Ravnik & Associates, Inc.
CIVIL ENGINEERING & LAND-USE PLANNING

JOHN P. RAVNIK
STATE OF WASHINGTON
25795
REGISTERED
PROFESSIONAL ENGINEER
EXPIRES 6·3·04

DRAINAGE ANALYSIS REPORT
for
PHILADELPHIA HARDWARE

Job No. 01041A
Prepared: August 19, 2002

NARRATIVE:

This drainage analysis report is provided to identify the existing and developed storm runoff rates that will occur on the subject property. The proposed development is for Philadelphia Hardware, which is a warehouse and retail facility for door, window, and cabinet hardware.

The site is located on Lot 7 within the Smokey Point Business Park, located on the northeasterly side of 166th Place N.E. approximately 800 feet east of Smokey Point Boulevard. The subject property contains 53,634 square feet, 1.23 acres, which will be developed in its entirety for this project. Refer to the end of this report for the Vicinity Map depicting this project location.

Peak storm water runoff rates will be calculated for the 2-, 10-, and 100-year storm intervals using the Santa Barbara Unit Hydrograph method, as incorporated into the Waterworks software. All developed storm runoff waters will be contained onsite and infiltrated into the underlying ground. Test pits were excavated and infiltration tests performed onsite to substantiate the high level of the underlying ground water table and the soil's infiltration capacity. For the purposes of this drainage design, the allowable infiltration rates per Table 7-1 of the 2001 Department of Ecology Puget Sound Storm Water Design Manual have been used. As identified in the attached Geotechnical Investigation report, and as referenced herein, the actual in-field infiltration rates are higher than the allowable infiltration rates identified in the D.O.E. manual.

There is a drainage system within the adjoining right of way, however the City of Arlington requires that all developments within this business park contain, manage, and infiltrate all storm runoff waters. The proposed, onsite, drainage improvements consist of a series of catch basins to receive surface runoff, storm pipes to convey runoff, oil/water separator vaults for treatment, detention facility to contain runoff, and an underlying infiltration system. This report details the design criteria for the treatment vaults, detention facility, and performance of the infiltration system.

108 E. GILKEY ROAD / P.O. BOX 361 · BURLINGTON, WA 98233
PH: (360) 707-2048 · FAX: (360) 707-2216

EXISTING CONDITIONS:

The subject property contains 1.23 acres and exists as an unmaintained, pasture-like, condition. There are no structures nor development onsite other than utilities that were installed to the project's frontage during development of this business park's infrastructure. The site is relatively flat, having a gradual slope downhill to the northeast, with the southwest street-frontage end near elevation 115.5 sloping to the rear at elevation 114.

As identified within the attached Geotechnical Investigation, the underlying soils are a combination of SAND and LOAMY SAND, having a maximum ground water elevation approximately five feet lower than the existing ground elevations. These conditions are consistent with the Drainage Analysis prepared by Two Rivers Engineering for the design of the business park's drainage system.

For the purposes of this drainage analysis, the following rainfall intensities and site conditions have been used to calculate the peak storm runoff rates. This information is provided to illustrate the differences between the existing and developed conditions, however since all developed runoff will be contained onsite, the existing peak runoff rates do not dictate sizing of the storm water management system.

STORM RAINFALL INTENSITIES

STORM INTERVAL	STORM INTENSITY
2-year	1.9 in./24 hour
10-year	2.7 in./24 hour
100-year	3.95 in./24 hour

Property Area: 1.23 acres
Drainage Basin: 1.23 acres

Pasture Site Coverage: CN = 85

Existing Time of Concentration:

75 linear feet of pasture sheet flow at S = 0.0225
100 linear feet of pasture shallow flow at S = 0.0150
Time of Concentration = 10.87 minutes

Based upon these site and rainfall conditions, peak storm runoff rates have been calculated as listed below

PEAK EXISTING STORM RUNOFF RATES

STORM INTERVAL	STORM RUNOFF
2-year	0.15 cfs
10-year	0.33 cfs
100-year	0.63 cfs

Existing Conditions Basin Summaries from the Waterworks Software are provided at the end of this drainage analysis report further describing the characteristics of these storm events.

DEVELOPED CONDITIONS

The entire property will be developed for this project. Storm runoff water from the proposed building's roof will be conveyed within pipes to the proposed detention facility. No treatment is provided for roof runoff prior to it entering the detention facility. All storm runoff from the exterior concrete and asphalt surfaces will be conveyed through oil/water separator vaults prior to entering the detention facility. All vaults will be sized to provide a minimum 15-minute retention period in order to debris and sediment to accumulate within the bottom of vault. In addition, all vaults will be outfitted with oil-absorbent pads. The project owner will be responsible for maintaining the water treatment and detention facilities as identified further on within this report.

Listed below are the site conditions for this proposed development and the criteria used in calculating the peak developed storm runoff rates.

Drainage Basin: 1.23 acres

Building Area:	0.43 acres @ CN = 98
Exterior Concrete & Asphalt Area:	0.59 acres @ CN = 98
Detention Pond Area:	0.07 acres @ CN = 100
Landscape Area:	0.14 acres @ CN = 86

The above-listed areas equate to 1.09 impervious acres at CN = 98.13 and 0.14 pervious landscaped acres at CN = 86.

Developed Time of Concentration:

76 lineal feet of asphalt sheet flow at S = 0.0158 to trench drain
80 lineal feet of channelized flow @ S = 0.0060 to CB #1
110 lineal feet of channelized flow @ S = 0.0010 to CB #2
190 lineal feet of channelized flow @ S = 0.0010 to CB #3

Time of Concentration = 5.56 minutes

Based upon these site and rainfall conditions, peak storm runoff rates have been calculated as listed below.

PEAK DEVELOPED STORM RUNOFF RATES

STORM INTERVAL	STORM RUNOFF
2-year	0.44 cfs
10-year	0.65 cfs
100-year	0.98 cfs

Developed Conditions Basin Summaries from the Waterworks Software are provided at the end of this drainage analysis report further describing the characteristics of these storm events.

STORM RUNOFF CONVEYANCE SYSTEM:

The proposed storm piping system is very flat between catch basins #1, #2, and #3. This is proposed to avoid excessive costs in filling the site with gravel, yet still utilize the proposed detention and infiltration facility. These storm pipes continue to have sufficient capacity for conveyance when flowing under a "head" condition. The 100-year developed runoff rate from the total property is approximately 1.1 cfs. Proportionately, the runoff rate to CB #1 and CB #2 represents 25% of the site, thereby having an estimated flow rate of 0.28 cfs. Listed below is the culvert-head formula used to substantiate that these proposed pipes, as flat as they are, are still capable of conveying the 100-year developed runoff rate of 0.28 cfs.

Culvert-Head Formula

$$H = \left\{1 + Ke + \frac{(29)(n^2)(L)}{R^{1.333}}\right\} \times \frac{V^2}{2g}$$

H = vertical depth of flow at the upstream end of the piping system to convey required flow rate
Ke = entrance coefficient, 0.50
n = manning roughness coefficient, 0.010 for smooth wall pipe
L = length of pipe system being analyzed, 300' from CB #1 to CB #3
R = hydraulic radius of 8" diameter pipe, which equal D/4 = 0.6667/4 = 0.16667
V = velocity of water flowing in pipe = Q/A = 0.28 cfs / 0.349 sq.ft. = 0.80 fps
g = acceleration due to gravity, 32.2

$$H = \left\{1 + (0.5) + \frac{(29)(0.010^2)(300)}{0.1667^{1.333}}\right\} \times \frac{0.80^2}{(2)(32.2)}$$

$$H = (10.98) \times (0.0124) = 0.14 \text{ feet of head}$$

During the 100-year storm event, the water-surface elevation within the pond is estimated to be 114.1. Based upon 0.28 cfs flowing through 300 l.f. of 8" pipe, the backed-up water surface elevation at CB #3 will be 114.24. This is a conservative approach, since it assumes the peak flow rate occurring when the pond is at its maximum estimated water surface elevation.

DETENTION & INFILTRATION SYSTEM

As previously stated, all storm runoff from the subject property is to be collected, managed, and infiltrated into the underlying soils. At the rear of the site, within the area containing the detention and infiltration system, the existing ground elevations vary between 115 and 114. For the purposes of this drainage design, and based upon there being 5 feet from the existing ground to the high level of ground water, the elevation of high ground water is estimated to be 109.5. Maintaining a minimum of three feet of vertical separation between the high ground water level and the bottom of the infiltration area, the bottom of the infiltration area cannot be lower than elevation 112.5.

The geotechnical investigation excavated test pits and performed in-field infiltration tests to substantiate the quality of the underlying soils and its infiltration capacity. Within the area comprising the infiltration and detention system, the underlying soils are classified as sand. The in-field infiltration tests within this area, test pit #2 and #3 per the geotechnical investigation, varied between 4.5 minutes per inch and 1 minute per inch respectively. These infiltration rates equate to 13.3 inches per hour and 60 inches per hour. Table 7.1 from the referenced D.O.E. manual identifies sand as having a recommended infiltration rate of 8 inches per hour. The geotechnical investigation's in-field tests substantiate that the underlying soils have an infiltration rate greater than the design rates established by D.O.E. For the purposes of this design however, only the design rates per D.O.E. will be used, incorporating a factor of safety equal to 2.0. No additional factor of safety is necessary because the in-field tests identified an infiltration rate at least 1.66 times greater than the recommended design rates from D.O.E. Incorporating a 2:1 safety factor, this drainage design is based upon an infiltration rate of 4 inches per hour, with the bottom of the infiltration area not being lower than elevation 112.5.

Infiltration is provided across the entire bottom of the detention facility. The detention facility is comprised of vertical concrete walls, extending from a top elevation of 116.5 to 111.0. The pond bottom elevation is 112.5, thereby providing 18 inches of cover from the bottom of the footing supporting the retaining wall encompassing the detention pond area. The pond area is 3,005 square feet, however the available area for infiltration is 2,412 square feet. This reduced area is to account for an 18-inch-wide footing covering part of the pond area.

Detention Pond Facility:

Infiltration Bottom Elevation = 112.5
Detention Pond Bottom Elevation = 113.0
Detention Pond Top Elevation = 115.5
Detention Pond Depth = 2.5 feet
Detention Pond Area = 3,005 square feet
Available Detention Volume = 3,005 sq. ft. x 2.5 feet = 7,512.5 cubic feet

Infiltration System Design Criteria:

Design Infiltration Rate = 4 inches per hour = 0.0000926 feet per second
Infiltration Area = 2,412 square feet
Available Infiltration Rate within Pond Bottom = 2,412 x 0.0000926 ft/sec = 0.22 cfs

The bottom six inches of the detention pond, from elevation 112.5 to 113.0 will be filled with clean, washed drainrock varying in size from 1-inch to 2-inches. The storage volume within the drainrock has not been included within the available detention volume.

Performance of Detention/Infiltration System

Listed below is a Level Pool Table Summary reflecting the storage elevations and volumes that occur during the 2-, 10-, and 100-year storm events as they flow into the detention/infiltration system.

STORM INTERVAL	DEVELOPED RUNOFF RATE	INFILTRATION RATE	STORAGE ELEVATION	STORAGE VOLUME
2-year	0.44 cfs	0.22 cfs	113.22	649 cu. ft.
10-year	0.65 cfs	0.22 cfs	113.47	1412 cu. ft.
100-year	0.98 cfs	0.22 cfs	114.10	3314 cu. ft.

WATER TREATMENT FACILITIES:

Storm runoff water from the proposed building will be conveyed directly to the detention pond without treatment. For the purposes of sizing oil/water separator vaults, the remainder of the site can be segregated into two basins. Oil/Water Vault #1 receives and treats all storm runoff entering the trench drain, CB #1, CB #2, and CB #3. Oil/Water Vault #2 receives and treats all storm runoff from CB #4 and CB #5. Listed below are the design criteria and performances of the two proposed Oil/Water Separator Vaults. According to the referenced D.O.E. manual, the vaults must be sized to accommodate the 6-month developed storm event. As a conservative approach, the vaults are designed using the 2-year developed storm event, thereby incorporating a safety factor of approximately 1.5.

Oil/Water Separator Vault #1

Drainage Area to Vault #1 = 0.37 acres = 30% of the site

2-year Developed Runoff Rate from Entire Site = 0.44 cfs

Therefore, 2-year Developed Runoff Rate to Vault #1 = 0.132 cfs

Converting to gallons per minute, = 59.14 gpm

Providing for a minimum 15-minute retention period, the vault capacity must not be less than 887 gallons.

Recommended Vault #1 Volume, using Utility Vault Product # 48SA = 1,000 gallons.

Actual retention period of 2-year storm event through Vault #1 = 16.9 minutes

Actual retention period of 6-month storm event through Vault #1 = 26.4 minutes

Oil/Water Separator Vault #2

Drainage Area to Vault #2 = 0.21 acres = 17% of the site

2-year Developed Runoff Rate from Entire Site = 0.44 cfs

Therefore, 2-year Developed Runoff Rate to Vault #2 = 0.075 cfs

Converting to gallons per minute, = 33.5 gpm

Providing for a minimum 15-minute retention period, the vault capacity must not be less than 503 gallons.

Recommended Vault #2 Volume, using Utility Vault Product # 577SA, = 800 gallons.

Actual retention period of 2-year storm event through Vault #2 = 23.8 minutes

Actual retention period of 6-month storm event through Vault #2 = 37.2 minutes

STORM SYSTEM MAINTENANCE:

Oil/Water Separator Vaults

One time per year, the property owner is responsible for having all vaults cleaned of accumulated debris. Each vault is outfitted with two oil-absorbent pads that are also to be removed and replaced with each vault cleaning. The specified pads are to have a minimum oil-retainage capacity of 2 gallons. The maintenance shall be contracted by the owner, and must be performed by personnel qualified to handle and dispose of the materials removed from the storm system.

Detention & Infiltration Facility

The detention area is comprised of an exposed, 6-inch depth of drainrock. This bed of drainrock covering the entire pond bottom is the infiltration system. The owner is responsible for periodically inspecting the detention facility to remove and debris and trash that enter this system. Since all surface runoff from driving surfaces is conveyed through oil/water separator vaults, no noticeable volume of sediment or silt should enter the detention pond. Maintenance of the vaults and piping as specified is essential to the continued function of the detention pond system. Failure to adequately maintain the vaults and piping will result in excessive debris and silt entering the detention pond and infiltration system, thereby requiring that all drainrock be removed from the pond-bottom and replaced.

Storm Piping System

All storm piping receiving runoff from driving surfaces and the building can be maintained from access points provided by catch basins and cleanouts. At the end of construction, the contractor is responsible for thoroughly cleaning all debris from storm pipes. Once every two years, after the vaults have been cleaned, the owner's contracted maintenance company for the storm vaults shall flush out all storm pipes receiving runoff from drive surfaces. After thoroughly flushing all storm pipes, the vaults shall be cleaned of all accumulated debris from the pipe-flushing.

CONCLUSION:

As identified herein and on the civil plans submitted for this project, all storm runoff waters from the building and exterior impervious surfaces will enter the proposed detention and infiltration facility. All runoff from the developed site will enter this site's detention and infiltration system. With the exception of a very small, entrance drivelane area contributing runoff into the street's drainage system, all of the site's runoff will be managed onsite. Although recent infiltration tests onsite documented infiltration rates greater than allowed by the Department of Ecology, the DOE rates have been used together with a 2:1 factor of safety. To assure clean waters entering the detention and infiltration system, oil/water separator vaults are to be installed to treat all runoff from exterior impervious surfaces prior to these waters entering the detention and infiltration facility. The vaults have purposely been oversized to assure that all debris, silt, and petroleum particulates are removed.

There are no offsite properties that contribute storm runoff waters onto this site. There are also no downstream nor upstream drainage facilities associated with this project, therefore there are no upstream nor downstream drainage impacts generated by this project.

DOCUMENTS ACCOMPANYING DRAINAGE REPORT

1. BASIN SUMMARIES FROM WATERWORKS SOFTWARE OF EXISTING AND DEVELOPED CONDITIONS.
2. VICINITY MAP
3. EXISTING CONDITIONS EXHIBIT
4. DEVELOPED CONDITIONS EXHIBIT
5. GEOTECHNICAL INVESTIGATION
6. TABLE 7-1 FROM THE 2001 D.O.E. MANUAL

BASIN SUMMARIES – EXISTING CONDITIONS

BASIN ID: EX2 NAME: **EXISTING CONDITION 2YR STORM**
SBUH METHODOLOGY
TOTAL AREA.......: 1.23 Acres BASEFLOWS: 0.00 cfs
RAINFALL TYPE....: TYPE1A PERVIOUS AREA
PRECIPITATION....: 1.90 inches AREA..: 1.23 Acres
TIME INTERVAL....: 10.00 min CN....: 85.00
TIME OF CONC.....: 10.87 min IMPERVIOUS AREA
ABSTRACTION COEFF: 0.20 AREA..: 0.00 Acres
CN....: 98.00
TcReach - Sheet L: 75.00 ns:0.1500 p2yr: 1.90 s:0.0225
TcReach - Shallow L: 100.00 ks:11.00 s:0.0150
PEAK RATE: 0.15 cfs VOL: 0.07 Ac-ft TIME: 480 min

BASIN ID: EX10 NAME: **EXISTING CONDITION 10YR STORM**
SBUH METHODOLOGY
TOTAL AREA.......: 1.23 Acres BASEFLOWS: 0.00 cfs
RAINFALL TYPE....: TYPE1A PERVIOUS AREA
PRECIPITATION....: 2.70 inches AREA..: 1.23 Acres
TIME INTERVAL....: 10.00 min CN....: 85.00
TIME OF CONC.....: 10.87 min IMPERVIOUS AREA
ABSTRACTION COEFF: 0.20 AREA..: 0.00 Acres
CN....: 98.00
TcReach - Sheet L: 75.00 ns:0.1500 p2yr: 1.90 s:0.0225
TcReach - Shallow L: 100.00 ks:11.00 s:0.0150
PEAK RATE: 0.33 cfs VOL: 0.14 Ac-ft TIME: 480 min

BASIN ID: EX00 NAME: **EXISTING CONDITION 100YR STORM**
SBUH METHODOLOGY
TOTAL AREA.......: 1.23 Acres BASEFLOWS: 0.00 cfs
RAINFALL TYPE....: TYPE1A PERVIOUS AREA
PRECIPITATION....: 3.95 inches AREA..: 1.23 Acres
TIME INTERVAL....: 10.00 min CN....: 85.00
TIME OF CONC.....: 10.87 min IMPERVIOUS AREA
ABSTRACTION COEFF: 0.20 AREA..: 0.00 Acres
CN....: 98.00
TcReach - Sheet L: 75.00 ns:0.1500 p2yr: 1.90 s:0.0225
TcReach - Shallow L: 100.00 ks:11.00 s:0.0150
PEAK RATE: 0.63 cfs VOL: 0.25 Ac-ft TIME: 480 min

BASIN SUMMARIES – DEVELOPED CONDITIONS

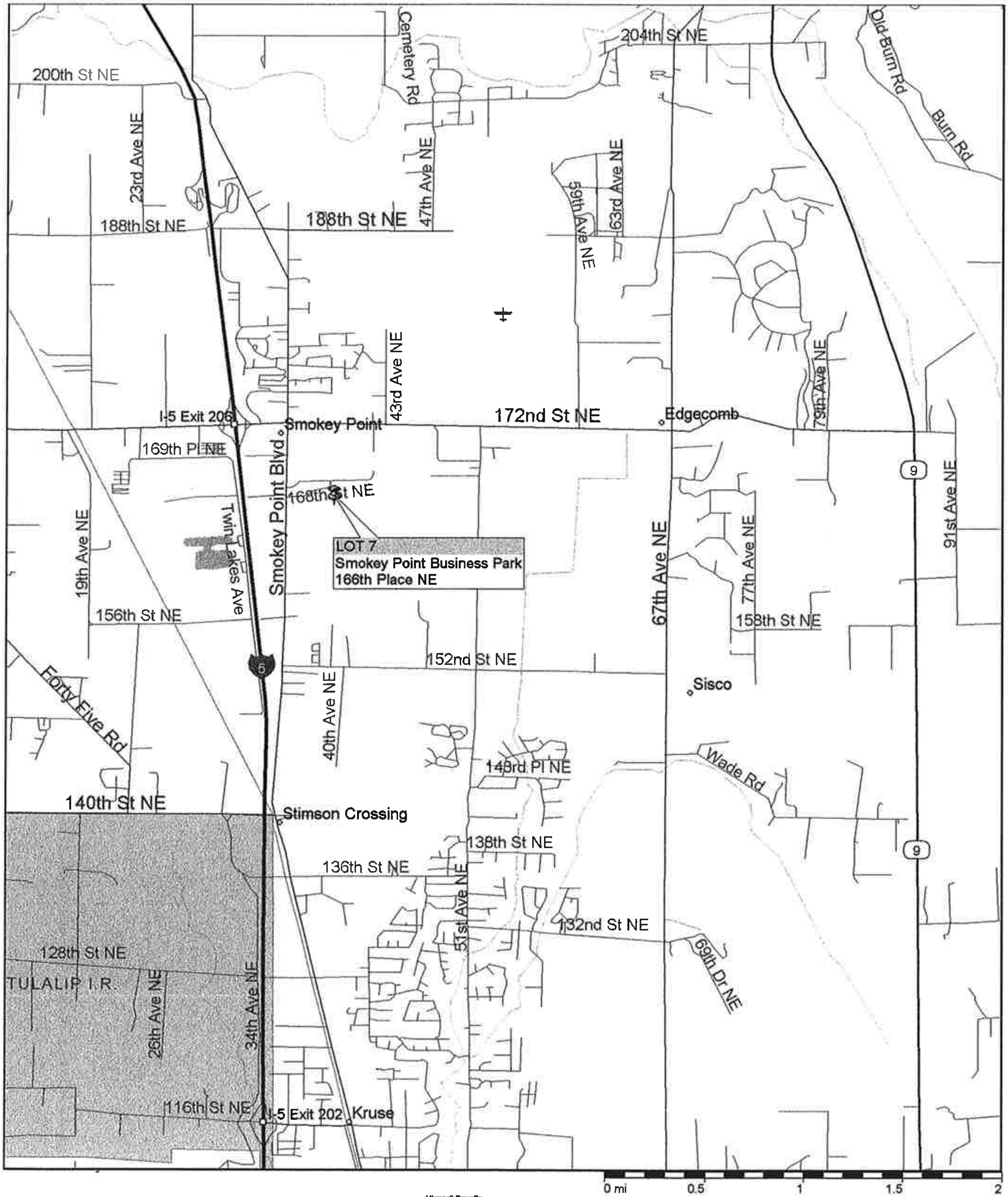
BASIN ID: DEV2 NAME: **DEVELOPED CONDITION 2YR STORM**
SBUH METHODOLOGY
TOTAL AREA.......: 1.23 Acres BASEFLOWS: 0.00 cfs
RAINFALL TYPE....: TYPE1A PERVIOUS AREA
PRECIPITATION....: 1.90 inches AREA..: 0.14 Acres
TIME INTERVAL....: 10.00 min CN....: 86.00
TIME OF CONC.....: 5.56 min IMPERVIOUS AREA
ABSTRACTION COEFF: 0.20 AREA..: 1.09 Acres
CN....: 98.13
TcReach - Sheet L: 76.00 ns:0.0110 p2yr: 1.90 s:0.0158
TcReach - Channel L: 80.00 kc:42.00 s:0.0060
TcReach - Channel L: 300.00 kc:42.00 s:0.0010
PEAK RATE: 0.44 cfs VOL: 0.16 Ac-ft TIME: 480 min

BASIN ID: DEV10 NAME: **DEVELOPED CONDITION 10YR STORM**
SBUH METHODOLOGY
TOTAL AREA.......: 1.23 Acres BASEFLOWS: 0.00 cfs
RAINFALL TYPE....: TYPE1A PERVIOUS AREA
PRECIPITATION....: 2.70 inches AREA..: 0.14 Acres
TIME INTERVAL....: 10.00 min CN....: 86.00
TIME OF CONC.....: 5.56 min IMPERVIOUS AREA
ABSTRACTION COEFF: 0.20 AREA..: 1.09 Acres
CN....: 98.13
TcReach - Sheet L: 76.00 ns:0.0110 p2yr: 1.90 s:0.0158
TcReach - Channel L: 80.00 kc:42.00 s:0.0060
TcReach - Channel L: 300.00 kc:42.00 s:0.0010
PEAK RATE: 0.65 cfs VOL: 0.24 Ac-ft TIME: 480 min

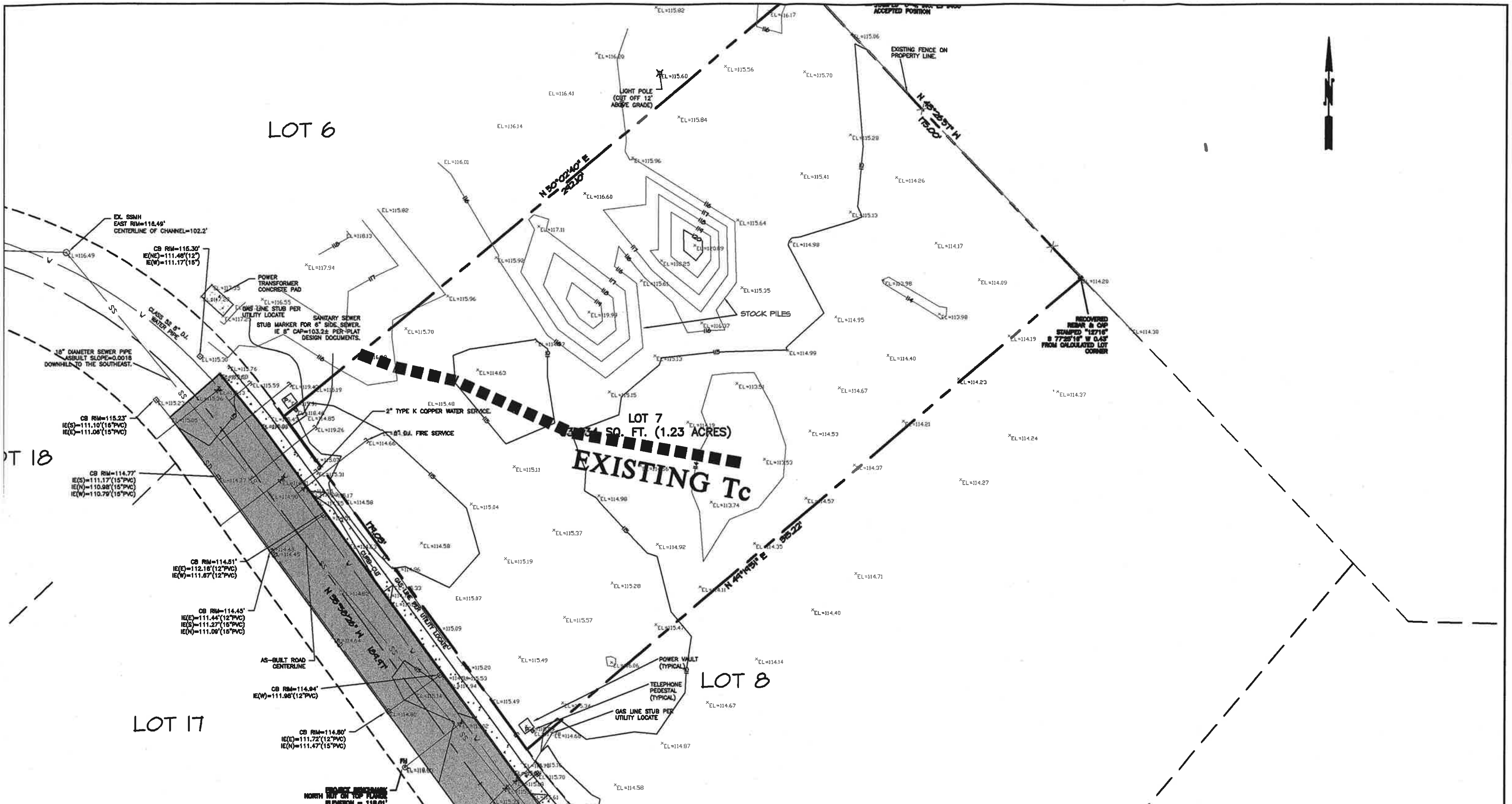
BASIN ID: DEV00 NAME: **DEVELOPED CONDITION 100YR STORM**
SBUH METHODOLOGY
TOTAL AREA.......: 1.23 Acres BASEFLOWS: 0.00 cfs
RAINFALL TYPE....: TYPE1A PERVIOUS AREA
PRECIPITATION....: 3.95 inches AREA..: 0.14 Acres
TIME INTERVAL....: 10.00 min CN....: 86.00
TIME OF CONC.....: 5.56 min IMPERVIOUS AREA
ABSTRACTION COEFF: 0.20 AREA..: 1.09 Acres
CN....: 98.13
TcReach - Sheet L: 76.00 ns:0.0110 p2yr: 1.90 s:0.0158
TcReach - Channel L: 80.00 kc:42.00 s:0.0060
TcReach - Channel L: 300.00 kc:42.00 s:0.0010
PEAK RATE: 0.98 cfs VOL: 0.37 Ac-ft TIME: 480 min

VICINITY MAP

LOT 7 SMOKEY POINT BUSINESS PARK



Microsoft Expedia
Streets98

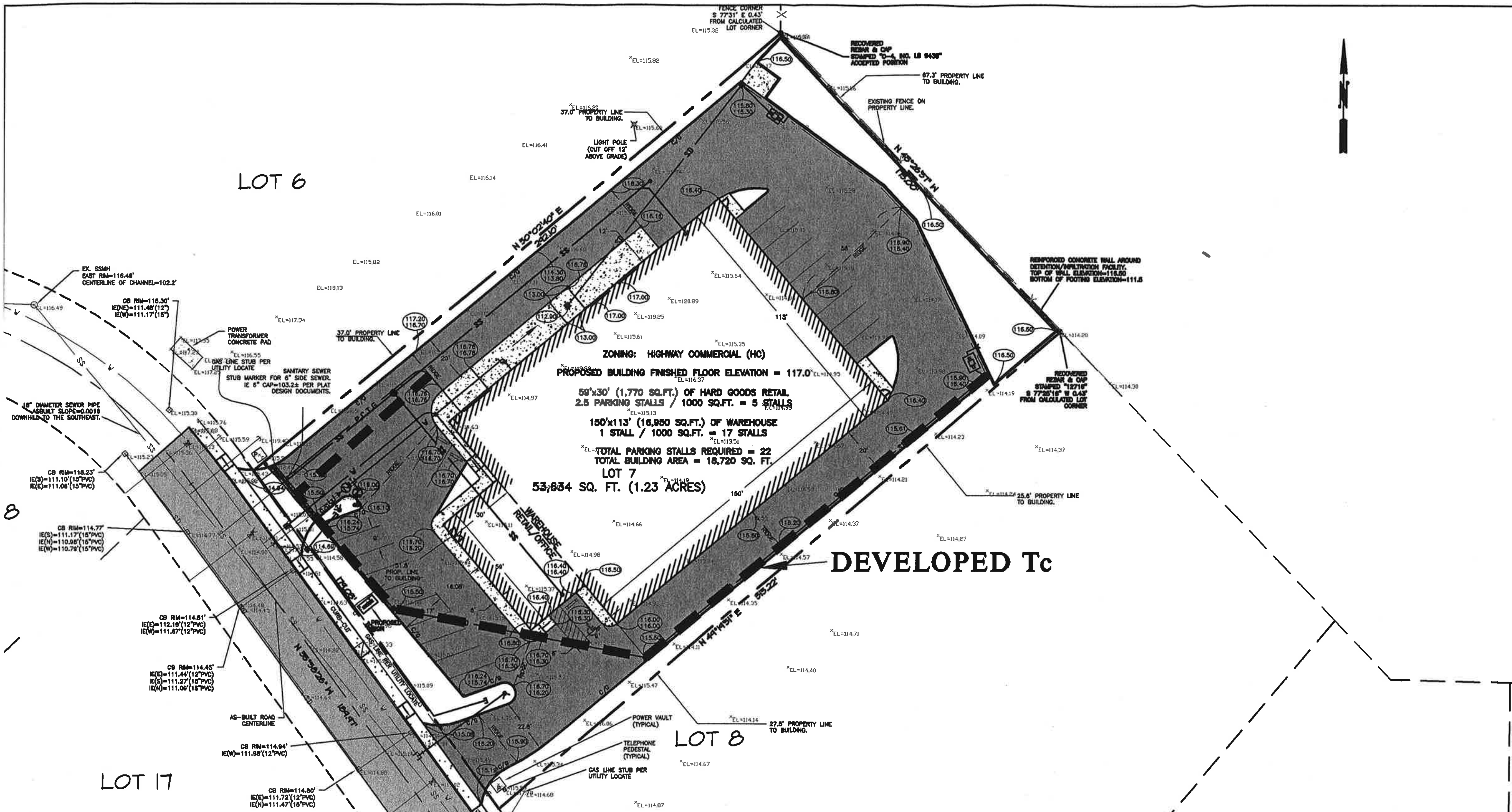


Ravnik & Associates, Inc. CIVIL ENGINEERING & LAND-USE PLANNING 108 E. GILKEY/P.O. BOX 361 BURLINGTON, WA 98233 PH: (360) 707-2048 FAX: (360) 707-2216	SHEET DESCRIPTION: **EXISTING CONDITIONS DRAINAGE PLAN**	SCALE: 1"=40' DRAWN BY: D. SVEUM CHECKED BY: J. RAVNIK DATE: 09/06/02	SHEET TITLE: **PHILADELPHIA HARDWARE FOR PERKES CONSTRUCTION** NW 1/4 OF S. 28, T. 31 N., R. 5 E., W.M.

DRAWING NO. 01041ASIT.dwg

JOB NO. 01041A

SHEET NO. 1 OF 2



SHEET DESCRIPTION:		SHEET TITLE:	
Ravnik & Associates, Inc. CIVIL ENGINEERING & LAND-USE PLANNING 108 E. GILKEY/P.O. BOX 361 BURLINGTON, WA 98233 PH: (360) 707-2048 FAX: (360) 707-2216	DEVELOPED CONDITIONS DRAINAGE PLAN	SCALE: 1"=40' DRAWN BY: D. SVEUM CHECKED BY: J. RAVNIK DATE: 09/06/02	PHILADELPHIA HARDWARE FOR PERKES CONSTRUCTION NW 1/4 OF S. 28, T. 31 N., R. 5 E., W.M.

DRAWING NO. 01041ASIT.dwg

JOB NO. 01041A

SHEET NO. 2 OF 2

Zipper Zeman Associates, Inc.
Geotechnical and Environmental Consulting

J-1418
August 1, 2002

ANP Construction
17713-B Dunbar Road
Mount Vernon, Washington 98273

Attention: Mr. Alan Perkes

Subject: Report of Geotechnical Services
Philadelphia Hardware Facility
166th Place NE
Arlington, Washington

Dear Mr. Perkes:

Zipper Zeman Associates, Inc. (ZZA) has completed a geotechnical evaluation for the proposed Philadelphia Hardware facility on 166th Place NE in Arlington, Washington. This report presents the results of our geotechnical evaluation relative to design and general construction considerations. These services were completed in general accordance with the scope of work in our proposal dated June 24, 2002, which you authorized by signature on June 25, 2002. The field evaluation was completed on June 28, 2002.

We understand that the project is in the planning stages, with the proposed building currently planned to be located in the central portion of the property. We understand that the building will consist of a steel frame structure supported on conventional spread footings, with concrete slab-on-grade floors. Portland concrete and asphalt concrete pavements are anticipated around the facility for general parking and truck access, and for a proposed loading ramp.

The purpose of our geotechnical evaluation is to assess subsurface conditions relative to the design and construction of the hardware facility. The scope of our services included an exploration program consisting of excavating seven test pit explorations at the site. In addition, four falling-head infiltration tests were performed to assess storm drainage considerations. Based on the subsurface conditions observed in the test pits, we performed geotechnical analyses and formulated recommendations that are presented in this report. Specific items addressed in this report include:

1. Description of the project site with exploratory locations shown on a site plan;
2. General subsurface conditions;
3. Earthwork and site preparation recommendations;

4. Structural fill and the suitability of on-site soils for use as structural fill;

5. Recommendations relative to the suitability and design of shallow spread footing foundations and slab-on-grade floors, including allowable bearing pressures and settlement estimates;

6. Provide assessed infiltration rates of site soils based on results of the infiltration tests performed on the site;

7. Preparation of pavement subgrades;

8. Wet weather construction considerations;

9. Erosion control and site drainage considerations;

10. General seismicity and liquefaction potential;

11. Recommendations for further study, if appropriate.

SITE DESCRIPTION

The proposed Philadelphia Hardware site is located on the north side of 166th Place NE. The lot is rectangular and encompasses approximately 1-¼ acres. Overall, the site slopes down gently to the east. It appears that some grading (stripping) has occurred in the eastern portion of the lot, with some soil mounds on the northwest side of the lot. The soil in these mounds appears to be the stripped material from the eastern portion. Vegetation consisting of principally grass and weeds covers the property.

FIELD EXPLORATION

The subsurface exploration program conducted for this study consisted of completing seven test pit explorations at the site on June 28, 2002. The approximate locations of the test pits are presented in Figure 1, Site and Exploration Plan. The test pit explorations were completed to depths ranging from 12 to 15 feet below the existing ground surface. Logs of the test pits are enclosed with this report. In addition to the test pits, four falling-head infiltration tests were performed adjacent to test pits TP-1, TP-2, TP-3, and TP-7.

Subsurface Conditions

In general, subsurface conditions in the test pits were consistent. Fill consisting of loose sandy gravel was observed in test pits TP-2 and TP-4 on the northern side of the lot. This fill extended to depths of 1 to 2 feet below the surface. Topsoil consisting of loose silty sand was observed below the surface in the remaining test pits and extended to a maximum depth of roughly 2 feet. Below the fill and topsoil were native layers of sand and silty sand. These native soils were generally in a medium dense to dense condition. The amount of fines (silt and clay) typically decreased with depth.

Soil descriptions presented in this report are based on the subsurface conditions observed at the specific test pit locations. Variations in subsurface conditions may exist between the exploration locations, and the nature and extent of variations between the explorations may not become evident until construction. If variations then appear, it may be necessary to reevaluate the recommendations presented in this report.

Groundwater

Groundwater was observed in test pits TP-1, TP-2, TP-3, TP-5, and TP-7 at the time of excavation. The depth to groundwater varied from approximately 6 to 7.5 feet below the existing ground surface. Test pits TP-4 and TP-6 were not excavated deep enough to encounter the groundwater table. It should be noted that groundwater conditions and soil moisture contents are expected to vary with changes in season, precipitation, site utilization, and other on- and off-site factors. We understand that the City of Arlington has indicated that the groundwater table rises to within approximately 5 feet of the surface in this area during the wet seasons of the year.

CONCLUSIONS AND RECOMMENDATIONS

Based on the subsurface exploration program, the project appears feasible utilizing conventional shallow foundation support. The following recommendations have been prepared for the design and construction of conventional spread footing foundations and concrete slab-on-grade floors.

Site Preparation

We understand that some fill will be required to raise existing site grades to design elevations. Site preparation should include the removal of vegetation (including root balls), existing fill, and organic soils (topsoil) and any other deleterious debris from building and paving areas, or those locations where "structural fill" is to be placed. Exposed soils following site preparation should consist of the native medium dense to dense sand or silty sand. Any excavations that extend below finish grades should be backfilled with structural fill as outlined subsequently in this report.

Preparation for site grading and construction should include procedures intended to drain ponded water and control surface water runoff. It may not be possible to successfully utilize on-site soils as "structural fill" if accumulated water is not drained prior to grading, or if drainage is not controlled during construction. Attempting to grade the site without adequate drainage control measures will reduce the amount of on-site soil effectively available for use, increase the amount of select import fill materials required, and ultimately increase the cost of the earthwork and foundation construction phases of the project.

After removal of existing fill, topsoil, and other deleterious material, and prior to placement of structural fill, we recommend that foundation and floor subgrade areas, pavement areas, and areas to receive structural fill be proofrolled and compacted to a firm and unyielding condition in order to achieve a minimum compaction level of 92 percent of the modified Proctor maximum dry density as determined by the ASTM:D-1557 test procedure. Proofrolling should be accomplished with a heavy compactor, loaded double-axle dump truck, or other heavy equipment under the observation of a representative from our firm. The need for or advisability of proofrolling due to soil moisture conditions should be determined at the time of construction. We recommend that a representative from our firm observe the soil conditions prior to and during proofrolling to evaluate the suitability of stripped subgrades.

Excavated site soils may not be suitable for use as structural fill depending on the moisture content and weather conditions at the time of construction. If soils are stockpiled for future reuse and wet weather is anticipated, the stockpile should be protected with plastic sheeting that is securely anchored. If on-site soils become unusable, it may become necessary to import clean, granular soils to complete wet weather site work.

Subgrade soils that become disturbed due to elevated moisture conditions should be overexcavated to expose firm, non-yielding, non-organic soils and backfilled with compacted structural fill. We recommend that the earthwork portion of this project be completed during extended periods of dry weather, if possible. If earthwork is completed during the wet season, it may be necessary to take extra precautionary measures to protect subgrade soils. Wet season earthwork may require additional mitigative measures beyond that which would be expected during the drier summer and fall months. This could include diversion of surface runoff around exposed soils, draining of ponded water on the site, and collection and rerouting of groundwater seepage from upgradient on- and off-site sources. Once subgrades are established, it may be necessary to protect the exposed subgrade soils from construction traffic. Placing quarry spalls, crushed recycled concrete, or clean pit-run sand and gravel over these areas would help protect the soils from construction traffic.

Structural Fill

All fill material placed in building, pavement, and non-landscaped areas should be placed as structural fill. Prior to placement, the exposed subgrade surfaces to receive structural fill should be prepared as previously described. All structural fill should be free of organic material, debris, or other deleterious material. Individual particle size should be less than 3 inches in maximum dimension.

Structural fill should be placed in lifts no greater than 8 inches in loose thickness. The structural fill should be compacted to at least 95 percent of the modified Proctor maximum dry density as determined by the ASTM:D-1557 test procedure in building areas and to a depth of 2 feet below the subgrade surface in pavement areas. Below a depth of 2 feet in pavement areas, the structural fill should be compacted to at least 90 percent of ASTM:D-1557. In the case of roadway and utility trench filling, the backfill should be placed and compacted in accordance with current local codes and standards.

The suitability of soils for use as structural fill use depends primarily on the gradation and moisture content of the soil when it is placed. As the amount of fines (that soil fraction passing the U.S. No. 200 sieve) increases, soil becomes increasingly sensitive to small changes in moisture content and adequate compaction becomes more difficult, or impossible, to achieve. Generally, soils containing more than about 10 percent fines by weight (based on that soil fraction passing the U.S. No. 4 sieve) cannot be compacted to a firm, non-yielding condition when the moisture content is more than a few percent from optimum. The optimum moisture content is that which yields the greatest soil density under a given compactive effort.

The native sand and silty sand observed in the test pits generally appears suitable for use as structural fill. However, some of these soils may contain a significant fine-grained fraction. Consequently, use of these soils as structural fill will require that strict control of moisture content be maintained during the grading process. Soil moisture conditions should be expected to change throughout the year. Drying of over-optimum moisture soils may be achieved by scarifying or windrowing surficial materials during extended periods of dry weather. Soils which are dry of optimum may be moistened through the application of water and thorough blending to facilitate a uniform moisture distribution in the soil prior to compaction.

In the event that inclement weather or wet site conditions prevent the use of on-site soil or non-select material as structural fill, we recommend that a "clean", free-draining pit-run sand and gravel be used. Such materials should generally contain less than 5 percent fines, based on that soil fraction passing the U.S. No. 4 sieve, and not contain discrete particles greater than 3 inches in maximum dimension. It should be noted that the placement of structural fill is, in many cases, weather-dependent. Delays due to inclement weather are common, even when using select granular fill. We recommend that site grading and earthwork be scheduled for the drier months, when possible.

Permanent Fill Slopes

We understand that finish grades will require some fill to be imported to the site. Permanent fill slopes should be constructed no steeper than 2H:1V. If the slopes are exposed to prolonged rainfall before vegetation becomes established, the surficial soils will be prone to erosion and possible shallow sloughing. Surficial repairs, such as protecting affected areas with quarry spalls, jute matting or other system, may be necessary until vegetation is established.

Temporary Cut Slopes

Temporary cut slopes are anticipated for utility trenches and the proposed loading ramp. Temporary slope stability is a function of many factors, including the following:

1. The presence and abundance of groundwater;
2. The type and density of the various soil strata;
3. The depth of cut;
4. Surcharge loadings adjacent to the excavation;
5. The length of time the excavation remains open.

It is exceedingly difficult under the variable circumstances to pre-establish a safe and "maintenance-free" temporary cut slope angle. Therefore, it should be the responsibility of the contractor to maintain safe slope configurations since the contractor is continuously at the job site, able to observe the nature and condition of the cut slopes, and able to monitor the subsurface materials and groundwater conditions encountered. It may be necessary to drape temporary cut slopes with plastic or to otherwise protect the slopes from the elements in order to minimize sloughing and erosion. We do not recommend unsupported vertical cuts deeper than 4 feet if worker access is necessary. The cuts should be adequately sloped or supported to prevent injury to personnel from local sloughing and spalling. The excavation should conform to applicable Federal, State, and local regulations.

For preliminary planning purposes only, the native sand and silty sand soils observed above the high water table (approximately 5 feet) in the test pits is anticipated to maintain cut slopes on the order of 1H:1V. We anticipate that shoring (trench boxes or other) will be required for any excavations that extend below the water table as the sands at these depths were observed to readily slough during excavation of the test pits. For all cut slopes, if seepage or surface runoff occurs and is not controlled, flatter temporary slopes may be necessary. These guidelines assume that surface loads, such as equipment loads and storage loads, will be kept a sufficient distance away from the top of the cut so that the stability of the excavation is not affected.

Foundations

All footings should be founded on the medium dense to dense native sand, silty sand, or on compacted structural fill that extends down to these competent native soils. Footings should not be founded on or within loose or disturbed native soil or fill unless it has been evaluated and approved by the geotechnical engineer. Continuous or column footings may be designed for a maximum allowable bearing pressure of 2,000 psf. A one-third increase in this bearing pressure may be used for short-term wind or seismic loading. Exterior footings should extend at least 18 inches below adjacent grade for frost protection, while interior footings should extend at least 12 inches below adjacent grade. We recommend that all continuous and isolated footings be at least 18 and 24 inches in width, respectively. We recommend using an allowable base friction value of 0.35 for footings supported on the native sand soils or on compacted granular fill.

We estimate that the total settlement of foundation members founded within the medium dense to dense sand or structural fill prepared as described above may approach 1 inch. Differential settlement of foundations founded within the same soil type could approach ½ inch over a distance of 30 feet. Settlements would occur elastically as the loads are applied.

Foundation settlement is oftentimes a function of the condition of the footing excavation subgrade. Under no circumstances should footings be cast atop loose or soft soil, slough, debris, or surfaces with standing water. We recommend that a representative from our firm observe the condition of the footing subgrades prior to pouring footing concrete in order to confirm that the bearing soils are undisturbed and that conditions are consistent with the recommendations contained within this report.

We recommend that perimeter footing drains with cleanouts be installed around the planned building. The drains should consist of a minimum 4-inch diameter perforated pipe embedded in at least a 24-inch wide envelope of clean, free-draining granular material containing less than 5 percent fines (material passing the U.S. No. 200 sieve). Footing drains should be directed toward appropriate storm water drainage facilities and not onto adjacent slopes. Roof drains should not be connected to the footing drains. We recommend that the ground surface adjacent to foundations be sloped to drain surface runoff away from the structure.

Slab-On-Grade Floors

Slab-on-grade floor subgrades should be prepared in accordance with the site preparation recommendations presented above. All slab-on-grade floors should be founded on the medium dense to dense native sand or silty sand, or on compacted structural fill extending down to these competent native soils. Slab-on-grade floors should not be founded on or within loose or disturbed native or fill soil unless it has been evaluated and approved by the geotechnical engineer. We recommend that at least 4-inches of clean coarse sand and gravel (containing less than 5 percent material passing the U.S. No. 200 sieve) be placed between the prepared subgrade and bottom of the concrete floors. This zone will serve for support, and as a capillary break and working surface.

In floor slab areas where moisture sensitive floor coverings are planned, an impermeable membrane (e.g. polyethylene sheet) should be placed directly beneath the floor slab to act as a vapor barrier. The impermeable membrane should be protected by two inches of fine, moist sand placed both above and below the membrane. The sand cover will provide protection for the membrane and will promote uniform curing of the concrete slab. The sand cover should be moistened and tamped prior to slab placement.

Seismic Criteria

Figure 16-J in the 1997 Uniform Building Code classifies the subject site as being within Seismic Zone 3. Based on the subsurface conditions encountered at the site and published geologic literature, it is our opinion that Soil Profile Type S_D should be used to describe the average soil properties within the upper 100 feet beneath the site. This designation describes a stiff soil profile with shear wave velocity between 600 to 1,200 feet per second, Standard Penetration Test values between 15 and 50, and undrained shear strength between 1,000 to 2,000 psf.

Soil liquefaction is a condition where loose, saturated granular soils loose strength during the ground shaking associated with an earthquake. Groundwater was encountered at the site at a depth on the order of 7 feet below grade, with city designated high water level approaching 5 feet below existing grades. Although no site specific liquefaction analyses was completed for this project, it is our opinion that the existing site soils are at least moderately susceptible to liquefaction due to the relatively high groundwater table, and overall clean nature of the sand observed near the groundwater elevation in the test pits. If further assessment of liquefaction potential is required, we recommend that additional field exploration (consisting of drilling at least one boring to a depth on the order of 30 to 40 feet) and analyses be performed. We are available to assist in this evaluation, if necessary.

Erosion Control

Erosion and sedimentation controls are recommended during construction to reduce the potential impacts to adjacent areas. Erosion control measures should be designed to prevent sediment transport. This may be accomplished by constructing water bars or utilizing other methods to control surface water runoff, and constructing silt fences to control sedimentation. If construction is accomplished during the winter months, we further recommend that temporary erosion protection be provided consisting of covering exposed soil areas with plastic sheeting, jute matting, and/or straw.

We recommend that all bare soil areas be planted or mulched as soon as possible. It may be necessary to provide temporary erosion protection until vegetation has been reestablished.

Pavements

The native medium dense to dense sand and silty sand are considered suitable for subgrade support of pavements. We recommend that the subgrade in pavement areas be prepared as recommended in the Site Preparation section of this report. The upper one-foot of pavement subgrade soils should be compacted to at least 95 percent of the maximum dry density determined in accordance with the ASTM D-1557 test method. The extent of any soil improvement or replacement can only be determined at the time of construction. Finished subgrade surfaces should be constructed to facilitate drainage and prevent ponding of water below the pavement section. We recommend that a layer of crushed base course be placed between the prepared subgrade or structural fill, and the pavement working surface material (asphalt or concrete). Base course material under Portland concrete and asphalt pavement should conform to Specification 9-03.9(3) of the WSDOT/APWA 2000 Standard Specifications, or current City of Arlington or Snohomish County standards. Base material should be moisture conditioned and compacted to a minimum of 95 percent of the maximum dry density per ASTM D-1557.

Stormwater Drainage

We understand that on-site stormwater disposal systems are being considered at the site. We recommend that all on-site drainage systems conform to applicable City of Arlington and/or Snohomish County regulations. To assist in design of these systems, we performed falling-head infiltration tests near test pit locations TP-1, TP-2, TP-3, and TP-7. These tests were performed at approximately 2 feet below the surface to maintain the 3-foot separation from the City designated high water level of 5 feet below the surface. In addition, we performed grain size testing on samples collected in TP-4 (west-middle side of lot) and TP-6 (east-middle side of lot) near the two-foot depth for reference to the USDA Textural Triangle classification.

Infiltration rates varied at the test locations and appeared to be directly correlative with the amount of fines (silt and clay) in the soil at the tested depths. Overall, the infiltration rates were observed to be slower on the western portion of the lots. A summary of test results is presented below:

Location	Assessed Infiltration Rate (minutes/inch)*
Near TP-1, southwest corner	10 min/inch
Near TP-2, northwest corner	4.5 min/inch
Near TP-3, northeast corner	< 1 min/inch
Near TP-7, southeast corner	4 min/inch

* These values are field results. Appropriate safety factors should be applied for design.

The grain-size tests performed in TP-4 and TP-6 near the 2-foot depth interval suggest a USDA classification of "Sandy Loam" for the TP-4 sample, and "Sand" for the TP-6 sample. These tests appear to confirm that overall, the upper soils on the west side of the lot typically contained more silt than those soils on the east side of the lot. Per the "Stormwater Management Manual for the Puget Sound Basin", infiltration rates on the order of 7.25 min/inch (for "Sand") and 25 min/inch ("Sandy Loam") are reported. The "Sand" is considered within Hydrologic Soil Group "A", while the "Sandy Loam" is listed as Hydrologic Soil Group "B".

CLOSURE

This report has been prepared in accordance with generally accepted geotechnical engineering practices for the exclusive use of ANP Construction for specific application to the planned Philadelphia Hardware facility on 166th Place NE in Arlington, Washington. The conclusions and recommendations presented in this report are based on the explorations accomplished for this study. The number, location, and depth of the explorations were completed within the site and scope constraints of the project so as to yield the information necessary to formulate our recommendations. The plans for this project were in the preliminary stage at the time this report was written. Under the circumstances, it is recommended that we be provided the opportunity for general review of the project plans and specifications in order to confirm that the recommendations and design considerations presented in this report have been properly interpreted and implemented into the project design package.

The integrity and performance of foundation systems depend greatly on proper site preparation and construction procedures. Field judgement by a qualified engineer will be necessary in order to determine the adequacy of the site drainage and foundation support systems. Therefore, because of our familiarity with the site soils, we recommend that Zipper Zeman Associates, Inc. be retained to provide geotechnical engineering services during earthwork and foundation construction of the facility. If variations in the subsurface conditions are observed at the time of construction, we would be able to provide additional geotechnical engineering recommendations to the contractor and owner in a timely manner as the project construction progresses.

We trust this information meets your current needs. If you have any questions, or we can be of further assistance, please contact us at (425) 771-3304. We appreciate this opportunity to be of service to you.

Respectfully submitted,

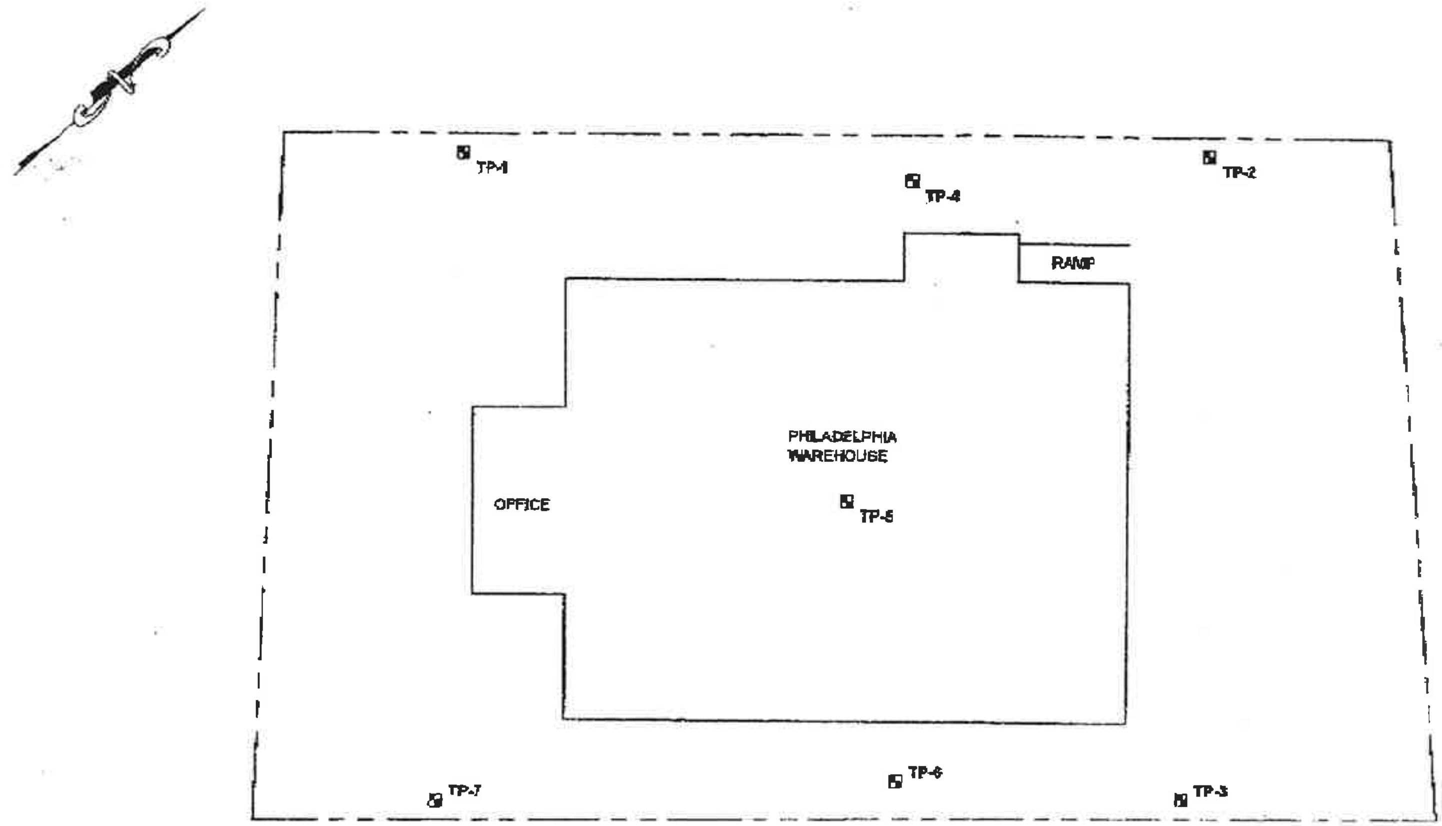
Zipper Zeman Associates, Inc.

Timothy H. Roberts, P.E., P.G.
Project Engineer

John E. Zipper, P.E.
President

JOHN E. ZIPPER
STATE OF WASHINGTON
22011
REGISTERED PROFESSIONAL ENGINEER
EXPIRES 1/24/03

Enclosures: Figure 1 – Site and Exploration Plan
Test Pit Logs (TP-1 through TP-7)
Gradation Test Results



LEGEND:

TP-1 APPROXIMATE TEST PIT LOCATION AND NUMBER

Zipper Zeman Associates, Inc. Geotechnical and Environmental Consulting 18905 33rd Avenue West, Suite 117 Lynnwood, Washington 98036 Tele: (425) 771-3304 Fax: (425) 771-3549	Project No: J-1418 Drawn by: J.Duncan Date: July, 2002 Scale: As Noted	PHILADELPHIA HARDWARE Arlington, Washington FIGURE 1 - SITE AND EXPLORATION PLAN

Test Pit TP-1

Depth (feet)	Material Description	Sample No.
0.0-1.0	Loose, damp, light brown, silty, fine SAND with organics.	S-1@0.5
1.0-2.0	Medium dense, moist, light to dark brown, silty SAND.	S-2@1.5
2.0-3.5	Medium dense to dense, moist, mottled orange-brown, silty SAND.	S-3@2.0-2.5
3.5-8.5	Medium dense, wet to saturated, gray, SAND with trace silt.	

Test pit terminated at 8.5 feet on 6/28/02.
Groundwater seepage observed at 7.5 feet.
Severe caving observed below 7.5 feet.

Test Pit TP-2

Depth (feet)	Material Description	Sample No.
0.0-1.0	Loose, dry, gray-brown, sandy GRAVEL. (Fill)	
1.0-2.5	Medium dense, moist, light brown, silty SAND. Grades to brown, silty, fine grained SAND with charcoal and minor roots to 2.0 feet.	
2.5-4.0	Medium dense, moist, mottled orange-brown, fine grained silty SAND.	
4.0-9.5	Medium dense, wet to saturated, gray, SAND with trace silt.	

Test pit terminated at 9.5 feet.
Groundwater seepage observed at 7.0 feet.
Severe caving observed below 7.0 feet.

Test Pit TP-3

Depth (feet)	Material Description	Sample No.
0.0-0.5	Loose, moist, light brown, silty SAND with organics (Topsoil).	
0.5-1.5	Medium dense, moist, mottled orange-brown, silty SAND with some gravel.	
1.5-6.5	Medium dense, moist to wet, gray-brown, silty SAND with some gravel. Grades to saturated at 6.0 feet.	S-1@2.5'

Test pit terminated at 6.5 feet on 6/28/02.
Groundwater seepage observed at 6.0 feet.
Severe caving observed below 6.0 feet.

Test Pit TP-4

Depth (feet)	Material Description	Sample No.
0.0-2.0	Loose, dry to moist, gray, sandy GRAVEL (Fill) with organics.	
2.0-6.0	Medium dense to dense, moist, dark brown, silty SAND. Grades to mottled orange-brown, silty SAND with charcoal at 2.5 feet.	
6.0-7.0	Medium dense, moist to wet, gray, SAND with trace to some silt.	S-1@3.0

Test pit terminated at 7.0 feet on 6/28/02.
Groundwater seepage observed at 6.5 feet.
Severe caving observed below 6.5 feet.

Test Pit TP-5

Depth (feet)	Material Description	Sample No.
0.0-1.0	Loose, dry, gray, silty SAND with some gravel. (Topsoil and Organics)	
1.0-3.0	Medium dense to dense, moist to wet, mottled orange-brown, medium SAND with trace to minor silt and gravel.	S-1@2.0
3.0-7.0	Medium dense, wet, gray, SAND with trace silt and gravel.	

Test pit terminated at 7.0 feet on 6/28/02.
Groundwater seepage observed at 6.5 feet.
Severe caving observed below 6.5 feet.

Test Pit TP-6

Depth (feet)	Material Description	Sample No.
0.0-0.5	Loose, damp, light brown, silty SAND with organics. (Topsoil)	
0.5-1.5	Loose to medium dense, brown mottled, silty SAND.	
1.5-3.0	Medium dense, moist, rust-gray, fine to medium SAND with minor silt.	S-1@2.5

Test pit terminated at 3.0 feet on 6/28/02.
No groundwater seepage observed.
No caving observed.

Test Pit TP-7

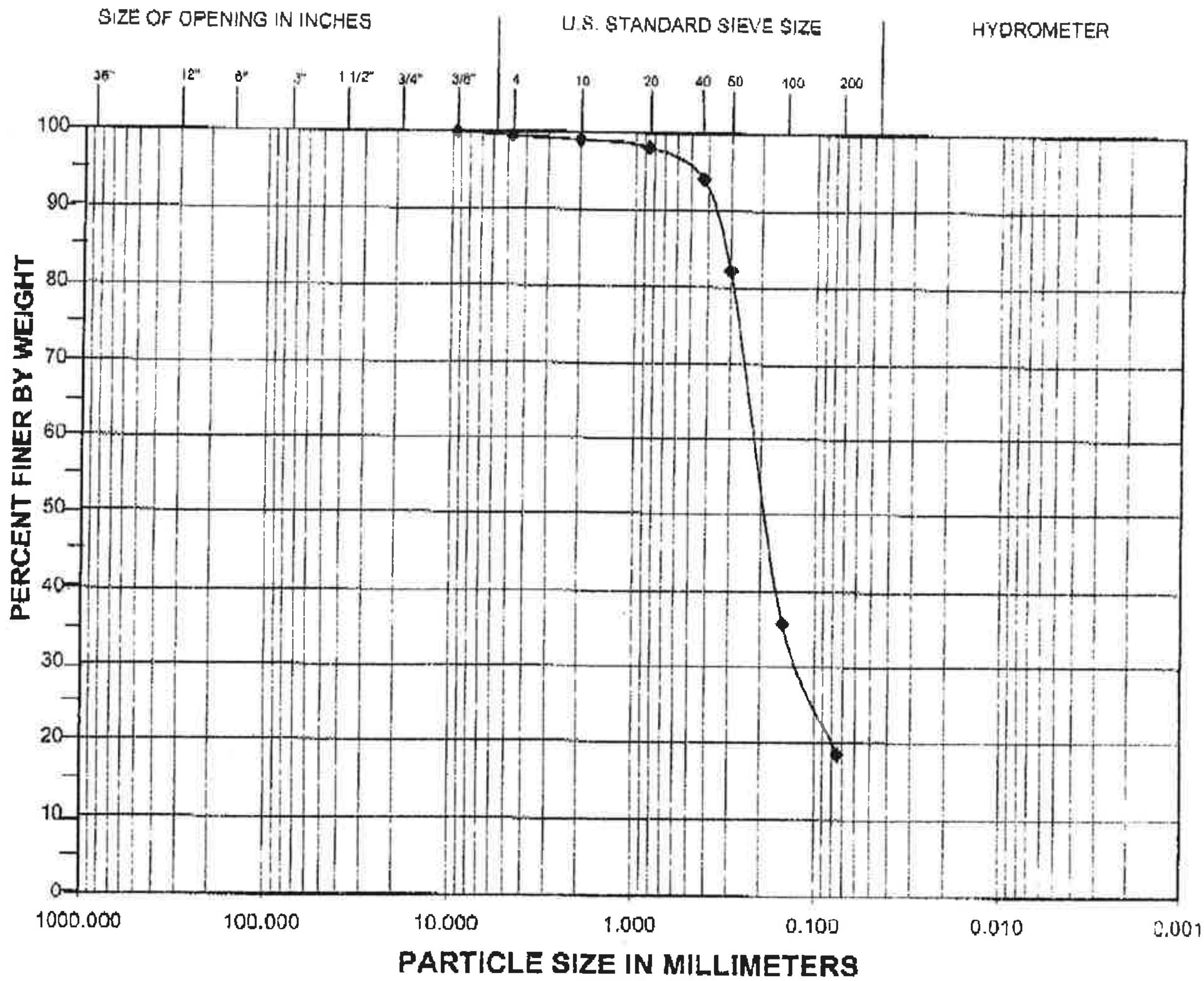
Depth (feet)	Material Description	Sample No.
0.0-2.0	Loose, damp, light brown, silty SAND with organics and some gravel. (Topsoil)	
2.0-5.0	Medium dense, moist, mottled orange-brown, silty SAND with trace to some gravel.	
5.0-8.0	Medium dense, wet to saturated, gray, gravelly, medium SAND.	

Test pit terminated at 8.0 feet on 6/28/02.
Groundwater seepage observed at 7.5 feet.
Severe caving observed below 7.5 feet

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D1140



		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
BOULDERS	COBBLES	GRAVEL		SAND			FINE GRAINED	

Comments:

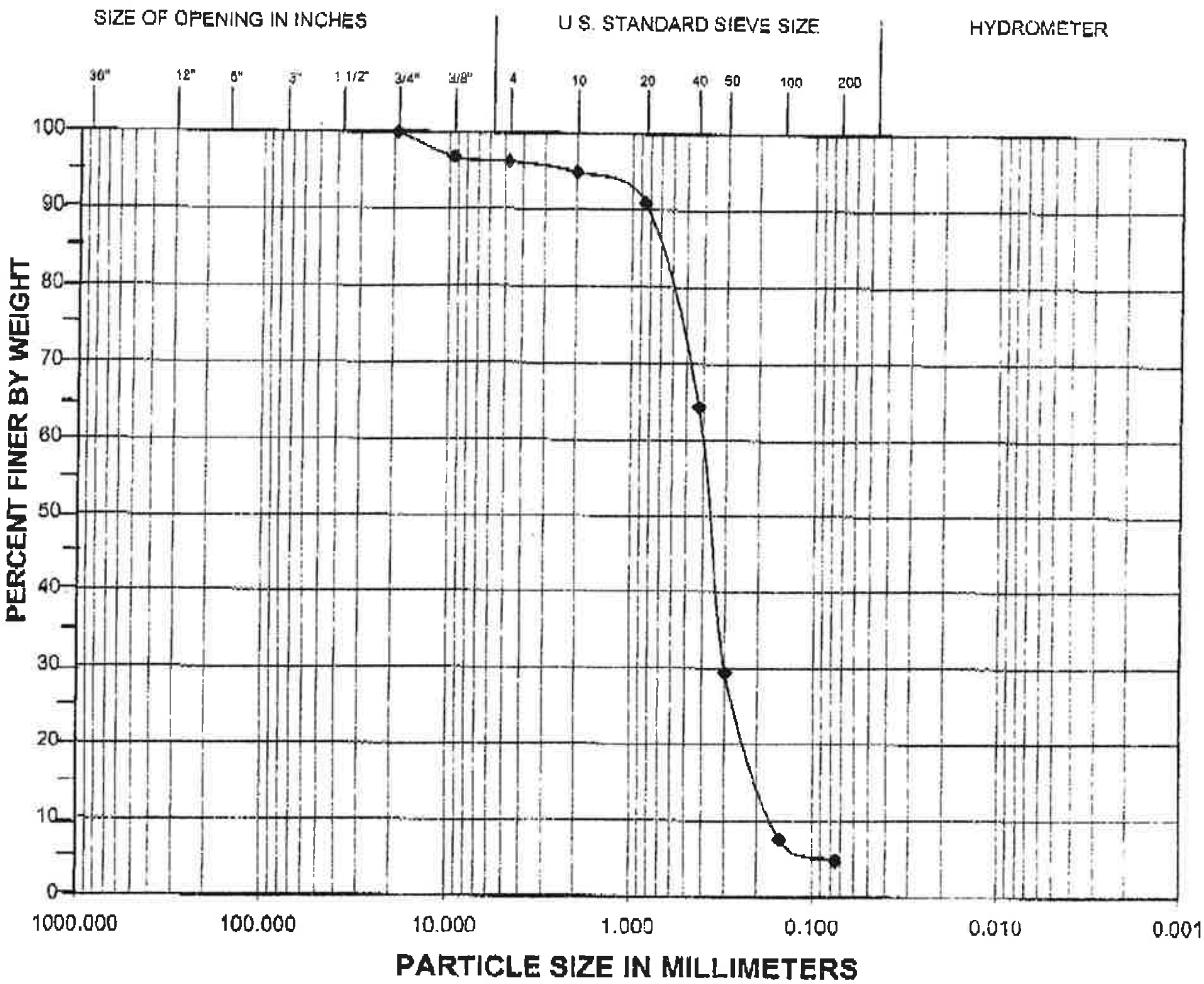
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
IT-2	S-1	2.0 - 2.5	21	18.4	SAND with some Silt

Zipper Zeman Associates, Inc. Geotechnical and Environmental Consulting	PROJECT NO: J-1416 DATE OF TESTING: 7/1/02	PROJECT NAME. Philadelphia Hardware

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D1140



BOULDERS	COBBLES	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
		GRAVEL		SAND			FINE GRAINED	

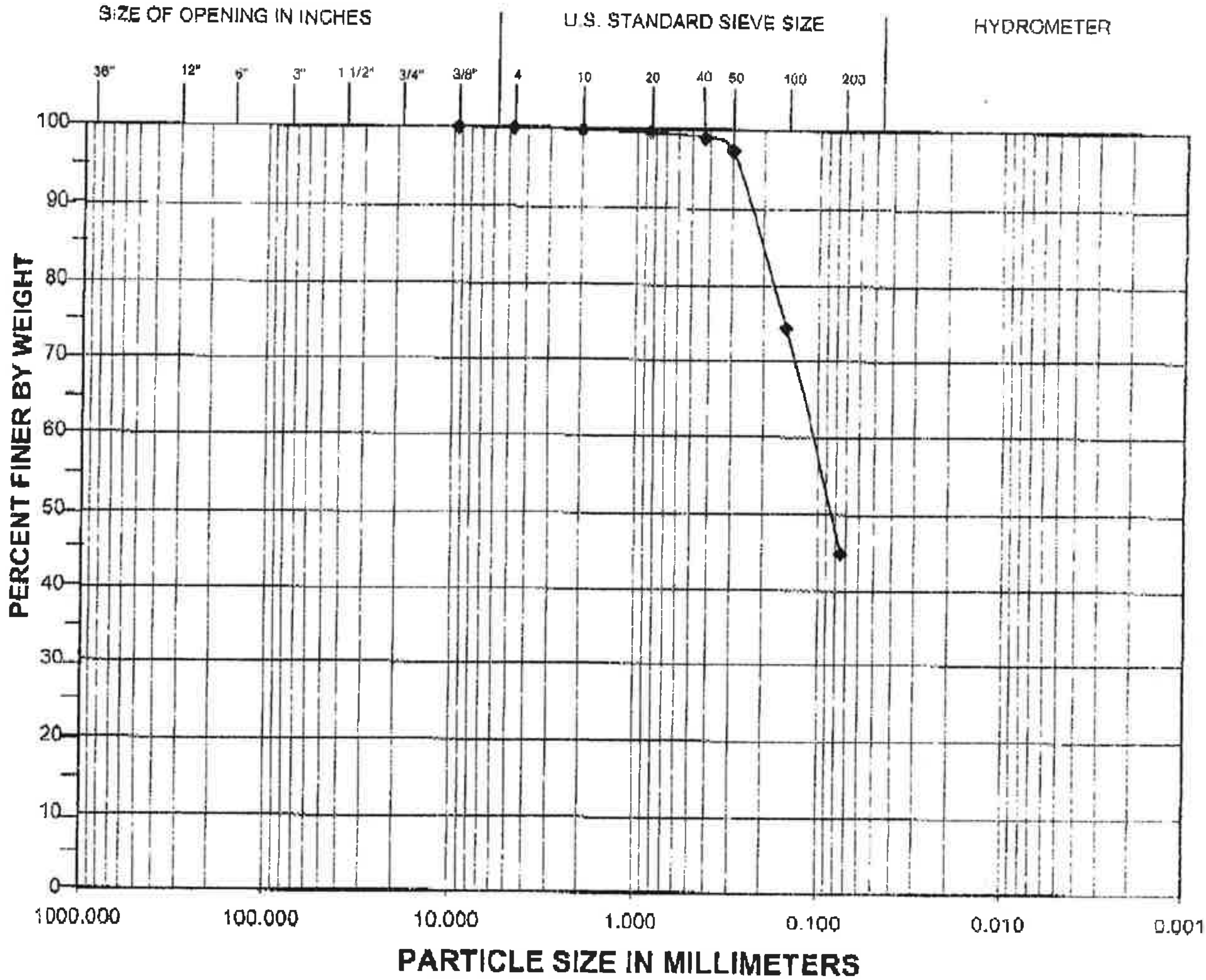
Comments:

Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
IT-4	S-1	2.0 - 2.5	12	4.6	SAND with trace gravel and silt

Zipper Zeman Associates, Inc. Geotechnical and Environmental Consulting	PROJECT NO: J-1418 DATE OF TESTING: 7/1/02	PROJECT NAME: Philadelphia Hardware

GRAIN SIZE ANALYSIS Test Results Summary

ASTM D1140



		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
BOULDERS	COBBLES	GRAVEL		SAND			FINE GRAINED	

Comments:

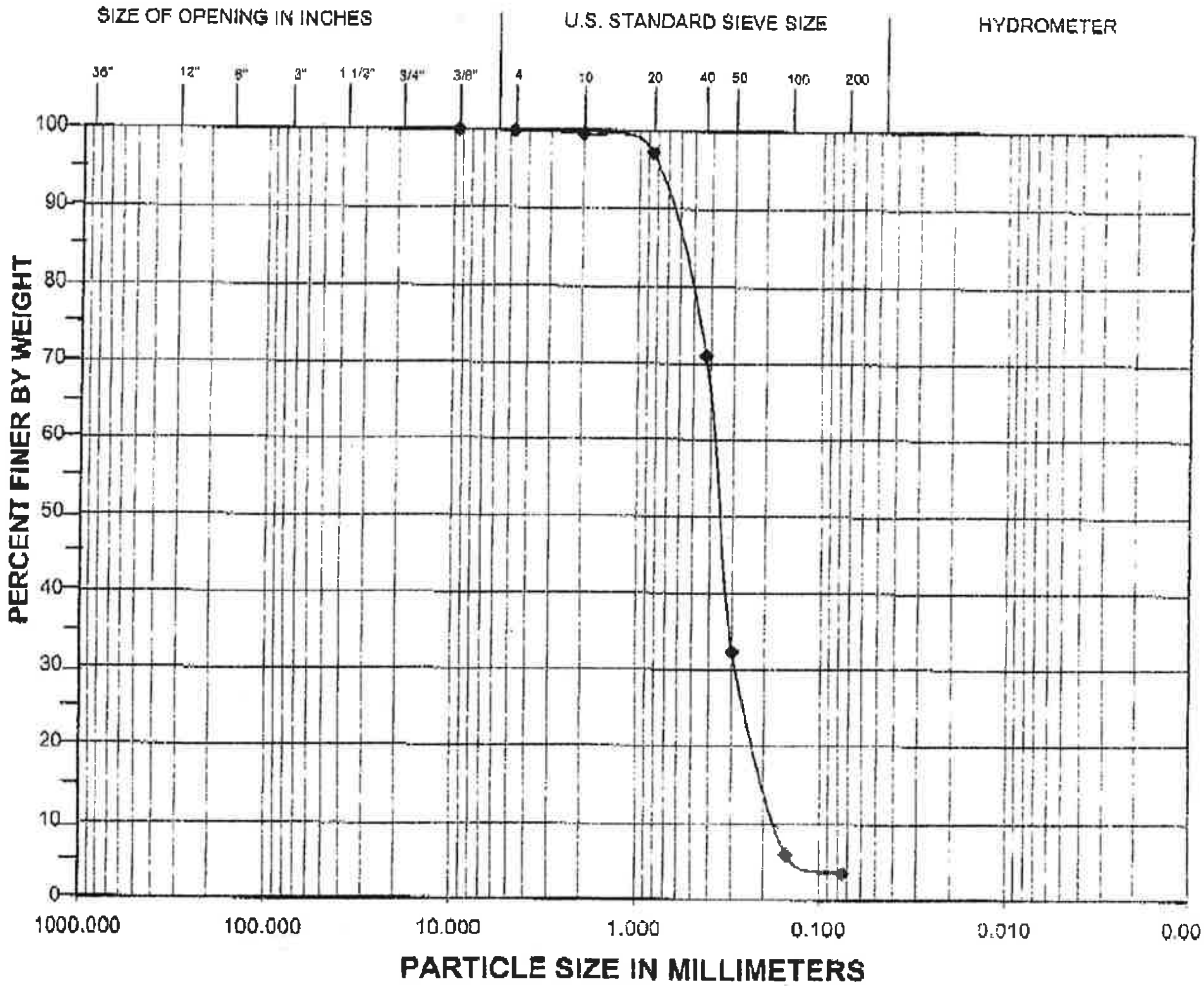
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
TP-4	S-1	2.0 - 2.5	21	44.8	silty SAND

Zipper Zeman Associates, Inc. Geotechnical and Environmental Consulting	PROJECT NO: J-1416 DATE OF TESTING: 7/1/02	PROJECT NAME: Philadelphia Hardware

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D1140



BOULDERS	COBBLES	GRAVEL		SAND			FINE GRAINED	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

Comments:

Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
TP-6	S-1	2.0 - 2.5	10	3.5	SAND with trace of silt

Zipper Zeman Associates, Inc. Geotechnical and Environmental Consulting	PROJECT NO: J-1418 DATE OF TESTING: 7/1/02	PROJECT NAME: Philadelphia Hardware

MANUAL, U.S. Department of Agriculture, October 1993, page 136). This manual only considers soil passing the #10 sieve (2 mm) (U.S. Standard) to determine percentages of sand, silt, and clay for use in Figure 7.1 (USDA Textural Triangle). However, many soil test laboratories use the ASTM soil size distribution test procedure (ASTM D422), which considers the full range of soil particle sizes, to develop soil size distribution curves. The ASTM soil gradation procedure must not be used with Figure 7.1.

Three Methods for Determining Long-term Infiltration Rate for Sizing the Infiltration Basin, Trench, or Swale

For designing the infiltration facility, the site professional should select one of the three methods described below that will best represent the long-term infiltration rate at the site. The long-term infiltration rate should be used for routing and sizing the basin/trench for the maximum drawdown time of 24 hours. It is suggested that Method 1 be used to corroborate and compare the infiltration rate estimates of the other methods, using the appropriate correction factors. Verification testing of the completed facility is strongly encouraged using Site Suitability Criterion (SSC) # 9.

Method 1 — USDA Soil Textural Classification

Table 7.1 correlates USDA soil texture and infiltration rates for homogeneous soils. It is based on the correlation developed by Rawls, et. al., with minor changes in the infiltration rates based on WEF/ASCE (1998). The infiltration rates provided in Table 7.1 represent short-term conservative rates for homogeneous soils which should be used for treatment soil suitability determinations. However, these rates do not represent the effects of site variability and long-term clogging due to siltation and biomass buildup in the infiltration facility.

Table 7.1 Recommended Infiltration Rates based on USDA Soil Textural Classification.

	***Short-Term Infiltration Rate (in./hr)**	**Correction Factor, CF**	**Estimated Long-Term (Design) Infiltration Rate (in./hr)**
Clean sandy gravels and gravelly sands (i.e., 90% of the total soil sample is retained in the #10 sieve)	20	2	10 **
Sand	8	4	2***
Loamy Sand	2	4	0.5
Sandy Loam	1	4	0.25
Loam	0.5	4	0.13

* From WEF/ASCE, 1998.

** Not recommended for treatment

*** Refer to SSC-4 and SSC-6 for treatment acceptability criteria