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3001 N Gantenbein Ave

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GEOTECHNICAL INVESTIGATION

LEGACY EMANUEL
BURN CENTER ADDITIONS
PORTLAND, OREGON

GT_002473

GEOCON
INCORPORATED
GEOTECHNICAL CONSULTANTS

PREPARED FOR

LEGACY HEALTH SYSTEMS
PORTLAND, OREGON

DECEMBER 2000

GEOCON
NORTHWEST

GEOTECHNICAL CONSULTANTS

Project No. P1009-05-08
December 11, 2000

Legacy Health Systems
1919 Northwest Lovejoy
Portland, Oregon 97209

Attention: Mr. Larry Hill

Subject: LEGACY EMANUEL BURN CENTER ADDITIONS
PORTLAND, OREGON
GEOTECHNICAL INVESTIGATION

Dear Mr. Hill:

In accordance with our proposal number P00-05-70, dated October 31, 2000, and your authorization, we have performed a geotechnical investigation for the proposed additions to the Legacy Emanuel Burn Center in Portland, Oregon. The accompanying report presents the findings of the investigation and conclusions and recommendations regarding the geotechnical aspects of the proposed addition. Based on the results of this investigation, it is our opinion that the additions can be constructed as proposed, provided the recommendations of this report are followed.

If you have questions regarding this report, or if we may be of further service, please contact the undersigned at your convenience.

Sincerely,

GEOCON NORTHWEST, INCORPORATED

Heather Devine

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Geotechnical Engineering Staff

Wesley Spang

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of the geotechnical investigation for the proposed additions to the Legacy Emanuel Burn Center. The burn center is located at 3001 N Gantenbein Avenue in Portland, Oregon, as shown on Figure 1, Site Vicinity Map. The purpose of the geotechnical investigation was to evaluate subsurface soil and geologic conditions at the site and, based on the conditions encountered, provide conclusions and recommendations pertaining to the geotechnical aspects of the construction of the proposed additions.

The scope of the field investigation consisted of a site reconnaissance, review of published geological literature, and two exploratory borings. A detailed discussion of the field investigation is presented in Section 4 of this report. Exploratory logs are presented in Appendix A.

Laboratory tests were performed on selected soil samples obtained during the investigation to evaluate pertinent physical properties. Appendix B presents a summary of the laboratory test results. The results of laboratory moisture content tests are presented on the boring logs, located in Appendix A.

The recommendations presented herein are based on an analysis of the data obtained during the investigation, laboratory test results and our experience with similar soil and geologic conditions. Geocon Northwest has completed geotechnical investigations for the new Legacy Emanuel parking structure, medical offices building, and north emergency entrance addition, and information from this previous work has been used to supplement the site-specific data acquired during this investigation. This report has been prepared for the exclusive use of Legacy Health Systems and their agents, for specific application to this project, in accordance with generally accepted geotechnical engineering practice. This report may not contain sufficient information for purposes of other parties or other uses.

2. SITE AND PROJECT DESCRIPTION

The Legacy Emanuel Burn Center site is located within the northeast quadrant of T.1N, R.1E, Section 27, in Multnomah County, Oregon, in the City of Portland. The existing burn center is located at 3001 N. Gantenbein Avenue. The additions will consist of an ambulance entrance with a cantilevered canopy from N. Commercial Avenue to the northwest corner of the building, and an elevator located on the south side of the burn center. The approximate location of the Legacy Emanuel Burn Center is shown in the Site Vicinity Map, Figure 1.

Historic fire insurance maps from 1950 and 1969 were reviewed to evaluate past usage of the site. Based on these maps it appears that the burn center site was a residential area as recently as 1950.

A slope, approximately four feet in height, was observed on the west side of the proposed ambulance entrance. Minor cuts and fills are anticipated in the proposed ambulance entrance location. Excavations on the order of 14 to 16 feet are expected in the location of the elevator.

3. GEOLOGY AND SEISMIC SETTING

3.1. Regional Geology

Based on the State of Oregon Department of Geology and Mineral Industries' (DOGAMI) publication *Geologic Map of the Portland Quadrangle, Multnomah and Washington Counties, Oregon and Clark County, Washington*, the site is mapped within an area of Pleistocene age fine-grained facies. These Pleistocene age deposits are characterized by brown to buff, unconsolidated beds and lenses of coarse-grained sand to silt. The fine-grained facies are slack water fluvial and/or lacustrine deposits resulting from repeated temporary inundation of the Willamette Valley by Late-Pleistocene glacial outburst floods. These glacial floods originated in the Missoula Valley of Montana, passed through eastern Washington, and followed the Columbia River downstream. When these large floods entered the Portland Basin they flowed up the Willamette River and its tributaries, flooding most of the Willamette and Tualatin Valleys up to an approximate elevation of 350 feet MSL. The last of these glacial floods, also thought to be one of the largest, occurred about 12,400 years ago, establishing the minimum age of the silt deposit. This alluvial deposit may range in thickness from as little as 10 feet to over 250 feet in the vicinity of the site. Below the surface deposit is a Pliocene age sandstone and conglomerate of inundated beds and lenses of well sorted sand and gravel, typically referred to as the Troutdale Formation. The Troutdale Formation occurs primarily in the valleys of the Willamette, Clackamas and Sandy Rivers, as well as along many of their tributaries. At depths below the Troutdale Formation, Sandy River Mudstone and Columbia River Basalt is mapped.

3.2. Seismic Setting

3.2.1. Earthquake Sources

The seismicity of Portland and adjacent areas, and hence the potential for ground shaking, is controlled by three separate fault mechanisms. These include the Cascadia Subduction Zone (CSZ), the mid-depth intraplate zone, and the relatively shallow crustal zone sources.

The Cascadia Subduction Zone is located offshore and extends from Northern California to British Columbia. Within this zone the oceanic Juan De Fuca Plate is being subducted beneath the continental North American Plate to the east. The interface between these two plates is located at a depth of approximately 15 to 20 kilometers. The seismicity of the CSZ is subject to several uncertainties, including the maximum earthquake magnitude and the recurrence intervals associated with various magnitude earthquakes. Anecdotal evidence of previous CSZ earthquakes has been observed within coastal marshes along the Oregon coast (Peterson et al. 1993). Sequences of interlayered peat and sands have been interpreted to be the result of large subduction zone earthquakes occurring at intervals on the order of 300 to 500 years, with the most recent event taking place approximately 300 years ago. A recent study by Geomatrix (1995) suggests that the maximum earthquake associated with the CSZ is moment magnitude (M_W) 8 to 9. This is based on an empirical expression relating moment magnitude to the area of fault rupture derived from earthquakes which have occurred within subduction zones in other parts of the world.

The intraplate zone encompasses the portion of the subducting Juan De Fuca Plate located at a depth of approximately 20 to 40 km below Western Oregon. Very low levels of seismicity have been observed within the intraplate zone in Oregon. However, much higher levels of seismicity within this zone have been recorded in Washington and California. Several reasons for this seismic quiescence in Oregon were suggested in the Geomatrix (1995) study and include changes in the direction of subduction between Oregon and British Columbia as well as the effects of volcanic activity along the Cascade Range. Historical activity associated with the intraplate zone includes the 1949 Olympia (magnitude 7.1) and the 1965 Puget Sound (magnitude 6.5) earthquakes.

The third source of seismicity that can result in ground shaking within the greater Portland area is near-surface crustal earthquakes occurring within the North

American Plate. The historical seismicity of crustal earthquakes in western Oregon is higher than the seismicity associated with the CSZ and the intraplate zone. The 1993 Scotts Mills (magnitude 5.6) and Klamath Falls (magnitude 6.0) were crustal earthquakes. Individual faults or fault zones, which have been mapped by the Oregon Department of Geology and Mineral Industries (1991) and (2000), and Geomatrix (1995) within the near-vicinity of the site, are indicated on Table 1: Area Faults.

Table 1: Area Faults

Fault System	Approx. Distance to Site (miles)
Portland Hills Fault	1.5
Oatfield Fault	4
East Bank Fault	<1
Bolton Fault	9
Grant Butte, Damascus-Tickle Creek Fault Zone	12
Helvetia Fault	12
Lacamas Creek Fault	13
Sandy River Fault	22
Mount Angel Fault	31
Newberg Fault	23
Gales Creek Fault	21

Seismic and geologic parameters such as slip rate, horizontal and vertical offset, rupture length, and geologic age have not been determined for the majority of the above faults. This is primarily due to the lack of surface expressions or exposures of faulting because of urban development and the presence of late Quaternary soil deposits that overlie the faults. The low level of historical seismicity (particularly for earthquakes greater than magnitude 5) and lack of paleo-seismic data results in large uncertainties when evaluating individual crustal fault maximum magnitude earthquakes and recurrence intervals. Thus it is considered prudent to also evaluate the potential for seismic shaking due to crustal earthquakes on a regional scale. Based on data presented by Geomatrix (1995) and DOGAMI (1991) the seismic exposure at the site from crustal zone sources is represented by an earthquake of magnitude 6.5.

3.2.2. Historical Seismicity

The historical seismicity of the site and the vicinity was determined based upon the review of the September 1993 and November 1995 issues of Oregon Geology, Bureau of Reclamation Scoggins Dam Seismic Study, and on the analysis of the 150 year Oregon earthquake catalog, DOGAMI Open-File Report O-94-4. OFR O-94-4 is a database of 15,000 Oregon earthquakes that occurred between 1833 and October 25, 1993. In order to establish an estimated Richter Magnitude for those seismic events that do not have such a recording, the Gutenberg and Richter, 1965 relationship, M = (2/3) MMI +1, was applied to those earthquakes that only had a Modified Mercalli Intensity (MMI). The MMI scale is a means of estimating the size of an earthquake using human observations and reactions to the earthquake. The MMI scale ranges from I to XII, with XII representing the highest intensity. A search of the database was conducted to determine the number and estimated magnitude of earthquakes that have taken place within 50 kilometers of the site. The information derived from the Oregon earthquake catalog indicates that several M5.0 to M5.7 earthquakes occurred within the search zone.

3.2.3. Relative Seismic Hazards

The Oregon Department of Geology and Mineral Industries has prepared earthquake hazard maps for the Portland Metropolitan Area. These maps were developed for regional evaluation and are not intended to be used in lieu of site-specific seismic evaluations.

Three seismic hazards were evaluated as a part of Interpretive Map Series IMS-1 (1997): amplification, liquefaction, and slope instability. Each hazard was divided into three or four categories with the greatest hazard given a value of 3 and the least hazard give a value of 1 or 0. The relative amplification hazard for the site is mapped within Zone 2. Amplification Zone 2 is characterized as areas with moderate amplification hazard. The site lies within the liquefaction hazard Zone 2 and within Zone 0 for slope instability hazard.

4. SUBSURFACE EXPLORATION AND CONDITIONS

4.1. Site Exploration

The subsurface soil conditions at the Legacy Emanuel Burn Center site were determined based on the literature review, previous work in the area, field exploration, and laboratory testing. The field exploration was completed on November 15, 2000 and consisted of a site reconnaissance and two exploratory borings.

The borings were advanced to depths of approximately 31.5 feet bgs with a trailer mounted drill rig equipped with solid stem auger. A member of Geocon Northwest's geotechnical engineering staff logged the subsurface conditions encountered within the borings. Standard penetration tests (SPT) were performed at regular intervals by driving a 2-inch outside diameter split spoon sampler 18 inches into the bottom of the boring, in general accordance with ASTM D 1586. The number of blows to drive the sampler the last 12 of the 18 inches are reported on the boring logs located in Appendix A at the end of this report. Disturbed bag samples were obtained from SPT testing. Drilling service providers subcontracted by Geocon Northwest completed the borings. Approximate exploration locations are shown in Figure 2, Site Plan.

4.2. Subsurface Conditions

The subsurface explorations were widely spaced across the site and it is possible that some local variations and possible unanticipated subsurface conditions exist. Based on the conditions observed during the reconnaissance and field exploration, the subsurface conditions, in general, consisted of the following:

TOPSOIL – Approximately four inches of topsoil was encountered within the borings.

FILL- Non-engineered fill was encountered below the topsoil in Boring B-2, located near the proposed elevator. Fill was encountered to a depth of approximately 14 feet bgs.

SANDY SILT/SILTY SAND- Interbeds of varying thicknesses of medium stiff to stiff, moist, light brown, fine-grained sandy silt and medium dense, light brown, silty fine-grained sand were encountered below the topsoil in Boring B-1 and below the fill in Boring B-2. This deposit extended to a depth of approximately 25 to 26 feet below the ground surface.

SAND – Dense, wet, dark gray, medium-grained sand was encountered below the sandy silt and silty sand layers. The borings were terminated within this deposit.

Subsurface conditions encountered during the field investigation appear to be consistent with the geologic conditions mapped within the region.

GROUNDWATER – Static groundwater was not encountered during the field investigation.

5. SEISMIC HAZARD EVALUATION

The primary geologic hazards associated with earthquakes are liquefaction, settlement, lateral spreading, fault rupture, landsliding, ground shaking, amplification, and seiche/tsunami. For many of these potential hazards, the subsurface conditions and topography of a site will dictate how a site will likely perform during a seismic event. Liquefaction typically takes place in loose, saturated sand. Seismic induced settlement generally occurs in loose granular soil. Lateral spreading is a form of slope failure that occurs in liquefiable sediments adjacent to an open-face (e.g. riverbank). Seismic induced landsliding generally takes place in over-steepened slopes that are at or near static equilibrium prior to the event. The level of ground shaking at a given site will depend on the magnitude of the seismic event and the distance from the source. Typically the level of ground shaking will attenuate as it propagates away from the source. However, depending on the earthquake motion characteristics and the subsurface conditions at the site, the level of ground shaking can be increased due to amplification. Seiche and tsunami hazards are seismically induced waves in lakes or inland bodies of water and oceans, respectively.

5.1. Ground Shaking

In their recent study, Geomatrix (1995) estimated peak bedrock horizontal accelerations in Portland of 0.19g, 0.27g, and 0.37g for return periods of 500, 1000, and 2500 years, respectively. The analyses were based on a total mean hazard comprised of crustal and subduction zone sources. The majority of these bedrock accelerations were attributed to crustal earthquake sources.

Based on the subsurface conditions encountered during the field investigation, it is recommended that a soil profile type S_D be used for design. A seismic zone factor of 0.3 for

Zone 3 should be used for seismic design. Seismic design coefficients of C_a equal to 0.36 and C_v equal to 0.54 are recommended.

5.2. Fault Displacement and Subsidence

Based on the literature review, identified faults were not mapped within the boundaries of the site or within adjacent properties. The Oregon Department of Geology and Mineral Industries (2000) has mapped the East Bank Fault, an individual fault of the Portland Hills Fault System, within one mile of the site. Field studies indicate an offset of the Pliocene age Troutdale Formation in the location of the mapped fault. There is no evidence of faulting in the more recent Pleistocene and Holocene age alluvial deposits, dating the most recent activity of the fault at approximately 1.6 million years ago. Reactivation of the fault is considered doubtful. Evidence was not encountered during the field investigation to suggest the presence of faults within the property boundaries. The potential for fault displacement and associated ground subsidence at the site is considered remote.

5.3. Slope Instability

Earthquake induced landslides generally occur on steep slopes composed of weak soil or bedrock. Among the factors that influence seismic induced landsliding include earthquake intensity, topographic relief, ground water, and soil or bedrock type. Earthquakes can also reactivate existing landslides. Based on the topography and field observations, the site is estimated to have low earthquake induced slope instability hazard.

5.4. Liquefaction

Liquefaction can cause aerial and differential settlement, lateral spreading, and sudden loss in soil strength. Soils prone to liquefaction are typically loose, saturated sands and, to a lesser degree, silt. These soils are generally young alluvial deposits and can be found along waterways such as the Willamette River. Due to the depth of groundwater, the natural moisture content of the near surface soils is less than saturation, and liquefaction potential at the site is considered negligible.

5.5. Lateral Spreading

Lateral spreading is a liquefaction related seismic hazard that may adversely impact some sites. Areas subject to lateral spreading are underlain by liquefiable sediments and are sloping sites or are flat sites adjacent to an open face. Because the site has low potential for liquefaction, it is estimated that the site has negligible potential for lateral spreading.

5.6. Seiche and Tsunami Inundation

There is not a potential for seiche- and tsunami-related damage at the site due to the distance of the site from waterways, lakes, and coastal areas.

6. LABORATORY TESTING

Laboratory testing was performed on selected soil samples to evaluate in situ moisture content. Laboratory determination of compaction characteristics was completed for earlier phases of work at Legacy Emanuel. Visual soil classification was performed both in the field and laboratory, in general accordance with the Unified Soil Classified System. Moisture content determinations (ASTM D2216) were performed on soil samples to aid in classifying the soil. Moisture contents are indicated on the boring logs, which are located in Appendix A of this report.

Test results of compaction characteristics are reproduced in Appendix B of this report.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1. General

7.1.1. It is our opinion that the proposed project is geotechnically feasible, provided the recommendations of this report are followed.

7.1.2. Non-engineered fill, to a depth of approximately 14 feet, was encountered in Boring B-2, near the proposed location of the elevator. Recommendations regarding the presence of fill in structural areas are provided in subsequent sections.

7.1.3. Moisture contents of near surface soils were wet of optimum at the time of the investigation. Recommendations for both dry and wet weather construction in moisture sensitive soils are provided, however, dry weather construction at this site is recommended. If construction is to occur during wet weather, it would be prudent to allow for cost increases for construction expenses associated with the subgrade stabilization techniques described in the wet weather construction paragraphs of this report.

7.2. Site Preparation

7.2.1. Prior to beginning construction, the areas of the site to receive fill, footings or pavement should be stripped of concrete, asphalt, vegetation, topsoil, non-engineered fill, previous subsurface improvements, debris, and otherwise unsuitable material, down to firm native soil. Excavations made to remove previous subsurface improvements should be backfilled with structural fill per Section 7.4 of this report.

7.2.2. Subsurface explorations indicate that approximately 14 feet of non-engineered fill is present in the proposed location of the elevator. Discussions with the project architect and structural engineer indicate that an excavation of approximately 14 to 16 feet is anticipated for elevator construction. Non-engineered fill that is present at proposed subgrade elevation of the elevator pit should be overexcavated to firm native soil and backfilled in accordance with Section 7.4 of this report. Although fill was not encountered in Boring B-1, it may be present in variable and unanticipated locations across the site. If non-engineered fill is encountered at subgrade level within the proposed ambulance entrance, such fill should be overexcavated a minimum of two feet and backfilled in accordance with Section 7.4. It is recommended that a member of Geocon Northwest's geotechnical staff be on site during site preparation to confirm adequate subgrade preparation.

7.2.3. Dry Weather Construction

Subgrades in pavement and structural areas that have been disturbed during stripping or cutting operations should be scarified to a depth of at least eight inches. The scarified soil should be moisture conditioned as necessary to achieve the proper moisture content, then compacted to at least 92% of the maximum dry density as determined by ASTM D-1557. Minimum compaction for the eight inches immediately underlying pavement sections should be 95%. Even during dry weather it is possible that some areas of the subgrade will become soft or may "pump," particularly in poorly drained areas. Soft or wet areas that cannot be effectively dried and compacted should be prepared in accordance with Section 7.2.4.

7.2.4. Wet Weather Construction

During wet weather, or when adequate moisture control is not possible, it may be necessary to install a granular working blanket to support construction equipment and provide a firm base on which to place subsequent fills and pavements. Commonly, the working blanket consists of a bank run gravel or pit run quarry rock (six to eight inch maximum size with no more that 5% by weight passing a No. 200

sieve). A member of Geocon Northwest's engineering staff should be contacted to evaluate the suitability of the material before installation.

The working blanket should be installed on a stripped subgrade in a single lift with trucks end-dumping off an advancing pad of granular fill. After installation, the working blanket should be compacted by a minimum of four complete passes with a moderately heavy static steel drum or grid roller. It is recommended that Geocon Northwest be retained to observe granular working blanket installation and compaction.

Construction practices can affect the amount of work pad necessary. By using tracked equipment and special site access roads, the work pad area can be minimized. Normally, the design, installation and maintenance of a work pad are the responsibility of the contractor.

7.3. Proof Rolling

7.3.1. Regardless of which method of subgrade preparation is used (i.e. wet weather or dry weather), it is recommended that, prior to fill placement or base course installation, the subgrade or granular working blanket be proof-rolled with a fully-loaded 10- to 12-yard dump truck. Areas of the subgrade that pump, weave or appear soft or muddy should be scarified, dried and compacted, or overexcavated and backfilled with structural granular fill per Section 7.4. If a significant length of time passes between fill placement and commencement of construction operations, or if significant traffic has been routed over these areas, the subgrade should be similarly proof-rolled before final placement of asphalt or concrete. It is recommended that a member of our geotechnical engineering staff observe the proof-roll operation.

7.4. Fills

7.4.1 Structural fills should be constructed on a subgrade that has been prepared in accordance with the recommendations in Section 7.2 of this report. Structural fills should be placed in horizontal lifts not exceeding about eight inches in thickness, and should be compacted to at least 92% of the maximum dry density for the native silt soils, and 95% for imported granular material. The top eight inches of foundation subgrade should be compacted to 95%, regardless of the material type. Compaction should be referenced to ASTM D-1557 (Modified Proctor). The compaction criteria may be reduced to 85% in landscape, planter or other non-structural areas.

7.4.2. During dry weather when moisture control is possible, structural fills may consist of native material, free of topsoil, debris and organic matter, which can be compacted to the preceding specifications. However, if excess moisture causes the fill to pump or weave, those areas should be scarified and allowed to dry, and then be recompacted, or removed and backfilled with compacted granular fill as discussed in Section 7.2 of this report.

7.4.3. The native, non-organic silty sand to sandy silt would generally be acceptable for structural fills if properly moisture conditioned. Near-surface moisture contents at the time of the field investigation ranged from approximately 23.7% to 31.6%. Based on past experience, moisture contents for the near-surface silty soils should be reduced to approximately 13% to 15% for compaction.

7.4.4 During wet-weather grading operations, Geocon Northwest recommends that fills consist of well-graded granular soils (sand or sand and gravel) that do not contain more than 5% material by weight passing the No. 200 sieve. In addition, it is usually desirable to limit this material to a maximum six inches in diameter for future ease in the installation of utilities.

7.5. Temporary Cut Slopes and Shoring

7.5.1. It is anticipated that the elevator excavation will be approximately 14 to 16 feet below existing grade. Temporary cut slopes should be sloped no steeper than 1H:1V. These values assume that the slopes will be protected from erosion and that significant drainage will not occur over the face of the slope. They further assume that no loads will be imposed within a horizontal distance of one-half of the slope height measured from the top of the slope face. Cut slopes should be constructed with a smooth bucket excavator to minimize subgrade disturbance. Slope drainage may be required if springs, seeps, or groundwater are encountered. Cut slopes greater than those anticipated should be designed on an individual basis.

7.5.2. It is understood that the elevator excavation may extend two to four feet below the existing basement floor. The portion of the excavation extending below the depth of the basement floor may require a temporary shoring system. The necessity for shoring (or underpinning) will depend on the horizontal and vertical distances of the elevator excavation from the existing structure. Geocon Northwest should be contacted once elevator plans have been finalized to evaluate the need for shoring and to provide recommendations if shoring is required.

7.6. Surface and Subsurface Drainage

7.6.1. During site contouring, positive surface drainage should be maintained away from foundation and pavement areas. Additional drainage or dewatering provisions may be necessary if soft spots, springs, or seeps are encountered in subgrades. Where possible, surface runoff should be routed independently to a storm water collection system.

7.6.2. It is recommended that the elevator structure be provided with waterproofing along the base and sides of the structure.

7.6.3. Drainage and dewatering systems are typically designed and constructed by the contractor. Failure to install necessary subsurface drainage provisions may result in premature foundation or pavement failure.

7.7. Foundations

7.7.1. Spread and perimeter foundation support for proposed structures may be obtained from the native silt soil or from structural fill installed in accordance with our recommendations. Non-engineered fill encountered below subgrade elevation should be excavated and backfilled in accordance with Section 7.4 of this report.

7.7.2. Based on discussions with kpff, structural engineers for the project, it is anticipated that uplift loads associated with the ambulance entrance canopy will be resisted by the mass of the foundations. Spread and perimeter footings that are at least 18 inches wide and embedded a minimum of 18 inches within firm native soils or engineered fill may be designed for an allowable bearing capacity of 3200 psf. An allowable bearing capacity of 4000 psf may be used for the design of footings that are to be embedded five feet or more below existing grade.

7.7.3. The allowable bearing pressures given above may be increased by one-third for short-term, transient loading, such as wind or seismic forces.

7.7.4. Lateral loads may be resisted by sliding friction and passive pressures. A base friction of 40% of the vertical load may be used against sliding. An equivalent fluid weight of 350 pcf may be used to evaluate passive resistance to lateral loads.

7.7.5. Foundation settlements for the loading conditions expected for this project are estimated to be less than one inch, with not more than one-half inch occurring as differential settlement.

7.8. Concrete Slabs-on-Grade

7.8.1. Subgrades in floor slab areas should be prepared in accordance with Section 7.2 of this report. Floor slab areas should be proof-rolled with a fully loaded 10- to 12-yard dump truck to detect areas that pump, weave, or appear soft or muddy. When detected, these areas should be overexcavated and stabilized with compacted granular fill.

7.8.2. A minimum six-inch thick layer of compacted ¾-inch minus material should be installed over the prepared subgrade to provide a capillary barrier and to minimize subgrade disturbance during construction. The crushed rock or gravel material should be poorly-graded, angular and contain no more than 5% by weight passing the No. 200 Sieve.

7.8.3. A modulus of subgrade reaction of 150 pci is recommended for design.

7.9. Retaining Walls and Lateral Loads

7.9.1. The tables presented in the following sections summarize the recommendations for design of retaining structures. These values represent estimates of the long-term pressures that will develop in an active or at-rest state of stress. These values do not include an allowance for hydrostatic pressures and assume that retaining structures will be provided with a drainage system in accordance with subsequent sections of this report. The design parameters in the following sections are for conventional retaining walls and do not include a factor of safety. They also do not include loading from traffic or other surcharges.

7.9.2. Restrained walls are those that are prevented from rotating more than 0.001H (where H equals the height of the retaining wall portion of the wall in feet) at the top of the wall. Most basement walls and walls that are rigidly connected to buildings or that make sharp bends fall into this category. Restrained walls should be designed for pressures derived from the criteria provided in Table 2.

Table 2: Restrained Wall Design Criteria

Backfill Slope H:V	Equivalent Fluid Weight lb/ft^3
Level	65
3H:1V	80
2H:1V	105

7.9.3. Non-restrained walls are not restrained at the top and are free to rotate about the base. Most cantilever retaining walls fall into this category. Non-restrained walls should be designed for pressures derived from the criteria provided in Table 3.

Table 3: Non-Restrained Wall Design Criteria

Backfill Slope H:V	Equivalent Fluid Weight lb/ft^3
Level	40
3H:1V	50
2H:1V	65

7.9.4. Retaining wall backfill should consist of free-draining granular material. To minimize pressures on retaining walls, the use of open-graded crushed rock backfill with less than 5% by weight passing the No. 200 Sieve is recommended. Retaining wall backfill should be compacted to 90% of ASTM D1557. Backfill, within approximately five feet of retaining structures, should be compacted with lightweight hand operated equipment. Use of other material and/or over-compaction of the backfill could increase wall pressures.

7.9.5. If backfill is in direct contact with the wall, pressures against the back of the wall can be assumed to act at a downward inclination of 20 degrees from the horizontal. If friction is prevented by drainage membranes or water proofing membranes, the pressures should be assumed to act horizontally.

7.9.6. Foundations or major loads should not be placed in a zone that extends back from the base of a retaining wall at a 1H:1V slope.

7.9.7 Retaining walls should be provided with drainage in order to alleviate lateral hydrostatic pressures that may accumulate behind the wall. Retaining wall drains should be positioned near the base of the retaining wall and should be protected by a filter fabric to prevent internal soil erosion and potential clogging.

7.10. Pavement Design

7.10.1. Near surface soil samples were evaluated to determine pavement design parameters. A CBR of 3 at 95% compaction and a resilient modulus of 4,500 were used for pavement design.

7.10.2. Alternate pavement designs for both asphalt and portland cement concrete (pcc) are presented in Tables 4 and 5. Pavement designs have been prepared in accordance with accepted AASHTO design methods. A range of pavement designs for various traffic conditions is provided in the tables. The designs assume that the top eight inches of pavement subgrade will be compacted to 95% ASTM D-1557. Specifications for pavement and base course should conform to current Oregon State Department of Transportation specifications. Additionally, the base rock should contain no more than 5% by weight passing a No. 200 Sieve, and the asphaltic concrete should be compacted to a minimum of 91% of ASTM D2041.

Table 4: Asphalt Concrete Pavement Design

Approx. Number of Trucks per Day (each way)	Approx. Number of 18 Kip Design Axle Load (1000)	Asphalt Concrete Thickness (inches)	Crushed Rock Base Thickness (inches)
Auto Parking	10	2.5	8
5	22	3.0	8
10	44	3.0	10
15	66	3.5	10
25	110	4.0	10
50	220	4.0	12
100	440	4.5	12
150	660	5.0	13

Table 5: Portland Cement Concrete Pavement Design

Approx. Number of Trucks per Day (each way)	Approx. Number of 18 Kip Design Axle Load (1000)	P.C.C. Thickness (inches)	Crushed Rock Base Thickness (inches)
25	110	6.0	6
50	220	7.0	6
100	440	8.0	6
150	660	8.5	6
200	880	8.5	6
250	1100	9.0	6

Pavement sections were designed using AASHTO design methods, with an assumed reliability level (R) of 90%. Terminal serviceability of 2.0 for asphaltic concrete, and 2.5 for portland cement concrete were assumed. The 18 kip design axle loads are estimated from the number of trucks per day using State of Oregon typical axle distributions for truck traffic and AASHTO load equivalency factors, and assuming a 20 year design life. The concrete designs were based on a modulus of rupture equal to 550 psi, and a compressive strength of 4000 psi. The concrete sections assume plain jointed or jointed reinforced sections with no load transfer devices at the shoulder.

7.10.3. If possible, construction traffic should be limited to unpaved and untreated roadways, or specially constructed haul roads. If this is not possible, the pavement design should include an allowance for construction traffic.

8. FUTURE GEOTECHNICAL SERVICES

The analyses, conclusions and recommendations contained in this report are based on site conditions as they presently exist, and on the assumption that the subsurface investigation locations are representative of the subsurface conditions throughout the site. It is the nature of geotechnical work for soil conditions to vary from the conditions encountered during a normally acceptable geotechnical investigation. While some variations may appear slight, their impact on the performance of structures and other improvements can be significant. Therefore, it is recommended that Geocon Northwest be retained to observe portions of this project relating to geotechnical engineering, including site preparation, grading, compaction, foundation construction and other soils related aspects of construction. This will allow correlation of observations and findings to actual soil conditions encountered during construction and evaluation of construction conformance to the recommendations put forth in this report.

A copy of the plans and specifications should be forwarded to Geocon Northwest so that they may be evaluated for specific conceptual, design, or construction details that may affect the validity of the recommendations of this report. The review of the plans and specifications will also provide the opportunity for Geocon Northwest to evaluate whether the recommendations of this report have been appropriately interpreted.

9. LIMITATIONS

Unanticipated soil conditions are commonly encountered during construction and cannot always be determined by a normally acceptable subsurface exploration program. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon Northwest should be notified so that supplemental recommendations can be given.

This report is issued with the understanding that the owner, or his agents, will ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans.

The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review should such changes occur.

If you have any questions regarding this report, or if you desire further information, please contact the undersigned at (503) 626-9889.

GEOCON NORTHWEST, INC.

Heather Devine

Heather Devine
Engineering Staff

Wesley Spang

Wesley Spang, Ph.D., P.E.
President

REFERENCES CITED

Bott, D.J., Wong, I.G., September 1993, "Historical Earthquakes in and Around Portland, Oregon," Oregon Geology, Vol. 55, No. 5

Geomatix, 1995, "Seismic Design Mapping, State of Oregon," prepared for Oregon Department of Transportation

Mabey, M.A., Black, G.L., Madin, I.P., Meier, D.B., Youd, T.S., Jones, C.F., Rice, J.B., 1997, IMS-1, Interpretive Map Series, "Relative Earthquake Hazard Map of the Portland Metro Region, Clackamas, Multnomah, and Washington Counties, Oregon"

Oregon Department of Geology and Mineral Industries, 2000, IMS-15, "Earthquake Scenario and Probabilistic Ground Shaking Maps for the Portland, Oregon Metropolitan Area. Portland Hills Fault M6.8 Earthquake, Peak Horizontal Acceleration (g) at the Ground Surface"

Oregon Department of Geology and Mineral Industries, 1994, Open-File Report O-94-4.

Oregon Department of Geology and Mineral Industries, 1991, GMS-75, "Geologic Map of the Portland Quadrangle, Multnomah and Washington Counties, Oregon, and Clark County, Washington"

Peterson, C.D., Darienzo, M.E., Burns, S.F., and Burris, W.K., September 1993, "Field Trip to Cascadia Paleoseismic Evidence Along the Northern Oregon Coast: Evidence of Subduction Zone Seismicity in the Central Cascadia Margin," Oregon Geology, Vol. 55, No. 5

Wong, I.G., Bott, D.J., November 1995, "A look Back at Oregon's Earthquake History, 1841-1994," Oregon Geology, Vol. 57, No. 6

United States Department of Interior, Bureau of Reclamation, April 1994, "Seismictectonic Evaluation Scoggins Dam Tualatin Project Final Report"

APPENDIX A

FIELD INVESTIGATION

The field investigation was performed on November 15, 2000, and consisted of a site reconnaissance and two exploratory borings. The approximate locations of the borings are shown in Figure 2.

Borings were drilled with a trailer mounted, solid stem auger. Standard penetration tests (SPT) were conducted at regular intervals within the borings. Disturbed bag samples were collected with a split spoon sampler and returned to the laboratory for further testing.

Subsurface logs of the conditions encountered are presented in the following pages.

PROJECT NO. P1009-05-08

BORING B 1

ELEV. (MSL.) ____________ DATE COMPLETED 11/15/00

EQUIPMENT TRLR MTD SOLID STEM

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					Approximately 4 inches TOPSOIL **WILLAMETTE SILT** Medium stiff to stiff, moist to wet, brown, SILT, trace clay, trace fine grained sand			
2	B1-1					6		23.7
4	B1-2					9		31.6
8	B1-3					14		26.3
10	B1-4				-Increasing fine grained sand	13		21.3
12								
14								
15	B1-5				Medium dense, moist, brown, Silty, fine grained SAND	11		27.9
20	B1-6					21		21.3
22								
24								34.2
26	B1-7				Stiff, saturated, brown, SILT	14		
27					Dense, wet, gray, medium grained SAND			
30	B1-8					37		10.7

BORING TERMINATED AT 31.5 FEET
STATIC GROUNDWATER WAS NOT ENCOUNTERED

Figure A-1, Log of Boring-B 1

LEBC

SAMPLE SYMBOLS
- ☐ ... SAMPLING UNSUCCESSFUL
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◧ ... STANDARD PENETRATION TEST
- ◪ ... CHUNK SAMPLE
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. P1009-05-08

BORING B 2

ELEV. (MSL.) ____________ DATE COMPLETED 11/15/00

EQUIPMENT TRLR MTD SOLID STEM

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					Approximately 4 inches TOPSOIL			
2					**FILL** Soft to medium stiff, wet, brown, Clayey SILT			
	B2-1					4		30.0
4								
	B2-2				-Saturated at 5 feet	4		26.7
6								
8	B2-3					7		32.8
					Medium stiff, moist to wet, brown/gray, SILT, some clay, trace fine grained sand			
10	B2-4					4		30.2
12								
14					**WILLAMETTE SILT** Medium stiff to stiff, moist, brown, fine grained Sandy SILT to medium dense, moist, brown, Silty, fine grained SAND			
	B2-5					9		25.7
16								
18								
20	B2-6					10		27.9
22								
24								
	B2-7A					9		23.5
26	B2-7B				Stiff, saturated, brown, SILT			38.6
					Dense, wet, gray, medium grained SAND			
28								
30	B2-8					36		12.1
					BORING TERMINATED AT 31.5 STATIC GROUNDWATER WAS NOT ENCOUNTERED			

Figure A-2, Log of Boring-B 2

LEBC

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	⊠ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

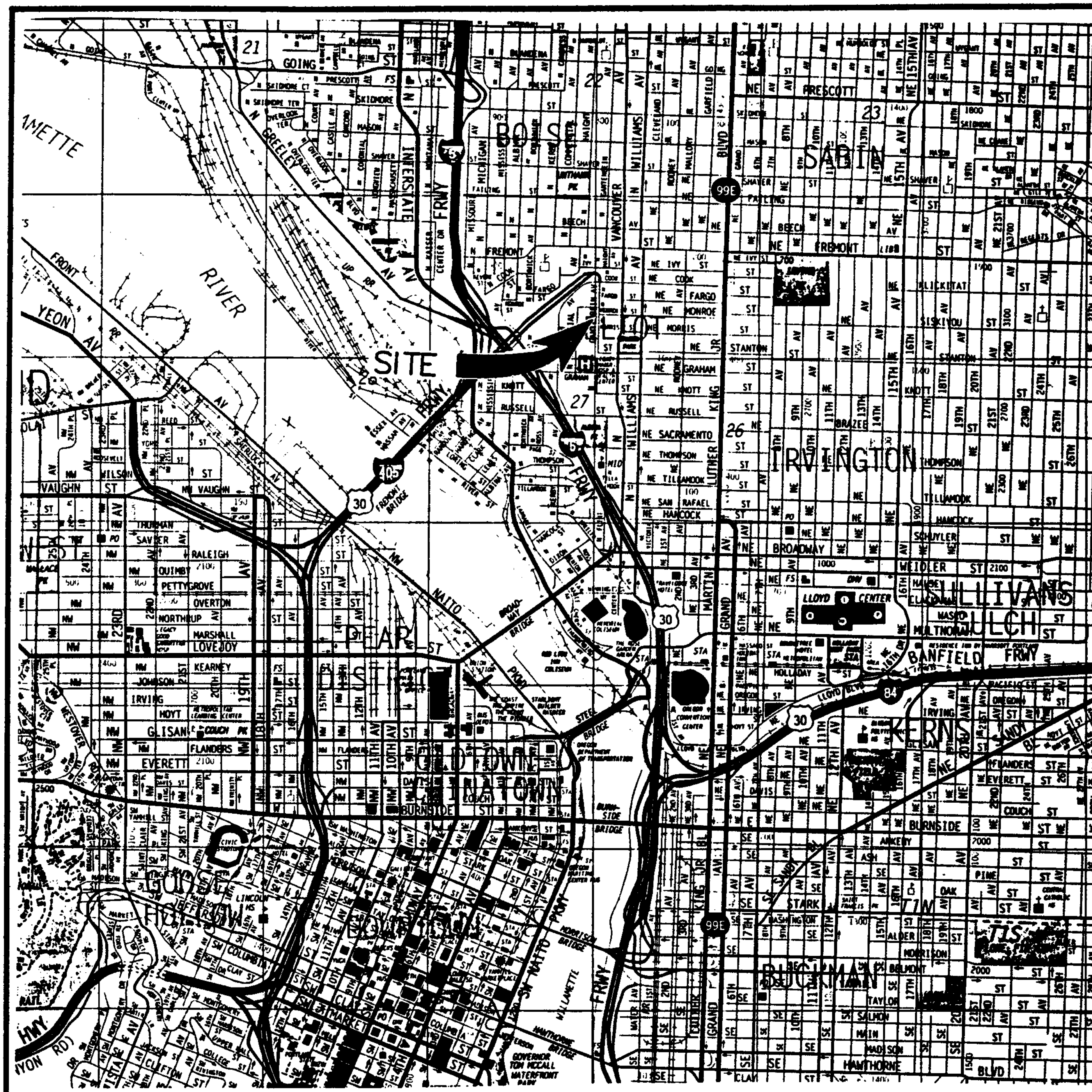
APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected soils samples were tested for in situ moisture content and compaction characteristics.

TABLE B-1
SUMMARY OF COMPACTION CHARACTERISTICS
ASTM D 1557

Sample No.	Depth (ft)	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
Composite	1-2	117	13



SOURCE: 2000 THOMAS BROTHERS MAP
PORTLAND METROPOLITAN AREA

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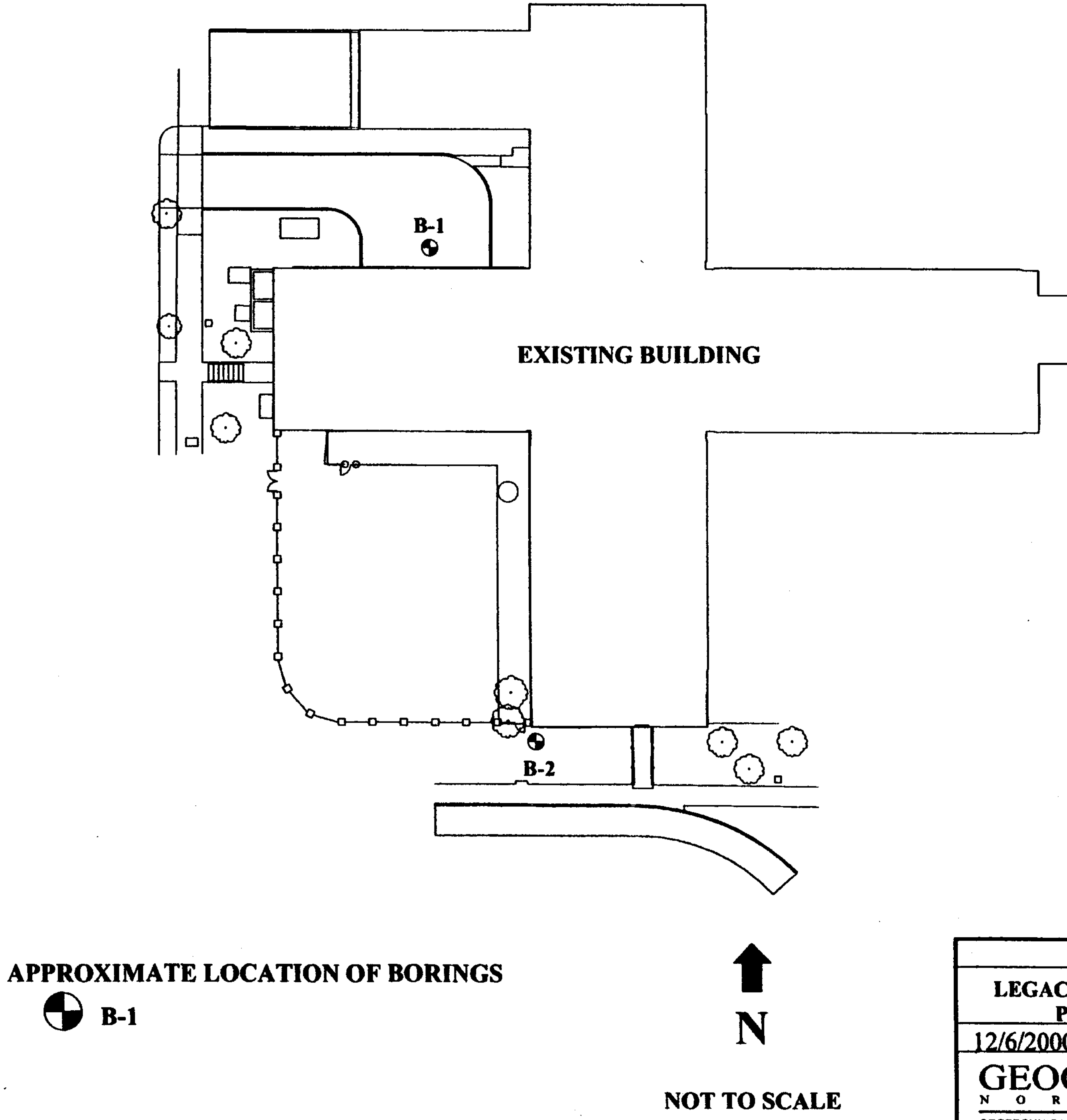
SITE VICINITY

LEGACY EMANUEL BURN CENTER

PORTLAND, OREGON

HD / RSS		DSK / D000D	12/6/2000	P1009-05-08	FIG. 1



SITE PLAN

LEGACY EMANUEL BURN CENTER
PORTLAND, OREGON

12/6/2000	P1009-05-08	FIGURE 2

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