier."	Von Hake and Cloud, 1967
25	Pier 6	"Similar problems to ***pier 5. There is subsidence of 6" to 12" at the land face of the pier."	MacPherson, 1965
26	2613 Marine Ave. S.W.	Damage to "basement floor and fresh water came in." (Mrs. J. Woodhouse, written commun., 1965).	University of Washington, unpub. data, 1965 From Hopper, 1981

[1]Locations plotted on Figure 2.

TABLE 2. GROUNDWATER DEPTHS

Region	Average Depth to Ground water (feet)	Maximum Depth (feet)	Minimum Depth (feet)	Standard Deviation (feet)
Duwamish River Valley (includes both upper and filled tideflats)	6.5	21.5	0.0	4.4
Alki Beach	10.5	25.0	6.0	3.7
Rainier Valley	8.7	26.0	0.0	8.3
Interbay (includes Lake Washington Ship Canal Qys)	6.4	13.0	0.0	4.3
West Point	15.0	30.0	9.0	6.9
Union Bay	4.0	6.5	2.0	1.3
Shilshole Bay	1.7	4.0	0.0	1.7

TABLE 3. SIGNIFICANT EARTHQUAKES IN THE SEATTLE REGION[1]

Source[2]	Year	Date Mo. Day	Time (PST) Hr:Min	Latitude[3] North (°)	Longitude[3] West (°)	Magnitude	Max Intensity (MM)	Depth (miles)	Epicentral Distance From Seattle (miles)
A,C	1880	8-22	13:25	48	122	–	VI	–	32 NNE
A,C	1880	12-12	20:40	47.5	122.5	–	VI	–	10 SW
A	1928	2-2	04:52	47.8	121.7	–	VI	–	33 NE
A	1931	12-31	07:25	47.5	123.0	–	VI	–	32 WSW
A	1932	8-6	14:16	47.7	122.3	–	VI	–	7 N
A	1939	11-12	23:46	47.4	122.6	5.75	VII	–	19 SW
B	1945	4-29	12:16	47.4	121.7	–	VII	–	33 ESE
A	1946	2-14	19:18	47.3	122.9	5.75	VII	–	34 SW
B,C,E	1949	4-13	11:56	47.1	122.7	7.1 (M_s)	VIII	44	39 SSW
A	1950	4-14	03:04	48.0	122.5	–	VI	–	29 NNW
A	1954	5-15	05:02	47.4	122.3	–	VI	–	14 S
B	1955	3-25	22:56	48.05	122.03	–	VI	–	34 NNE
B	1960	4-10	22:48	47.57	122.25	–	VI	–	5 SE
A	1963	1-24	13:43	47.4	122.1	–	VI	–	17 SE
B,E	1965	4-29	07:29	47.4	122.3	6.5 (m_b)	VI-VIII	37	14 S
B,C	1965	10-23	08:28	47.5	122.4	4.8	VI	–	8 SSW
B	1975	4-22	15:04	47.08	122.65	4.0 (m_b)	VI	29	40 SSW
B	1976	9-8	00:21	47.38	123.08	4.6 (m_b)	VI	30	38 WSW

Notes:

1. Earthquakes selected for this tabulation have maximum intensities of VI or greater and have occurred within about 60 km (37 miles) of Seattle. The intent of this table is to provide a general indication of seismicity in the region; it is not a complete list of all earthquakes.
2. The following sources were used in compiling the earthquake data:
   - A. Coffman and Von Hake (1973)
   - B. United States Earthquakes
   - C. U.S.G.S. (1975)
   - D. Stover, et al. (1978)
   - E. Weaver and Baker (1988)
3. The range of uncertainty for epicentral locations may be taken as about $\pm 0.5^{\circ}$ for earthquakes prior to 1960 and as about $\pm 0.2^{\circ}$ for those since 1960.
4. Basis of magnitudes were not provided in the above references for events prior to 1975.

TABLE 4. SUMMARY OF SPT DATA FOR GLACIALLY OVERCONSOLIDATED DEPOSITS

Depth Range (ft.)	Undetermined			Till			Lacustrine		
	n	N	10	n	N	10	n	N	10
0-5	1	40	–	–	–	–	–	–	–
5-10	2	100	–	3	103	–	–	–	–
10-15	8	100	34	5	120	82	1	68	–
15-20	15	91	50	6	170	150	2	104	–
20-25	20	71	32	6	115	33	5	109	38
25-30	16	101	38	13	118	50	13	113	36
30-35	27	92	28	7	98	42	7	91	34
35-40	26	107	39	8	133	100	5	57	31
40-45	31	135	46	3	120	39	6	102	31
45-50	32	124	39	2	125	–	4	60	32

LEGEND
n-Number of samples.
N-Average standard penetration resistance (blows/ft.).
10-10 percentile of SPT N-values.

TABLE 5. LIQUEFACTION RATING COMPARISON

Unit	Threshold Criteria (0.30g)	Thickness Criteria (0.15g)	Thickness Criteria (0.30g)	Assigned Rating	Historical Liquefaction	Youd and Perkins (1978) Relative Ranking
FILLS						
Duwamish	High	High	High	High	Numerous	Very high
Interbay	High	High	High	High	None	
Other	N/A	N/A	N/A	High	Occasional	
HOLOCENE ALLUVIUM						
Mouth of Duwamish	High	High	High	High	Numerous	High
Flood Plain	Moderate	Moderate	Moderate	Moderate	Occasional	Moderate
HOLOCENE BEACH	Moderate	Moderate	Low	Moderate	Occasional	Moderate
PLEISTOCENE ALLUVIUM	Low	N/A	N/A	Low	None	Low
PLEISTOCENE GLACIAL	Very low	N/A	N/A	Very low	None	Very Low

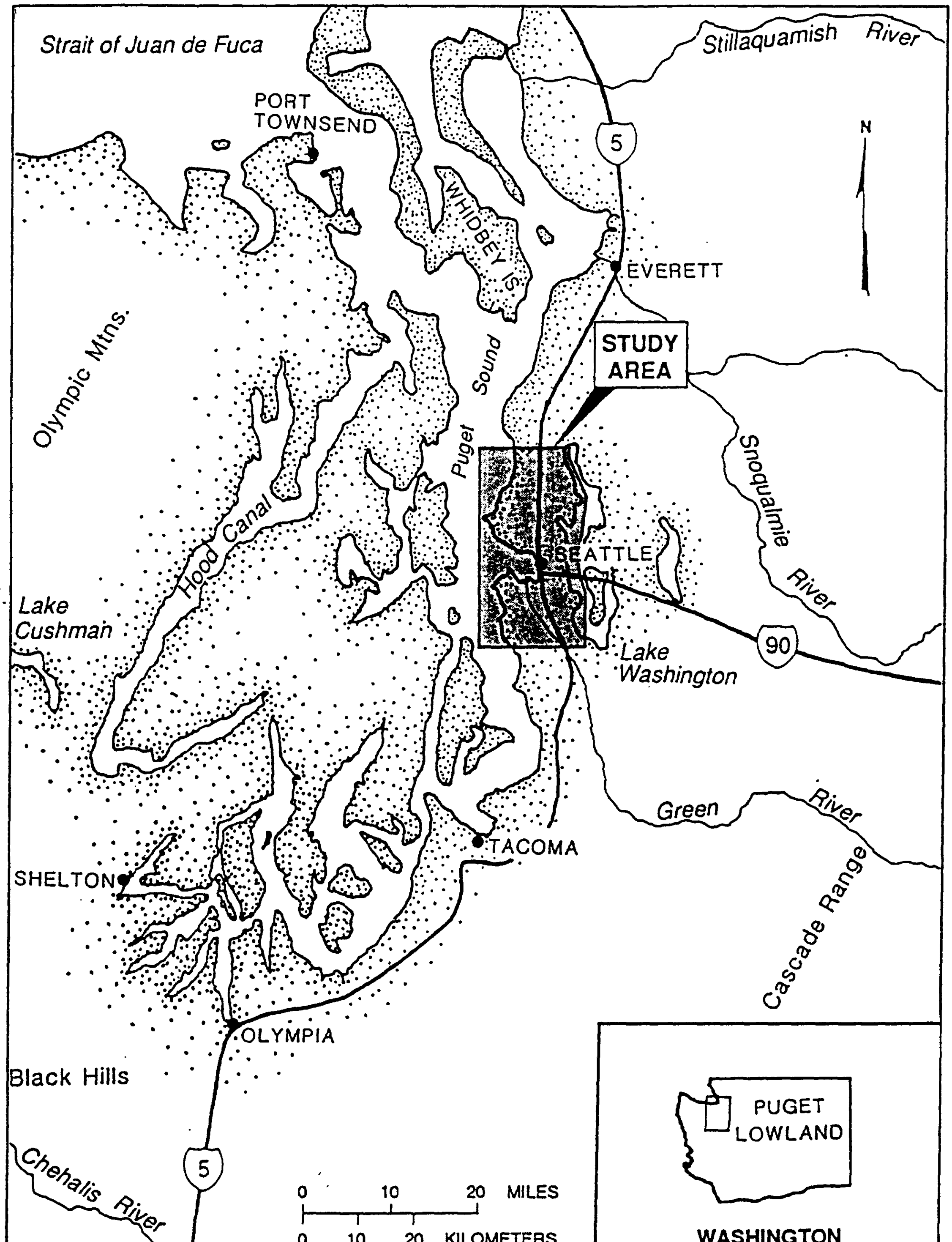


FIGURE 1. —Location of study area.

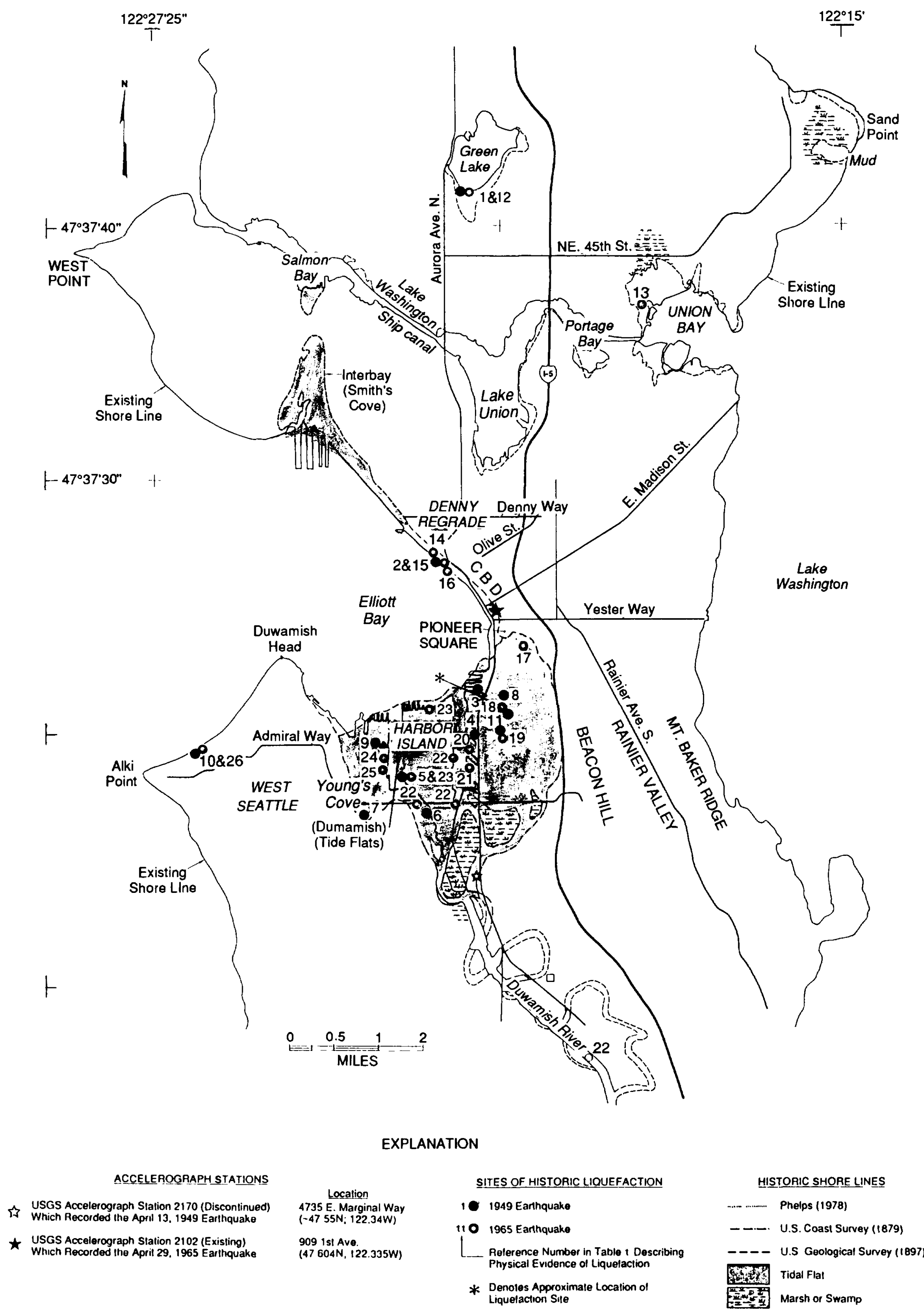


Figure 2. Sites of historical liquefaction.

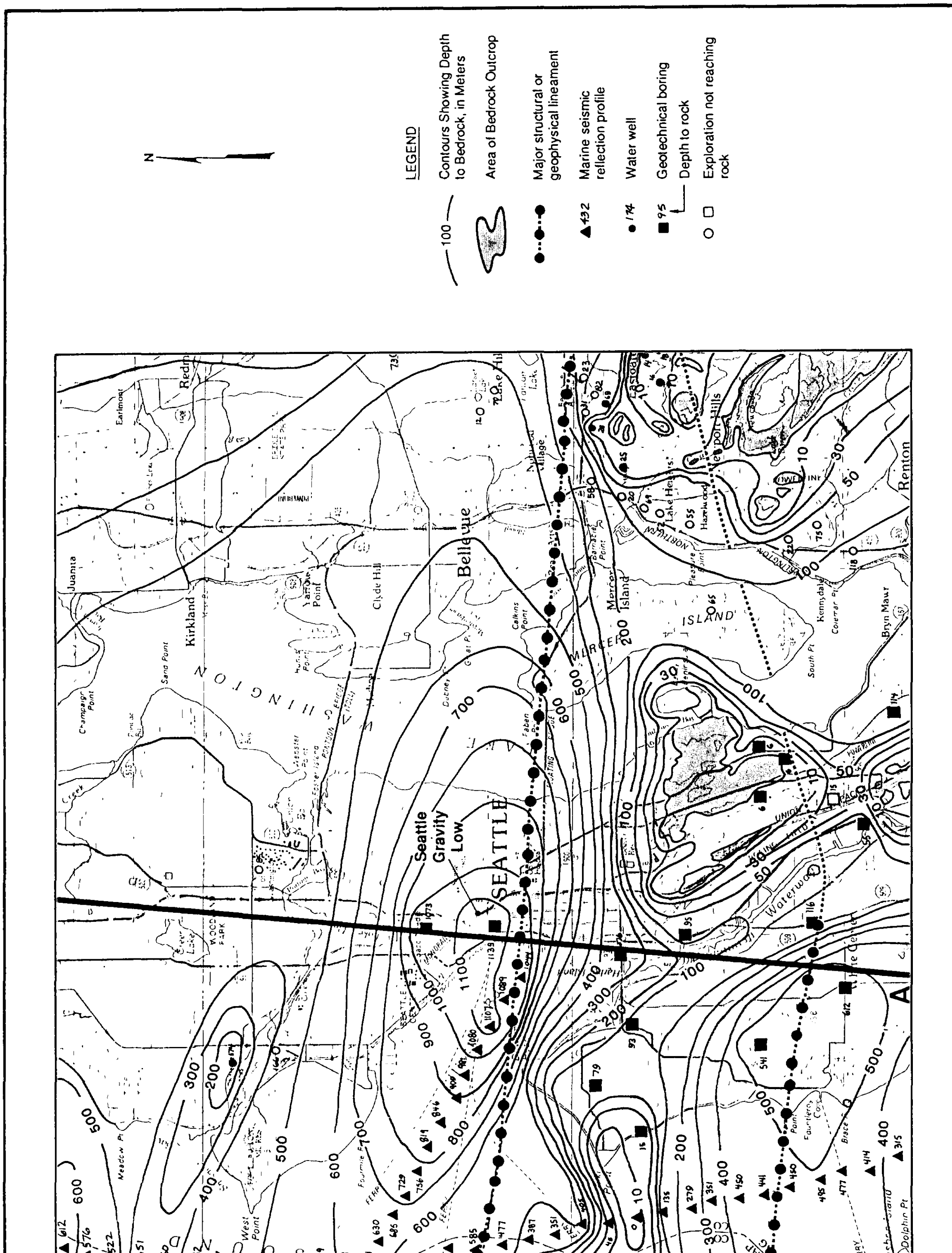


Figure 3. Depth to bedrock map (from Yount et al. 1985)

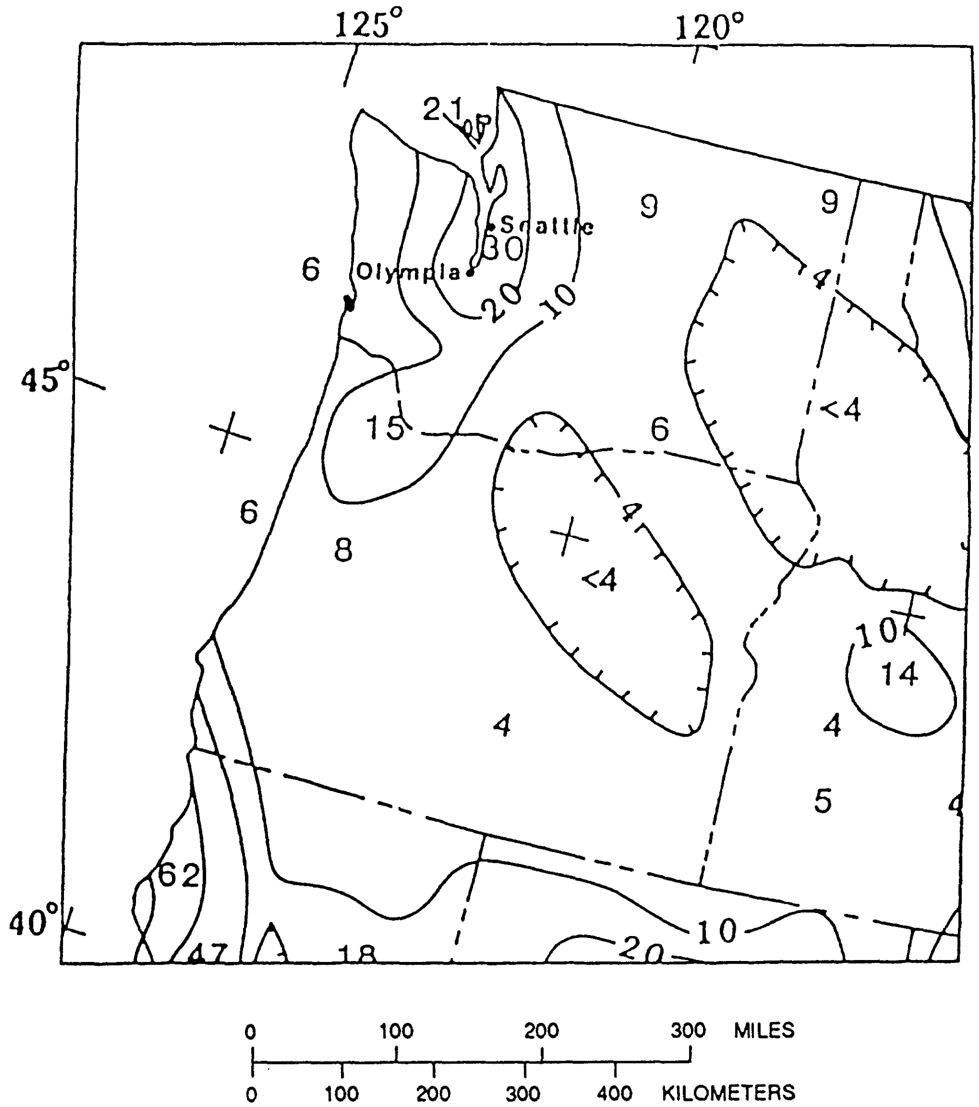


FIGURE 4. —Regional peak ground acceleration map (from Algermissen, 1988a). Map presents peak acceleration in %g on rock for 10% chance of exceedance in 50 years (475 year return interval).

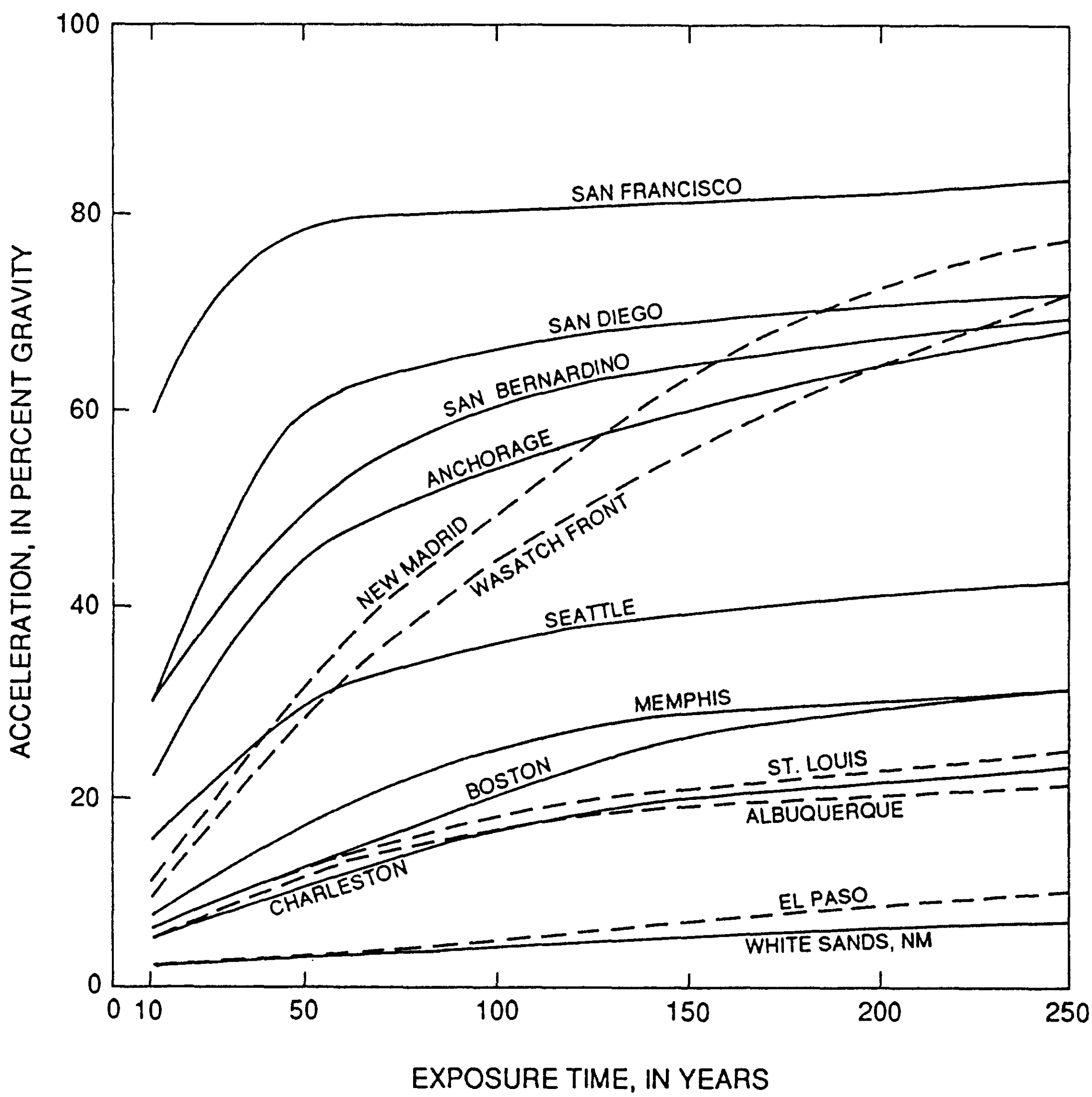


FIGURE 5. —Probabilistic Seismic Exposure (from Algermissen, 1988b).

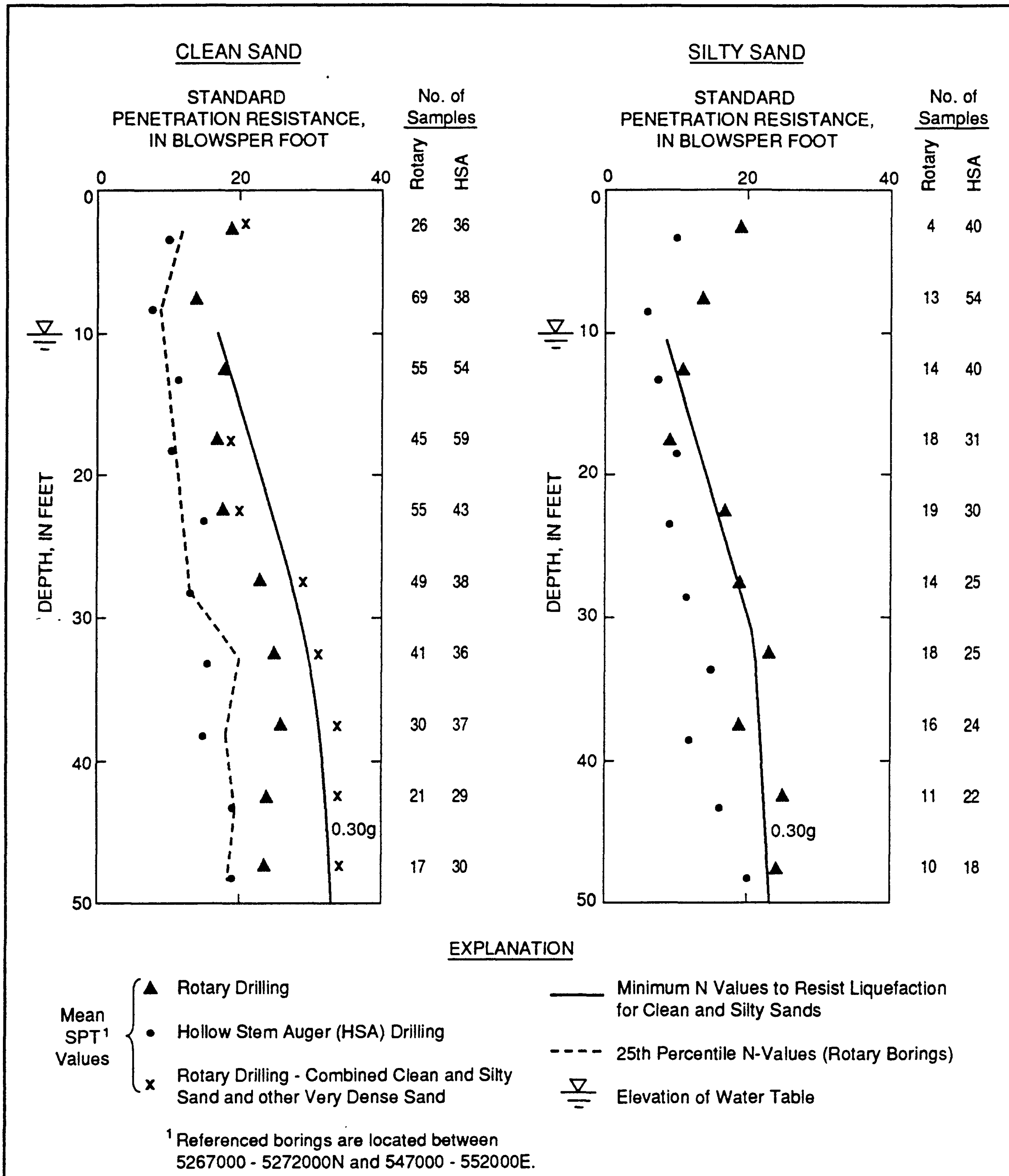
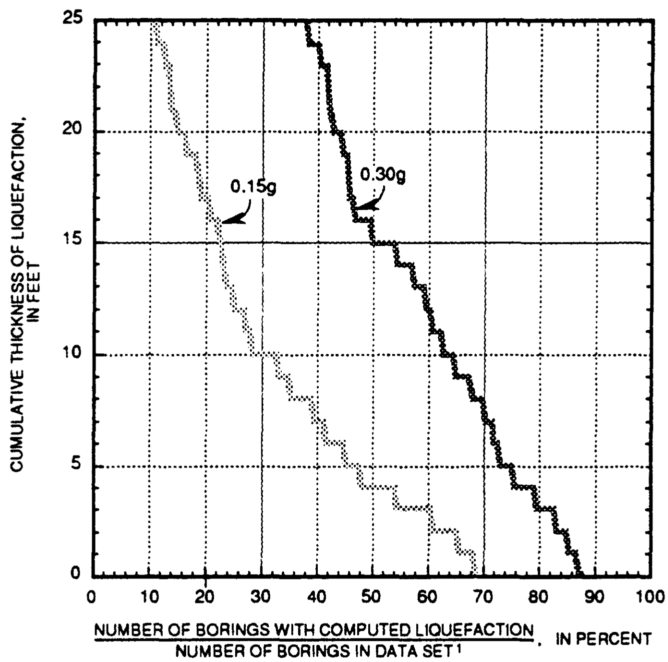


FIGURE 6. —Liquefaction evaluation, Duwamish tideflats fill, threshold criteria.



[1] 172 borings in Duwamish tideflats fill data set

FIGURE 7. — Liquefaction Evaluation, Duwamish tideflats fill, thickness criteria. Each curve in the figure represents the cumulative thickness of liquefaction for the indicated ground acceleration.

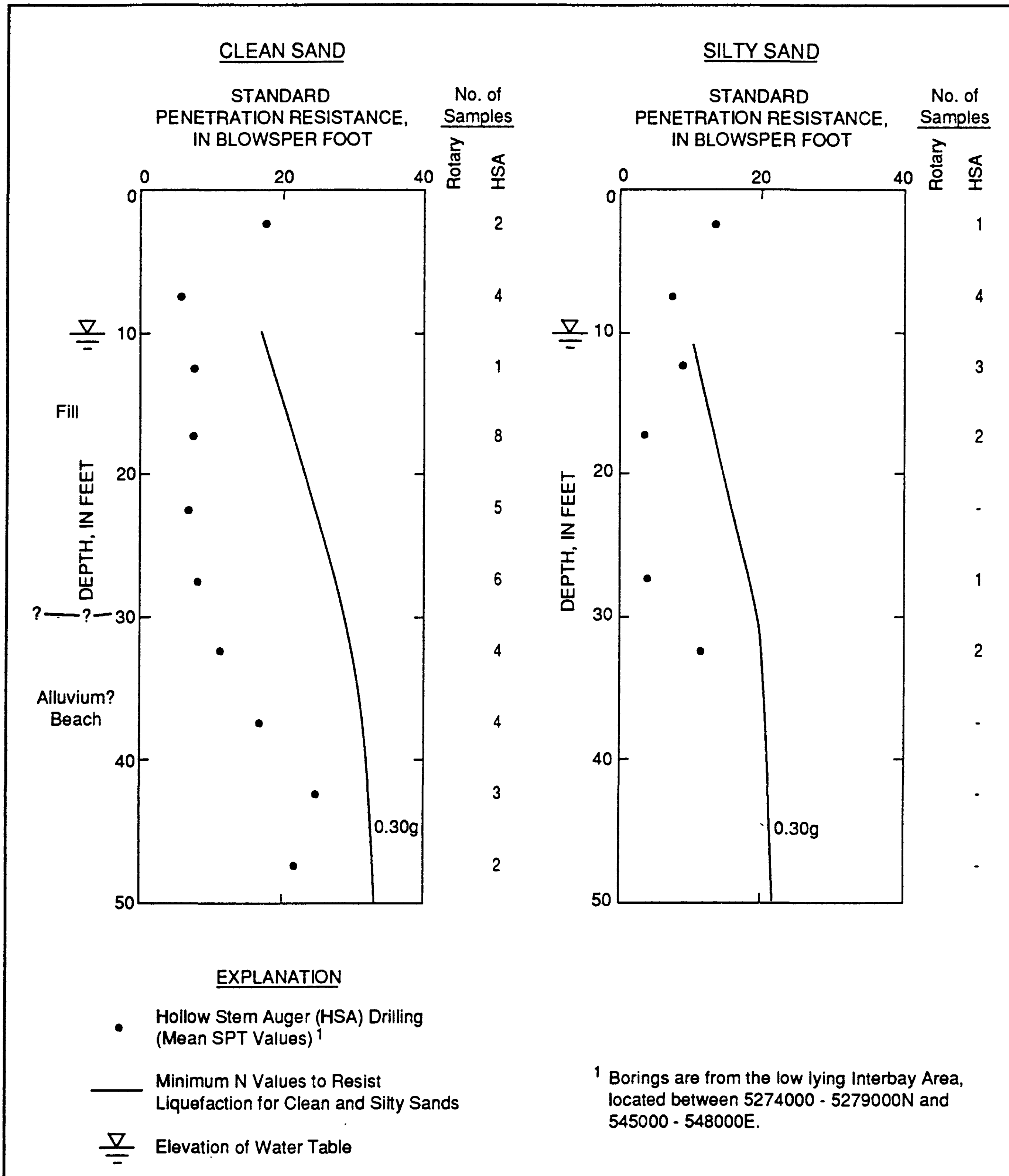
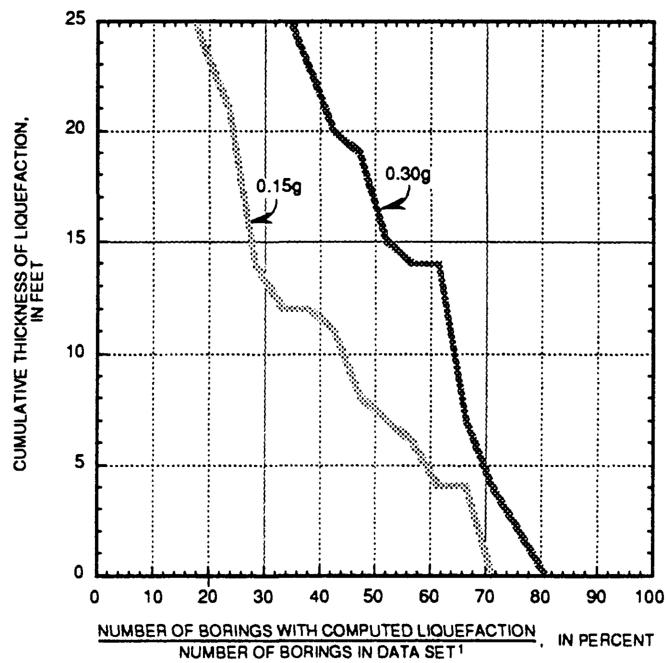


FIGURE 8. —Liquefaction evaluation, Interbay fill, threshold criteria.



[1] 21 borings in Interbay fill data set

FIGURE 9. —Liquefaction Evaluation, Interbay fill, thickness criteria. Each curve in the figure represents the cumulative thickness of liquefaction for the indicated ground acceleration.

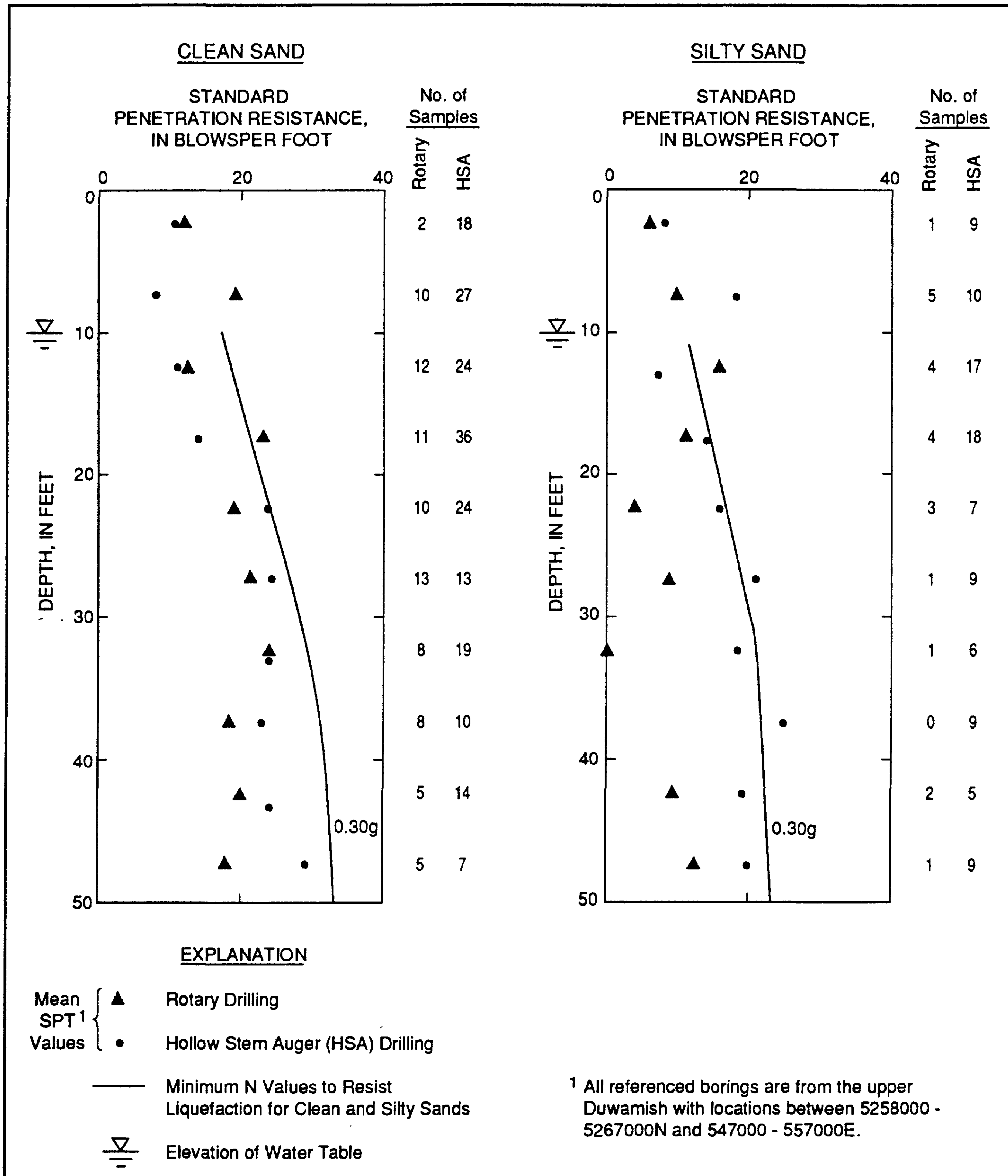
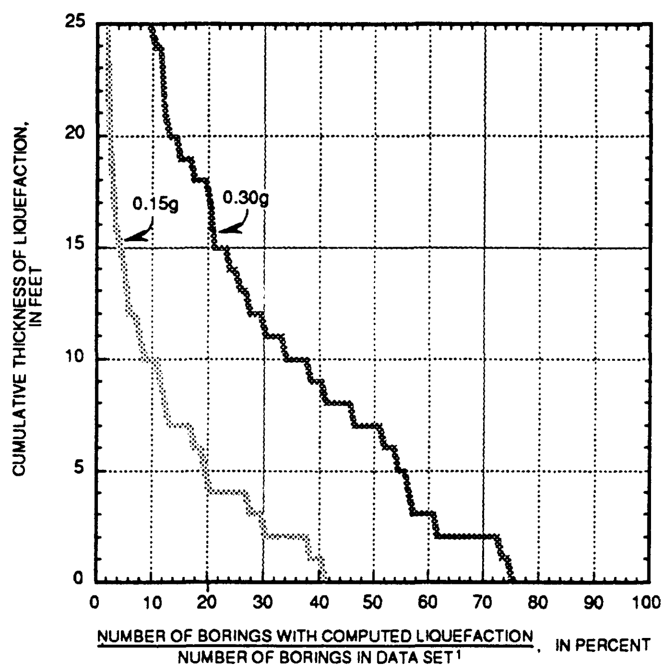


FIGURE 10. —Liquefaction evaluation, Holocene alluvium, threshold criteria.



[1] 111 borings in upper Duwamish area data set

FIGURE 11. —Liquefaction Evaluation, Holocene alluvium, thickness criteria. Each curve in the figure represents the cumulative thickness of liquefaction for the indicated ground acceleration.

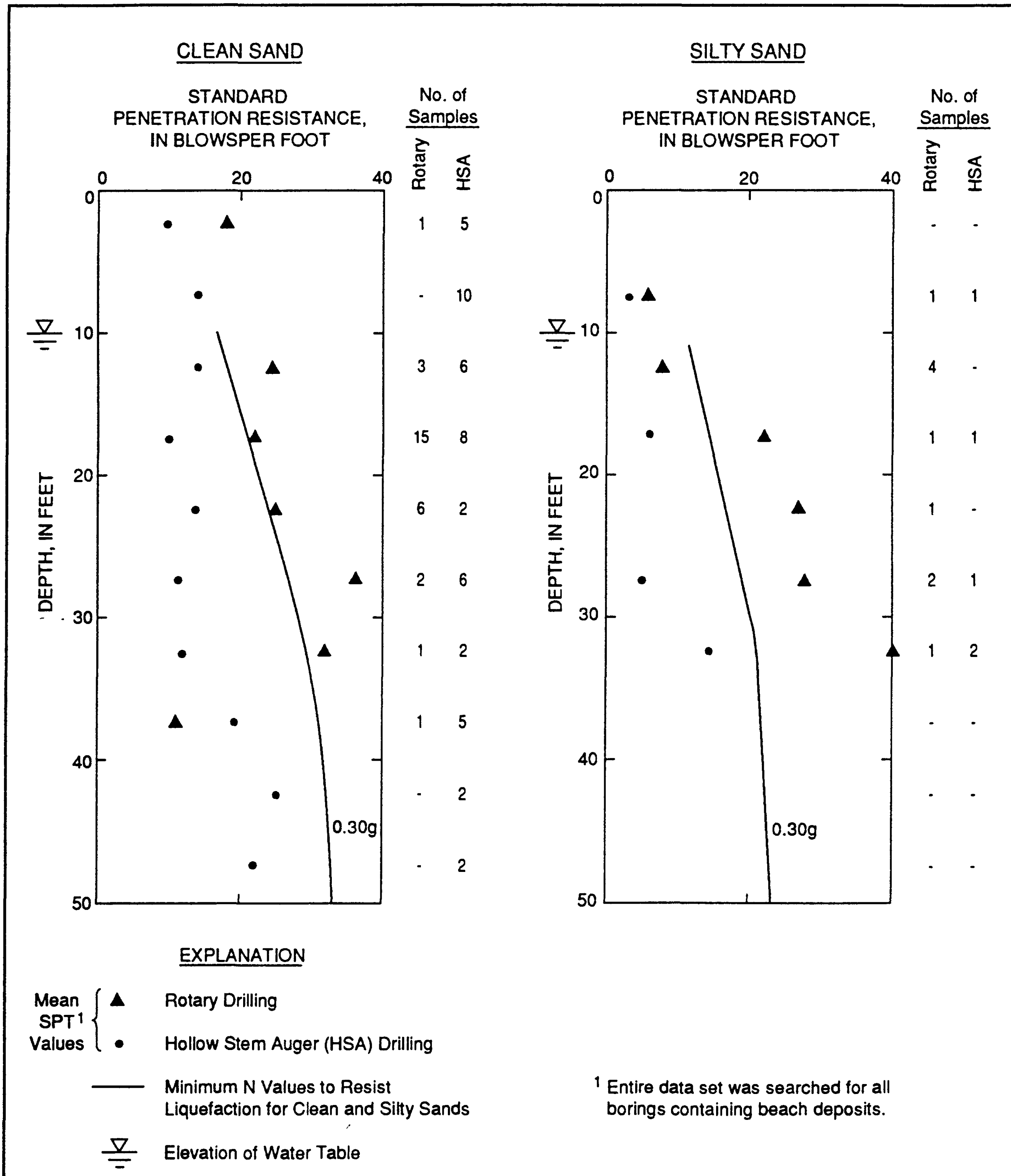
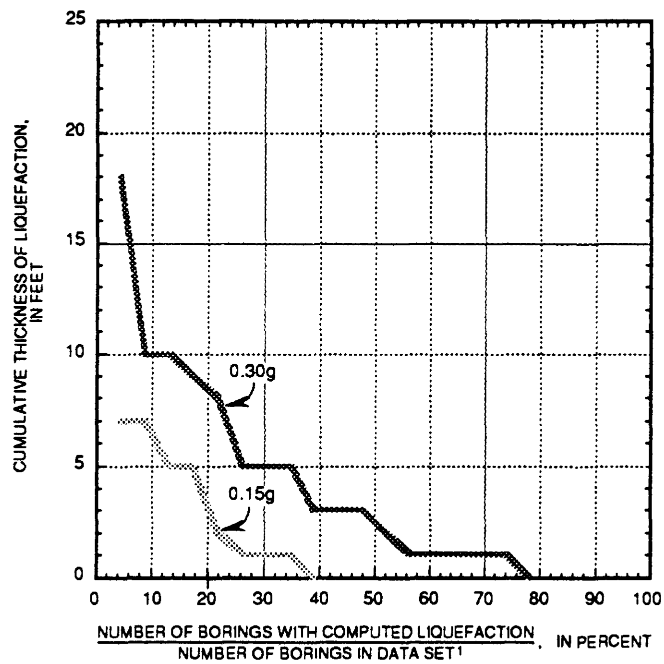


FIGURE 12. —Liquefaction evaluation, Holocene beach deposits, threshold criteria.



[1] 23 borings in Alki Beach area data set

FIGURE 13. —Liquefaction Evaluation, Holocene beach deposits, thickness criteria. Each curve in the figure represents the cumulative thickness of liquefaction for the indicated ground acceleration.

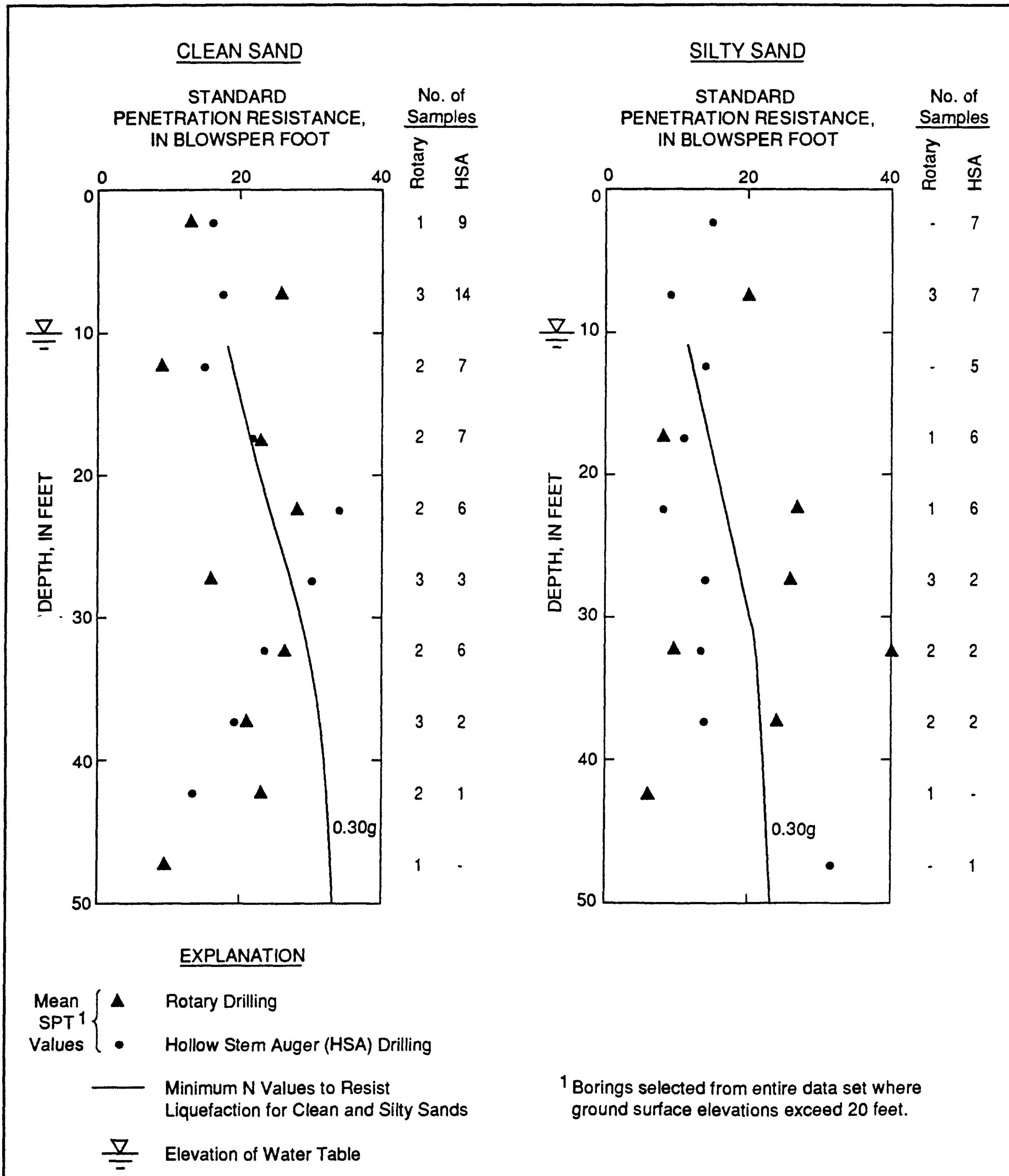


FIGURE 14. —Liquefaction evaluation, Pleistocene alluvium, threshold criteria.