Report

Geotechnical Engineering Services

Proposed Church Addition

Port Townsend Ward

Chimacum, Washington

August 12, 1993

For

The Church of Jesus Christ

of Latter-Day Saints

GeoEngineers

August 12, 1993

Geotechnical, Geoenvironmental and Geologic Services

The Church of Jesus Christ of Latter-Day Saints
c/o Gary W. Hochstrasser Associates, Architects
4115 Bridgeport Way West
Tacoma, Washington 98466

Attention: Mr. Gary W. Hochstrasser

We are pleased to submit 18 copies of our report of geotechnical engineering services for the proposed church addition at the Port Townsend Ward meetinghouse in Chimacum, Washington. Our services were conducted in accordance with the scope described in our confirming agreement dated July 12, 1993. We discussed our findings and preliminary conclusions and recommendations with Mr. Gary Warner of Warner Engineering, the project civil engineer, on August 3, 1993.

We appreciate the opportunity to be of service to you on this project. Please contact us if you have any questions regarding our findings and report.

Yours very truly,

GeoEngineers, Inc.

Kenneth G. Buss

Kenneth G. Buss, P.E.
Associate

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GeoEngineers, Inc.
8410 154th Avenue N.E.
Redmond, WA 98052
Telephone (206) 861-6000
Fax (206) 861-6050

Printed on recycled paper

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REPORT
GEOTECHNICAL ENGINEERING SERVICES
PROPOSED CHURCH ADDITION
PORT TOWNSEND WARD
CHIMACUM, WASHINGTON
LDS PROJECT NO. 516-5520

INTRODUCTION

This report presents the results of our geotechnical engineering services for the proposed church building addition and associated parking lot expansion at the existing Port Townsend Ward meetinghouse. The site is located at 4141 Rhody Drive in Chimacum, Washington. The location of the site is shown on the Vicinity Map, Figure 1.

Based on information provided by Mr. Gary Hochstrasser of Gary W. Hochstrasser Associates, we understand that the proposed addition will be a single story, 4,898 square-foot, wood frame structure housing a chapel, foyers and offices. Foundation and floor loads are expected to be relatively light. The proposed addition will be attached to the east end of the existing church building, in an area presently occupied by a lawn and gravel-surfaced driveway. We understand that no modifications of the existing septic system are planned. A new parking area covering approximately 25,000 square feet will extend east from the existing parking area around the north, east and south sides of the proposed church addition. Also, a new driveway entrance is to be constructed at the southwest corner of the existing parking area. From information provided by Mr. Gary Warner of Warner Engineering, the project civil engineer, we understand that a new stormwater drainfield is to be located along the east side of the proposed new parking area.

SCOPE

The purpose of our services is to explore surface and subsurface soil and ground water conditions at the site to develop geotechnical recommendations for site development, foundation and pavement design. The specific scope of our services includes the following:

1. Explore subsurface conditions in the areas of the proposed addition, parking lot and driveway entrance by drilling a total of nine test borings to depths ranging from approximately 5 to 7 feet using a small power auger and hand tools.
2. Perform two percolation tests near the new stormwater drainfield location, in general accordance with the EPA (U.S. Environmental Protection Agency) falling head percolation test procedure.
3. Evaluate pertinent physical and engineering properties of foundation and subgrade soils based on classification, laboratory moisture content, and gradation tests performed on soil samples obtained from the explorations.

4. Provide recommendations for earthwork including stripping, removal of soft, organic or other unsuitable material, grading and backfill compaction, and subgrade requirements for support of slab-on-grade floors. This includes evaluating the effects of weather and construction equipment on site soils.
5. Develop foundation design requirements including allowable soil bearing pressures and settlement estimates for shallow spread footings.
6. Provide lateral earth pressures for design including active pressures for below-grade walls, passive earth pressures on footings and the coefficient of base friction against sliding.
7. Provide recommendations for pavement design.
8. Provide recommendations for surface and subsurface drainage systems based on the soil and ground water conditions encountered in the explorations, including soil infiltration rates for use in design of the stormwater drainfield.
9. Present the results of our study in a written report in general accordance with the Church's Project Soil Information Guidelines dated January 1993.

SITE CONDITIONS

SURFACE CONDITIONS

The church property is relatively flat and is situated in a broad valley at an elevation of approximately 120 feet above sea level (U.S. Geological Survey datum). The existing church building and parking areas are located on the western portion of the site, as shown on the Site Plan, Figure 2. The existing entrance driveway is located near the northwest corner of the site and provides access to an asphalt paved parking area on the north side of the church building. A gravel-surfaced driveway is located on the east side of the church building and connects the north parking lot with an asphalt paved parking area on the south side of the church building. A lawn area at approximately the same grade as the existing church floor is situated between the church building and the gravel driveway. The south parking lot is bordered to the south by a strip of dense vegetation that includes evergreen trees with 12-inch trunk diameters and brush.

The eastern portion of the site consists of open grassy areas and scattered groves of large evergreen trees and brush. We observed several picnic tables and water faucets in this area, and understand that this portion of the site is presently used as a picnic area and campground. A storage building is located near the north site boundary.

The site is generally well drained based on the type of native vegetation observed on the site and surrounding areas. No streams, seeps, or springs were observed on the church property at the time of our site visit on July 28, 1993.

SUBSURFACE CONDITIONS

Subsurface soil and ground water conditions at the site were explored on July 28, 1993 by drilling a total of nine test borings to depths ranging from approximately 5 to 7 feet using a small power auger and hand tools. Borings B-1 through B-3 were drilled within the footprint of the

proposed addition, B-4 through B-8 were drilled within the proposed new parking area, and B-9 was drilled at the location of the proposed new driveway entrance. The relatively small number of borings drilled for the building addition was based on the size of the addition and our knowledge of the relatively uniform glacial outwash soils that underlie the site vicinity. The approximate locations of the borings are shown in Figure 2. The boring logs, geotechnical laboratory test results, and descriptions of the subsurface exploration and geotechnical laboratory testing programs are presented in the Appendix.

The subsurface conditions, as encountered in our borings, are relatively uniform across the site. In general, the borings encountered a surficial layer of sod and topsoil ranging from approximately 2 to 6 inches thick. Boring B-3, which is located within the gravel-surfaced driveway, encountered gravel surfacing approximately 4 inches thick. The underlying soils encountered in the borings generally consist of granular, loose to medium dense recessional outwash material deposited in streams associated with the melting and retreat of glaciers from the area approximately 13,000 years ago. Weathered outwash generally consisting of loose to medium dense sand with silt and occasional gravel and roots typically extends to depths ranging from approximately 1 to 2.5 feet. The weathered material is underlain by unweathered, medium dense gravelly sand with a trace of silt that extends to the completion depths of the borings. The outwash material encountered in B-6 has a higher percentage of gravel than the material encountered in the other borings.

The shallow weathered material encountered in B-8 includes ash and charcoal fragments. Based on discussions with Mr. Cecil Quakenbush, an LDS Church representative with historical knowledge of the Chimacum church, we understand that the site was formerly used to generate charcoal for a nearby smelter. Mr. Quakenbush indicated that numerous pockets of ash with charcoal and/or partially burned organic material are likely scattered across the site.

Ground water seepage was not encountered in any of the borings. We expect that the regional ground water table is several tens of feet deep in the vicinity of the site. In general, ground water conditions at the site should be expected to fluctuate as a function of precipitation, season and other factors.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

We conclude that geotechnical site conditions are favorable for development of the site as proposed. Based on the results of our study, it is our opinion that the proposed church addition may be satisfactorily supported on conventional shallow spread footings bearing on medium dense native sand and gravel. It is also our opinion that the unweathered, on-site native soils are adequate to be used for any structural fill that might be needed for site grading. Our specific geotechnical design recommendations for the proposed church addition, parking lot, new entrance driveway and stormwater drainfield are presented in the following sections of this report.

SITE PREPARATION

The existing topsoil, vegetation and root systems should be stripped from the proposed building and pavement areas. Based on our observations, the required stripping depth should generally be about 6 inches. However, greater stripping depths may be required to remove localized zones of soft or organic soils, pockets of ash and charcoal, or deep root systems. We anticipate the required stripping depth in the area of the new driveway entrance will be approximately 12 to 18 inches to adequately remove forest duff, topsoil and root systems in this area. Actual stripping depths across the site should be determined based on field observations at the time of construction. The stripped material should be placed in nonsettlement-sensitive areas (e.g., areas to be landscaped), or disposed of off-site. All existing utilities should also be removed from beneath proposed foundation locations, and consideration should be given to the relocation of existing water lines and faucets located within the proposed pavement area.

After stripping has been completed, the exposed subgrade should be evaluated by thoroughly proofrolling with heavy rubber-tired construction equipment (dry weather construction) or by probing (wet weather construction). During dry weather, we recommend that all loose or soft areas be compacted to a firm, nonyielding condition. If the subgrade soil is too wet to respond to compaction and aeration is not possible, we recommend that the soil be excavated to depths determined by a representative from our firm and replaced with structural fill. The structural fill should be placed and compacted as recommended in a subsequent section of this report.

Evaluation of the need and extent of overexcavation should be accomplished during the grading operations. For planning purposes, we expect that limiting the depth of excavation of unsuitable areas to firm soil or to 2 feet, whichever is less, will be sufficient. The excavated material should be removed from the site or used in landscaping areas.

WET WEATHER CONSIDERATIONS

The native on-site soils generally contain a sufficiently small amount of fines (material passing the U.S. Standard No. 200 sieve) to be only slightly moisture-sensitive. These soils will typically provide adequate support for construction equipment even when wet. However, care should be taken to avoid unnecessary or excessive disturbance to these soils during periods of wet weather.

Construction during wet weather should be coordinated so that footings and floor slabs are constructed immediately after subgrade preparation. Subgrade areas being prepared should be limited in size such that the exposure of the subgrade to wet weather is minimized.

STRUCTURAL FILL

Due to the nature of the existing site grades, it appears that little to no new fill will be needed for site grading. If any fill is needed within the proposed building and parking areas, such as backfill for overexcavated areas, the fill should be placed as compacted structural fill.

The suitability of material for use as structural fill depends on the gradation and moisture content of the material. As the amount of fines increases, soil becomes increasingly more sensitive to small changes in moisture content and adequate compaction becomes more difficult, if not impossible, to achieve. Structural fill material should consist of crushed rock, crushed gravel, sand and gravel, or well-graded sand with less than 5 percent fines based on the fraction passing the 3/4-inch sieve, and should be free of rock fragments larger than 6 inches and debris or any organic contaminants. In our opinion, the unweathered, on-site native sand and gravel will be suitable for use as structural fill. The weathered, on-site native sand with silt and gravel may be suitable for use as structural fill during periods of relatively dry weather. If this material is considered for use as structural fill during wet weather construction, we recommend that the specific material be evaluated by a representative from our firm prior to fill placement. Likewise, if design site grades require that fill from off site be used, we recommend that the suitability of structural fill material from proposed borrow sources be evaluated by a representative from our firm before the earthwork contractor is allowed to transport the material to the site.

Any structural fill placed within 2 feet below the design footing and pavement subgrade elevations should be compacted to at least 95 percent of the maximum dry density as determined in accordance with ASTM D-1557. Structural fill placed at depths greater than 2 feet below these subgrade elevations should be compacted to at least 90 percent of the same standard. We recommend that structural fill be placed in layers not exceeding 10 inches in loose thickness. Each layer should be uniformly compacted to the recommended density before placing additional lifts of fill.

We recommend that a representative from our firm be present during site preparation and structural fill placement. Our representative will observe the work, evaluate subgrade performance under proofrolling or probing, perform representative in-place density tests during filling to determine if the required compaction is being achieved and provide recommendations for any modifications to procedures which may be appropriate for the prevailing conditions.

FOUNDATION SUPPORT

We recommend supporting the proposed building addition on conventional spread footings bearing on undisturbed, medium dense native sand or gravel, or compacted structural fill. Structural fill placed for support of footings should extend laterally beyond the edge of the footings a distance equal to the thickness of the fill. Footing excavations should be probed prior to fill placement and soft or loose areas should be remediated in accordance with the recommendations of this report.

Exterior footings should be founded at least 18 inches below the lowest adjacent finished grade. The bottom of interior footings should be at least 12 inches below the adjacent finished floor grade. Based on available published information and our experience in the area, these recommended footing embedment depths are below the maximum expected frost depth.

Continuous strip footings should be at least 18 inches wide, and isolated column footings should be at least 24 inches wide. An allowable bearing pressure of 2,000 psf (pounds per square foot) can be used for footings designed in accordance with the above recommendations. This bearing pressure applies to the sum of all dead plus long-term live loads, excluding the weight of the footing and any overlying backfill. This value may be increased by one-third when earthquake or wind loads are considered.

We estimate that postconstruction settlements for interior column and perimeter wall footings will be less than 1 inch. The maximum differential settlement between adjacent, comparably loaded column footings is expected to be 1/2 inch or less. Similarly, we expect that differential settlements along continuous wall footings will not exceed about 1/2 inch measured along 25 feet of continuous footing or between adjacent, comparably loaded isolated footings. Because of the granular nature of the existing subgrade soils, most of the settlement should occur shortly after application of the new load.

We recommend that all footing excavations be observed by a representative from our firm immediately prior to placement of reinforcing steel and structural concrete to confirm that the subsurface conditions are as expected and that the bearing surface has been prepared in a manner consistent with our recommendations.

FLOOR SLAB SUPPORT

Satisfactory subgrade support for building floor slabs can be obtained on undisturbed medium dense native sand or gravel. We recommend that the floor slab be directly underlain by a layer of gravel or crushed rock to provide uniform support and act as a capillary break. This layer should consist of a 4-inch minimum thickness of crushed rock or clean, well-graded sand and gravel compacted to at least 95 percent of the maximum dry density (ASTM D-1557). The capillary break material should have a maximum particle size of 3/4-inch, with not more than 80 percent passing the U.S. No. 4 sieve and less than 5 percent fines. A vapor barrier such as a visqueen type plastic sheet should be used to provide added protection against upward migration of moisture into the slab. Also, a 2-inch thick layer of sand can be placed over the vapor barrier to protect the vapor barrier from damage during concrete placement, and to aid in uniform curing of the concrete.

We expect settlement of floor slabs supporting the anticipated design loads and constructed as recommended to not exceed 1/2 inch over a 25-foot distance.

LATERAL RESISTANCE

Below-grade walls should be designed for active earth pressures based on an equivalent fluid density of 35 pcf (pounds per cubic foot). This value is based on the assumption that (1) the walls will not be restrained against rotation when the backfill is placed, (2) the backfill is level, (3) the backfill consists of free-draining, granular material, and (4) the backfill is drained. If below-grade walls will be restrained against rotation during backfilling, they should be designed

for an at-rest earth pressure based on an equivalent fluid density of 55 pcf. A surcharge pressure corresponding to one foot of increased wall height should be made for each 100 psf of floor load that may be imposed behind the wall. Other surcharge loads should be considered as appropriate.

Backfill should be placed and compacted as recommended for structural fill, with the exception of backfill placed immediately adjacent to the walls. Backfill adjacent to the walls should be compacted to a lesser standard to reduce the potential for development of excess pressure on the walls. We recommend that fill placed within a horizontal distance equal to the height of the wall be compacted to approximately 90 percent of the maximum dry density as determined by ASTM D-1557. If slabs or pavement will be placed adjacent to the wall, we recommend that the upper 2 feet of fill be compacted to 95 percent of the maximum dry density (ASTM D-1557). The contractor should avoid overcompaction of the backfill so that damage to the walls does not occur.

Lateral loads and base shear forces transmitted to the building footings by seismic events or wind may be resisted by passive resistance on the sides of the footings and by friction on the base of the footings and slab. Passive resistance may be evaluated using an equivalent fluid density of 300 pcf (pounds per cubic foot) provided that the footings are surrounded by granular native soil or structural fill, compacted to at least 95 percent of the maximum dry density (ASTM D-1557) and extending laterally a distance of at least twice the depth of the footing. Passive resistance should be calculated from the bottom of the adjacent floor slabs or at a depth of 1 foot below the ground surface, if the adjacent area is unpaved. Frictional resistance of footings and the building slab should be evaluated using 0.4 for the coefficient of base friction. The above values incorporate a factor of safety of about 1.5.

SEISMIC CONSIDERATIONS

According to the 1991 Uniform Building Code, the site is located within Seismic Zone 3; the corresponding seismic zone factor, Z, is 0.30. The appropriate site coefficient, or S factor, is 1.2 for this site.

The region in which the site is located is a seismically active area. Based on the results of our explorations, however, we conclude that the medium dense native soils are generally not liquefiable.

PAVEMENT RECOMMENDATIONS

The pavement subgrade should be prepared in accordance with the previously described site preparation, wet weather construction and structural fill recommendations.

We anticipate that traffic at the site will consist of passenger cars and occasional highway-legal trucks. We recommend a minimum pavement section consisting of at least a 2-inch thickness of AC (asphalt concrete) over a minimum 4-inch thickness of compacted crushed rock base course for the parking area, and at least a 3-inch thickness of AC over a minimum 6-inch thickness of compacted crushed rock base course for the entrance driveway. These pavement

design considerations are based on the assumption that the native soils have a CBR (California Bearing Ratio) value of 15. If imported structural fill is placed within pavement areas, we recommend that the imported material have a comparable or greater CBR value.

The asphalt concrete pavement and crushed rock base should conform to applicable requirements of the Standard Specifications for Road, Bridge and Municipal Construction, Washington State Department of Transportation, 1991 Edition. The base course should be placed in one lift and compacted to not less than 95 percent of the maximum dry density as determined by ASTM D-1557.

DRAINAGE CONSIDERATIONS

We recommend that all surfaces be sloped to drain away from the proposed building area. Pavement surfaces and open space areas should be sloped such that surface water runoff is collected and routed to suitable discharge points. All roof drains should be connected to tightlines that discharge into the designed storm sewer disposal system. For sites such as this one, which is underlain by free-draining, granular soils and where the design floor grade will be at or above the surrounding site grades, installation of perimeter footing drains is generally not warranted.

We evaluated soil infiltration rates at two locations near the proposed new stormwater drainfield. Our evaluation consisted of percolation tests conducted in general accordance with the falling head percolation test procedure outlined in the EPA (U.S. Environmental Protection Agency) Design Manual for Onsite Wastewater Treatment and Disposal Systems, as described in the Appendix. The tests were conducted at the locations of borings B-4 and B-5 at depths of 5 and 6 feet, respectively. The results of our tests indicate that the vertical infiltration rate within the unweathered sand and gravel is less than 1 minute per inch.

LIMITATIONS

We have prepared this report for use by The Church of Jesus Christ of Latter-Day Saints and members of their design team in the design of a portion of this project. The data and report should be provided to prospective contractors for bidding or estimating purposes. However, our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions. The conclusions and recommendations in this report should be applied in their entirety.

If there are any changes in the design loads, grades, location, configuration or type of facility to be constructed, the conclusions and recommendations presented in this report might not be fully applicable. If any such changes are made, we should be retained to review our conclusions and recommendations and to provide written modifications or verifications of these recommendations. We also recommend that our firm be retained to review the appropriate final design drawings and specifications to confirm that our recommendations have been interpreted and implemented as intended.

Variations in subsurface conditions are possible between the locations of the explorations and can also occur with time. A contingency for unanticipated conditions should be included in the project budget and schedule for such an occurrence. We recommend that sufficient monitoring, testing and consultation be provided by our firm during construction to confirm that the conditions encountered are consistent with those indicated by the explorations and to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted geotechnical practices in this area at the time the report was prepared. No other conditions, express or implied, should be understood.

We appreciate the opportunity to be of continued service to The Church of Jesus Christ of Latter-Day Saints. Please call if you have any questions regarding this report or require additional information.

Respectfully submitted,

GeoEngineers, Inc.

Michael D. Hylland
Senior Staff Geologist

Kenneth G. Buss, P.E.
Associate

KENNETH G. BUSS
STATE OF WASHINGTON
10999
REGISTERED
PROFESSIONAL ENGINEER

EXPIRES 6/94

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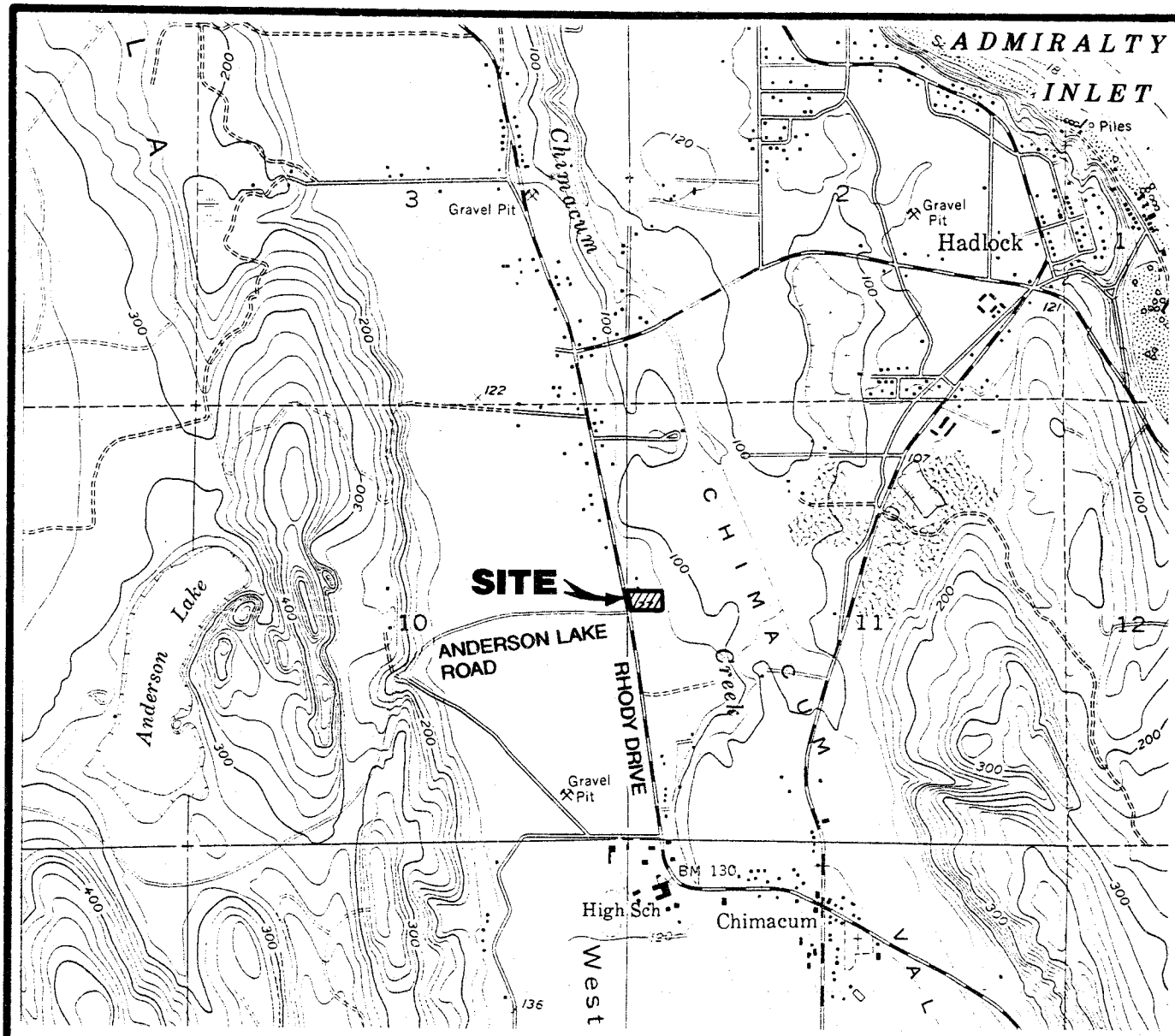
Attachments

Eighteen copies submitted



0 2000 4000
SCALE IN FEET

Reference: USGS 7.5' topographic quadrangle map "Port Townsend South, Wash.," photorevised 1973.

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VICINITY MAP

FIGURE 1

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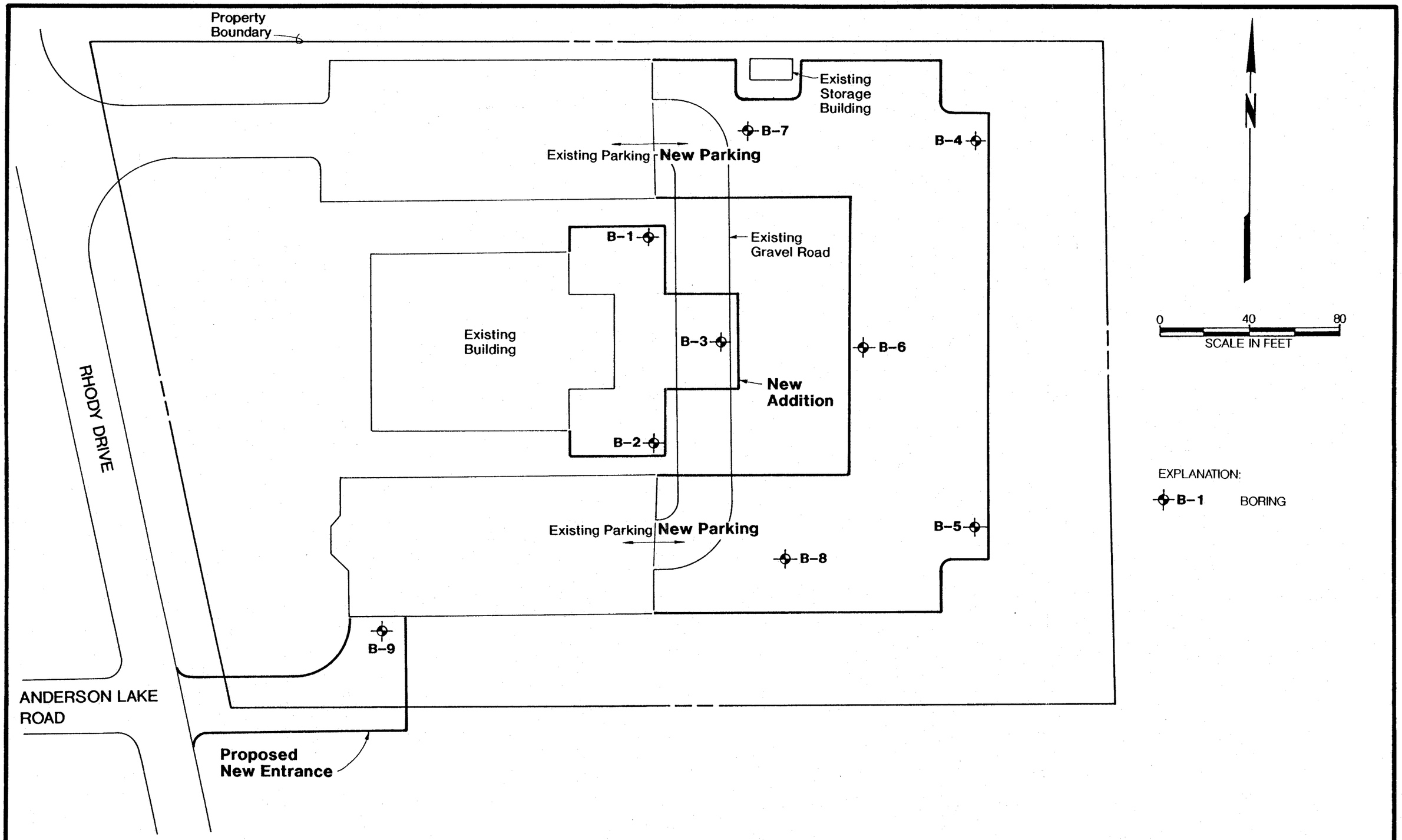

Reference: Drawing entitled "Addition to the Port Townsend Ward, Port Angeles Washington Stake," by Gary W. Hochstrasser Associates, Architects, dated 06/93.

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SITE PLAN

FIGURE 2

1314-051-R10 MDH:BDH 8/2/93

APPENDIX A

APPENDIX

FIELD EXPLORATIONS

Subsurface soil and ground water conditions at the site were explored on July 28, 1993 by drilling nine test borings to depths ranging from approximately 5 to 7 feet below the existing ground surface. The borings were drilled to a depth of 5 feet using a 9-inch diameter solid-stem auger mounted on a rubber-tired "Bobcat" loader. Deeper borings were extended using a hand auger with a 6-inch diameter sampling bit. The borings were backfilled with the cuttings upon completion. The approximate locations of the explorations are shown in Figure 2. The explorations were located in the field by pacing from existing site features.

The borings were accomplished by an engineering geologist from our firm who visually classified the soils encountered, obtained representative soil samples, observed ground water seepage conditions and prepared a detailed log of each exploration. Soils encountered were classified in general accordance with the classification system described in Figure A-1. A key to the symbols used on the borings logs is presented in Figure A-2, and the boring logs are presented in Figures A-3 through A-11.

PERCOLATION TESTS

We conducted two percolation tests at the site on July 28, 1993 in general accordance with the falling head percolation test procedure outlined in the EPA (U.S. Environmental Protection Agency) Design Manual for Onsite Wastewater Treatment and Disposal Systems, dated October 1980. The tests were conducted at the locations of borings B-4 and B-5 at depths of 5 and 6 feet, respectively. Each test consisted of extending a 6-inch diameter boring to the depth indicated, filling the boring twice with a 12-inch depth of water, and then adjusting the water surface to 6 inches above the bottom of the hole and measuring the rate of fall of the water surface. Long-term soaking was not necessary because of the granular nature of the soils. The results of the percolation tests are presented on the boring logs.

LABORATORY TESTING

All soil samples obtained from the borings were sealed and returned to our laboratory for further examination and testing. Selected samples were tested to determine their natural moisture content in accordance with ASTM D2216-90, and grain size distribution (gradation) in accordance with ASTM C136-84a.

The results of the moisture content tests are presented on the boring logs. The results of the grain size analyses are presented as gradation curves in Figures A-12 and A-13.

SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE GRAINED SOILS More Than 50% Retained on No. 200 Sieve	GRAVEL More Than 50% of Coarse Fraction Retained on No. 4 Sieve	CLEAN GRAVEL	GW	WELL-GRADED GRAVEL, FINE TO COARSE GRAVEL
			GP	POORLY-GRADED GRAVEL
		GRAVEL WITH FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SAND More Than 50% of Coarse Fraction Passes No. 4 Sieve	CLEAN SAND	SW	WELL-GRADED SAND, FINE TO COARSE SAND
			SP	POORLY-GRADED SAND
		SAND WITH FINES	SM	SILTY SAND
			SC	CLAYEY SAND
FINE GRAINED SOILS More Than 50% Passes No. 200 Sieve	SILT AND CLAY Liquid Limit Less Than 50	INORGANIC	ML	SILT
			CL	CLAY
		ORGANIC	OL	ORGANIC SILT, ORGANIC CLAY
	SILT AND CLAY Liquid Limit 50 or More	INORGANIC	MH	SILT OF HIGH PLASTICITY, ELASTIC SILT
			CH	CLAY OF HIGH PLASTICITY, FAT CLAY
		ORGANIC	OH	ORGANIC CLAY, ORGANIC SILT
HIGHLY ORGANIC SOILS			PT	PEAT

NOTES:

1. Field classification is based on visual examination of soil in general accordance with ASTM D2488-90.
2. Soil classification using laboratory tests is based on ASTM D2487-90.
3. Descriptions of soil density or consistency are based on interpretation of blow count data, visual appearance of soils, and/or test data.

SOIL MOISTURE MODIFIERS:

Dry - Absence of moisture, dusty, dry to the touch

Moist - Damp, but no visible water

Wet - Visible free water or saturated, usually soil is obtained from below water table

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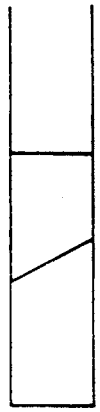
SOIL CLASSIFICATION SYSTEM

FIGURE A-1

LABORATORY TESTS:

AL	Atterberg limits
CP	Compaction
CS	Consolidation
DS	Direct shear
GS	Grain – size
%F	Percent fines
HA	Hydrometer analysis
SK	Permeability
SM	Moisture content
MD	Moisture and density
SP	Swelling pressure
TX	Triaxial compression
UC	Unconfined compression
CA	Chemical analysis

SOIL GRAPH:



SM Soil Group Symbol (See Note 2)

Distinct Contact Between Soil Strata

Gradual or Approximate Location of Change Between Soil Strata

Water Level

Bottom of Boring

BLOW-COUNT/SAMPLE DATA:

Blows required to drive a 2.4-inch I.D. split-barrel sampler 12 inches or other indicated distances using a 300-pound hammer falling 30 inches.

- 22 ■ Location of relatively undisturbed sample
- 12 ⊠ Location of disturbed sample
- 17 □ Location of sampling attempt with no recovery

Blows required to drive a 1.5-inch I.D. (SPT) split-barrel sampler 12 inches or other indicated distances using 140-pound hammer falling 30 inches.

- 10 ◨ Location of sample obtained in general accordance with Standard Penetration Test (ASTM D-1586) procedures
- 26 ◫ Location of SPT sampling attempt with no recovery

Location of grab sample

"P" indicates sampler pushed with weight of hammer or against weight of drill rig.

NOTES:

1. The reader must refer to the discussion in the report text, the Key to Boring Log Symbols and the exploration logs for a proper understanding of subsurface conditions.

2. Soil classification system is summarized in Figure A-1.

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KEY TO BORING LOG SYMBOLS

FIGURE A-2

BORING B-1

TEST DATA						DESCRIPTION	
Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Samples	Group Symbol	Surface Elevation (ft.):	
						Sod and topsoil	
SM	12				SP–SM	Brown fine to medium sand with silt, occasional gravel and a trace of organic matter (medium dense, moist) (weathered recessional outwash)	
					SP	Grayish brown gravelly fine to coarse sand with a trace of silt and occasional cobbles (medium dense, moist) (recessional outwash)	
						Boring completed at 7.0 feet on 07/28/93 No ground water seepage encountered	

DEPTH IN FEET: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

Note: See Figure A-2 for explanation of symbols

:MDH:CMS 8/9/93

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LOG OF BORING

FIGURE A-3

BORING B-3

TEST DATA						DESCRIPTION
Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Samples	Group Symbol	Surface Elevation (ft.):
						Gravel surfacing
					SP	Grayish brown gravelly fine to medium sand with a trace of silt, occasional coarse sand and occasional cobbles (medium dense, moist) (recessional outwash)
SM	7					

DEPTH IN FEET: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

Boring completed at 7.0 feet on 07/28/93
No ground water seepage encountered

Note: See Figure A-2 for explanation of symbols

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LOG OF BORING

FIGURE A-5

:MDH:CMS 8/9/93

1314-051-R10

BORING B-9

TEST DATA

Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Samples	Group Symbol	DESCRIPTION
						Surface Elevation (ft.):
						Sod
					SP–SM	Brown fine to medium sand with silt, occasional gravel and roots (loose, moist) (weathered recessional outwash)
SM	8					
					SP	Grayish brown fine to medium sand with a trace of silt and occasional coarse sand and gravel (medium dense, moist) (recessional outwash)
						Boring completed at 5.0 feet on 07/28/93 No ground water seepage encountered

DEPTH IN FEET: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

Note: See Figure A-2 for explanation of symbols

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LOG OF BORING

FIGURE A-11

:MDH:CMS 8/9/93

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1314-051-R10 MDH:KGB:vvv 08/09/93 (131451B4)

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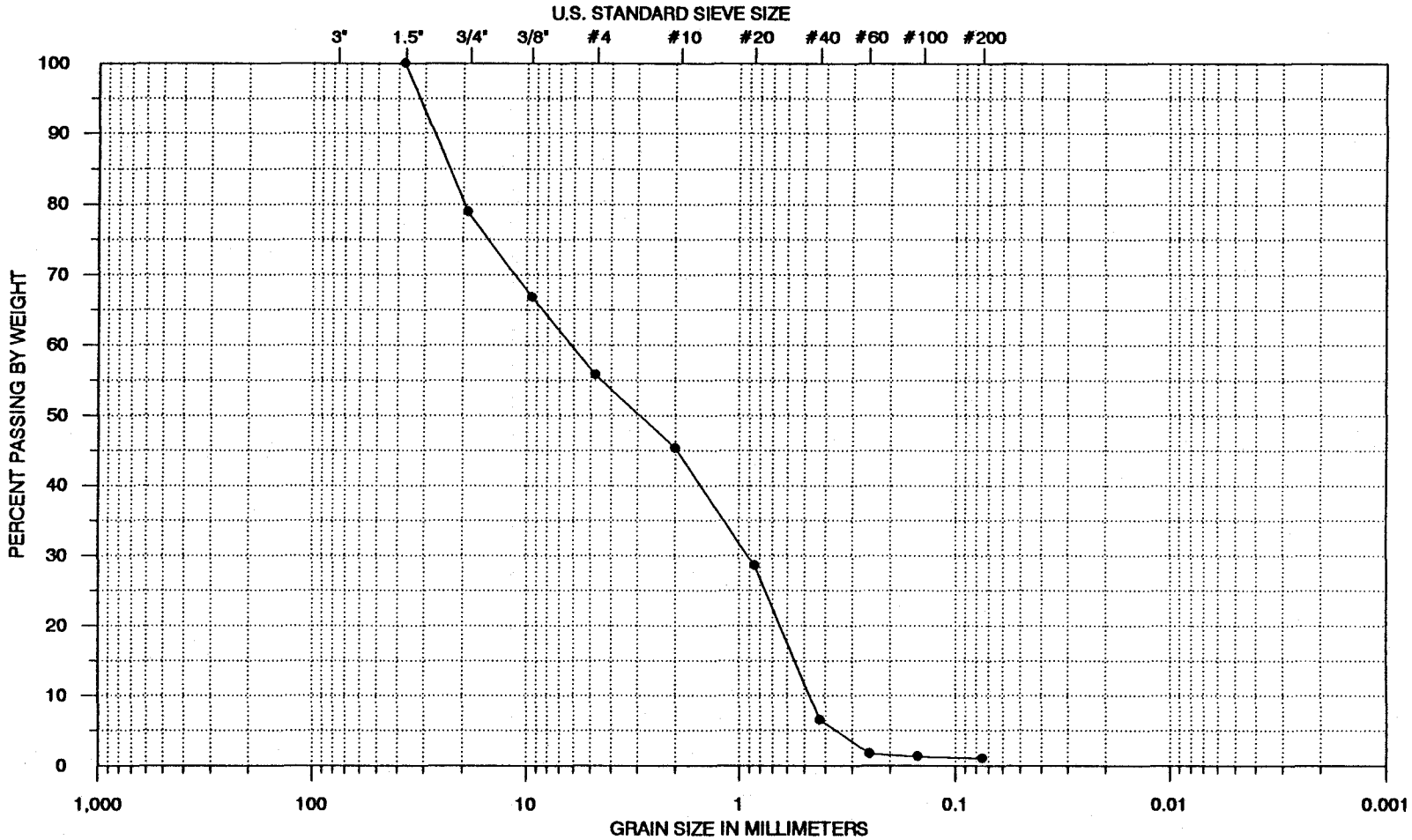

COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	SOIL DESCRIPTION
●	B-4	5	Brownish gray gravelly medium to coarse sand with a trace of silt and occasional fine sand (SP)

GRADATION CURVES

FIGURE A-12

1314-051-R10 MDH:KGB:vvv 08/09/93 (131451B5)

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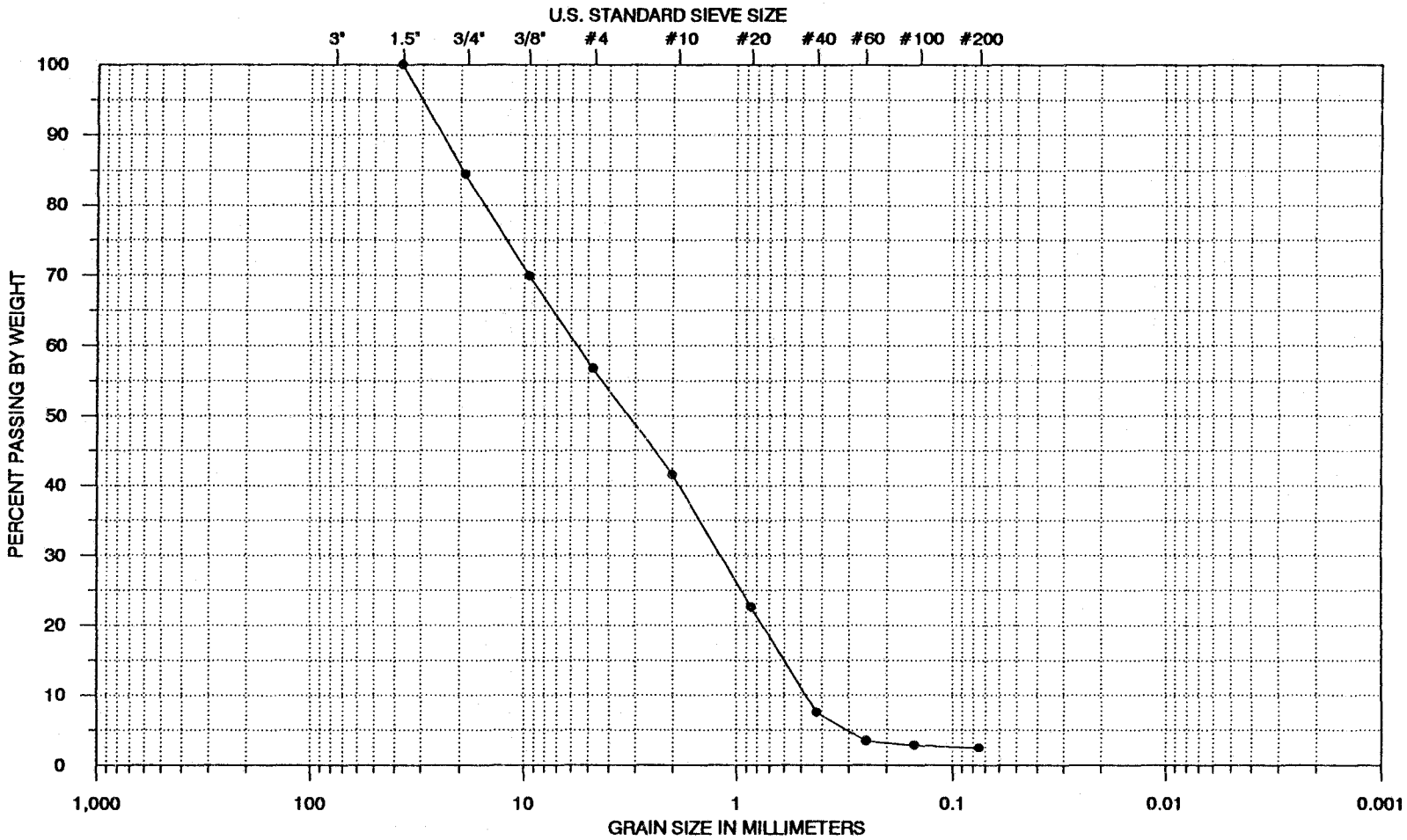

COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	SOIL DESCRIPTION
●	B-5	4	Grayish brown gravelly fine to coarse sand with a trace of silt (SP)

GRADATION CURVES

FIGURE A-13