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GEOTECHNICAL & SEISMIC HAZARD STUDY
COLLEGE OF URBAN & PUBLIC AFFAIRS
PORTLAND STATE UNIVERSITY

February 12, 1997

Prepared for

PORTLAND STATE UNIVERSITY
Director of Facilities
Portland, Oregon

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GEOTECHNICAL & SEISMIC HAZARD STUDY
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1. INTRODUCTION

1.1. Purpose

In accordance with our proposal of November 18, 1996, and our Retainer Agreement Supplement No. 90-96-49-01, dated November 26, 1996, we have completed a geotechnical investigation and site specific seismic hazards study for the referenced project. The purposes of the geotechnical investigation are to evaluate subsurface conditions at selected locations on the site and to assist with the design as it relates to earthwork, foundations and pavements. The purpose of the seismic hazards investigation is to evaluate the vulnerability of the project site to seismic geologic hazards so that measures to mitigate these hazards can be taken. This report presents the results of the geotechnical investigation and site specific seismic hazards investigation.

This report was prepared for your use in the design of the subject facility and should be made available to potential contractors and/or the Contractor for information on factual data only, i.e., field boring logs and samples. This report should not be used for contractual purposes as a warranty of interpreted subsurface conditions such as those indicated by the formal boring logs, and/or discussion of subsurface conditions contained herein.

1.2. Site Description

The site is located on the Portland State University (PSU) Campus on the block bounded by SW Mill and Montgomery Streets and between SW 5th and 6th Avenues as shown on the Vicinity Map, Fig. 1. The block currently contains an L-shaped building that extends along the entire length of SW 5th Avenue and most of SW Mill Street, and a small commercial building at the southwest corner. Existing features are shown in Fig. 2.

The ground slopes down from west to east and from south to north, with the high at the southwest corner at about elevation 140 feet, and the low at the northeast corner at about elevation 128 feet. The ground floor of the existing school facility is at about elevation 131 feet. A narrow asphalt-surfaced parking area separates the existing building from SW 6th Avenue. A level asphalt parking area (elevation 129 feet) about 90 by 105 feet in plan dimension occupies the interior of the "L" and fronts SW Montgomery Street. A 10-foot high retaining wall separates the level parking area from the commercial property.

1.3. Project Understanding

We understand the proposed new building will house the College of Urban and Public Affairs. It will be an irregular, but roughly U-shaped structure. The approximate foot print has been superimposed on the Site Plan, Fig. 2. The main structure, nestled in the northeast corner of the block will be eight stories high with a full basement with the floor at elevation 117 feet. The smaller segment, located along Sixth Avenue near the southwest corner, will be three stories high with no basement. Here ground floor elevation will be 131 feet. Loads will be transmitted to foundation level through a series of columns and bearing walls. In the tower section, loading, we understand, will be in the order of 500 kips and 20 kips/foot. In the three-story section, loads will be in the order of 200 kips and 8 kips/foot.

2. FIELD EXPLORATIONS AND LABORATORY TESTING

2.1. Field Explorations

The field exploratory program consisted of four borings made at the locations shown on the Site Plan, Fig. 2. The borings, designated B-1, B-1A, B-2 and B-3, were drilled on January 13 and 14, 1997, under subcontract with Geo-Tech Explorations with a truck-mounted drill rig using mud rotary drilling techniques. A Fujitani Hilts & Associates, Inc., technician was present throughout the explorations to collect samples and log the exploratory holes. The borings were each drilled to depths of 51.5 feet below the ground surface except B-1 which was terminated at depth 10.5 feet after penetrating into an abandoned buried fuel tank at depth 3 feet. The hole was then moved to the B-1A location (Fig. 2), the only nearby area not blocked by parked cars. Locations of the holes shown on the Site Plan, Fig. 1, are approximate and were determined using a cloth tape from nearby reference points. Elevations were determined by interpolation from the Site Plan topography provided by the Project Architect.

Samples were obtained in the borings at 2.5- to 5-foot depth intervals using a standard 2-inch O.D. split-spoon sampler. Standard Penetration testing was performed in accordance with ASTM D 1586 in conjunction with the split-spoon sampling to measure in-situ relative density and consistency. A few relatively undisturbed 3-inch O.D. thin-wall Shelby tube samples were also obtained at selected depths in lieu of the penetration test samples. All samples were sealed to retain moisture and returned to our laboratory for additional examination and testing.

When completed, each hole was backfilled with bentonite chips and the surface was sealed with 6 inches of concrete in accordance with City and State standards.

The boring logs are presented in Figs. 3, 4, 5 and 6. Soil descriptions and interfaces on the logs are interpretive, and actual changes may be gradual. The left-hand portion of the boring logs gives ground-water information. Our interpretation of the soils encountered during the field exploration program is given in the central portion of the logs. The right-hand, graphic portion, shows sample locations, the resulting Standard Penetration N-values.

2.2. Laboratory Testing

All samples were returned to our laboratory and visually examined in our laboratory to refine the field classifications. Samples were visually classified in accordance with ASTM D 2488 (Unified Soil Classification System). Water contents were determined for all applicable samples, and in-place densities of the soil retained in the Shelby tube samples were measured. The results of the water content determinations are shown on the boring logs, Figs. 3 through 6. The in-place density measurements from the Shelby tube samples are also shown on the logs and in the table below. Sophisticated tests to measure strength and compressibility were considered unnecessary, in our opinion.

Table 1. Density measurements from Shelby tube samples.

Sample ID		Depth (ft)		Wet Density (lbs per ft^3)	Dry Density (lbs per ft^3)
Boring No.	Sample No.	From	To		
B-1	S-5	12.5	14.5	95.4	78.3
B-1	S-8	20.0	22.0	102.1	85.8
B-2	S-1	2.5	4.5	115.2	88.2
B-2	S-4	10.0	12.0	101.5	83.2
B-3	S-2	5.0	7.0	100.0	77.4
B-3	S-7	17.0	19.0	112.4	89.2

3. SUBSURFACE INTERPRETATION

3.1. Geology

During the late Pleistocene, repeated catastrophic flood waters back-flooded from the Columbia River. These floods deposited great gravel bars across east Portland, sand and silt in tributary valleys, and silt with sand in the Tualatin basin and the Willamette Valley. In the vicinity of downtown Portland and the Portland State University campus, these young sediments consist predominantly of sand and silt. Based on mapping by Madin (1990)[1], the flood deposits are approximately 90 feet thick beneath the project site and overlie the Troutdale Formation.

3.2. Soils

In boring B-1, located near the southwest corner of the existing PSU building, an underground storage tank was encountered beneath 3.0 feet of fill. The drill rod penetrated through the tank and stopped on the bottom at depth 10.5 feet. The tank appears to contain about 16 inches of oil and water in the bottom. That bore hole was plugged and the drill rig was moved 12 feet east and the hole (B-1A) re-drilled to the full depth of 51.5 feet. Fill material was also encountered in the first 15 feet of the Boring B-1A. The fill consists of mottled brown silty sand and sandy silt. It appears to be loose to medium dense based upon standard penetration test N-values in the

[1] See list of references in the Appendix

order of 8 to 14 blows per foot. This was probably native soil, excavated and reused as backfill around the tank and/or behind the retaining wall located 8 feet east of the boring.

The borings revealed that the site (and fill) are underlain by brown, stratified silt, fine sandy silt, and silty fine sand to the full depth of the borings at 51.5 feet. The soils were generally loose to medium dense in the first 8 to 15 feet where N-values were in the order of 8 to 15 blows per foot. The fine sands and silts become medium dense between 28 and 40 feet (N = 15 to 30 blows per foot), and dense below about 40 feet (N > 30 blows per foot). Individual strata from 1 inch to 10-inches in thickness were exposed in several samples, and have been noted on the boring logs. These thin layers were typically upward fining, i.e. they graded upward from fine sand at the base of the layer to silt at the top of the layer. Some samples, particularly S-11 and S-12 in Boring B-2, did not exposed the contacts between inter-stratified layers, suggesting that individual strata may be up to 8 or 10 feet in thickness.

3.3. Groundwater

Groundwater was not observed in the borings. Groundwater probably fluctuates with the time of year, being highest in late winter or early spring and lowest in late summer or early fall. Data from nearby borings indicates that static water levels are on the order of 70 feet below the ground surface at this site.

4. GEOTECHNICAL DESIGN RECOMMENDATIONS

4.1. Foundation Options

It is our opinion that conventional spread footing support is acceptable. Two conditions are recognized: 1) basement footings that will bear in the medium dense phase of the fine sands and silts, and 2) ground floor footings that will bear within the less dense phases of these soils. Bearing for ground floor footings can be improved by removing the looser soils and replacing them with compacted structural fill. This option is discussed later in this report.

4.1.1. Allowable Bearing Capacity

Allowable bearing capacity curves have been developed for continuous and square footings at each of the two levels based upon the following assumptions:

Soil/Design Parameter	Basement Footings	Ground Flr. Footings
Max. Footing Elevation (ft)	115	129
Saturated unit weight, γ (pcf)	110	110
Average N-value (blows/foot)	18	10
Cohesion, c (pcf)	0	0
Bearing Capacity Factor, N_q/ N_γ	20.3/17.0	10.4/6.3
Factor of Safety	3.0	3.0
Limiting Total Settlement (ins)	1.0	1.0
Differential Settlement (ins)	0.5	0.5

For a given set of soil parameters, maximum settlement, and factor of safety, allowable bearing varies with footing size and depth (measured below the lowest adjacent grade - usually the floor slab). Allowable bearing and footing loads for continuous and for square basement footings are given in Figs. 7 and 8, respectively. Similar data for ground floor footings is given in Figs. 9 and 10. For either building area, we recommend a minimum footing depth of 1.5 feet, and minimum footing widths of 1.5 feet and 2.0 feet for continuous and individual footings respectively.

Allowable bearing for ground floor footings can be increased by over-excavating and replacing the loose fill or native soils with structural fill. (Structural fill specifications are given later in this report.) This concept is shown in Fig. 11. Allowable bearing values derived from the upper curves on Figs. 9 and 10 can be increased 50% provided that, 1) the excavation ans structural fill geometry given in Fig. 11 is satisfied, 2) the structural fill thickness beneath continuous footings equals at least ½ of the footing width, 3) the fill thickness beneath square footings equals at least ¼ of the footing width, and, 4) for either case, allowable bearing does not exceed 4,000 psf.

4.1.2. Footing Preparation

Each footing excavation should be evaluated by a qualified Geotechnical Engineer to confirm suitable bearing conditions and to determine that all loose materials, organics, unsuitable fill and softened subgrade, if present, have been removed. If miscellaneous fills or unsuitable soils are encountered at footing locations, we recommend that the fill or unsuitable soil be removed. To keep footings at a conventional grade, the excavated fill beneath footings may be replaced with compacted structural fill to grade as shown by Fig. 11. To minimize the potential for disturbance during excavation for the footings, we recommend that excavations be made with a smooth bucket (no teeth) backhoe, that the final 3 to 4 inches be excavated by hand, or that the exposed subgrade be compacted to a dry density of at least 95 percent of the Modified Proctor maximum dry density (ASTM D 1557) within the upper 8 inches. If excavation for the footings is accomplished during wet weather, we recommend over-excavating the bottom by about 3 to 4 inches and backfilling with a granular material to provide a stable base for footing construction.

4.2. Retaining Walls

4.2.1. Allowable Bearing Capacity

Footings supporting basement and independent retaining walls may be designed in accordance with Section 4.1. The average pressure across the footing should not exceed the allowable bearing values determined from the appropriate continuous footing charts. The resultant earth pressure force should pass through the middle 1/3 of footing base.

4.2.2. Lateral Earth Pressures

We recommend the parameters in the following table for lateral earth pressure determinations for backfill consisting of on-site soil (sandy silt) and imported granular backfill, a level surface both in front of and in back of the walls:

Soil Parameter	Native Soil	Imported Granular
Friction Angle, F, degrees	30	38
Moist Unit Weight, γ, pcf	110	110
Active Earth Pressure Coefficient, K_a	0.33	0.24
At-Rest Earth Pressure, K_o Coefficient	0.50	0.44
Passive Earth Pressure Coefficient, K_p	3.00	4.20
Coefficient of Base Friction, f	0.35	0.45

The unit lateral earth pressure (equivalent fluid pressure) can be determined from:

$$\textbf{Unit Lateral Pressure} = \mathbf{K \times \gamma}$$

The use of the active or at-rest pressure will depend upon the degree of wall restraint when the backfill is placed. Generally, if the wall is restrained so that the deflection of the top of the wall will be less than 0.001 times the free-standing height of the wall, the at-rest pressure should be used. Surcharge effects can be included by adding additional height of soil behind the wall by an amount equal to the surcharge (psf) divided by the soil unit weight in the above table.

4.2.3. Stability

The stability of independent retaining walls should be sufficient to provide for a factor of safety against sliding and overturning of 1.5 or more. For basement walls supported at the top global stability is not an issue, in our opinion. Resistance to lateral forces for independent cantilevered retaining walls should be developed entirely by base friction, in our opinion, unless there is no risk of future excavation (utility trenches, for example) in front of the wall that will reduce or remove passive resistance.

4.2.4. Drainage

Drainage we believe is necessary to protect against saturation of the backfill due to leakage from broken water or sewer lines, or misdirected drainage. Recommendations for backfill and drainage behind basement and small cantilever retaining walls are shown in Fig. 12. Perimeter drain lines should be adequately sloped to allow the water to drain under gravity. Failure to adequately dispose of the water behind walls could lead to significantly higher lateral pressures than anticipated.

4.3. Floor Slabs

All floor slabs-on-grade, should be founded on a minimum 6-inch layer of free-draining well-crushed aggregate with a maximum particle size of 1-1/2 inches and containing not more than 2 percent passing the No. 200 sieve (based on a wet sieve analysis) compacted to a dry density of at least 95 percent of the modified Proctor maximum dry density (ASTM D 1557). A vapor barrier beneath the slab is unnecessary, in our opinion.

4.4. Earthwork

4.4.1. Site Preparation

The subgrade preparation should include the stripping and removal of all surficial organic soil (sod, topsoil, etc.), trees/roots, pavements, unsuitable fill, if any, and demolition debris from the building areas as determined by a qualified representative of the Owner. After stripping, the site should be graded reasonably level, and the building areas should be proof-rolled with a half loaded dump truck or similar vehicle in the presence of a qualified representative of the Client. Proof rolling should be deferred until just prior to construction in specific areas. Any soft or disturbed areas that are detected by the proof-rolling should be removed and backfilled with structural fill. The actual amount of soft or disturbed material to be excavated will probably need to be determined in the field, and we recommend that the specifications include a unit cost bid item for any over excavation.

The subgrade soils on the site are generally fine grained and, in our opinion, are moisture sensitive. Hence, the earthwork should be scheduled for the dry summer months if at all possible when the risk of shut downs or special provisions is less. Otherwise, during wet conditions special techniques including the possible use of a geotextile will be required during construction to minimize disturbance to the subgrade by construction equipment.

4.4.2. Excavation

Site excavation can be accomplished with conventional excavation equipment, in our opinion. The stability of excavation slopes should be the responsibility of the Contractor in our opinion, as this is related to job safety. However, for planning purposes, assume temporary excavation slopes up to about 10 feet high in the existing fine sands and silts will be no steeper than 1 Vertical on 1.25 Horizontal. All unsupported and shored excavations and trenches should be accomplished in accordance with OR-OSHA revised excavation rules adopted April 9, 1990 and effective September 1, 1990. Unsupported excavation and shoring should be based upon soil "Type C" as defined in Appendix A to Subdivision P of OAR 437.

4.4.3. Materials

On-site soils free of debris and deleterious materials are acceptable as structural fill beneath non-footing areas, in our opinion, subject to the limitations discussed below. Imported granular structural fill should be used beneath footings. It should consist of a reasonably well graded crushed aggregate containing less than 15 percent fines under dry weather conditions. If wet weather or wet subgrade conditions are anticipated or encountered, fines should be limited to less than 5 percent. Fines are defined as that portion, based on weight, passing the 200 mesh sieve using the wet sieve method. The fines should be non-plastic. Maximum particle size will depend upon placement restrictions, but generally should not exceed 3 inches. Under wet conditions, on-site soils are not acceptable for structural fill, in any areas, in our opinion, because workability and quality control will be too difficult.

4.4.4. Compaction

Structural fill should be placed in thin lifts and compacted to a dry density of at least 95 percent of the Modified Proctor maximum dry density (ASTM D 1557) within building areas, beneath footing and within a 2-foot depth of any pavement or sidewalk section. All fills outside of these limits could be compacted to 90 percent of the maximum dry density. The thickness of the lifts will need to be determined in the field, but generally for self propelled compactors, the lifts should not exceed about 9 to 12 inches as measured in a loose condition. For small hand compactors, the lifts may need to be reduced to about 4 inches loose measure to achieve the required compacted density.

4.4.5. Quality Control

Site preparation and the placement and compaction of all structural fill should be monitored by a qualified representative of the Owner who is experienced in earthwork, and can evaluate construction methods and end results.

5. SITE-SPECIFIC SEISMIC HAZARDS EVALUATION

5.1. General

Amendment OAR 918-460-015 to the Oregon Structural Specialty Code requires that essential facilities, hazardous facilities, major structures and special occupancy structures be evaluated on a site specific basis for vulnerability to seismic geologic hazards and provides minimum requirements for the investigation and report.

The purpose of the seismic site hazard investigation is to evaluate, on a site specific basis, the vulnerability of the site to seismically induced geologic hazards. The evaluation should include, but not be limited to, the source of earthquake, earthquake induced ground shaking, landslide, liquefaction, tsunami, seiche, fault displacement, and subsidence. The scope of work for the seismic site hazard investigation is generally outlined by the amendment to OAR 918-460-015. The scope of work includes a literature review of the regional seismic or earthquake history to assist in the determination of potential seismic sources, the maximum credible earthquake, recurrence intervals, etc., investigation of geologic conditions at the site and an evaluation of the ground response to design earthquakes identified by the investigation.

5.2. Geology

5.2.1. Geologic Setting

During the geologic period known as the middle Miocene (about 17 to 6 million years ago), the greater Portland area was inundated by great floods of basalt lava that were erupted from long linear fissure systems in northeastern Oregon, eastern Washington, and western Idaho. Now known as the Columbia River Basalt Group, these very fluid basalts filled in low lying areas leaving peaks and ridges of older formations rising above the lava plain. Almost as soon as the lava flows were emplaced, complex folding and faulting of the new rocks began, eventually pushing up the Tualatin Mountains.

As the mountains were uplifted, down warps developed to both the east and the west forming the Portland and Tualatin basins, respectively. Later, during the Pliocene epoch (about 5.3 to 1.6

million years ago) the basins became filled with sedimentary deposits. Silt and clay filled the basins first, the Helvetia Formation (Schlicker and Deacon, 1967) in the Tualatin basin, and the Sandy River Mudstone (Trimble, 1963) in the Portland basin. These fine sediments were in turn overlain by sand and gravel of the Troutdale Formation (Trimble, 1963).

During the late Pleistocene, repeated catastrophic flood waters back-flooded from the Columbia River. These floods deposited great gravel bars across east Portland, sand and silt in tributary valleys, and silt with sand in the Tualatin basin and the Willamette Valley. In the vicinity of downtown Portland and the Portland State University campus, these young sediments consist predominantly of sand and silt. Based on mapping by Madin (1990), the flood deposits are approximately 90 feet thick beneath the project site and overlie the Troutdale Formation.

5.2.2. Subsurface Profile.

Site specific subsurface information for this project was evidenced by the field exploratory program, which consisted of four borings made at the locations shown on the Site Plan, Fig. 1. The borings provided data to a depth of 51.5 feet. Subsurface information below this depth is based on nearby well logs and published geologic studies (Brown, 1963; Madin 1990; Beeson and Others 1991). Our interpretation of the subsurface profile at the site, listed from the surface down, is as follows:

Depth from Surface (ft)	Geologic Formation
0 to 90	Catastrophic flood deposits consisting of interlayered medium dense silt, sandy silt, silty sand, and sand of late Pleistocene age.
90 to 230	Troutdale Formation; compact fluvial deposits consisting of sandstone and conglomerate of middle Miocene to Pliocene age.
230 +	Columbia River Basalt; a medium hard bedrock consisting of basalt flows of middle Miocene age.

5.2.3. Groundwater

Groundwater was not observed in any of the borings. Groundwater is anticipated to fluctuate with the time of year, being highest in late winter or early spring and lowest in late summer or early fall. A review of data from nearby borings indicates that static water levels are on the order of 70 feet below the ground surface at this site.

5.2.4. Geologic Structure.

Beeson and others (1989, 1991) mapped numerous faults in the West Hills. A Geologic Map showing the structural features is shown in Fig. 13. The longest and most prominent of the mapped faults (actually several aligned fault segments) in the area is known as the Portland Hills Fault (PHF). The PHF includes a series of northwest-trending subsurface faults that extend for a distance of about 40 km (25 mi) along the eastern margin of the Portland Hills. The PHF is not defined by historical seismicity, but is judged to be potentially active on the basis of possible deformation of late Pleistocene sediments (inferred from subsurface sediment thickness data), and the topographic expression of the Portland Hills (Geomatrix, 1995). The mapped trace of the PHF lies approximately 300 m east of the Urban Planning Center site (Beeson and others, 1991).

Other faults in the Portland-metro area were identified by Geomatrix (1995) as potentially active, including the Lackamas Creek Fault, the Sandy River Fault, the Grant Butte and Damascus-Tickle Creek Fault Zone, and the Bolton Fault. Of all of the mapped faults in the Portland area, the PHF was considered by Geomatrix (1995) as the fault with the highest probability of being active. Of the potentially active faults, it is also the nearest to the subject site.

5.3. Seismicity

The region centered on the City of Portland is possibly the most active seismic region in the state of Oregon (Jacobson, 1986). However, based on a 100 or so year history, the level of seismic activity in the Portland region is generally lower when compared to seismic regions in Washington and California.

5.3.1. Sources.

In the past 10 years, several studies have been made of potential earthquake sources in the Pacific Northwest, and these studies have identified three general zones in the earth's crust that are capable of major earthquakes. They are:

1. **Large subduction zone earthquakes** could be generated within the Cascadia Subduction Zone (CSZ). The nearest segment of that zone to Portland lies approximately 120 km (62 miles) to the west (Wong, Silva and Madin, 1993). An earthquake with a moment magnitude of 8.5 (M_w=8.5) was selected as the likely maximum plausible event from that part of the subduction zone by Shedlock and Weaver (1991).

2. **Deep-focus earthquakes** could originate from within the subducting oceanic plate. Such events, called intraplate earthquakes, could be as large as M_w=7.5 originating 65 km (40 miles) from Portland. However, ground shaking produced by such intraplate earthquakes would be less intense and less prolonged in the Portland area than ground motions generated by great subduction zone events as noted above (Mabey and Madin, 1994).

3. **Shallow-focus, crustal earthquakes** could be generated by fault rupture within the crust of the North American plate beneath the Portland area. Mabey and Madin (1994) have concluded from their analysis of local geologic features that crustal earthquakes are possible up to magnitudes of M_w=6.5 virtually anywhere in the Portland area.

5.3.2. Historical Seismicity.

Historically (past 150 years), at least six events greater than a local magnitude of 5.0 ($M_L \geq 5.0$) have occurred in the Portland area (Bott and Wong, 1993), most of which are thought to be of the shallow crustal type. The Scotts Mills earthquake, at magnitude M_L=5.6, is the second largest recorded historical earthquake to occur in Oregon and was located in the continental crust. The epicenter of the Scotts Mills earthquake was located near the community of Scotts Mills, approximately 20 km (12.4 mi) southeast of Woodburn.

Two of the largest historic earthquakes in the Pacific Northwest, which occurred near Olympia in 1949 (M_L~7.1) and near Tacoma in 1965 at depths between 50 and 60 kilometers, are examples of the intraplate (within the subducted oceanic plate) type of event. Since the installation of a modern seismograph network in Oregon in the 1970's, numerous small intraplate earthquakes have been recorded from beneath the Coast Range of Oregon. In southwest Washington and Oregon, it has only been possible to distinguish intraplate earthquake for a few decades, so that it is not possible to determine the frequency of such events. The geologic structure that is thought to be capable of producing up to M_L=7.5 earthquakes in the Puget Sound area, is also thought to be present beneath western Oregon and northern California to some degree (Madin, 1993). However, the area between approximately Salem and the California border has had remarkably little recorded intraplate activity (Weaver and Shedlock, 1994). As a result, we consider the intraplate source to be the least active of the three source zones evaluated in this study. Although there is no historical record of large subduction zone earthquakes in the Pacific Northwest, sufficient geologic evidence exists in coastal estuaries to hypothesize great Cascadia subduction zone earthquakes with an average recurrence interval as low as 500 years (Crouse, 1994). The evidence is in the form of repeated catastrophic flooding and associated uplift and down drops in the geologic record.

5.3.3. Design Earthquakes

The Design Earthquake referred to in the Oregon Specialty Code and defined in the UBC should, as a minimum, be one having a 10% probability of occurrence in 50 years (1 every 500 years). However, in our opinion, there is insufficient seismic history/data to develop a probability based design earthquake, and as a result, a deterministic approach has been (and typically is in Portland) used to develop the design earthquakes. The design earthquakes are based on the above source zones; parameters for design earthquakes are summarized in the following table.

PSU Urban Affairs Project Design Earthquakes

Type	Magnitude	Distance	Mean Peak Ground Acceleration	Significant Cycles (Seed & Idriss 1982)	Duration (Seed & Idriss 1982)
Crustal	Mw=6.0	5 km	0.30 g	5	10 - 20 sec
CSZ	Mw=8.5	120 km	0.18 g	26	1 - 4 min.

Using the empirical attenuation relationships developed by Sadigh, as reported in Wong, Silva and Madin (1993), it is estimated that such a local event with a hypocentral distance of 5 km (3 mi) from the site (M_W=6.0, R=5 km) would produce *mean* peak ground accelerations on the order of 0.30g. The subduction zone earthquake selected (M_W=8.5, R=120 km) is estimated to result in peak horizontal ground accelerations of 0.18g in the Portland area, based on the attenuation equations developed by Crouse (1991).

For perspective, we have provided our opinion of the estimated probability that the selected design earthquake will occur:

> *Bott and Wong (1993) suggested that a M_W=6.0 or larger quake should occur somewhere (R~75 km?) in the Portland region every 300 to 350 years (or approximately 15% in 50 years). For a M_W=6.0 earthquake at just 5 km from the site, the probability would be considerably lower (reduced area and number of faults), and in our opinion considerably less than 10% in 50 years, i.e., we consider the design earthquake to be somewhat conservative from a probability point of view.*

5.4. Ground Response

The mean peak ground surface accelerations tabulated above for the crustal design earthquake are equivalent to the current UBC Zone 3 requirement of Z=0.3 (g).

We recommend a site coefficient (UBC, Table 16-J) of $S_{2/3}$=1.35, because the subsurface soil consistencies are borderline S_2-S_3.

Based on our understanding of the project subsurface conditions, we have concluded that the UBC Normalized Response Spectra Shape for cohesionless or stiff clay soil sites (Soil Type 2) is applicable for the site conditions evidenced and may be used in conjunction with the above design event. Alternatively, the maximum C value (2.75) as defined in the UBC could be used.

5.5. Seismic Hazards

5.5.1. General.

Based on the available information, there is no reason to believe the following specific hazards are a risk to this site:

1. Landslide (seismic induced slope instability).
2. Tsunami (tidal/ocean wave).
3. Seiche (lake or river wave).
4. Subsidence.

5.5.2. Fault Displacement.

Although many faults have been identified and mapped in the Portland area, the hazard of rupture on any specific fault is unknown. Mabey and others (1993) have reported that the magnitude range of 6.0 to 6.5 is the threshold at which fault rupture commonly begins to become apparent, and because a 6.0 to 6.5 is the likely maximum magnitude for any crustal fault in the Portland area, fault rupture is likely to be absent altogether, or of very limited extent. The number of structures affected and the severity of the effects, therefore, will also be limited.

5.5.3. Liquefaction and Lateral Spreading

Liquefaction can be described as a sudden loss of shear strength during a seismic event, (although some low value of shear strength is usually retained). The occurrence of liquefaction is restricted to loose, saturated granular soils. Loose soils do occur in the first 10 feet or so beneath this site, but ground water levels are sufficiently deep that saturated conditions are very unlikely. In our opinion, liquefaction hazards at this site are very low.

6. LIMITATIONS OF REPORT

The analyses, conclusions and recommendations contained in this report are based on site conditions as they presently exist and assume the exploratory holes are representative of the subsurface conditions throughout the site. If, during construction, subsurface conditions different from those encountered in the exploratory holes are observed or appear to be present beneath excavations, we should be advised at once so that we may review these conditions and reconsider our recommendations where necessary.

This report was prepared for your use in the design of the subject facility. This report should not be used for contractual purposes as a warranty of interpreted subsurface conditions such as those indicated by the formal boring logs, and/or discussion of subsurface conditions contained herein.

If there is a substantial lapse of time between the submission of this report and the start of work at the site, if conditions have changed due to natural causes of construction operations at or adjacent to the site, or if the basic project scheme is significantly modified from that assumed, it is recommended that this report be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

FUJITANI HILTS & ASSOCIATES

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by ____________________
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OREGON
NOVEMBER 15, 1974
DAVID E. HILTS

d:\fha documents\projects\2875.01\rpsu urban center.doc

APPENDIX

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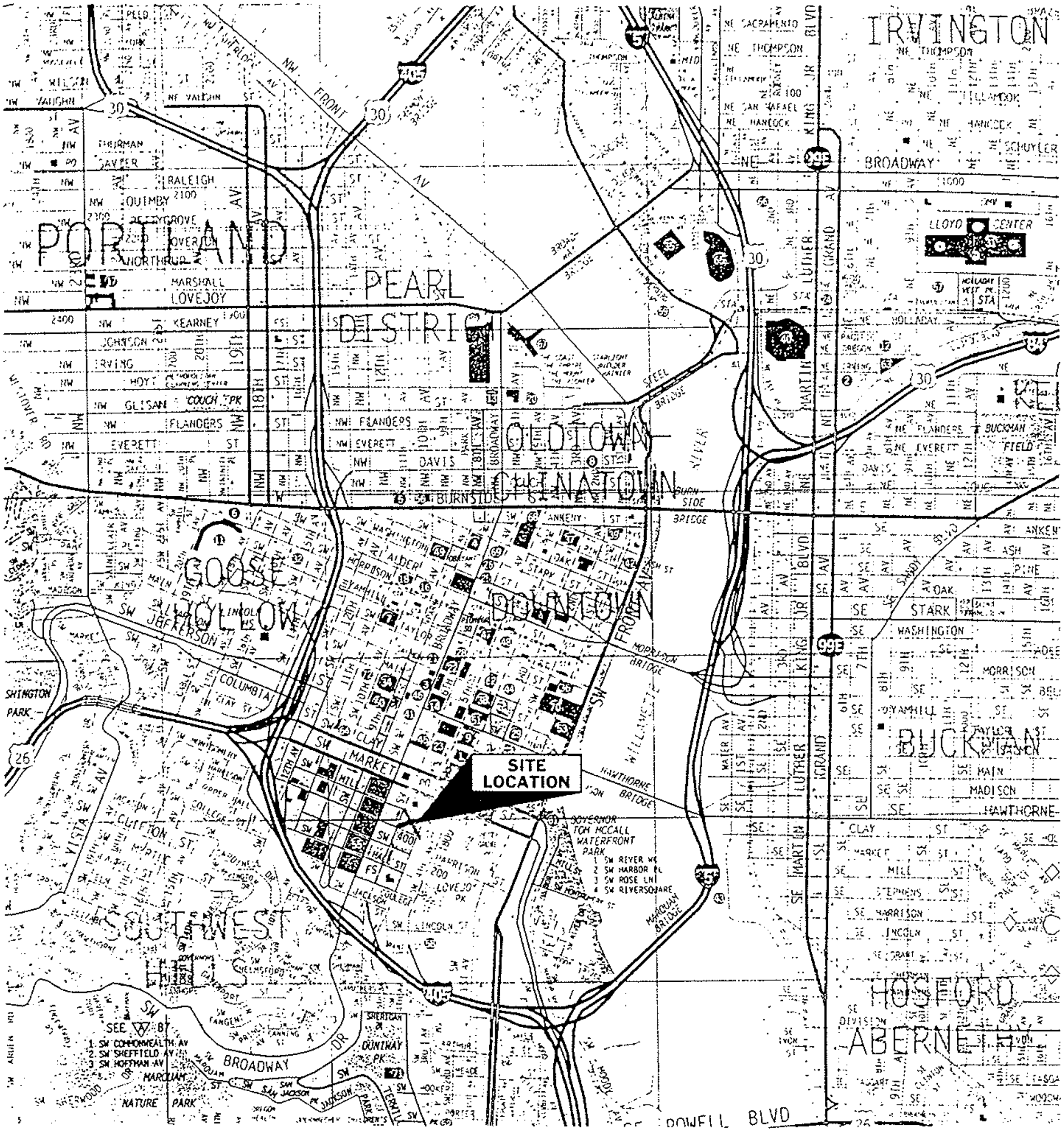

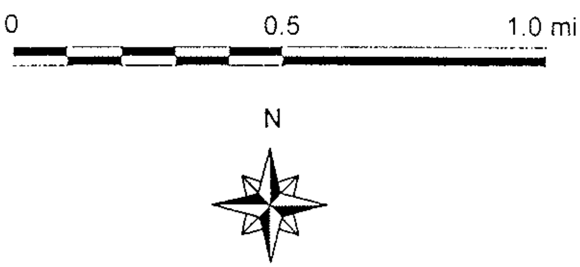

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VICINITY MAP

February 1997

2875.01

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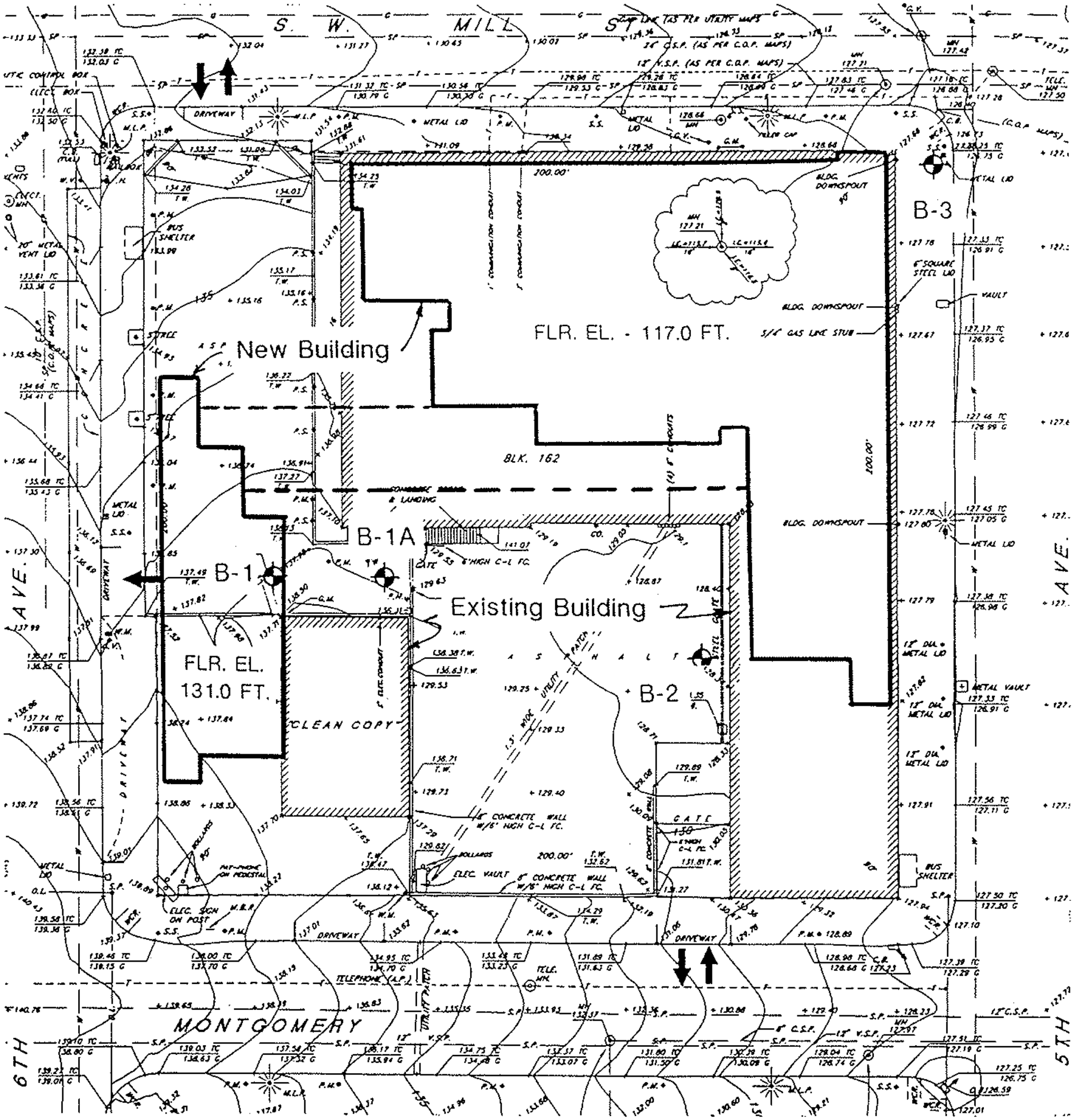
FIG. 1



LEGEND

⊕ Boring by FHA Jan 1997

Reproduced from Drawing Nos. L0.1 and L0.2 prepared by Thomas Hacker and Associates, Architects P.C., 12/18/96.

N

Scale: 1 in = 40 ft

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BORING LOCATION PLAN

February 1997 — 2875.01

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FIG. 2

Ground Water	Remarks	Elev. Depth Feet 138.0	CLASSIFICATION OF MATERIAL	Log	Depth in Feet	Samples	▲ SPT N-Value ● Moisture, % (0 – 25 – 50)
		137.6 0.4	**PAVEMENT** consisting of 2" of AC, 3 of AC treated base, and 12 inches of crushed rock base.		0		
		136.4 1.6	**GRAVEL** with trace of sand and trace of silt. Medium dense, gray, fine to coarse, moist, (FILL).				
		135.0 3.0	Top of underground storage tank.			I	▲
					5		
▼	1/13/97 16" of water and oil in tank.				10		
		127.5	Bottom of Storage Tank.				
		10.5	Bottom of Boring, Completed 1/13/97				

0 50 100

☐ Recovery, % ⊞ RQD, %

LEGEND

- I = 2.0" O.D. Split Spoon Sample
- II = 3.0" O.D. Thin-Walled Sample
- • = Sample Not Recovered
- ⊠ = Grab Sample: Drill Cuttings
- ▮ = Core Rock Sample

NOTE:
Lines between soil/rock units are approximate and transition may be gradual.

Impervious Seal (Bentonite)
Cement Grout
Random Backfill
Granular Backfill
▽ Ground Water Level on Date Shown
Piezometer/Inclinometer Tubing
Perforated Zone

ATTERBERG LIMITS
Liquid Limit
Natural Water Content
Plastic Limit

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LOG OF BORING B-1

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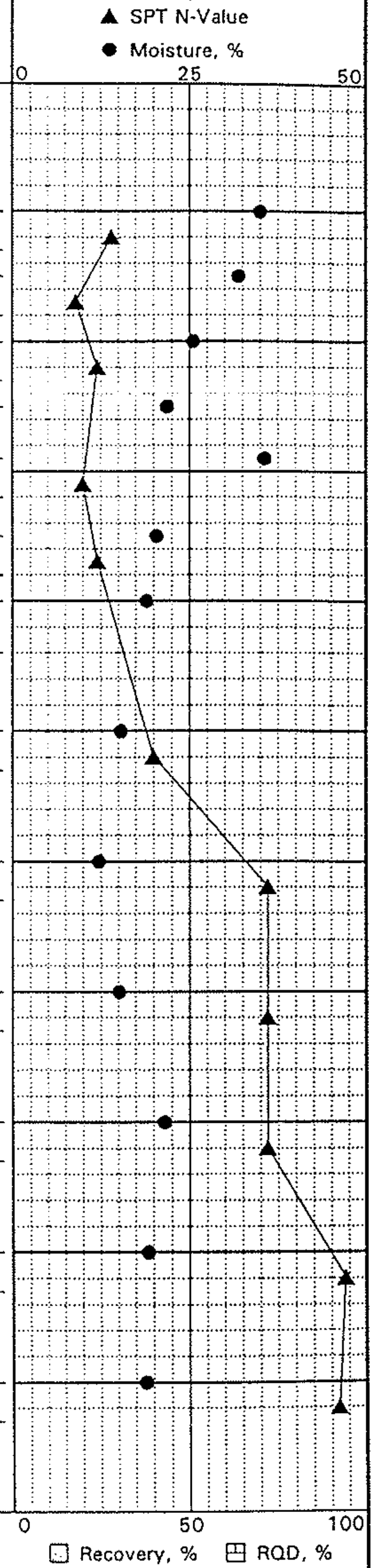
FIG. 3

FHA

WLG PSUUA 3/6/97

Ground Water	Remarks	Elev. Depth Feet 138.0	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
None Observed		137.5 0.5	ASPHALT over 1-1/2 inch crushed rock.		0	
			SILT to SANDY SILT, medium dense, brown to mottled brown, moist, non-plastic, micaceous (FILL).		5	S2
						S3
					10	S4
						S5
	Dry unit wt. 78 pcf	122.5 15.5	**SILTY SAND**, medium dense to dense, brown, moist, nonplastic, fine grained, micaceous.		15	S6
						S7
	Dry unit wt. 86 pcf		Two thin silt layers in S8, 1-inch and 4-inches thick.		20	S8
					25	S9
					30	S10
			4-inch thick fine sandy silt layer.		35	S11
					40	S12
			Formation becomes denser below 43.5 feet.		45	S13
		86.5 51.5	4-inch thick silt layer. Bottom of Boring, Completed 1/13/97		50	S14



LEGEND

- 2.0" O.D. Split Spoon Sample
- 3.0" O.D. Thin-Walled Sample
- • = Sample Not Recovered
- = Grab Sample: Drill Cuttings
- = Core Rock Sample

Imprevious Seal (Bentonite)
Cement Grout
Random Backfill
Granular Backfill
Ground Water Level on Date Shown
Piezometer/Inclinometer Tubing
Perforated Zone

ATTERBERG LIMITS
Liquid Limit
Natural Water Content
Plastic Limit

NOTE:
Lines between soil/rock units are approximate and transition may be gradual.

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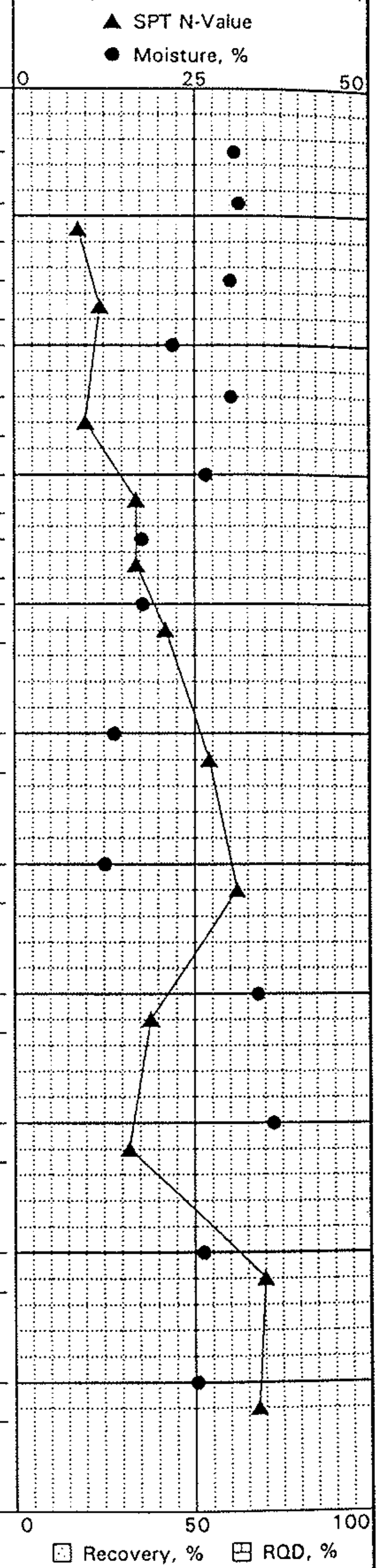
FIG. 4

FHA

WLG PSUUA 3/5/97

Ground Water	Remarks	Elev. Depth Feet 128.5	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
None Observed		127.8 / 0.7	**ASPHALT** over 3/4-inch base rock.		0	
			SILT with SAND, loose, brown to mottled brown, moist, non-plastic, micaceous.			
	Dry unit wt. 90 pcf					S1
			9-inch thick fine sand layer.		5	S2
		120.5 / 8.0	**SANDY SILT to SILTY SAND**, medium dense, brown, moist, fine grained, micaceous.			S3
	Dry unit wt. 83 pcf				10	S4
						S5
			Contains occassional interbedded layers of fine sandy silt up to 8-inches thick.		15	S6
						S7
					20	S8
					25	S9
					30	S10
		94.5 / 34.0	**SILT**, medium dense, mottled brown, moist, very low plasticity, micaceous.		35	S11
					40	S12
		85.5 / 43.0	**SILTY SAND**, dense, brown, moist, non-plastic, micaceous.		45	S13
					50	S14
		77.0 / 51.5	Bottom of Boring, Completed 1/14/97			



LEGEND

- = 2.0" O.D. Split Spoon Sample
- = 3.0" O.D. Thin-Walled Sample
- • = Sample Not Recovered
- = Grab Sample: Drill Cuttings
- = Core Rock Sample

Impervious Seal (Bentonite)
Cement Grout
Random Backfill
Granular Backfill
Ground Water Level on Date Shown
Piezometer/Inclinometer Tubing
Perforated Zone

ATTERBERG LIMITS
Liquid Limit
Natural Water Content
Plastic Limit

NOTE:
Lines between soil/rock units are approximate and transition may be gradual.

WLG PSUUA 3/5/97

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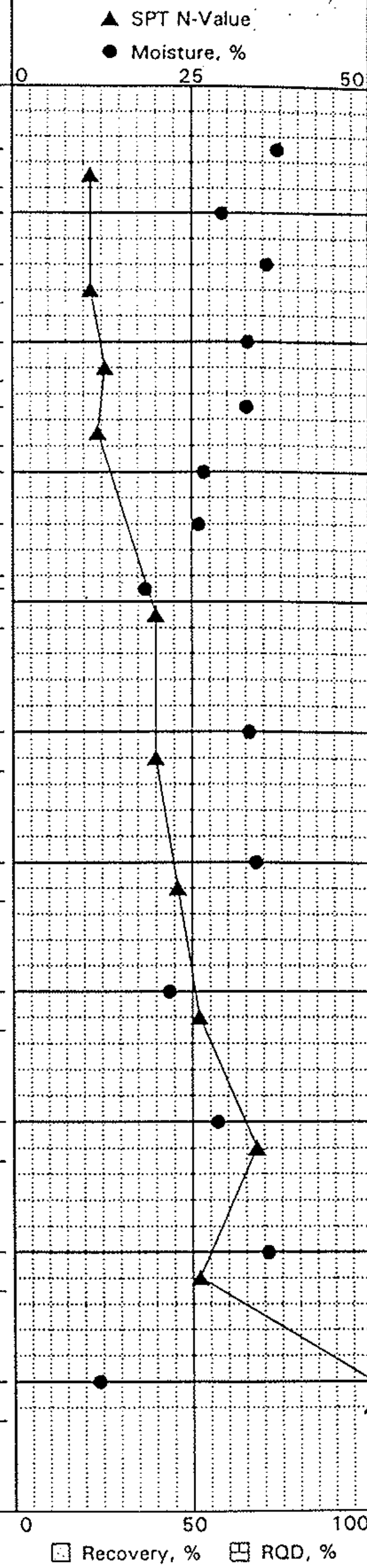
FIG. 5

FHA

Ground Water	Remarks	Elev. Depth Feet 127.5	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
None Observed		126.7 / 0.8	**CONCRETE** sidewalk over 1-1/2 inch crushed base rock.		0	
	Dry unit wt. 78 pcf	119.0	**SILT with SAND**, medium dense, brown to mottled brown, moist low to non-plastic, micaceous.		5	S1, S2, S3
	Dry unit wt. 89 pcf	8.5	**SANDY SILT to SILTY SAND**, medium dense, brown, moist, non-plastic, fine grained, micaceous. Contains occassional interbedded layers of silt to fine sandy silt up to 10-inches thick. Becomes dense at 40 feet. Becomes medium dense at 45 feet. Becomes very dense at 50 feet.		10, 15, 20, 25, 30, 35, 40, 45, 50	S4, S5, S6, S7, S8, S9, S10, S11, S12, S13, S14
		76.0 / 51.5	Bottom of Boring, Completed 1/14/97			



LEGEND

- 2.0" O.D. Split Spoon Sample
- 3.0" O.D. Thin-Walled Sample
- • = Sample Not Recovered
- = Grab Sample: Drill Cuttings
- = Core Rock Sample

Impervious Seal (Bentonite)
Cement Grout
Random Backfill
Granular Backfill
Ground Water Level on Date Shown
Piezometer/Inclinometer Tubing
Perforated Zone

ATTERBERG LIMITS
Liquid Limit
Natural Water Content
Plastic Limit

NOTE:
Lines between soil/rock units are approximate and transition may be gradual.

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FIG. 6

FHA

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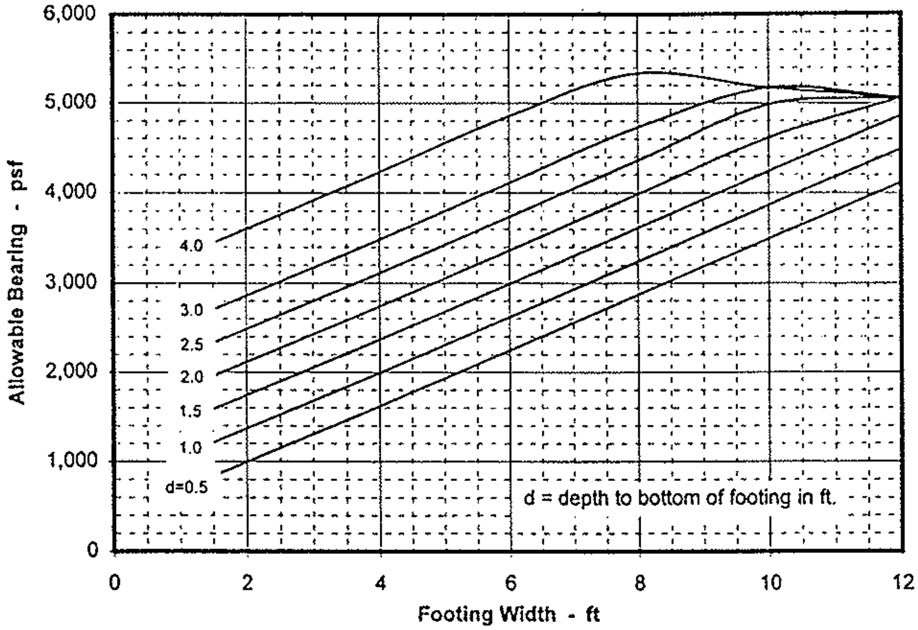
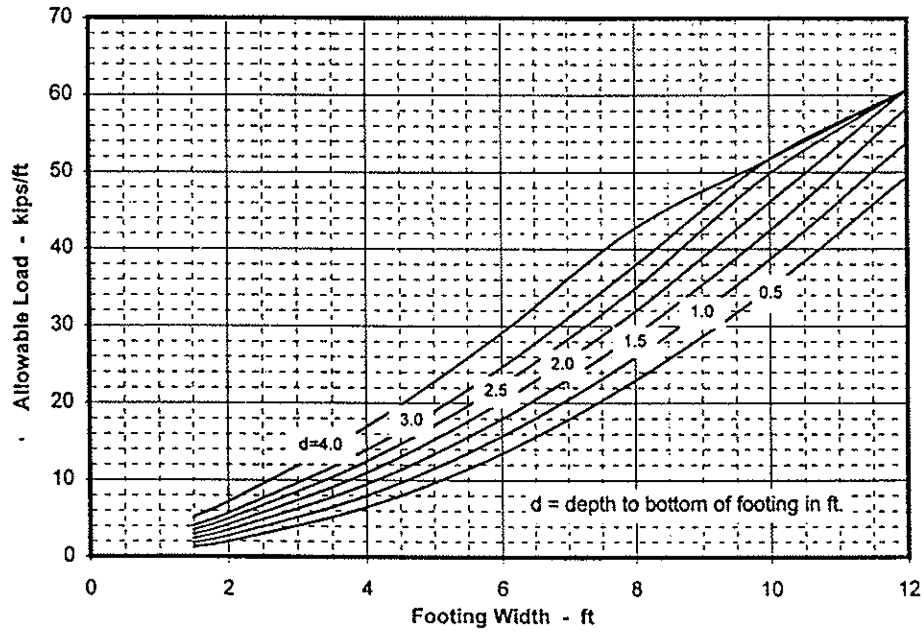
Allowable Bearing for Continuous Basement Footings



Allowable Load for Continuous Basement Footings



Note: Assumes footing elevation is at or below El. 117 feet.

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CONTINUOUS BASEMENT FOOTING CRITERIA

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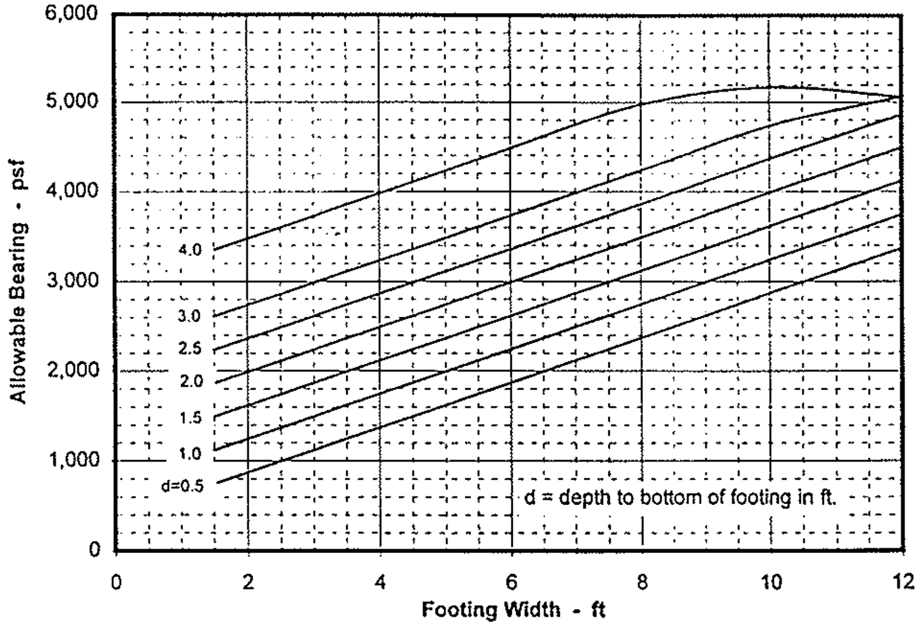
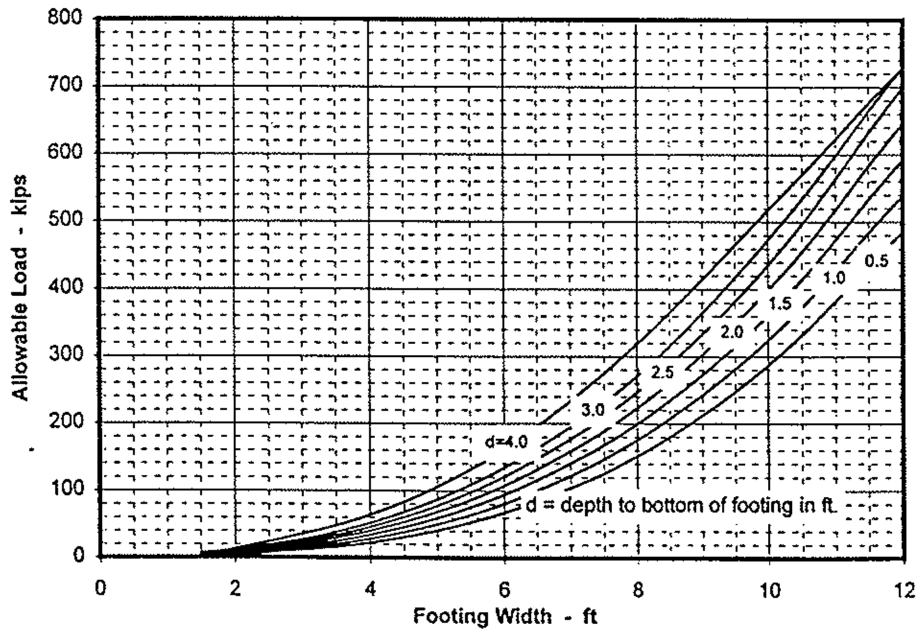
FIG. 7

Allowable Bearing for Square Basement Footings



Allowable Load for Square Basement Footings



Note: Assumes footing elevation is at or below El. 117 feet.

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SQUARE BASEMENT FOOTING CRITERIA

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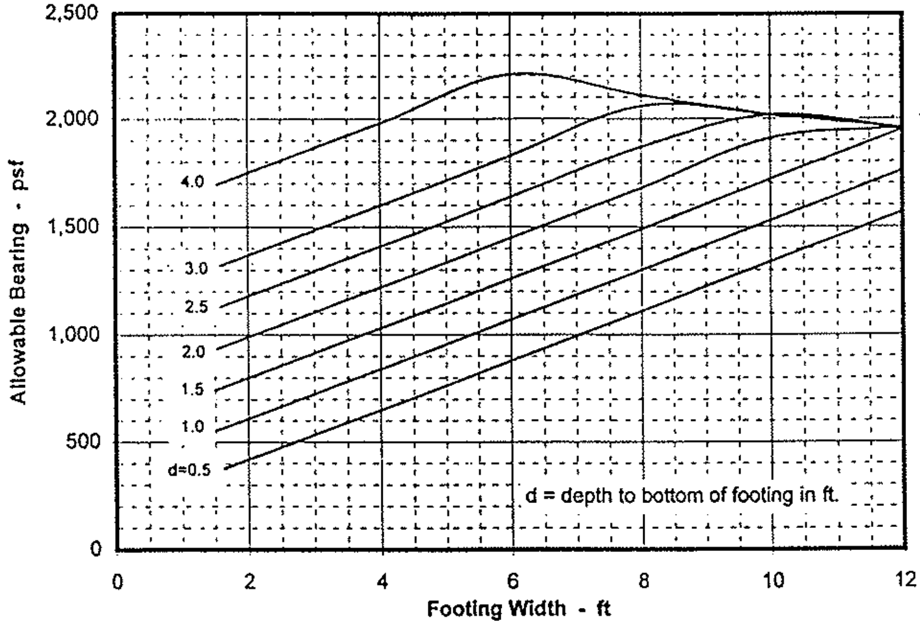
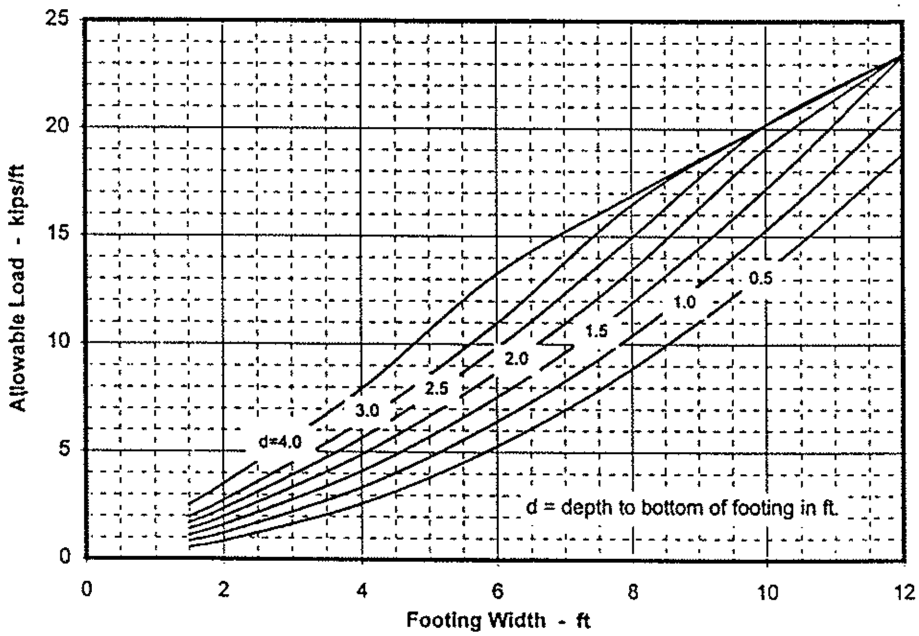
FIG. 8

Allowable Bearing for Continuous Ground Floor Footings



Allowable Load for Continuous Ground Floor Footings



Note: Assumes footing elevation is at or below El. 127 feet.

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CONTINUOUS GROUND FLOOR FOOTING CRITERIA

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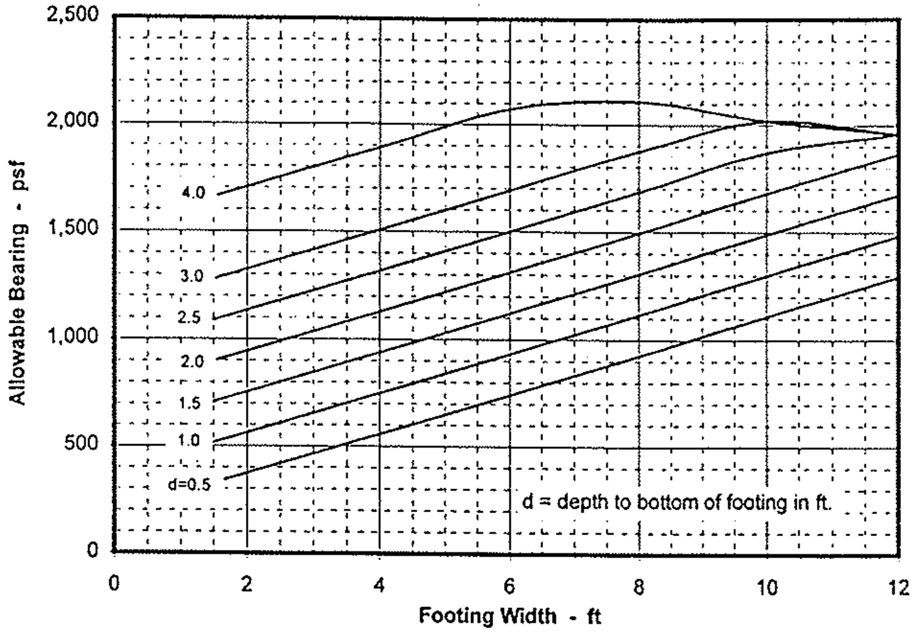
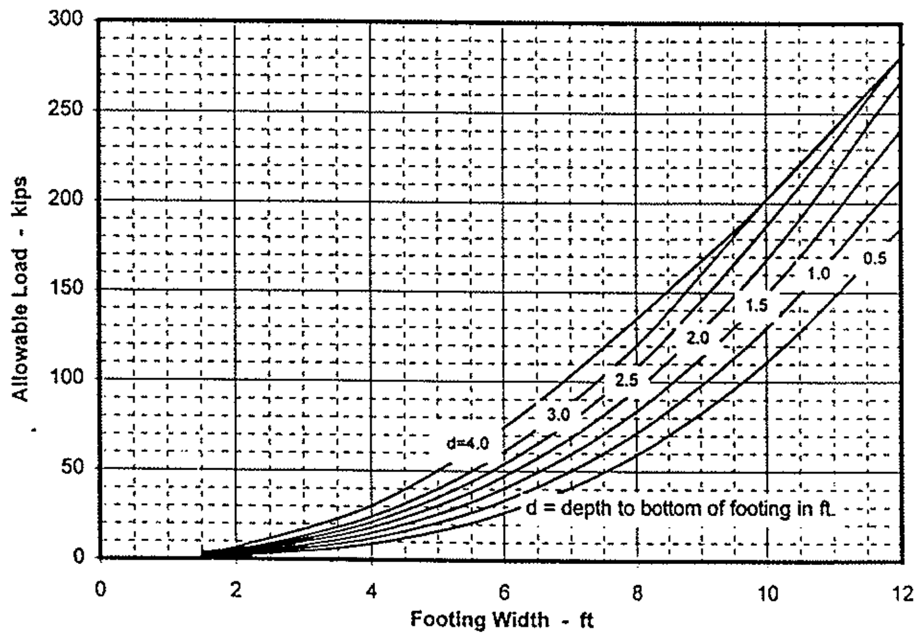
FIG. 9

Allowable Bearing for Square Ground Floor Footings



Allowable Load for Square Ground Floor Footings



Note: Assumes footing elevation is at or below El. 127 feet.

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SQUARE GROUND FLOOR FOOTING CRITERIA

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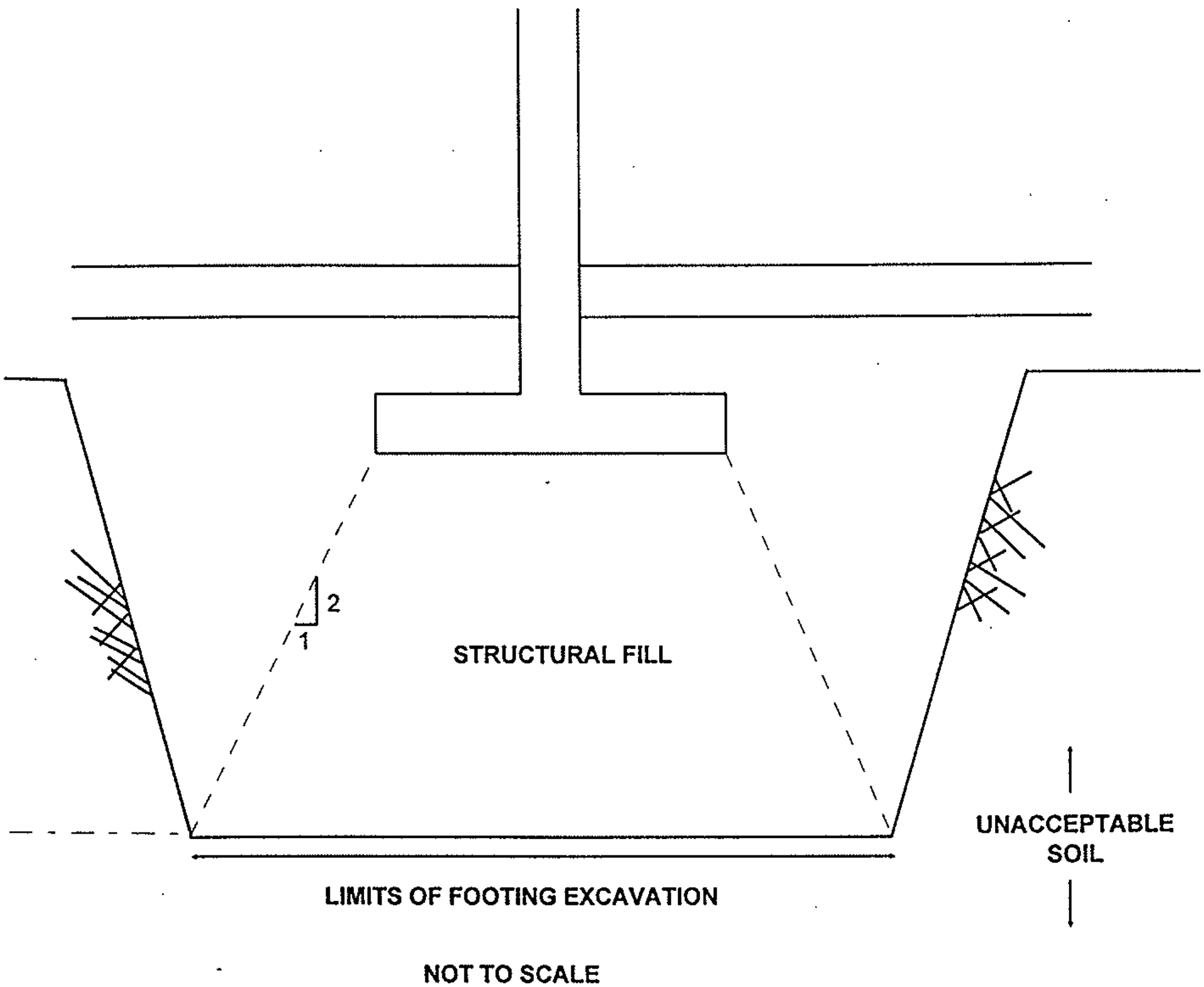
FIG. 10



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GROUND FLOOR FOOTING OVER-EXCAVATION CRITERIA

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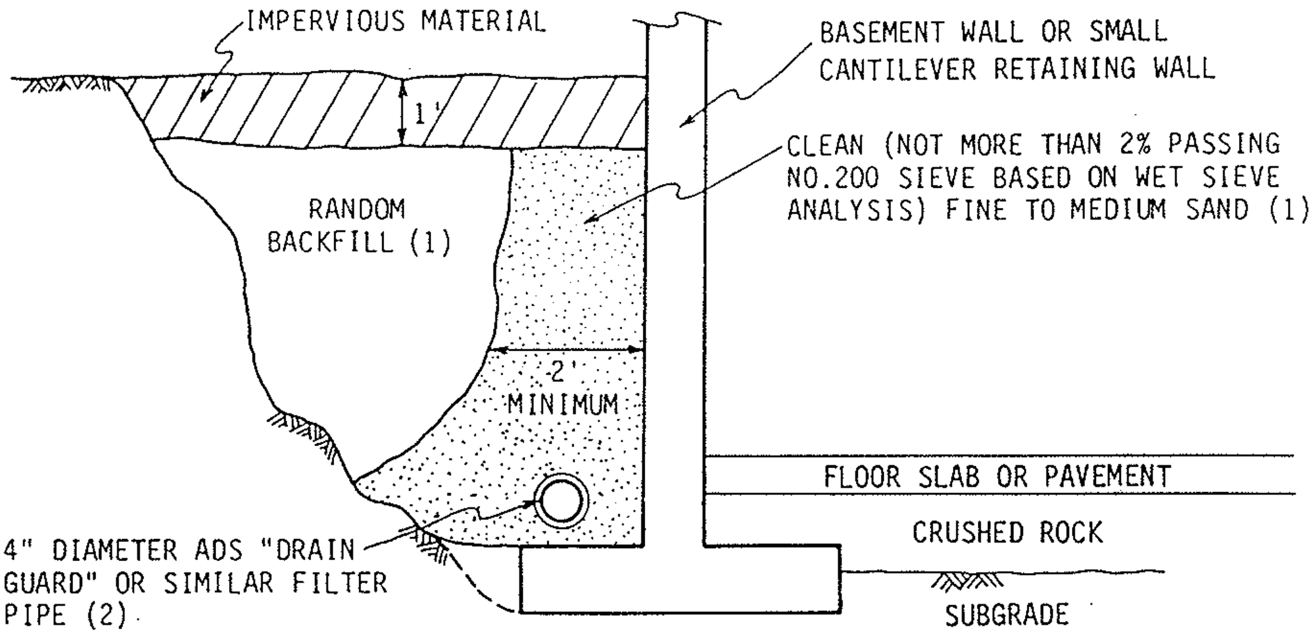
FIG. 11



NOTES:

1. COMPACT RANDOM BACKFILL AND CLEAN SAND TO 92 PERCENT OF STANDARD PROCTOR MAXIMUM DRY DENSITY.
2. PERFORATED PIPE SURROUNDED BY A 6" ENVELOPE OF 1½" MINUS CRUSHED ROCK MAY BE USED IN LIEU OF FILTER PIPE.

NOT TO SCALE

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RETAINING WALL DRAINAGE CRITERIA

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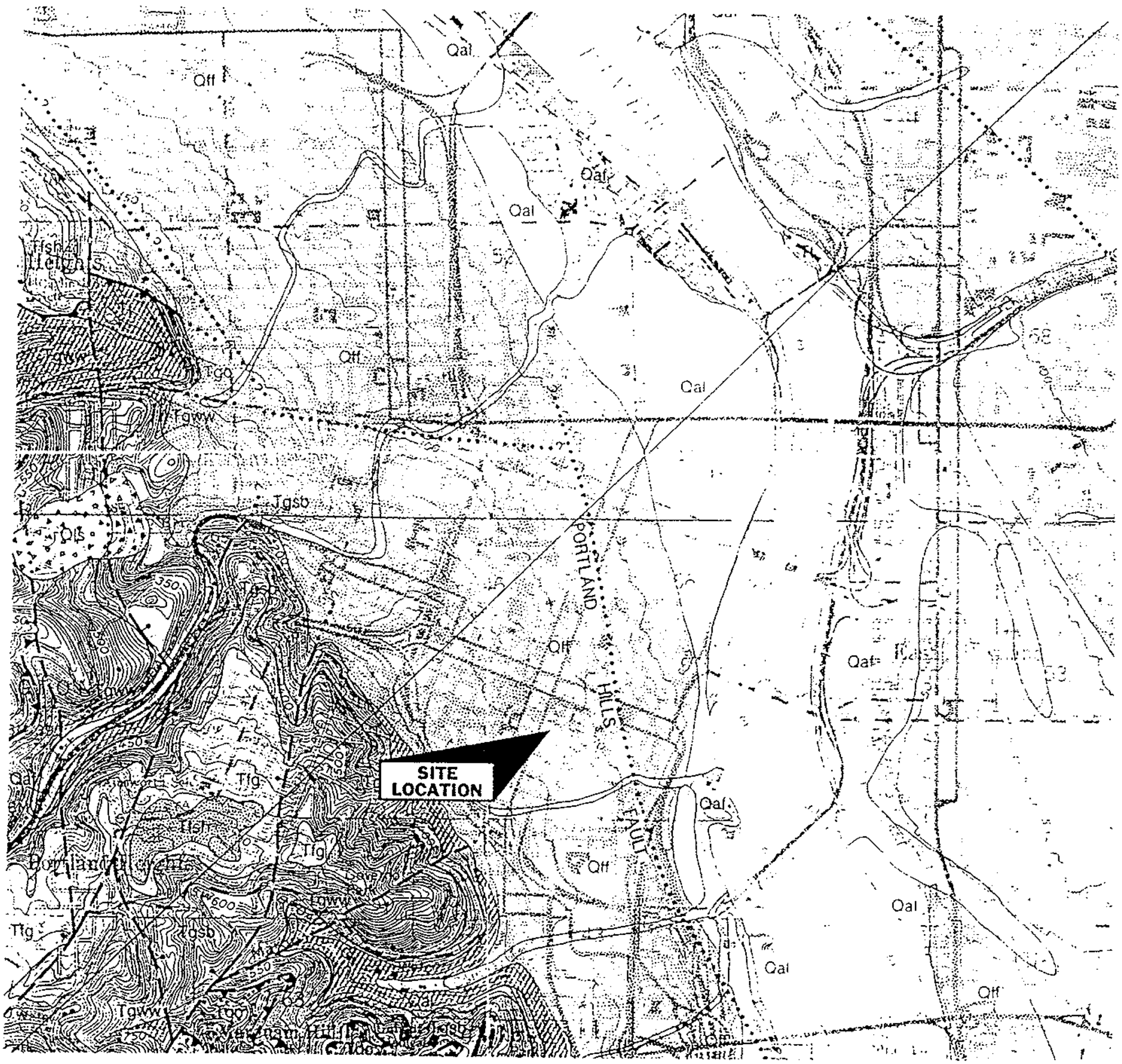
FIG. 12



MAP SYMBOLS

Qaf	Artificial fill	Qfch	Catastrophic flood deposits; channel facies.		Contact--Approximatley located.
Qls	Landslide deposits	Qff	Catastrophic flood deposits; fine grained	?— • • •	Fault--Dashed where inferred, dotted where concealed, queried where doubtful, ball and bar on downthrown side.
Qal	Alluvium	Tfsh Tfg Tgww Tgsb	Columbia River Basalt Group.	▲? ▲ •▲•	Thrust fault--Dashed where inferred; dotted where concealed; queried where doubtful; sawteeth on upper plate.

NORTH

0 2,000 4,000

SCALE FEET

Beeson, M.H., Tolan, T.L., Madin, I.P., 1991, Geologic Map of the Portland quadrangle, Multnomah and Washington Counties, Oregon, and Clark County, Washington: Oreg. Dept. Geol. and Miner. Ind. Geological Map Serie.s GMS-75, scale 1:24,000.

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AREA GEOLOGY

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Portland, Oregon

FIG. 13

NOTICE TO WATER WELL CONTRACTOR
The original and first copy of this report are to be filed with the STATE ENGINEER, SALEM 10, OREGON within 30 days from the date of well completion.

WATER WELL REPORT
STATE OF OREGON
(Please type or print)

State Well No.
State Permit No.

(1) OWNER:
Name NORTHWEST HOSPITAL SERVICE
Address 1320 S.W. BROADWAY
PORTLAND 4 ORE.

(2) LOCATION OF WELL:
County MULT Driller's well number 4132
¼ ¼ Section T. R. W.M.
Bearing and distance from section or subdivision corner

(3) TYPE OF WORK (check):
New Well ☐ Deepening ☐ Reconditioning ☐ Abandon ☐
If abandonment, describe material and procedure in Item 12.

(4) PROPOSED USE (check):
Domestic ☐ Industrial ☐ Municipal ☐
Irrigation ☐ Test Well ☐ Other ☐

(5) TYPE OF WELL:
Rotary ☐ Driven ☐
Cable ☐ Jetted ☐
Dug ☐ Bored ☐

(6) CASING INSTALLED: Threaded ☐ Welded ☒
12 " Diam. from 0 ft. to 206 ft. Gage 330
" Diam. from ft. to ft. Gage
" Diam. from ft. to ft. Gage

(7) PERFORATIONS: Perforated? ☒ Yes ☐ No
Type of perforator used STAR
Size of perforations 1/4 in. by 1 1/4 in.
540 perforations from 160 ft. to 169 ft.
420 perforations from 178 ft. to 185 ft.
perforations from ft. to ft.
perforations from ft. to ft.
perforations from ft. to ft.

(8) SCREENS: Well screen installed ☐ Yes ☒ No
Manufacturer's Name
Type Model No.
Diam. Slot size Set from ft. to ft.
Diam. Slot size Set from ft. to ft.

(9) CONSTRUCTION:
Well seal—Material used in seal CEMENT GROUT
Depth of seal 50 ft. Was a packer used? NO
Diameter of well bore to bottom of seal 16 in.
Were any loose strata cemented off? ☐ Yes ☒ No Depth
Was a drive shoe used? ☒ Yes ☐ No
Was well gravel packed? ☐ Yes ☒ No Size of gravel:
Gravel placed from ft. to ft.
Did any strata contain unusable water? ☐ Yes ☒ No
Type of water? Depth of strata
Method of sealing strata off

(10) WATER LEVELS:
Static level 138 ft. below land surface Date 9/21/62
Artesian pressure lbs. per square inch Date

(11) WELL TESTS: Drawdown is amount water level is lowered below static level
Was a pump test made? ☒ Yes ☐ No If yes, by whom? RJ STRASSER
Yield: 340 gal./min. with 25 ft. drawdown after 24 hrs.
" " " "
" " " "
Bailer test gal./min. with ft. drawdown after hrs.
Artesian flow g.p.m. Date
Temperature of water 55 Was a chemical analysis made? ☐ Yes ☐ No

(12) WELL LOG: Diameter of well below casing 12
Depth drilled 207 ft. Depth of completed well 207 ft.
Formation: *Describe by color, character, size of material and structure, and show thickness of aquifers and the kind and nature of the material in each stratum penetrated, with at least one entry for each change of formation.*

MATERIAL	FROM	TO
SANDY (DRY)	0	12
SAND AND SILT	12	98
CEMENTED GRAVEL	98	124
LOOSE SAND AND GRAVEL	124	169
BLUE CLAY	169	173
SAND AND GRAVEL	173	185
BLUE BLACK CLAY	185	189
BROKEN ROCK	189	206
BLACK ROCK	206	207

Work started Aug 31 1962 Completed SEPT 25 1962
Date well drilling machine moved off of well SEPT 26 1962

(13) PUMP:
Manufacturer's Name
Type: H.P.

Water Well Contractor's Certification:
This well was drilled under my jurisdiction and this report is true to the best of my knowledge and belief.
NAME RJ STRASSER DRILLING CO
(Person, firm or corporation) (Type or print)
Address 8110 SE SUNSET LANE PORTLAND ORE
Drilling Machine Operator's License No. 56
[Signed] Robert [illegible]
(Water Well Contractor)
Contractor's License No. 10 Date OCT 3, 1962

(USE ADDITIONAL SHEETS IF NECESSARY)

* * * WATER RIGHT INFORMATION * * *

Application #: G-2387 Permit #: G-2205 Certificate #: 33769

ell Owner:

USER-ID: 10739

STATE OF OREGON; BOARD OF HIGHER EDUCATION
PO BOX 3175
EUGENE OR 97403

Original Water Right Holder:
NORTHWEST HOSPITAL SERVICE
1320 SW BROADWAY
PORTLAND OR 97201

WELLS ASSOCIATED WITH RIGHT

Note: The POD-ID is an arbitrary number used for computer purposes only.)

A WELL (POD-ID 22684)
Permitted Use of Water: AIR CONDIT
Rate of Use: 0.8600 cubic feet per second
Priority Date: 7/17/1962
Well Location Information:
Township 1 S Range 1 E
NE Quarter of SE Quarter of Section 4
4 FT S & 18 FT E FM SW COR, BLOCK 161, PORTLAND ADDITION, S4
ossible Tax Lot # 667716410 (MULT COUNTY)

* * * * * * * * *
PUMP TEST DUE ON
7/17/1992
* * * * * * * * *

PSU - WATER SERVICES BLDG