Betrozoff Jones
Preliminary
Stormwater Site Plan

May 13, 2013
Revised November 5, 2013

Prepared for

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Chapter 1.0 Project Overview

The proposed Betrozoff Jones project is located in the southwest quarter of Section 26, Township 26 North, Range 5 East, Willamette Meridian, in King County, Washington. More specifically, the project site is located at the 11845 and 11818 Woodinville-Redmond Road Northeast, Redmond, WA 98052. The overall project includes two parcels (942850-0070 and 942850-0065) totaling 8.04 acres. Parcel 942850-0070 will be revised via a boundary line adjustment.

Currently, the project site is developed with two residences with multiple building structures located on the western portion of the site. The site is covered with native forested area and accessed via paved driveway roads. See Figure 1.2 – Predeveloped Basin Map for details. According to the Geotechnical Engineering Report dated February 6, 2013 by Robinson Noble, the site is covered with older alluvium and glacial till.

The proposed development consists of 31 single family residential lots and the associated roadways and landscaped areas. Furthermore, this project is classified as a "Large Project" where the project is required to meet the Minimum Requirements #1-9 in Chapter 2 of the Stormwater Notebook and comply with requirements on Chapter 6 of the 2012 City of Redmond Stormwater Management Technical Notebook (SWMTN). See the attached Figure 1.1 – Vicinity Map for the specific location and Figure 1.3 – Drainage Basin, Sub-basins and site characteristics.

The project was also evaluated for Low Impact Development (LID), however based on the Geotechnical Engineering Report, infiltration through existing soils was not considered feasible. Furthermore, due to the proposed development type and density, LID features such as dispersion, rain gardens, and retention of native growth are also not practical. In summary, the proposed LID feature is to use compost amended soils in landscaping.

Stormwater runoff will be collected via catch basins and conveyed through a piped conveyance system towards the stormwater quality and detention vault. The release rate from the vault will match the pre-developed forested condition of the overall project site. From there the stormwater will be conveyed to the existing ditch along 154th Place NE and continue along existing ditches until runoff enters the Sammamish River few miles downstream. See Section 4 for the downstream analysis.

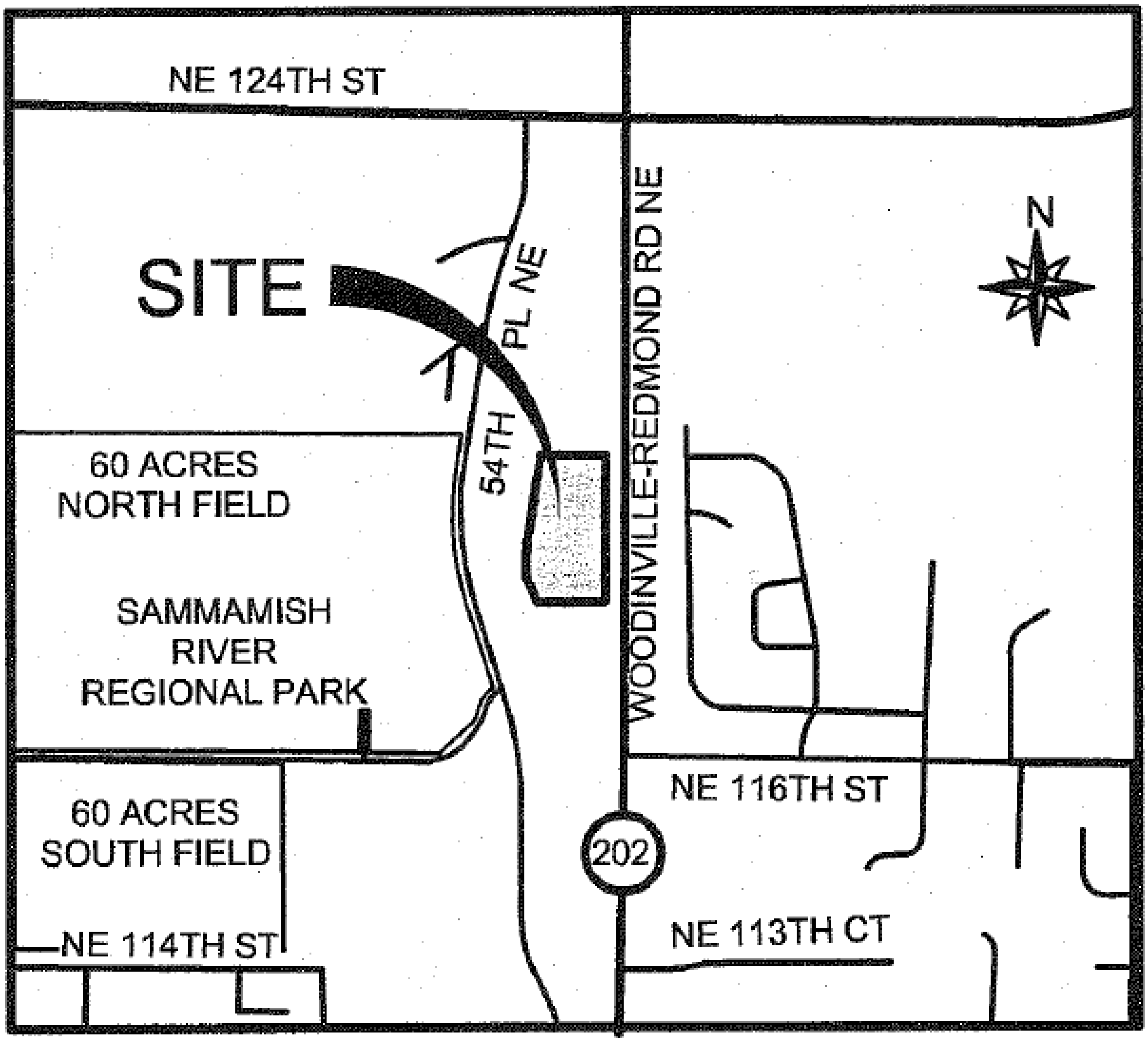
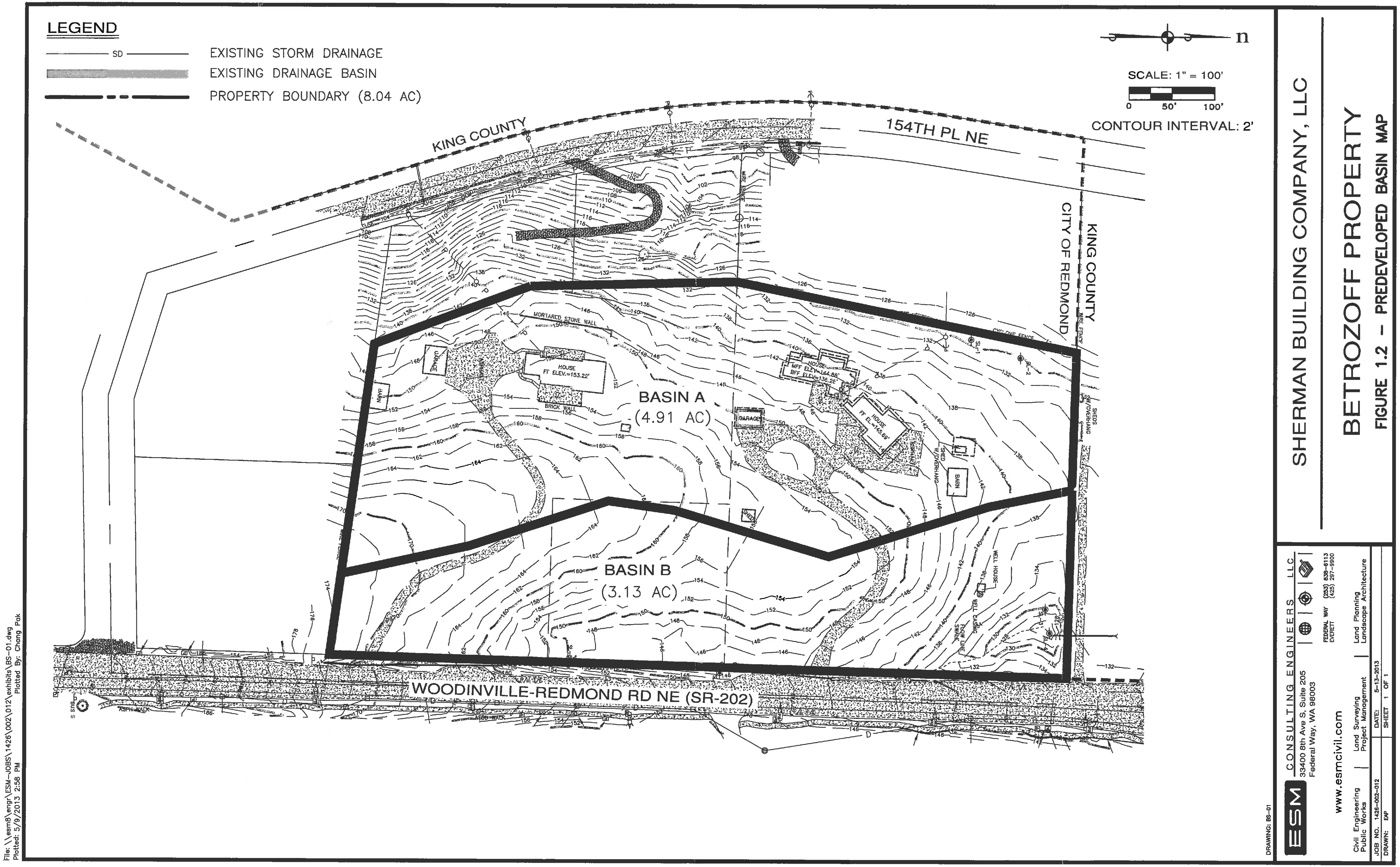


Figure 1.1 Vicinity Map



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LEGEND

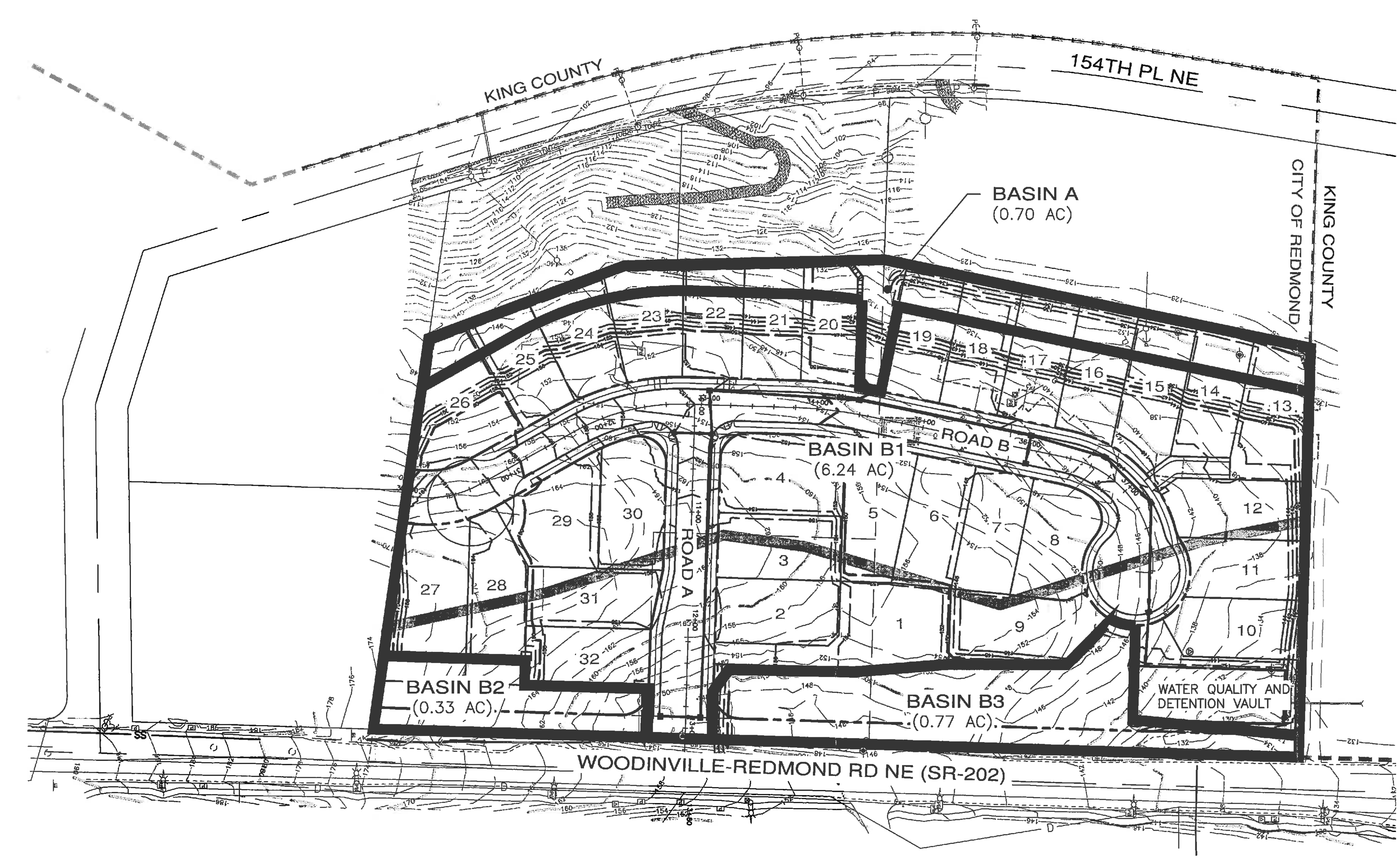
—— SD ——	EXISTING STORM DRAINAGE
——→	PROPOSED 12"ø STORM DRAINAGE
(grey band)	EXISTING DRAINAGE BASIN
(heavy line)	PROPOSED DRAINAGE BASIN

n

SCALE: 1" = 100'

0 50' 100'

CONTOUR INTERVAL: 2'



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SHERMAN BUILDING COMPANY, LLC

BETROZOFF PROPERTY

FIGURE 1.3 – DRAINAGE BASIN, SUB-BASINS, AND SITE CHARACTERISTICS

DRAWING: BS-02

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Chapter 2.0 Discussion of Minimum Requirements

The proposed project will address the minimum requirements #1 through #9 as documented below.

Minimum Requirement #1: Preparation of a Stormwater Site Plans
This report meets this minimum requirement.

Minimum Requirement #2: Construction Stormwater Pollution Prevention Plan (SWPPP)
The report will be provided with the final submittal, prior to construction.

Minimum Requirement #3: Source Control of Pollution
The Temporary Erosion and Sedimentation Control (TESC) Design will be shown on the final construction plans and additional information will be provided with the final Stormwater Site Plan.

Minimum Requirement #4: Preservation of Natural Drainage Systems and Outfalls
The proposed project will follow the existing natural drainage system and outfalls. For more detail see Chapters 4.0 and 5.0.

Minimum Requirement #5: On-Site Stormwater Management
On-Site stormwater management is described in Chapter 5.0.

Minimum Requirement #6: Runoff Treatment
Runoff treatment will be provided in a proposed stormwater vault. For more information, see Chapter 5.0.

Minimum Requirement #7: Flow Control
Flow control will be provided in a proposed stormwater vault. For more information, see Chapter 5.0.

Minimum Requirement #8: Wetlands Protection
Wetlands will not be impacted by the proposed project improvements.

Minimum Requirement #9: Operation and Maintenance
O & M Manual will be provided at time of final engineering plan review.

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Chapter 3.0 Existing Conditions Summary

The 8.04-acres existing site is densely covered forest area with two existing single family residences located on the western portion of the site. There are two natural drainage basins on the project site that connect within a quarter mile downstream. Basin A has a natural discharge point located on the northwestern portion of the project site and Basin B has a natural discharge point where it flows towards existing 12-inch diameter culvert. See Figure 1.2 Predeveloped Basin Map which flows north following the natural topography. Both discharge flows combine within a quarter mile further downstream and ultimately flow into Sammamish River northwest of the site. Table 3.1 Existing Land Use displays the current area breakdowns and Figure 1.2 Predeveloped Basin in Section 1.0 of this report. For more information see Section 4.0.

Table 3.1 – Existing Land Use	**Total**
Basin A – modeled as forested, C	4.91
Basin B – modeled as forested, C	3.13
Overall Basin (acres)	**8.04**

According to the Geotechnical Engineering Report dated February 6, 2013 by Robinson Noble, the site is covered with older alluvium and glacial till. A copy of the Geotechnical Engineering Report can be found in Appendix A.

The adjacent properties are developed with single family and multi-family residences.

There are no known historical drainage problems such as flooding, erosion, etc. There are also no known difficult site conditions, sensitive areas, critical areas, fuel tanks, groundwater wells, or septic systems located on the property. The project site is not located in an aquifer recharge area, wellhead protection area, a Superfund area or a 100-year flood hazard zone.

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Chapter 4.0 Offsite Analysis

An offsite analysis for the property has been completed as part of the Preliminary Stormwater Site Plan for the project.

The following Level 1 downstream analysis reviews the four tasks (outlined in the 2005 Western Washington DOE Stormwater Design Manual). These tasks were completed in an effort to avoid any negative downstream impacts to the existing drainage system.

The four tasks outlined under this review are:

- Task 1 – Define and map the study area
- Task 2 – Review all available information on the study area
- Task 3 – Field inspect the study area
- Task 4 – Describe the drainage system, and its existing and predicted problems

Task No. 1. Define and map the study area

The site is located in the Sammamish River Basin and found within the Sammamish River Sub-Basin boundary. The project site is not located in a landslide hazard area, flood plains, geologic sensitive area, critical drainage area, or landslide hazard area.

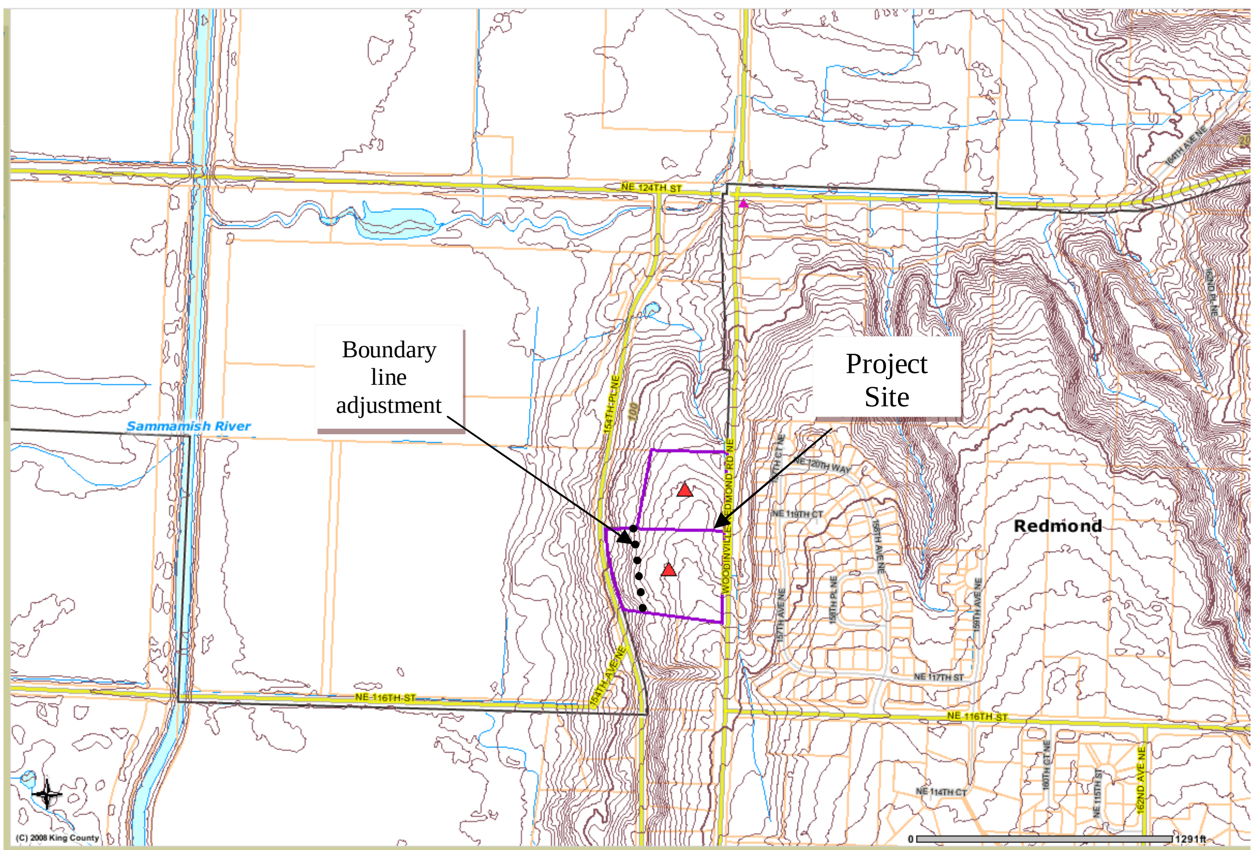

Figure 3.1.1 Offsite Analysis Overview

Task No. 2. Resource Review

The following resources from the King County iMAP website were used in conjunction with the Level 1 analysis for the project site:

Adopted basin plans

- The site is located in the Sammamish River Basin and found within the Sammamish River Sub-Basin boundary.

Sensitive Areas Folio

- Wetlands – None mapped on the project site.
- Streams and 100-Year Floodplains – None mapped on the project site.
- Erosion Hazard Areas – None mapped on the project site.
- Seismic Hazard Areas – None mapped on the project site.
- Coal Mine Hazard Areas – None mapped.

For more information, see the attached maps following this section.

Drainage Complaints

- The drainage complaint list has been attached following this section. The drainage complaints have been obtained and none were applicable for the project site.

U. S. Department of Agriculture, King County Soils Survey

- The soils on the project site are Kitsap silt Loam 2 to 8 percent and 15 to 30 percent, with a Hydrologic Soil Group C. Furthermore, the Geotechnical Engineering Report by Robinson Noble advises that the site is underlain by deposits of older alluvium and glacial till.

Flow Control Facility

- Flow control device is required for proposed project and stormwater discharges shall match developed discharge durations to pre-developed durations for the range of pre-developed discharge rates from 50% of the 2-year peak flow up to the full 50-year peak flow. The pre-developed condition to be matched shall be a forested land cover.

Water Quality Facility

- The project site is shown in the Water Quality Treatment Area as Basic Water Quality.

Task No. 3.   Field Inspection

A site reconnaissance was performed on February 14, 2013, for the purpose of analyzing the proposed project site and its upstream and downstream corridors. The weather conditions were dry, 46 degrees Fahrenheit and partly cloudy.

Upstream Analysis – There are no upstream flows to the project site.

Downstream Analysis – Runoff from the site (Basins A and B) gradually sheet flows north, through mowed pasture, at the boundary edges of the site. Flows continue along a natural grassy channel with slope of approximately 12 percent. Figure 3.3.1 shows the approximate location of each reach point with descriptions. Ultimately the combined flow from Basins A and B described in Section 3 site will be conveyed into Sammamish River located directly west from project site, approximately quarter mile downstream.

All existing channels along the 154th Place appeared to be fully functional without any apparent backups.

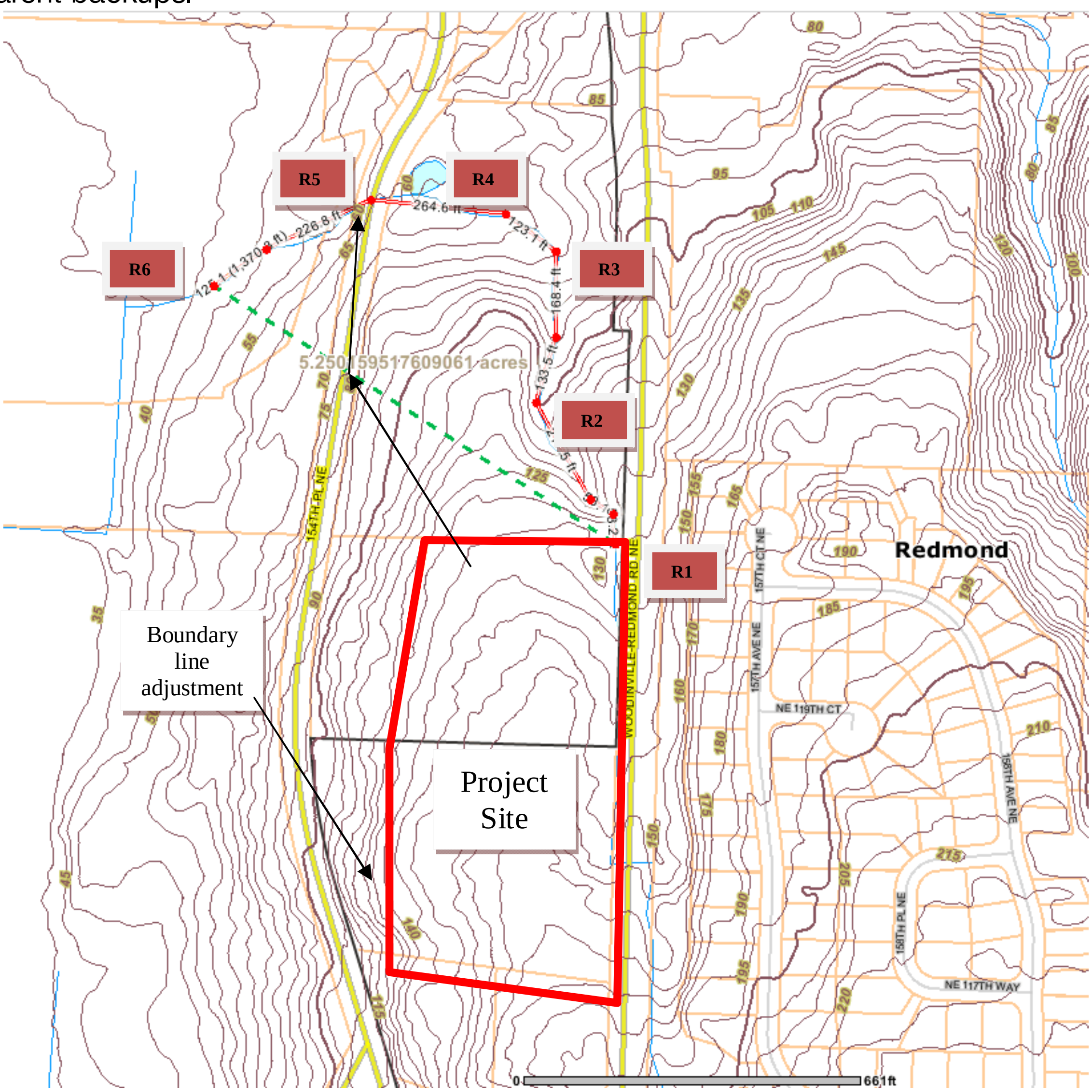

Figure 3.3.1 Site Contour Map with Reach Points

Figure 3.3.2 Current Offsite Condition
(Existing site conditions, looking from the northeast corner of the site towards the northwest corner)

Reach 1 (0 to 10'+)
Runoff from the existing Basin B is conveyed by this culvert at the northeast corner of the site. From there flow enters the natural grassy channel with approximate 12 percent slope which flows north. A small amount of flow was visible at the time of visit.

Figure 3.3.3 Looking south at 12" Culvert under existing driveway

Figure 3.3.4 Close up of existing 12" Culvert

Reach 2 (10' to 250'+)
A grassy channel continues to carry runoff north. Standing water was observed in portions of the channel.

Figure 3.3.5 Grassy Channel
(Looking south from 250' south of Reach 1)

Reach 3 (250' to 540'+)
Tje grassy channel continues to flow northwest towards a small pond. At this point there were visible signs of additional flows being combined. The combined flow continues northwest.

Figure 3.3.6 Continuation of Grassy Channel
(Looking south 290' of Reach 2)

Reach 4 (540' to 845'+)
At the small pond, water was observed approximately 2 feet deep with flows out of a rectangular riser to convey flow west across 154th Place NE.

Figure 3.3.7 Small Pond
(Looking east from the pond outlet point)

Figure 3.3.8 Small Pond Outlet

Reach 5 (845' to 1,160'+)
After flow reaches the west side of 154th Place NE, it continues down to another small pond located west of Reach 4.

Figure 3.3.9 Small Pond
(Located west of 154th Place NE)

Reach 6 (1,160'+ to Sammamish River)
After flow reaches the west side of 154th Place NE, the King County IMAP shows it continues down approximately a half mile until it drains into Sammamish River. Access to this portion of the downstream analysis was not available.

Task 4 – Describe the drainage system, and its existing and predicted problems

During the site visit, there did not appear to be any problems with the offsite drainage system or any backed up runoff.

The drainage complaint list has been attached following this section. The drainage complaints have been obtained and none were applicable for the project site.

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Chapter 5.0 Permanent Stormwater Control Plan

EXISTING SITE HYDROLOGY

Following are the assumptions and site parameters for the pre-developed conditions:

The site is currently developed with two current residences with multiple out buildings and surrounding covered in native trees with paved driveways accessing off Woodinville-Redmond Road Northeast. The overall existing drainage basin is 8.04 acres with two natural drainage discharge points located northern portion of the site (Basins A and B). The overall basin was modeled as hydrologic group C, moderate forested condition in WWHM3, matching the original condition of the overall site. Both Basins A and B flows will combine further downstream from the site, within a quarter mile, towards the Sammamish River. See Table 3.1 Existing Land Use in Section 3 of this report for area breakdown.

The associated stormwater existing condition flows are shown in Table 5.1 below.

Table 5.1 – Existing Conditions WWHM Flows	**2-year** (cfs)	**5-year** (cfs)	**10-year** (cfs)	**25-year** (cfs)	**50-year** (cfs)	**100-year** (cfs)
Basin A	0.0800	0.1248	0.1492	0.1741	0.1888	0.2010
Basin B	0.1258	0.1962	0.2346	0.2736	0.2968	0.3159
Overall Basin (cfs)	0.2056	0.3207	0.3833	0.4471	0.4851	0.5163

DEVELOPED SITE HYDROLOGY

The proposed stormwater drainage basins were defined based on the existing topography of the site, the proposed site grading, and the proposed drainage system. Figure 1.3 – Drainage Basin, Sub-basins and site characteristics shows the overall drainage basin and the sub basins.

The impervious areas in the right-of-way consist of roadway and sidewalk. The impervious areas on the lots consist of building roof, driveway, walkway, and deck areas, estimated at 3,000 square feet per lot. The impervious and pervious areas - assume road, driveway, and building roof areas to be 100 percent impervious and the landscaped areas to be 100 percent pervious.

Table 5.2 below summarizes the proposed basin areas.

Table 5.2 – Basin Area	Pervious, C (acres)		Impervious (acres)	Total (acres)
Basin A (landscaped)	0.70			0.70
Basin B1		2.84	3.40	6.24
Basin B2 (forested)	0.33			0.33
Basin B3 (forested)	0.77			0.77
Overall Basin (acres)	1.80	2.84	3.40	8.04
Total Vault		**2.84**	**3.40**	**6.24**

The associated overall basin stormwater flows are shown in Table 5.3 below.

Table 5.3 – Proposed Conditions WWHM Flows	**2-year** (cfs)	**5-year** (cfs)	**10-year** (cfs)	**25-year** (cfs)	**50-year** (cfs)	**100-year** (cfs)
Basin A	0.0388	0.0608	0.0773	0.1003	0.1188	0.1386
Basin B1	1.0704	1.3349	1.5093	1.7301	1.8956	2.0623
Basin B2	0.0084	0.0132	0.0157	0.0184	0.0199	0.0212
Basin B3	0.0197	0.0307	0.0367	0.0428	0.0465	0.0494
Overall Basin (cfs)	1.1008	1.3812	1.5669	1.8029	1.9803	2.1594

While the developed Basin A is smaller than pre-developed (4.92 acres to 0.70 acres) and Basin B is larger (3.13 acres to B1, B2, and B3 – 7.34 acres), both basins combine within a quarter mile downstream to the northwest from the site. Furthermore, the neighbor to the north has expressed a strong request that the existing small pond on his property retains water throughout the year. We believe our proposed basin layout will not be detrimental to downstream conditions and assist with the neighbor's request.

Basin A is modeled as hydrologic group C, landscaped, and consists of a small, negligible area (0.70 acres) that serves as grassy landscape for individual homes. Basins B2 and B3 (0.33 acres and 0.77 acres, respectively) are modeled as hydrologic group C, forest and will remain undisturbed after construction of this project.

PERFORMANCE GOALS AND STANDARDS

The stormwater design standards for the proposed Betrozoff Jones project are based on the SWMTN. The detention vault and water quality facility are designed to meet the SWMTN.

FLOW CONTROL SYSTEM

The flow control facility is required for the proposed project. The facility is designed to detain stormwater and release to the pre-developed duration discharge rates from 50% of the 2-year peak flow up to the full 50-year peak flow. The pre-developed condition to be matched shall be a forested land cover.

The detention portion of the facility requires a minimum volume of 74,150 cubic feet with a volume of 75,000 cubic feet is provided. The detention portion of the vault size is 150 feet in length, 50 feet in width and 10 feet in depth.

WATER QUALITY SYSTEM

The water quality facility is required for the proposed project and to meet the basic water quality requirement based on SWMTN. The SWMTN requires that all water quality treatment facilities to exceed the minimum requirement of 91 percent of the runoff volume as estimated by an approved continuous runoff model will be treated.

The water quality portion of the facility requires a minimum volume of 23,632 cubic feet with a volume of 24,000 cubic feet is provided. The water quality portion of the vault size is 150 feet in length, 50 feet in width and 4 feet in depth.

CONVEYANCE SYSTEM ANALYSIS AND DESIGN

The proposed stormwater conveyance system will be consist of stormwater generated from the proposed site sheet flow along the road and collected via nearby catch basins and flow through underground pipes. The underground pipe network is designed to flow toward the water quality detention vault to be treated and to be detained. The stormwater drainage conveyance system will be sized to convey the 10 year design storm event and to contain the 50 year design storm event. A detailed of the Conveyance System Analysis and Design will be provided with the final Stormwater Site Plan.

TEMPORARY EROSION AND SEDIMENT CONTROL (TESC) DESIGN

The Temporary Erosion and Sedimentation Control (TESC) Design will be shown on the final construction plans and additional information will be provided with the final Stormwater Site Plan.

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Chapter 6.0 Construction Stormwater Pollution Prevention Plan

The Construction Stormwater Pollution Prevention Plan will be provided with the final Stormwater Site Plan.

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Chapter 7.0 Special Reports and Studies

A wetland report was completed for the Betrozoff Jones project site, by Mark Rigos, dated December 3, 2012 and is included with this submittal under separate cover.

A geotechnical engineering report was also completed for the project site, by Robinson Noble, dated February 6, 2012 and is also included in Appendix A.

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Chapter 8.0 Other Permits

The National Pollutant Discharge Elimination System (NPDES) NPDES permit will be prepared with the final construction plans.

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Chapter 9.0 Operations and Maintenance Manual

The Operations and Maintenance Manual will be provided with the final Stormwater Site Plan.

Chapter 10.0 Bond Quantities Worksheet

The Bond Quantities Worksheet will be provided with the final Stormwater Site Plan.

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Appendix A

Geotechnical Engineering Report

REVISED GEOTECHNICAL ENGINEERING REPORT
BETROZOFF-JONES PLAT
REDMOND, WASHINGTON
FOR
SHERMAN BUILDING COMPANY, LLC

FEBRUARY 2013

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ROBINSON NOBLE

February 6, 2013

Mr. Todd Sherman
Sherman Building Company, LLC
2100 124th Avenue NE, Suite 100
Bellevue, WA 98005

Revised Geotechnical Engineering Report
Betrozoff-Jones Property
Redmond, Washington
RN File No. 2777-001A

Dear Mr. Sherman:

This letter serves as a transmittal for six copies of our report for the Betrozoff-Jones Property residential project. The site is located on King County Parcels 9428500065 and 9428500070 in Redmond, Washington. The project will consist of the development of 32 residential lots, two stormwater detention facilities, and two associated access roads. The site soils are compatible with the planned development.

We appreciate the opportunity of working with you on this project. If you have any questions regarding this report, please contact us.

Sincerely,

Rick B Powell, PE
Principal Engineer

BAG:RBP:am

Six Copies Submitted
Seven Figures
Appendix A

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INTRODUCTION

This report presents the results of our geotechnical engineering investigation for the 32-lot subdivision in Redmond, Washington. The site consists of King County Parcels 9428500065 and 9428500070 and is located between Woodinville-Redmond Road and 154th Place NE, as shown on the Vicinity Map in Figure 1.

You have requested that we complete this report to evaluate subsurface conditions and provide recommendations for residential construction. For our use in preparing this report, we have been provided with a Preliminary Plat Site Plan dated January 7, 2013, prepared by ESM Consulting Engineers, which shows the planned lot layout, site topography, and the locations of existing structures on-site.

We understand from conversations with you that if infiltration is not feasible for stormwater detention ponds, precast stormwater detention vaults are planned in the northwest and northeast corners of the site at depths of approximately 12 feet.

SCOPE

The purpose of this study is to explore and characterize the subsurface conditions and present recommendations for site development. Specifically, our scope of services as outlined in our Services Agreement, dated December 27, 2012, includes the following:

- Explore the subsurface soil and groundwater conditions with an excavator provided by you. You have requested that we complete 8 test pits.
- Evaluate pertinent physical and engineering characteristics of the soils encountered in our explorations based on field test results, laboratory results and our experience.
- Prepare a geotechnical report containing the results of our subsurface explorations, and our conclusions and recommendations for geotechnical design elements of the project. Our report will include:
    - Description of the geologic materials encountered.
    - Depth to groundwater, if encountered.
    - Discussion of seismicity at the site along with seismic design parameters including Site Class and site coefficients based on current IBC criteria.
    - Recommendations for earthwork and site preparation.
    - Recommendations for temporary and permanent excavation cuts.
    - Recommendations for shallow foundations including allowable soil bearing values, minimum footing sizes and soil parameters for lateral load resistance.
    - Estimate the total and differential settlements of conventional footings within the building.
    - Recommendations for roadway subgrade preparation.
    - Detention pond recommendations including preliminary infiltration estimates based on grain-size distribution.

SITE CONDITIONS

Surface Conditions

The roughly rectangular-shaped project site is about 9.06 acres in size and has maximum dimensions of approximately 600 feet in the east-west direction and 875 feet in the north-south direction. Access to the site is provided by Woodinville-Redmond Road, bordering the east side of the site. The site is also bordered by existing residential acreage to the north and south. 154th Place Northeast borders the site to the west. A layout of the site is shown on the Site Plan in Figure 2.

The ground surface within the site slopes gently down to the north and gently to steeply down to the west. The site is vegetated with a grass lawn, landscaping bushes, several stands of small- to- medium sized trees and several larger trees. Two single family residences with associated outbuildings and asphalt drives currently within the site are to be removed.

Geology

Most of the Puget Sound Region was affected by past intrusion of continental glaciation. The last period of glaciation, the Vashon Stade of the Fraser Glaciation, ended approximately 14,000 years ago. Many of the geomorphic features seen today are a result of scouring and overriding by glacial ice. During the Vashon Stade, areas of the Puget Sound region were overridden by over 3,000 feet of ice. Soil layers overridden by the ice sheet were compacted to a much greater extent than those that were not. Part of a typical glacial sequence within the area of the site includes the following soil deposits from newest to oldest:

Artificial Fill (af) – Fill material is often locally placed by human activities, consistency will depend on the source of the fill. The thickness and expanse of this material will be dependent of extent of fill required to grade land to the desired elevations. Density of the fill will depend on earthwork activities and compaction efforts made during the placement of the material.

Recessional Outwash (Qvr) – These deposits were derived from the stagnating and receding Vashon glacier and consist of mostly of stratified sand and gravel, but include unstratified ablation and melt-out deposits. Recessional deposits were not compacted by the glacier and are typically not as dense as those that were.

Vashon Till (Qvt) – The till is a non-sorted mixture of clay, sand, pebbles, cobbles and boulders, all in variable amounts. The till was deposited directly by the ice as it advanced over and eroded irregular surfaces of previously deposited formations and sediments. The till was well compacted by the advancing glacier and exhibits high strength and stability. Drainage is considered very poor in the till.

Older Alluvium (Qoal) – Older alluvium consists of sand, silt, gravel and cobbles that may include landslide debris and colluvium at margins. These deposits form terraces along the valley sides.

The geologic units for this area are mapped on the Geologic Map of Kirkland Quadrangle, Washington, by James P. Minard (U.S. Geological Survey, 1983). The site is mapped as being underlain by deposits of older alluvium and glacial till. Our site explorations encountered older alluvium and glacial till.

Explorations

We explored subsurface conditions within the site on January 11, 2013 by excavating eight test pits with an excavator provided by you. The test pits were excavated to depths of 9.0 to 17.0 feet below the ground surface. The test pits were located in the field by you and an engineer from this firm. Our engineer also examined the soils and geologic conditions encountered, and maintained logs of the explorations. The approximate locations of the test pits are shown on the Site Plan in Figure 2. The soils were visually classified in general accordance with the Unified Soil Classification System, a copy of which is presented as Figure 3. The logs of the test pits are presented in Figures 4 through 6.

Subsurface Conditions

A brief description of the conditions encountered in our explorations is included below. For a more detailed description of the soils encountered, review the test pit logs in Figures 4 through 6.

Our explorations generally encountered a surficial layer of topsoil that ranged in thickness from ½ to 1½ feet. The topsoil was underlain in Test Pits 1 through 3 and Test Pit 8 by medium stiff to stiff silt with trace sand, which we interpreted as weathered older alluvium and which extended to depths ranging from 7 to 13 feet below ground surface (bgs). Below the weathered alluvium, we encountered very stiff to hard older alluvium, which extended to the depths explored of 15 to 17 feet bgs. Test Pit 7 disclosed about 3 feet of loose silty sandy gravel, interpreted as fill, that was underlain by a weathered zone of loose to medium dense silty sand. Below the weathered zone we encountered medium dense silty sand that was interpreted as weathered or ablated till. The topsoil was underlain in Test Pits 4 through 6 by silty sand with varying amounts of gravel that was interpreted to be weathered glacial till. Below the weathered till we encountered dense to very dense glacial till, which extended to the depths explored of 9 to 13 feet bgs.

Overall, the glacial till was encountered in the test pits excavated in the upland portions of the site. These test pits were generally located east of the planned north-south access road. The older alluvium was revealed in the test pits excavated at the lower elevations. These test pits were generally located in the western and northern portions of the site.

Hydrologic Conditions

Minor to moderate perched groundwater seepage was encountered in Test Pits 1, 2, 4, 5 and 8 at depths ranging from 3 to 5½ feet bgs. Groundwater seepage was not observed in the other test pits. The medium dense to very dense glacial till deposits and the very stiff to hard older alluvium deposits interpreted to underlie the site are considered poorly draining. During the wetter times of the year, we expect perched water conditions will occur as pockets of water on top of these layers. Perched water does not represent a regional groundwater "table" within the upper soil horizons. Volumes of perched groundwater vary depending upon the time of year and the upslope recharge conditions.

CONCLUSIONS AND RECOMMENDATIONS

General

It is our opinion that the site is compatible with the planned residential structures. The underlying medium dense to very dense glacial till deposits and stiff older alluvium deposits are capable of supporting the proposed structures. We recommend that the foundations for the structures extend through any fill, topsoil, loose, or disturbed soils, and bear on the underlying medium dense or firmer native glacial till deposits, the underlying stiff or firmer older alluvial soils, or on structural fill extending to these soils. These soils were generally encountered at depths ranging from 3 to 5 feet bgs. We have not been provided with a grading plan. However, based on our site explorations, we anticipate that these soils will generally be encountered at or within a few feet of typical footing depths on the upland portion of the site; this depth increases to the north and west in the lower portions of the site. We recommend that test pits be excavated at the time of construction or that a representative from our firm observe the grading operations to evaluate the need to overexcavate foundation soils. We expect that some type of overexcavation and replacement scheme will be needed, at least in the lower northern and western portions of the site.

Site Preparation and Grading

The first step of site preparation should be to strip the vegetation, topsoil, or loose soils to expose medium dense or firmer native soils in pavement and building areas. The excavated material should be removed from the site, or stockpiled for later use as landscaping fill. The resulting subgrade should be compacted to a firm, non-yielding condition. Areas observed to pump or yield should be repaired prior to placing hard surfaces.

The on-site glacial till deposits and stiff older alluvium deposits likely to be exposed during construction are considered highly moisture sensitive, and the surface will disturb easily when wet. We expect these soils would be difficult, if not impossible, to compact to structural fill specifications in wet weather. We recommend that earthwork be conducted during the drier months. Additional expenses of wet weather or winter construction could include extra excavation and use of imported fill or rock spalls. During wet weather, alternative site preparation methods may be necessary. These methods may include utilizing a smooth-bucket trackhoe to complete site stripping and diverting construction traffic around prepared subgrades. Disturbance to the prepared subgrade may be minimized by placing a blanket of rock spalls or imported sand and gravel in traffic and roadway areas. Cutoff drains or ditches can also be helpful in reducing grading costs during the wet season. These methods can be evaluated at the time of construction.

Geologic Hazards

Erosion Hazard

The erosion hazard criteria used for determination of affected areas includes soil type, slope gradient, vegetation cover, and groundwater conditions. The erosion sensitivity is related to vegetative cover and the specific surface soil types (group classification), which are related to the underlying geologic soil units. We reviewed the Web Soil Survey (WSS) on the U.S. Department of Agriculture's Natural Resources Conservation Service (NRCS) website for the King County Area, Washington to determine the erosion hazard of the on-site soils. The site surface soils were classified using the NRCS classification system as Kitsap silt loam with 2 to 8 percent slopes and 15 to 30 percent slopes (KpB and KpD). The corresponding description for parent material for these soils is listed as lacustrine which is similar to the low energy alluvial soils encountered in half of our site explorations. The soils east of Woodinville-Redmond Road were classified as Alderwood gravelly sandy loam (AgC). The corresponding description for parent material for these soils is listed as basal till which is in agreement with the soils encountered in half of our site explorations. The erosion hazard for the soil is listed as being slight for the gently sloping conditions at the site and moderate for the moderately sloping conditions at the site.

Seismic Hazard

It is our opinion based on our subsurface explorations that the Soil Profile in accordance with the 2009 and 2012 International Building Code (IBC) is Site Class C with Seismic Design Category D. We used the US Geological Survey program "U.S. Seismic Design Maps Web Application." The design maps summary reports for the 2009 and 2012 IBC are included in this report as Appendix A.

Additional seismic considerations include liquefaction potential and amplification of ground motions by loose and soft soil deposits. The liquefaction potential is highest for loose sand with a high groundwater table. The underlying dense and hard soils are considered to have a very low potential for liquefaction and amplification of ground motion.

Steep Slope Hazard

General: We observed fine-grained soils with a blocky structure in Test Pit 8. Based on our observations, in our opinion the steep slope area in the northwest region of the site may not be stable with respect to deep-seated slope failures. In addition, some surficial sloughing could occur on the steeper portions of the slope. We, therefore, are recommending setbacks from the top of the steepest portions of the slopes. Those setbacks are described in the **Slope Setback** portion of the report.

Slope Setback: To protect the planned residences from shallow sloughing failures over the lifetime of the structures, we recommend a 35-foot horizontal distance, as presented in Figure 7, from the slope face to the footings for the planned residences on lots 20 through 26. It is possible that further testing of the slope soils could justify a reduced distance.

Slope Protection: Protection of the setback and steep slope areas should be performed as required. It should be understood that the closer the site disturbance and development are to the slope, the more risk there is of affecting slope stability. Care should be taken to minimize disturbance to the slope face.

From a geotechnical standpoint, selective pruning and thinning of vegetation should be acceptable. Cutting and pruning of trees located on the slope can be performed, if allowed by the City, but certain precautions should be taken. We recommend that the root bundle/stump of fallen trees be left in place. Pruned materials and debris should be removed from the area and not allowed to remain on the slope. Any disturbed areas should be immediately re-stabilized through vegetation planting or other approved means. Soil, sod, clippings or other matter should not be placed on the slope.

Of great importance to the long-term stability of the slope is the control of surface and near-surface water, and erosion protection. We recommend that all drains, including foundation, roof and yard drains, be directed away from the top of slope and outfall at an approved area. Surface drainage over the slope should not be permitted.

Structural Fill

General: All fill placed beneath buildings or other settlement sensitive features should be placed as structural fill. Structural fill, by definition, is placed in accordance with prescribed methods and standards, and is observed by an experienced geotechnical professional or soils technician. Field observation procedures would include the performance of a representative number of in-place density tests to document the attainment of the desired degree of relative compaction.

Materials: Imported structural fill should consist of a good quality, free-draining granular soil, free of organics and other deleterious material, and be well graded to a maximum size of about 3 inches. Imported, all-weather structural fill should contain no more than 5 percent fines (soil finer than a Standard U.S. No. 200 sieve), based on that fraction passing the U.S. 3/4-inch sieve.

The use of on-site soil as structural fill will be dependent on moisture content control. Some drying of the native soils may be necessary in order to achieve compaction. During warm, sunny days this could be accomplished by spreading the material in thin lifts and compacting. Some aeration and/or addition of moisture may also be necessary. We expect that compaction of the native soils to structural fill specifications would be difficult, if not impossible, during wet weather.

Fill Placement: Following subgrade preparation, placement of the structural fill may proceed. Fill should be placed in 8- to 10-inch-thick uniform lifts, and each lift should be spread evenly and be thoroughly compacted prior to placement of subsequent lifts. All structural fill underlying building areas, and within a depth of 2 feet below sidewalk and access road subgrade, should be compacted to at least 95 percent of its maximum dry density (MDD). Maximum dry density, in this report, refers to that density as determined by the ASTM D1557 compaction test procedure. Fill more than 2 feet beneath sidewalks and pavement subgrades should be compacted to at least 90 percent of the maximum dry density. The moisture content of the soil to be compacted should be within about 2 percent of optimum so that a readily compactable condition exists. It may be necessary to overexcavate and remove wet surficial soils in cases where drying to a compactable condition is not feasible. All compaction should be accomplished by equipment of a type and size sufficient to attain the desired degree of compaction.

Temporary and Permanent Slopes

Temporary cut slope stability is a function of many factors, such as the type and consistency of soils, depth of the cut, surcharge loads adjacent to the excavation, length of time a cut remains open, and the presence of surface or groundwater. It is exceedingly difficult under these variable conditions to estimate a stable temporary cut slope geometry. 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.

For planning purposes, we recommend that temporary cuts in the near-surface fill and alluvial soils be no steeper than 1.5 Horizontal to 1 Vertical (1.5H:1V). Cuts in the medium dense to very dense till may stand at a 0.5H:1V inclination or possibly steeper. If groundwater seepage is encountered, we would expect that flatter inclinations would be necessary.

If possible, the detention vaults in the northwest and northeast portions of the site should be planned to allow for safe excavation cuts. If the vaults need to be excavated closer to the property line, shoring may be required.

We recommend that cut slopes be protected from erosion. Measures taken may include covering cut slopes with plastic sheeting and diverting surface runoff away from the top of cut slopes. We do not recommend vertical slopes for cuts deeper than 4 feet, if worker access is necessary. We recommend that cut slope heights and inclinations conform to local and WISHA/OSHA standards.

Final slope inclinations for granular structural fill and the native glacial soils should be no steeper than 2H:1V. Lightly compacted fills, common fills, native alluvial soils or structural fill predominately consisting of fine grained soils should be no steeper than 3H:1V. Common fills are defined as fill material with some organics that are "trackrolled" into place. They would not meet the compaction specification of structural fill. Final slopes should be vegetated and covered with straw or jute netting. The vegetation should be maintained until it is established.

Foundations

Conventional shallow spread foundations should be founded on undisturbed, medium dense or firmer soil or undisturbed stiff or firmer soil. If the soil at the planned bottom of footing elevation is not suitable, it should be overexcavated to expose suitable bearing soil or compacted to at least 95% MDD. Footings should extend at least 18 inches below the lowest adjacent finished ground surface for frost protection. Minimum foundation widths should conform to IBC requirements. Standing water should not be allowed to accumulate in footing trenches. All loose or disturbed soil should be removed from the foundation excavation prior to placing concrete.

For foundations constructed on stiff alluvium soil, we recommend an allowable design bearing pressure of 1,500 pounds per square foot (psf) be used for the footing design. For foundations constructed on medium dense or firmer till soil, or on structural fill compacted to at least 95% MDD, we recommend an allowable design bearing pressure of 2,500 pounds per square foot (psf) be used for the footing design. IBC guidelines should be followed when considering short-term transitory wind or seismic loads. Potential foundation settlement using the recommended allowable bearing pressure is estimated to be less than 1-inch total and ½-inch differential between footings or across a distance of about 30 feet. Higher soil bearing values may be appropriate with wider footings. These higher values can be determined after a review of a specific design.

Lateral Loads

The lateral earth pressure acting on retaining walls is dependent on the nature and density of the soil behind the wall, the amount of lateral wall movement, which can occur as backfill is placed, and the inclination of the backfill. Walls that are free to yield at least one-thousandth of the height of the wall are in an "active" condition. Walls restrained from movement by stiffness or bracing are in an "at-rest" condition. Active earth pressure and at-rest earth pressure can be calculated based on equivalent fluid density. Equivalent fluid densities for active and at-rest earth pressure of 35 pounds per cubic foot (pcf) and 55 pcf, respectively, may be used for design for a level backslope. These values assume that imported granular fill is used for backfill, and that the wall backfill is drained. The preceding values do not include the effects of surcharges, such as due to foundation loads or other surface loads. Surcharge effects should be considered where appropriate. The above drained active and at-rest values should be increased by a uniform pressure of 7.1H and 17.8H psf, respectively, when considering seismic conditions using the 2009 IBC seismic parameters. The above drained active and at-rest values should be increased by a uniform pressure of 7.6H and 18.8H psf, respectively, when considering seismic conditions using the 2012 IBC seismic parameters. H represents the wall height.

The above lateral pressures may be resisted by friction at the base of the wall and passive resistance against the foundation. A coefficient of friction of 0.5 may be used to determine the base friction in the native glacial soils. An equivalent fluid density of 360 pcf may be used for passive resistance design in the native glacial soils. A coefficient of friction of 0.34 may be used to determine the base friction in the native alluvial soils. An equivalent fluid density of 220 pcf may be used for passive resistance design in the native alluvial soils. To achieve this value of passive pressure, the foundations should be poured "neat" against the native dense soils, or compacted fill should be used as backfill against the front of the footing, and the soil in front of the wall should extend a horizontal distance at least equal to three times the foundation depth. A factor of safety of 1.5 has been applied to the passive pressure to account for required movements to generate these pressures. The friction coefficient also includes a factor of safety of 1.5.

All wall backfill should be well compacted. Care should be taken to prevent the buildup of excess lateral soil pressures due to overcompaction of the wall backfill.

Slabs-On-Grade

Slab-on-grade areas should be prepared as recommended in the **Site Preparation and Grading** subsection. Slabs should be supported on medium dense or firmer native soils, or on structural fill extending to these soils. Where moisture control is a concern, we recommend that slabs be underlain by 6 inches of pea gravel for use as a capillary break. A suitable vapor barrier, such as heavy plastic sheeting, should be placed over the capillary break. An additional 2-inch-thick damp sand blanket can be used to cover the vapor barrier to protect the membrane and to aid in curing the concrete. This will also help prevent cement paste bleeding down into the capillary break through joints or tears in the vapor barrier. The capillary break material should be connected to the footing drains to provide positive drainage.

Infiltration

We understand that project plans include the use of either stormwater detention ponds or detention vaults. We obtained soil samples from the test pits located in the planned stormwater detention areas. We have used the United States Department of Agriculture (U.S.D.A.) soil group classification (Figure 3.27) as presented in the "Storm Water Management Manual for Western Washington", (Ecology 2005) to classify the soil samples analyzed. Based on the sieve results, this material is classified as silt. Based on this manual, infiltration is not considered feasible, as indicated on Table 3.7 for silt soils.

Drainage

We recommend that runoff from impervious surfaces, such as roofs, driveway and access roadways, be collected and routed to an appropriate storm water discharge system. The finished ground surface should be sloped at a gradient of 5 percent minimum for a distance of at least 10 feet away from the buildings, or to an approved method of diverting water from the foundation. Surface water should be collected by permanent catch basins and drain lines, and be discharged into the existing storm drain system.

We recommend that footing drains be used around all of the structures where moisture control is important. The underlying till and fine-grained alluvial soils may pond water that could accumulate in crawlspaces. It is good practice to use footing drains installed at least 1 foot below the planned finished floor slab or crawlspace elevation to provide drainage for the crawlspace. At a minimum, crawlspaces should be sloped to drain to an outlet tied to the drainage system. If drains are omitted around slab-on-grade floors where moisture control is important, the slab should be a minimum of 1 foot above surrounding grades.

Where used, footing drains should consist of 4-inch-diameter, perforated PVC pipe that is surrounded by free-draining material, such as pea gravel. Footing drains should discharge into tightlines leading to an appropriate collection and discharge point. Crawlspaces should be sloped to drain, and a positive connection should be made into the foundation drainage system. For slabs-on-grade, a drainage path should be provided from the capillary break material to the footing drain system. Roof drains should not be connected to wall or footing drains.

Due to the impermeable nature of the underlying silt in the northern and western portions of the site, we recommend a perforated pipe below–slab collection system that can flow by gravity to a suitable discharge location. On a preliminary basis, we recommend these drains on 25-foot horizontal spacing. The drains, with cleanouts, should consist of a minimum 4-inch diameter perforated pipe that is surrounded by free-draining material, such as pea gravel. The drain invert should be at least 1 foot below the base of the slab, with the pipe sloped to drain. The need for below-slab drainage should be more fully evaluated during construction.

Detention Vault

Because the soils in the planned stormwater facility areas are not conducive to infiltration, we understand that stormwater detention vaults are planned. The stormwater detention vaults may be supported on footing foundations bearing on the underlying hard alluvial soils. We recommend a soil bearing pressure of 4,000 pounds per square foot (psf) for the design of vault footings poured on undisturbed very stiff to hard alluvial and a footing width of at least 3 feet.

We recommend that footing drains be installed on the outside of perimeter footings. The footing drains should be at least 4 inches in diameter and should consist of perforated or slotted, rigid, smooth-walled PVC pipe, laid at the bottom of the footings. The drain line should be surrounded with free-draining pea gravel or coarse sand and wrapped with a layer of non-woven filter fabric. A vertical drainage blanket at least 12 inches thick, consisting of compacted pea gravel or other free-draining granular soils, should be placed against the walls. A vertical drain mat, such as Miradrain 6000 by Mirafi Inc., may be placed against the walls in lieu of the vertical drainage blanket. Structural fill is then placed behind the vertical drainage blanket or drain mat to backfill the walls. The vertical drainage blanket or drain mat should be hydraulically connected to the drain line at the base of the walls. Sufficient number of cleanouts at strategic locations should be installed for periodical cleaning of the wall drain line to prevent clogging.

The perimeter walls of the concrete vault with a lid would be restrained at their top from horizontal movement and should be designed for at-rest lateral soil pressure, while the perimeter walls of a vault without a lid would be unrestrained at the top and may be designed for active lateral soil pressure. Active earth pressure and at rest earth pressure can be calculated based on equivalent fluid density. Equivalent fluid densities for active and at rest earth pressure of 35 pcf and 55 pcf, respectively, may be used for design for a level backslope. These values assume that granular soils are used for backfill, and that the wall backfill is drained. The preceding values do not include the effects of surcharges due to foundation loads, traffic or other surface loads. Surcharge effects should be considered where appropriate. Recommended seismic lateral loading is provided in the **Lateral Load** section of this report. For undrained soil conditions, the active and at-rest pressures should be increased to 78 pcf and 88 pcf, respectively. Undrained conditions may occur in the lower portion of the vault if there is not suitable fall to place a wall drain at the footing elevation.

All wall backfill should be well compacted. Care should be taken to prevent the buildup of excess lateral soil pressures due to overcompaction of the wall backfill.

Utilities

Our explorations indicate that deep dewatering will not be needed to install standard depth utilities. Anticipated groundwater is expected to be handled with pumps in the trenches. We also expect that some groundwater seepage may develop during and following the wetter times of the year. We expect this seepage to mostly occur in pockets. We do not expect significant volumes of water in these excavations.

The soils likely to be exposed in utility trenches after site stripping are considered highly moisture sensitive. We recommend that they be considered for trench backfill during the drier portions of the year. Provided these soils are within 2 percent of their optimum moisture content, they should be suitable to meet compaction specifications. During the wet season, it may be difficult to achieve compaction specifications; therefore, soil amendment with kiln dust or cement may be needed to achieve proper compaction with the on-site materials.

Pavement Subgrade

The performance of access road pavement is critically related to the conditions of the underlying subgrade. We recommend that the subgrade soils within the roadways be prepared as described in the **Site Preparation and Grading** subsection of this report. Prior to placing base material, the subgrade soils should be compacted to a non-yielding state with a vibratory roller compactor and then proof-rolled with a piece of heavy construction equipment, such as a fully-loaded dump truck. Any areas with excessive weaving or flexing should be overexcavated and recompacted or replaced with a structural fill or crushed rock placed and compacted in accordance with recommendations provided in the **Structural Fill** subsection of this report.

CONSTRUCTION OBSERVATION

We should be retained to provide observation and consultation services during foundation excavation 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. As part of our services, we would also evaluate whether or not earthwork and foundation installation activities comply with contract plans and specifications.

USE OF THIS REPORT

We have prepared this report for Sherman Building Company, LLC and its agents, for use in planning and design of this project. The data and report should be provided to prospective contractors for their bidding and estimating purposes, but our report, conclusions and interpretations should not be construed as a warranty of subsurface conditions.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractors' methods, techniques, sequences or procedures, except as specifically described in our report, for consideration in design. There are possible variations in subsurface conditions. We recommend that project planning include contingencies in budget and schedule, should areas be found with conditions that vary from those described in this report.

Within the limitations of scope, schedule and budget for our services, we have strived to take care that our services have been completed in accordance with generally accepted practices followed in this area at the time this report was prepared. No other conditions, expressed or implied, should be understood.

We appreciate the opportunity to be of service to you. If there are any questions concerning this report or if we can provide additional services, please call.

Sincerely,

Robinson Noble, Inc.

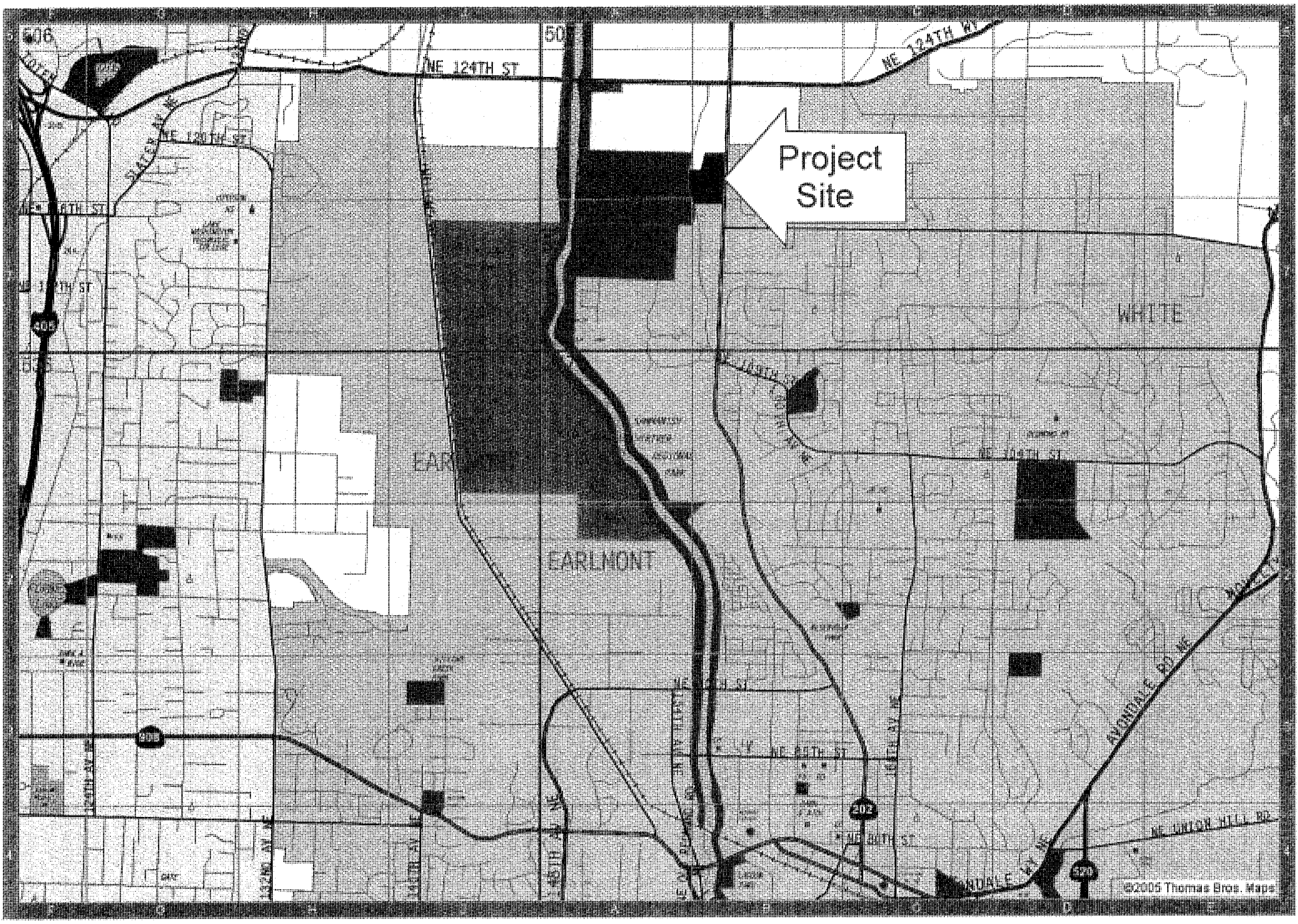
2/6/2013

Barbara A. Gallagher, PE
Senior Project Engineer

Rick B. Powell, PE
Principal Engineer

BAG:RBP:am

Six Copies Submitted
Seven Figures
Appendix A



ROBINSON
NOBLE

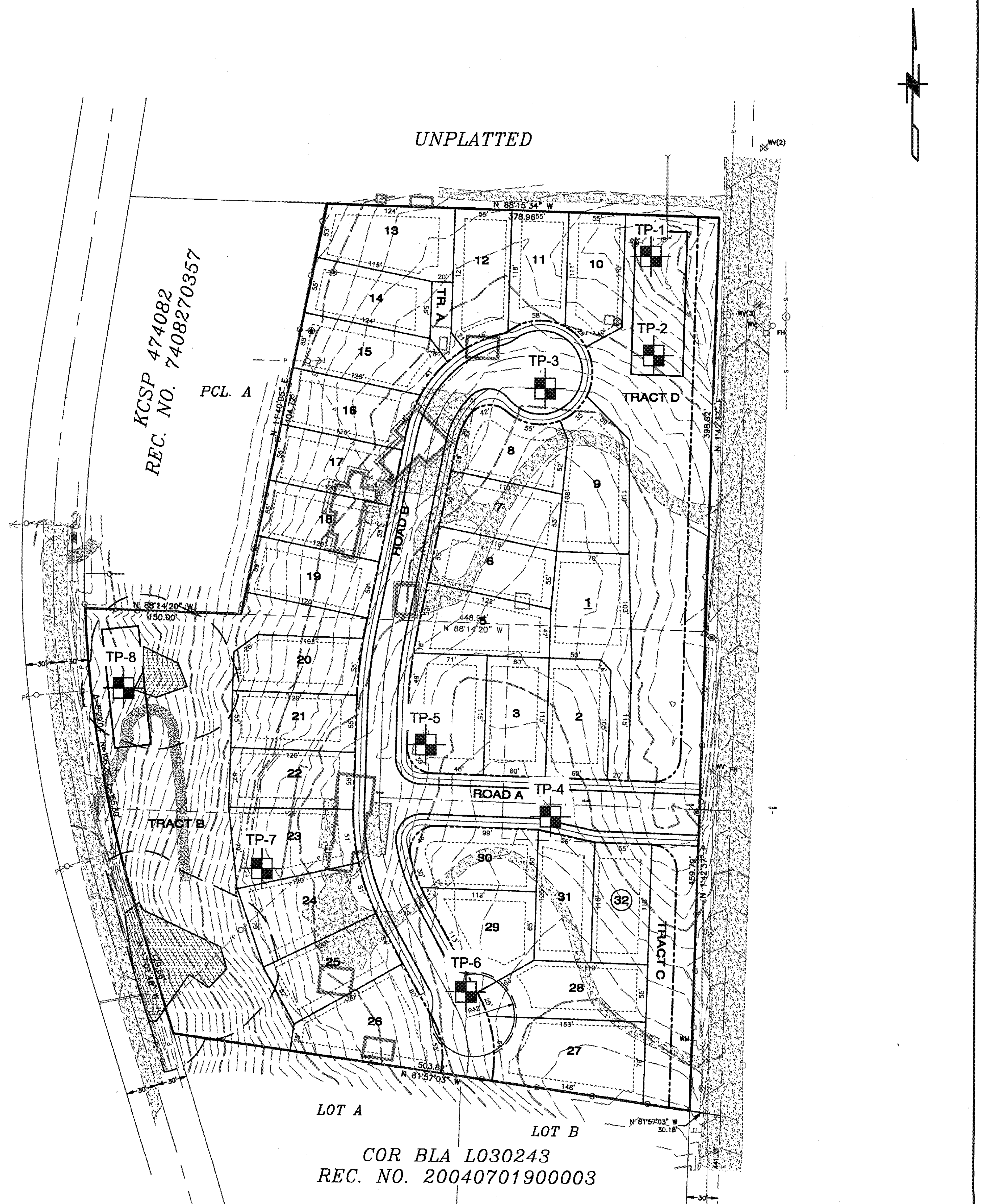
PM: RBP
February 2013
2777-001A

Figure 1

Vicinity Map

Sherman Building Company, LLC: Metzroff Jones Plat

ATTACHMENT 34



LEGEND

TP-1 Number and Approximate Location of Test Pit

0' 100'
Approximate Scale

ATTACHMENT 34

ROBINSON NOBLE

Note: Basemap taken from "Preliminary Plat Site Plan" prepared by ESM Consulting Engineers dated 1/7/2013.	PM: RBP February 2013 2777-001A	Figure 2 Site Plan Sherman Building Company, LLC: Betrozoff-Jones Plat

Unified Soil Classification System

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE - GRAINED SOILS MORE THAN 50% RETAINED ON number 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 D 2488-83.
2) Soil classification using laboratory tests is based on ASTM D 2487-83.
3) Descriptions of soil density or consistency are based on interpretation of blowcount 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

ROBINSON NOBLE	PM: RBP February 2013 2777-001A	King County	Figure 3 Unified Soil Classification System Sherman Building Compnay, LLC: Betrozoff-Jones Plat

ATTACHMENT 34

LOG OF EXPLORATION

DEPTH	USC	SOIL DESCRIPTION
TEST PIT ONE		
0.0 – 1.0	ML	Dark brown silt with roots (soft, moist) **(Topsoil)**
1.0 - 4.5	ML	Brown silt with trace sand (soft, moist)
4.5 – 7.0	ML	Grayish brown mottled silt with trace fine sand (medium stiff to stiff, moist to wet) MC = 26.7% at 5.0 feet
7.0 – 13.0	ML	Gray silt with trace fine sand (very stiff, moist) MC = 25.8% at 8.0 feet
13.0 – 15.5	ML	Gray silt with trace fine sand (hard, moist) (PP=3.5 tsf) **(Older Alluvium)** MC = 22.2% at 15.5 feet
		Samples were collected at 5.0, 8.0 and 15.5 feet Moderate groundwater seepage was encountered at 4.0 feet Test pit caving was not encountered Test pit was completed at 15.5 feet on 1/11/2013
TEST PIT TWO		
0.0 – 0.5	ML	Dark brown silt with roots (soft, moist) **(Topsoil)**
0.5 – 5.0	ML	Brown silt with trace sand (soft to medium stiff, moist) MC = 27.6% at 5.0 feet
5.0 – 5.5	SP	Brown fine to coarse sand with trace silt (medium dense, moist)
5.5 – 8.0	ML	Brown silt with trace fine sand (stiff to very stiff, moist)
8.0 – 15.5	ML	Gray silt with trace fine sand (very stiff to hard, moist) (PP=2.5 tsf) **(Older Alluvium)** MC = 26.4% at 15.5 feet
		Samples were collected at 5.0 and 15.5 feet Minor groundwater seepage was encountered at 3.0 feet Test pit caving was not encountered Test pit was completed at 15.5 feet on 1/11/2013
TEST PIT THREE		
0.0 – 0.5	ML	Dark brown sandy silt (soft, moist) **(Topsoil)**
0.5 – 1.5	ML	Reddish-brown silt with fine to medium sand (soft, moist)
1.5 – 3.0	ML	Brown slightly mottled silt with trace fine sand (soft to medium stiff, moist)
3.0 – 13.0	ML	Brown slightly mottled silt with trace fine sand (stiff to very stiff, moist) MC = 24.7% at 12.0 feet
13.0 – 15.0	ML	Gray silt with trace fine sand (very stiff to hard, moist) (PP=2.5 tsf) **(Older Alluvium)** MC = 22.6% at 15.0 feet
		Sample was collected at 12.0 and 15.0 feet Groundwater seepage was not encountered Test pit caving was not encountered Test pit was completed at 15.0 feet on 1/11/2013

MC = Moisture Content

LOG OF EXPLORATION

TEST PIT FOUR

DEPTH	USC	SOIL DESCRIPTION
0.0 – 0.5	ML	Dark brown sandy silt with roots (soft, moist) **(Topsoil)**
0.5 – 5.5	SM	Brown silty fine sand with tree roots to 3 feet (medium dense, moist)
5.5 – 11.0	SM	Grayish-brown silty gravelly fine sand with cobbles (dense to very dense, moist) **(Weathered Till)**
11.0 – 13.0	SM	Gray silty fine gravelly sand with cobbles (very dense, moist) **(Glacial Till)** MC = 9.4% at 13.0 feet

Sample was collected at 13.0 feet
Groundwater seepage was encountered at 5.5 feet
Test pit caving was not encountered
Test pit was completed at 13.0 feet on 1/11/2013

TEST PIT FIVE

DEPTH	USC	SOIL DESCRIPTION
0.0 – 1.5	ML	Brown sandy silt with roots (soft, moist) **(Topsoil)**
1.5 – 4.0	SM	Reddish-brown silty sand with trace cobbles and boulders (loose to medium dense, moist)
4.0 – 10.5	SM	Grayish-brown silty gravelly fine sand with trace cobbles (dense to very dense, moist) **(Weathered Till)** MC = 8.6% at 5.0 feet
10.5 – 11.5	SM	Gray silty fine gravelly fine sand with trace cobbles (very dense, moist) **(Glacial Till)** MC = 8.8% at 11.5 feet

Samples were collected at 5.0 and 11.5 feet
Slight groundwater seepage was encountered at 5.0 feet
Test pit caving was not encountered
Test pit was completed at 11.5 feet on 1/11/2013

TEST PIT SIX

DEPTH	USC	SOIL DESCRIPTION
0.0 – 0.5	ML	Dark brown sandy silt with moss and grass roots (soft, moist) **(Topsoil)**
0.5 – 3.0	SM	Reddish-brown silty fine to medium sand (loose to medium dense, moist)
3.0 – 8.0	SM	Grayish-brown slightly mottled silty fine to medium sand with gravel and trace cobbles (dense, moist) **(Weathered Till)**
8.0 – 9.0	SM	Grayish-brown silty fine to medium sand with gravel and trace cobbles (very dense, moist) **(Glacial Till)** MC = 15.4% at 9.0 feet

Sample was collected at 9.0 feet
Groundwater seepage was not encountered
Test pit caving was not encountered
Test pit was completed at 9.0 feet on 1/11/2013

MC = Moisture Content

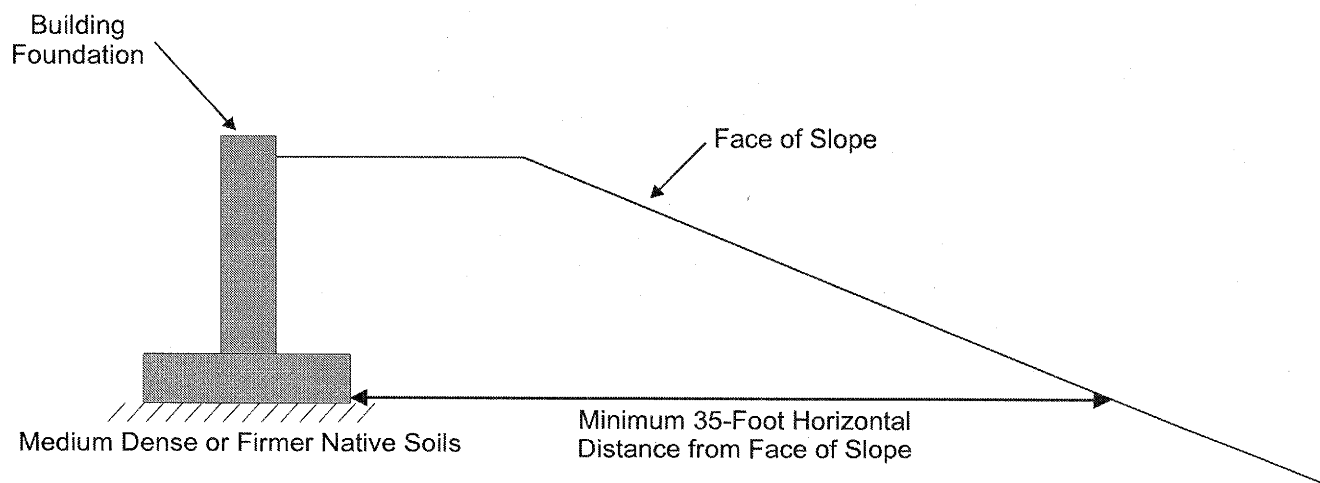
LOG OF EXPLORATION

DEPTH	USC	SOIL DESCRIPTION
TEST PIT SEVEN		
0.0 – 3.0	SM	Brown silty sandy gravel with trace roots (loose, moist) **(Fill)**
3.0 – 5.0	SM	Brown and reddish brown silty fine to medium sand with roots (loose to medium dense, moist)
5.0 – 11.5	SM	Grayish brown slightly mottled silty fine sand (medium dense, moist) **(Weathered Till)** MC = 20.9% at 11.5 feet
		Sample was collected at 11.5 feet Groundwater seepage was not encountered Test pit caving was not encountered Test pit was completed at 11.5 feet on 1/11/2013
TEST PIT EIGHT		
0.0 – 1.0	ML	Dark brown sandy silt with roots (soft, moist) **(Topsoil)**
1.0 – 3.0	ML	Brown silt with fine sand (soft to medium stiff, moist to wet)
3.0 – 5.0	ML	Brown slightly mottled silt with trace fine sand (medium stiff, moist) (PP=0.5 tsf) MC = 29.4% at 5.0 feet
5.0 – 13.0	ML	Brown silt with trace fine sand (medium stiff to stiff, moist) (PP=0.75 tsf) MC = 42.6% at 7.0 feet
13.0 – 17.0	ML	Gray silt with trace clay and trace sand (blocky) (very stiff to hard, moist) (PP=2.5 tsf) **(Older Alluvium/Colluvium)** MC = 29.8% at 17.0 feet
		Samples were collected at 5.0, 7.0 and 17.0 feet Groundwater seepage was encountered at 3.0 feet Test pit caving was not encountered Test pit was completed at 17.0 feet on 1/11/2013

MC = Moisture Content

Typical Foundation Embedment Detail with Horizontal Distance from Slope Face

(Not to Scale)



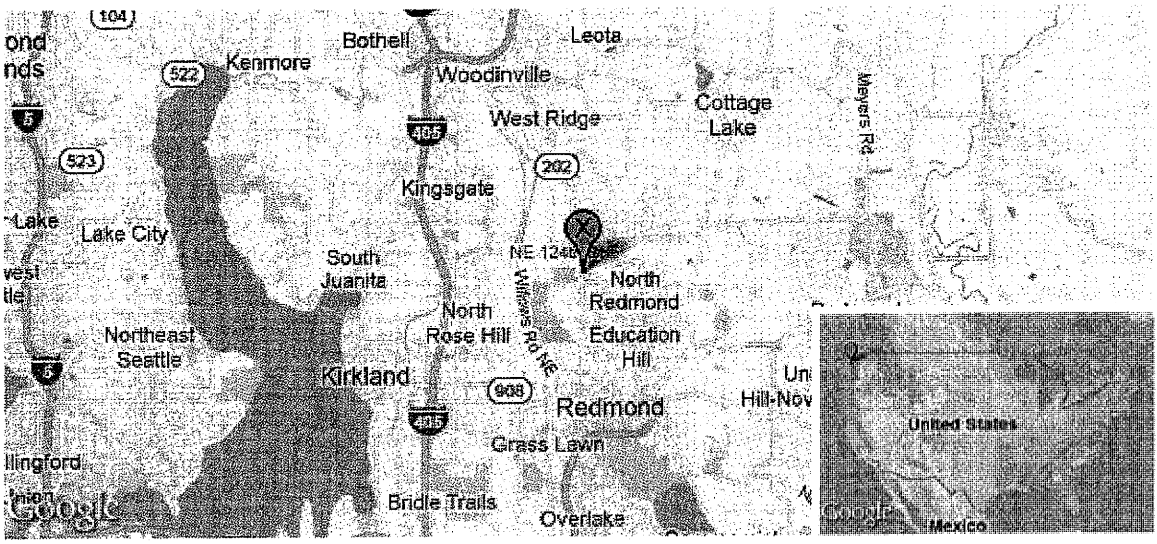
ROBINSON NOBLE		PM: RBP January 2013 2777-001A	King County Not to Scale	Figure 7 Foundation Embedment Detail Sherman Building Company, LLC: Betrozoff-Jones Plat

Appendix A

USGS Design Maps Summary Report

User–Specified Input

Report Title	Betrozoff-Jones Mon January 21, 2013 18:46:07 UTC
Building Code Reference Document	2012 International Building Code (which makes use of 2008 USGS hazard data)
Site Coordinates	47.70593°N, 122.13186°W
Site Soil Classification	Site Class C – "Very Dense Soil and Soft Rock"
Risk Category	I/II/III

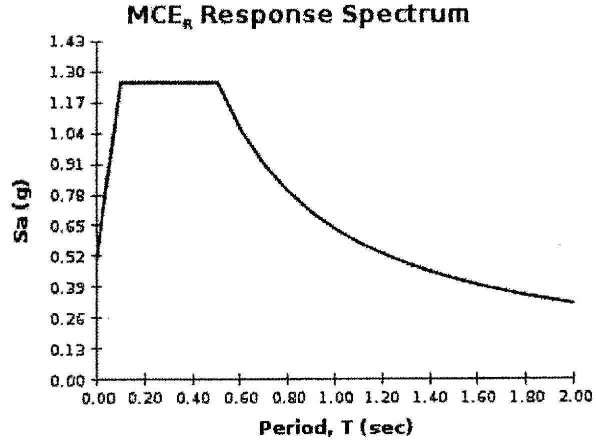


USGS–Provided Output

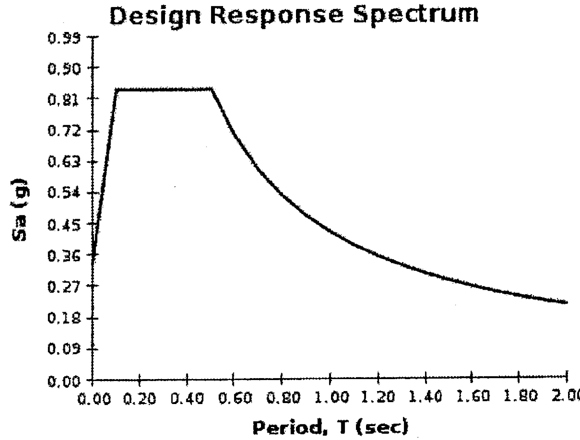
$S_S =$ 1.254 g	$S_{MS} =$ 1.254 g	$S_{DS} =$ 0.836 g
$S_1 =$ 0.482 g	$S_{M1} =$ 0.636 g	$S_{D1} =$ 0.424 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.

MCE_R Response Spectrum



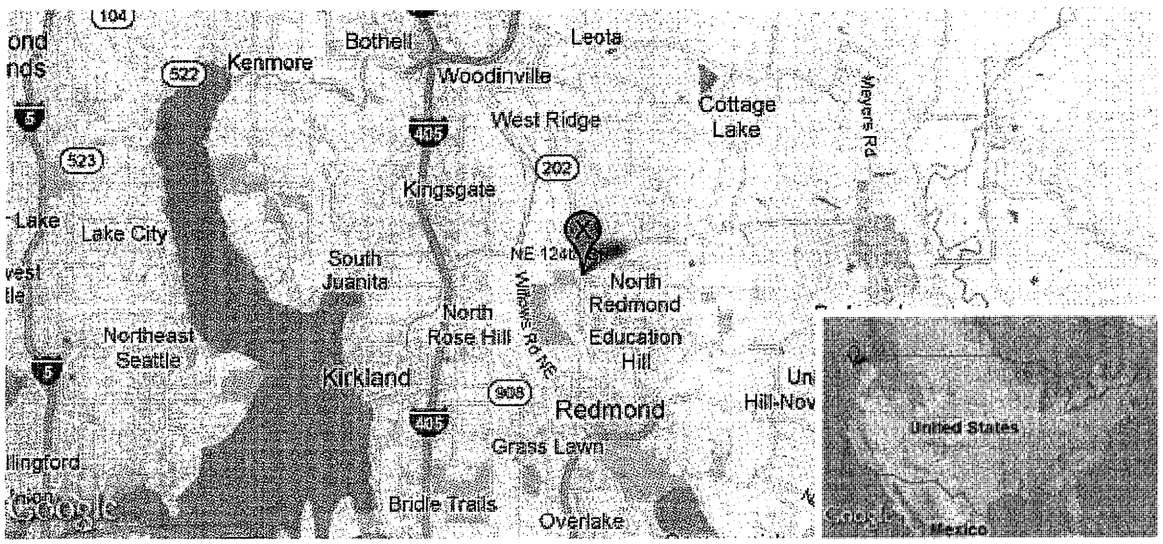
Design Response Spectrum



USGS Design Maps Summary Report

User–Specified Input

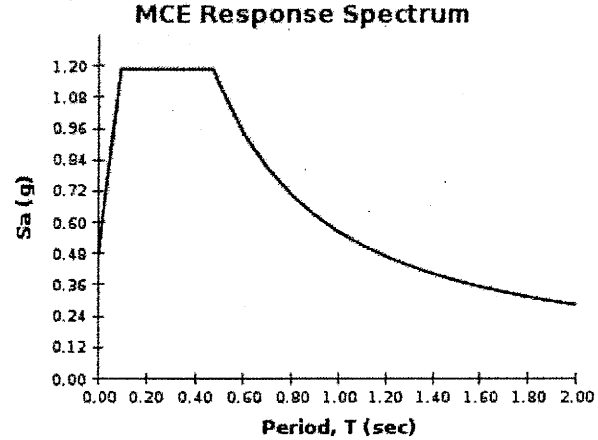
Report Title	Betrozoff-Jones Mon January 21, 2013 18:46:38 UTC
Building Code Reference Document	2006/2009 International Building Code (which makes use of 2002 USGS hazard data)
Site Coordinates	47.70593°N, 122.13186°W
Site Soil Classification	Site Class C – "Very Dense Soil and Soft Rock"
Occupancy Category	Occupancy Category I



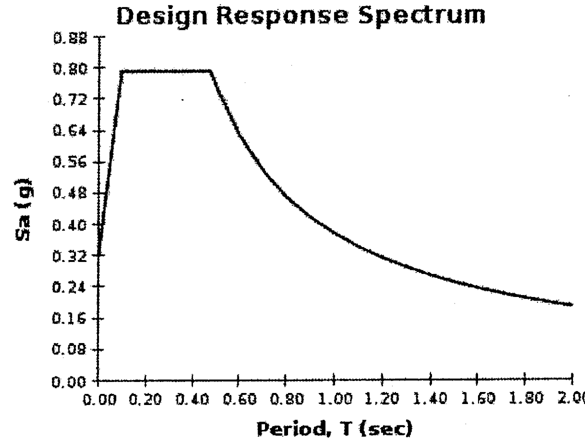
USGS–Provided Output

S_S =	1.187 g	S_{MS} =	1.187 g	S_{DS} =	0.791 g
S_1 =	0.403 g	S_{M1} =	0.563 g	S_{D1} =	0.376 g

MCE Response Spectrum



Design Response Spectrum



Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter knowledge.

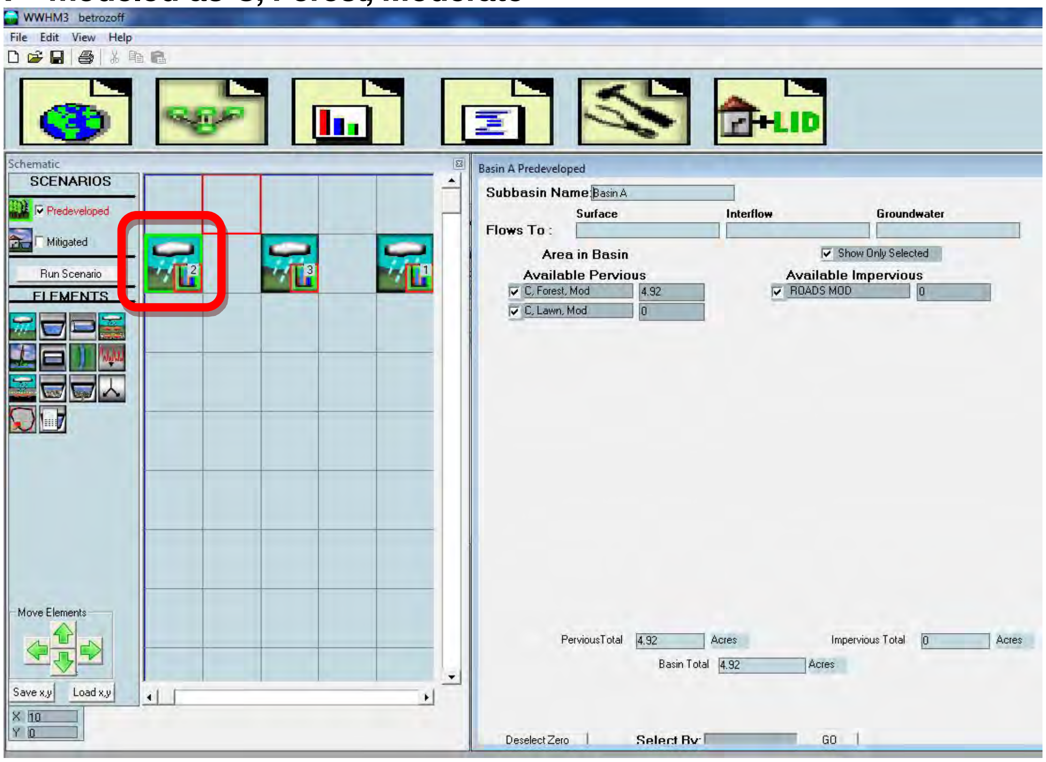
ATTACHMENT 34

Appendix B

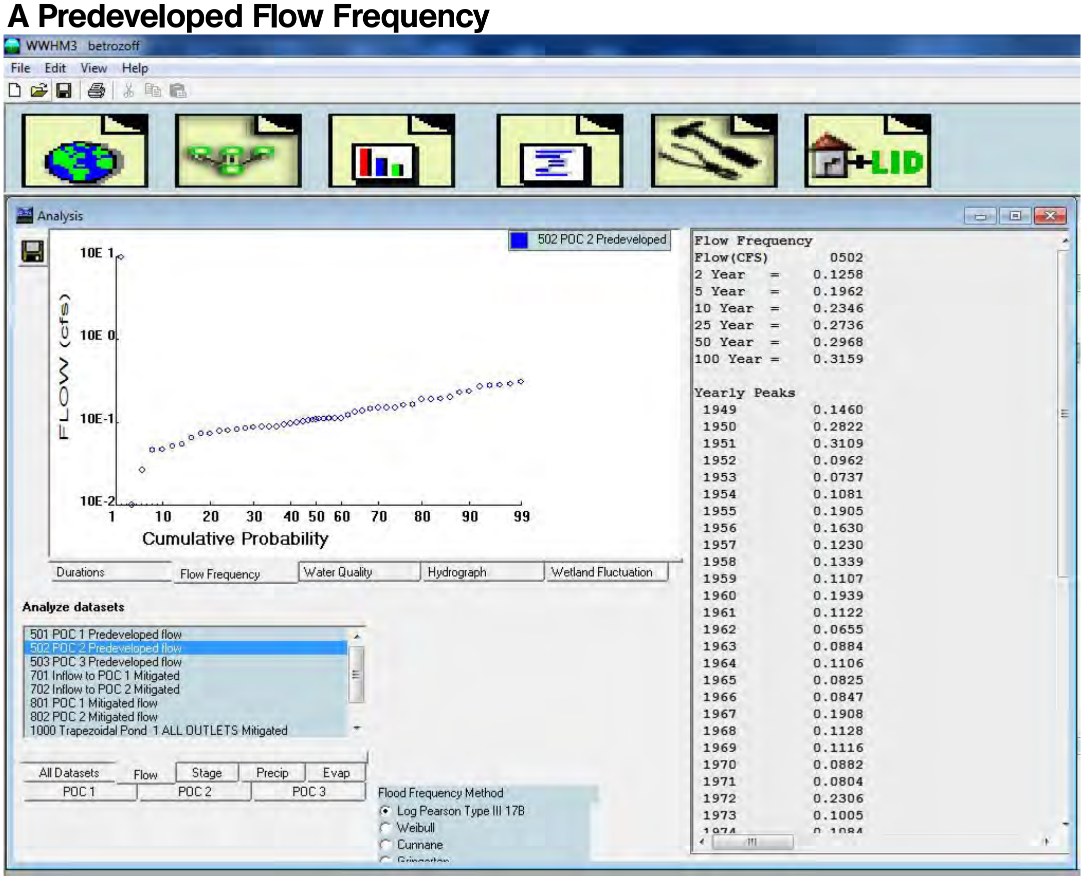
Stormwater Calculations

Predeveloped Basins

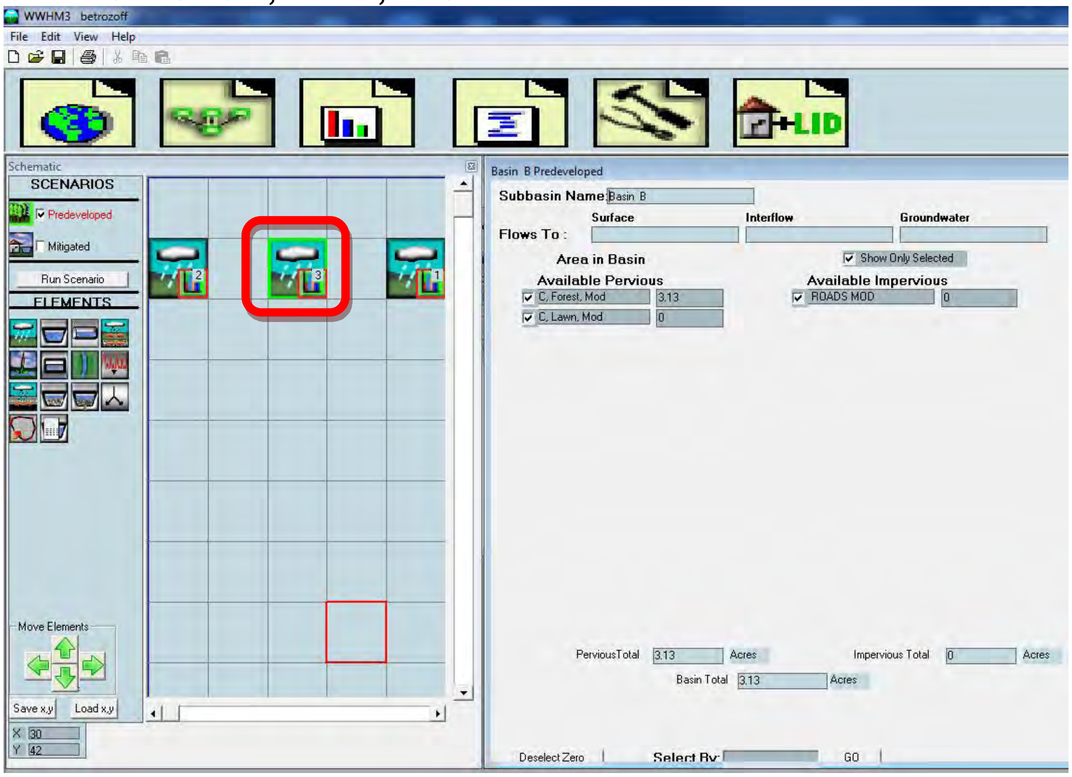
Basin A – modeled as C, Forest, Moderate



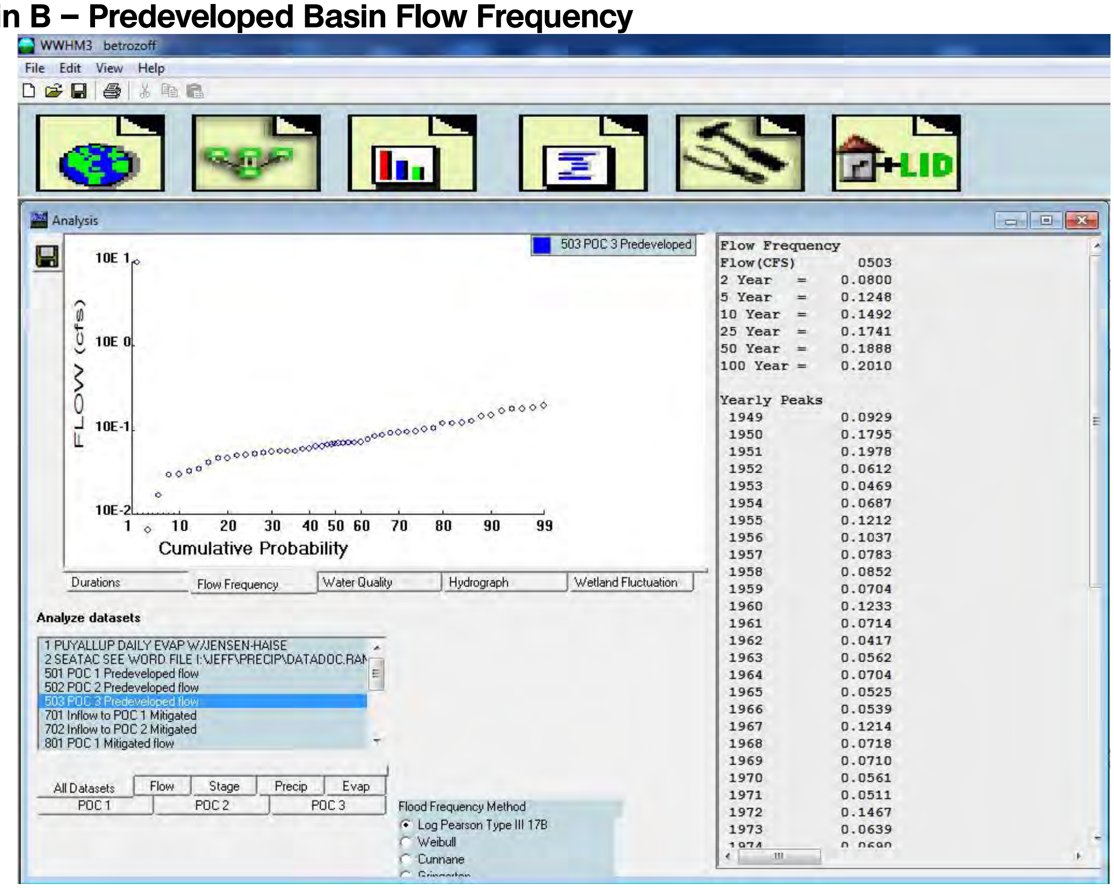
Basin A Predeveloped Flow Frequency



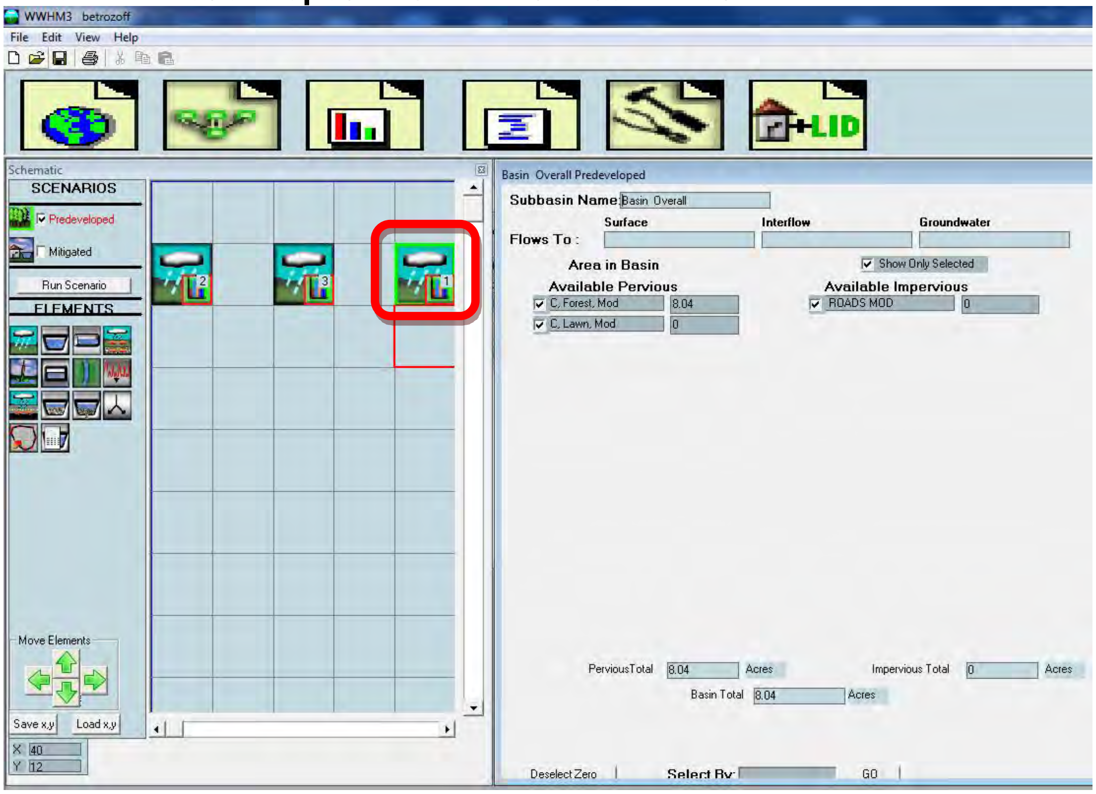
Basin B – modeled as C, Forest, Moderate



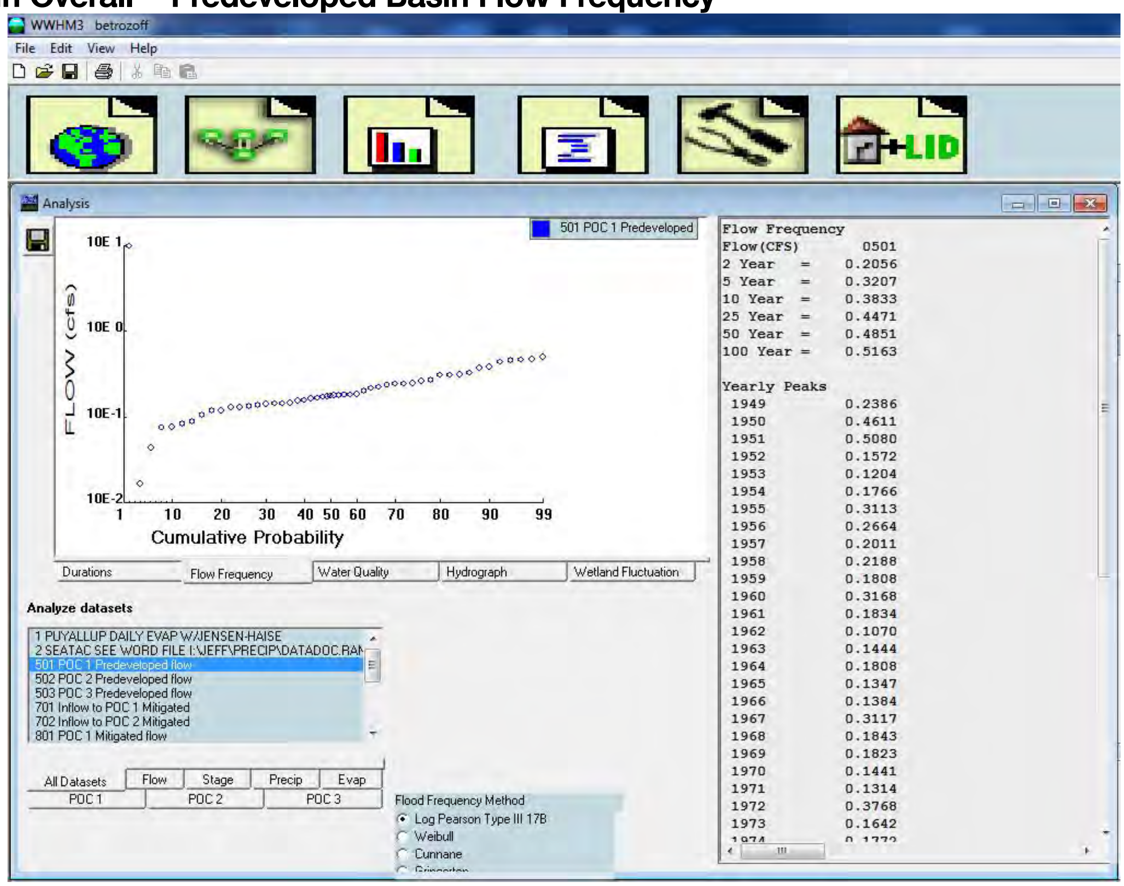
Basin B – Predeveloped Basin Flow Frequency



Basin Overall – Predeveloped Basin A and B combined

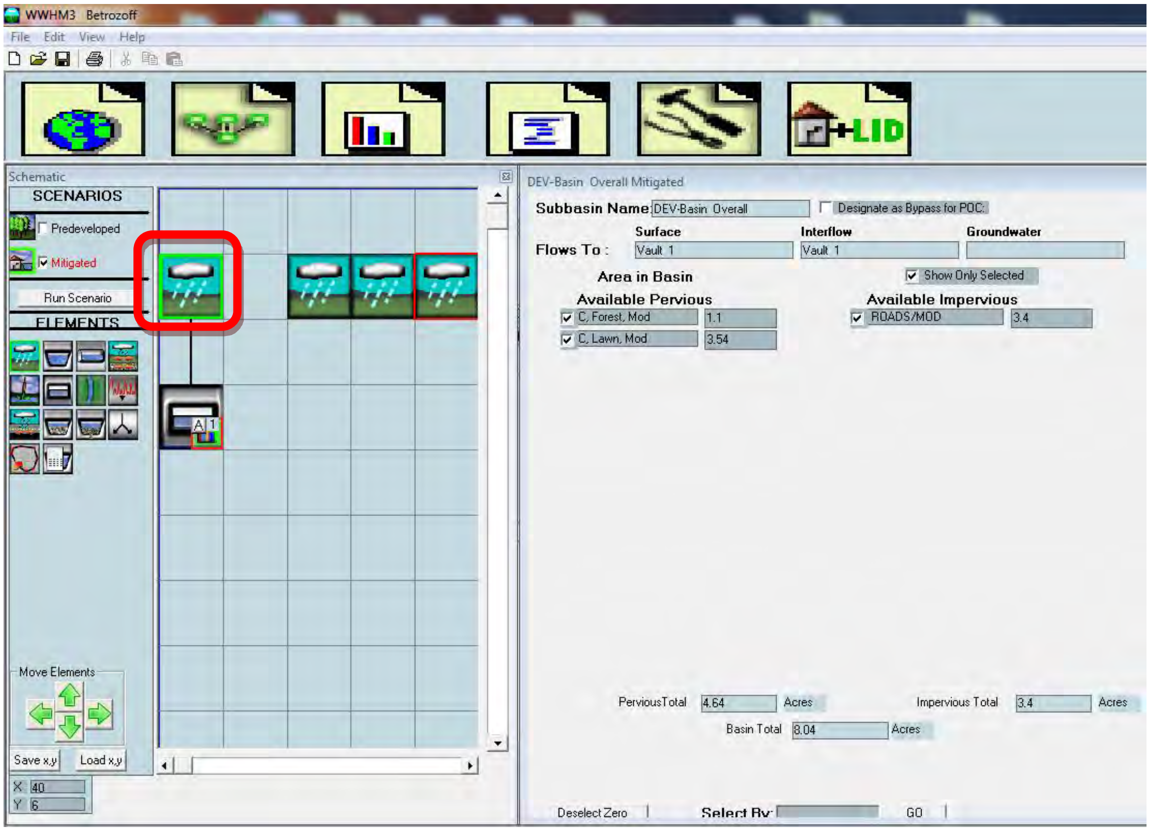


Basin Overall – Predeveloped Basin Flow Frequency

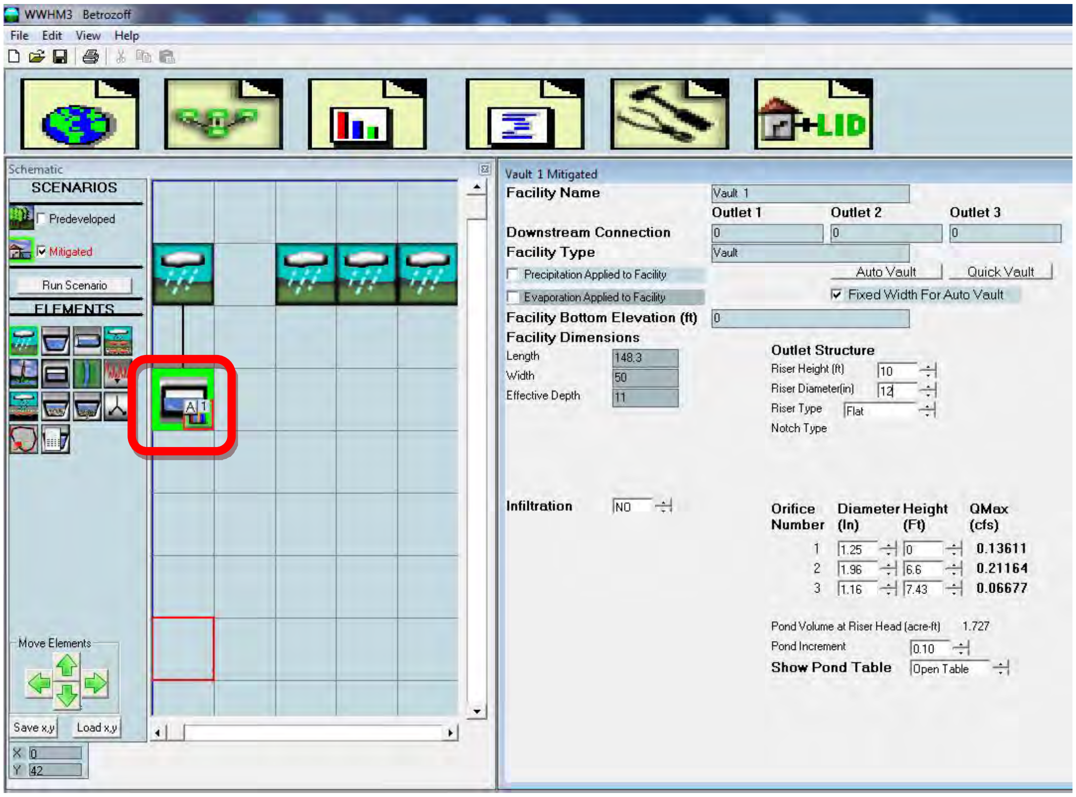


Developed Basins

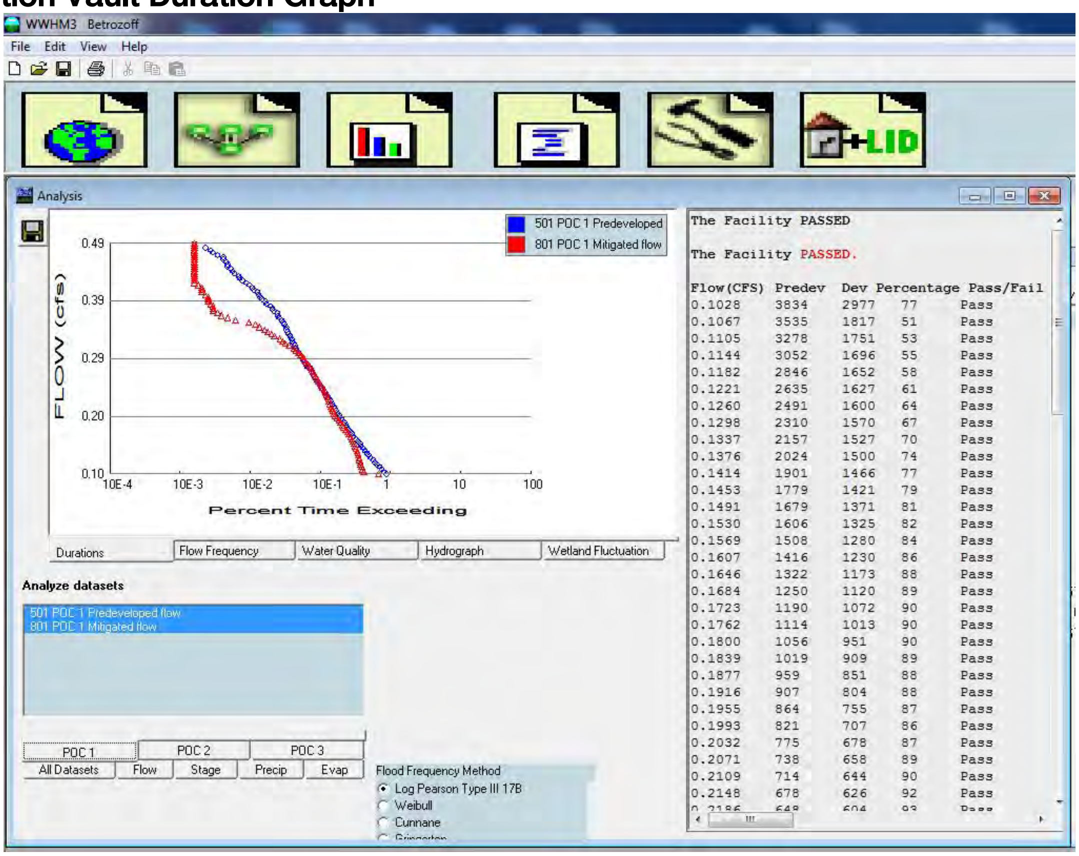
Basin B1 Developed Basin



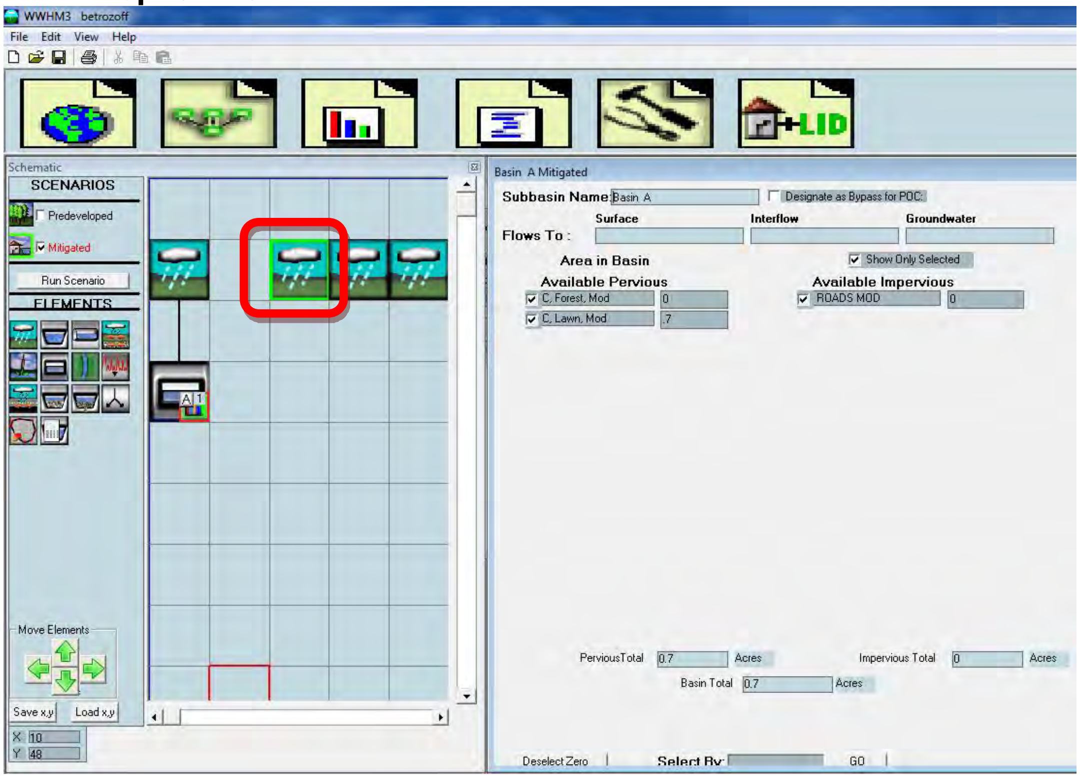
Detention Vault



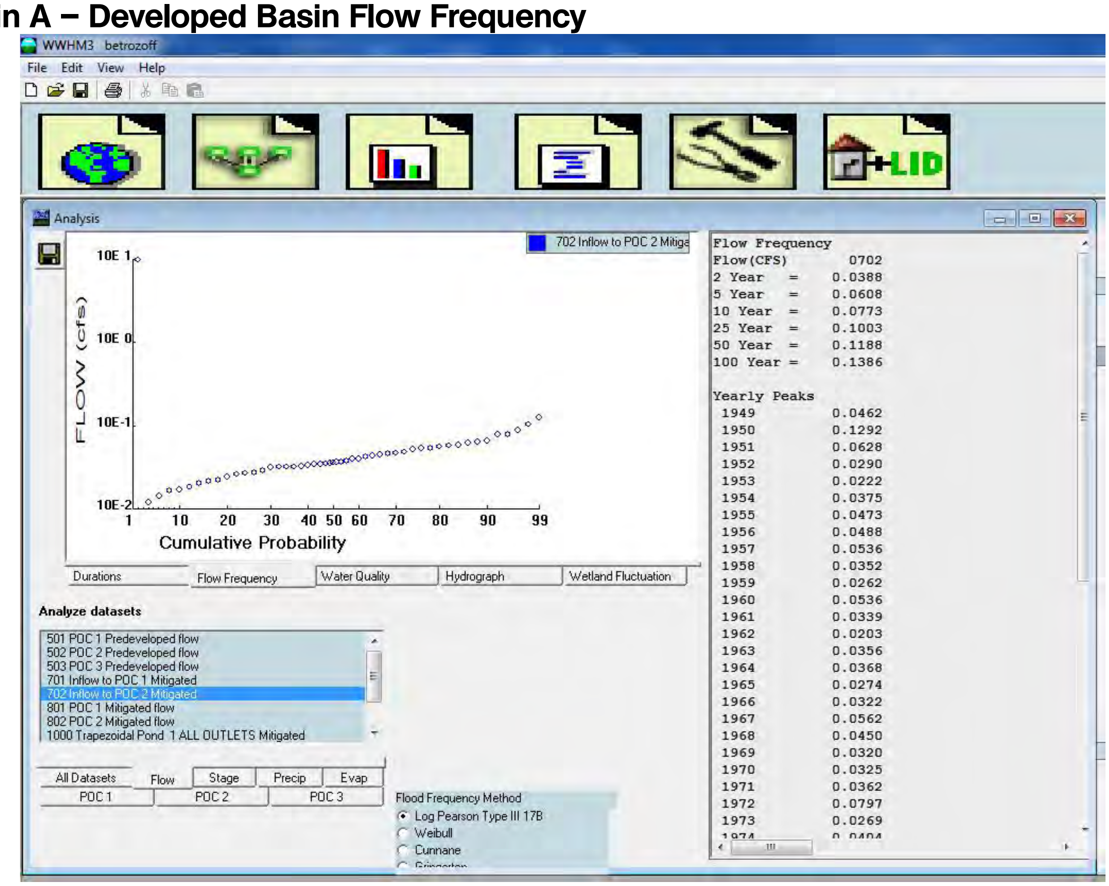
Detention Vault Duration Graph



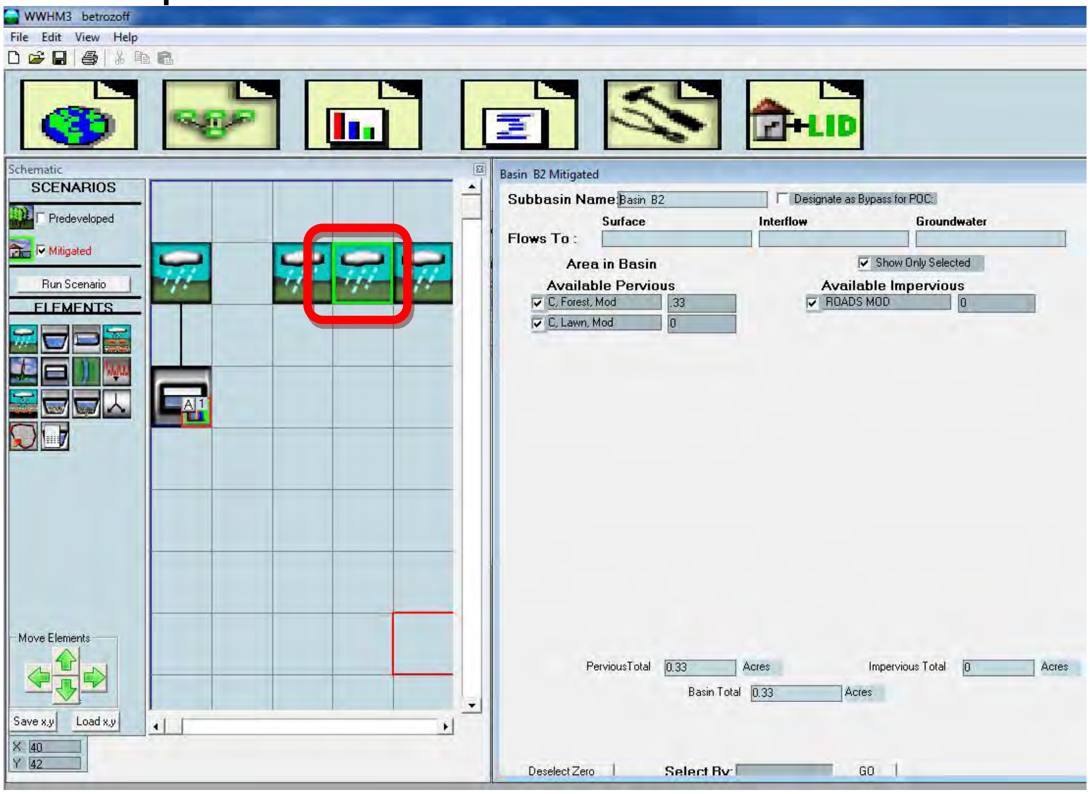
Basin A Developed Basin



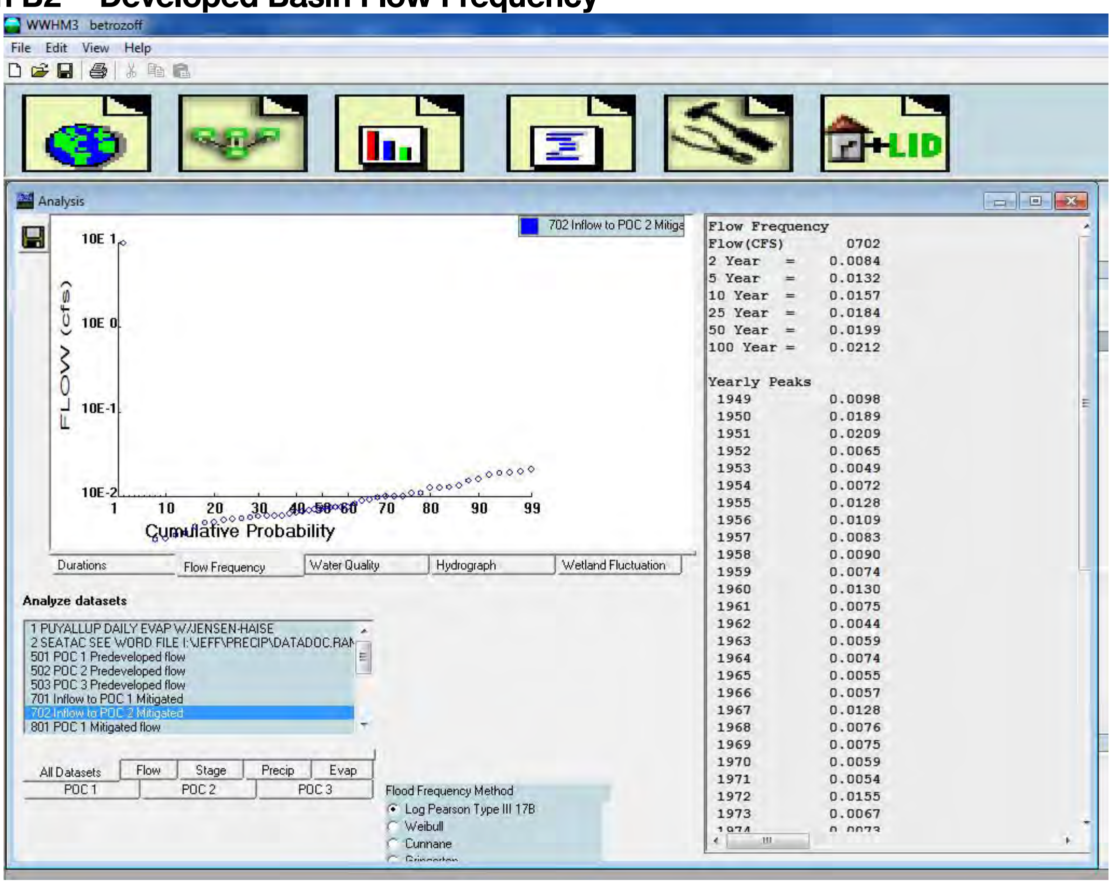
Basin A – Developed Basin Flow Frequency



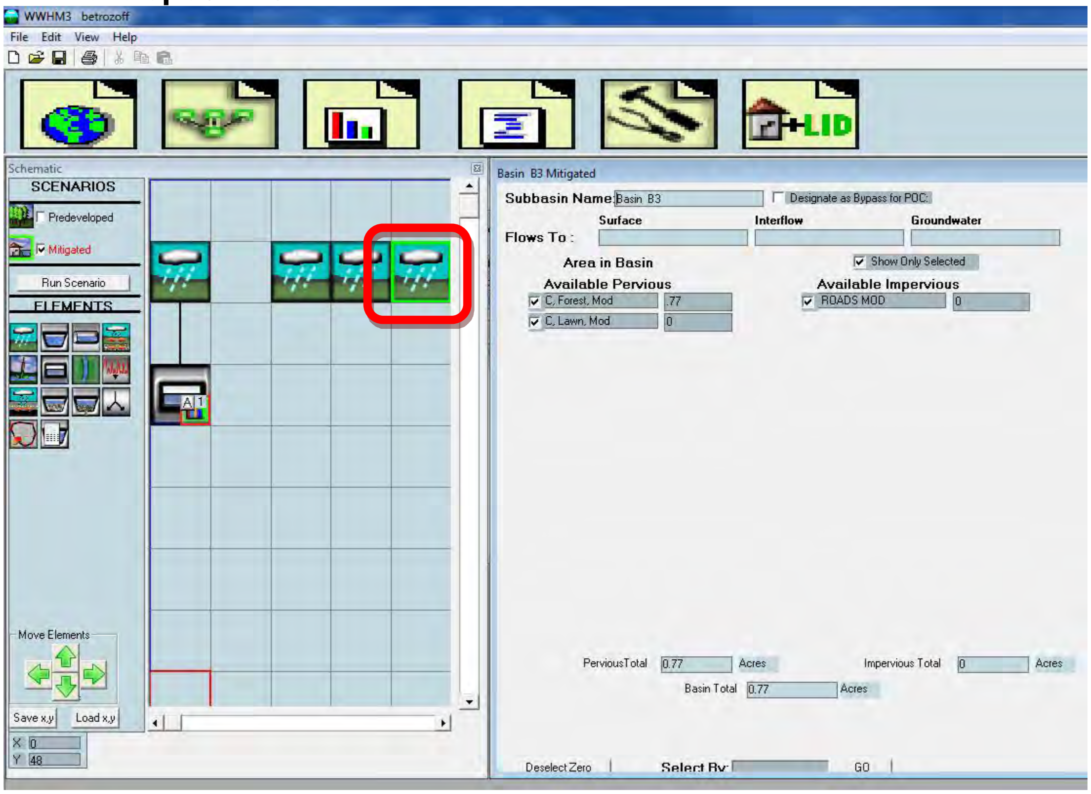
Basin B2 Developed Basin



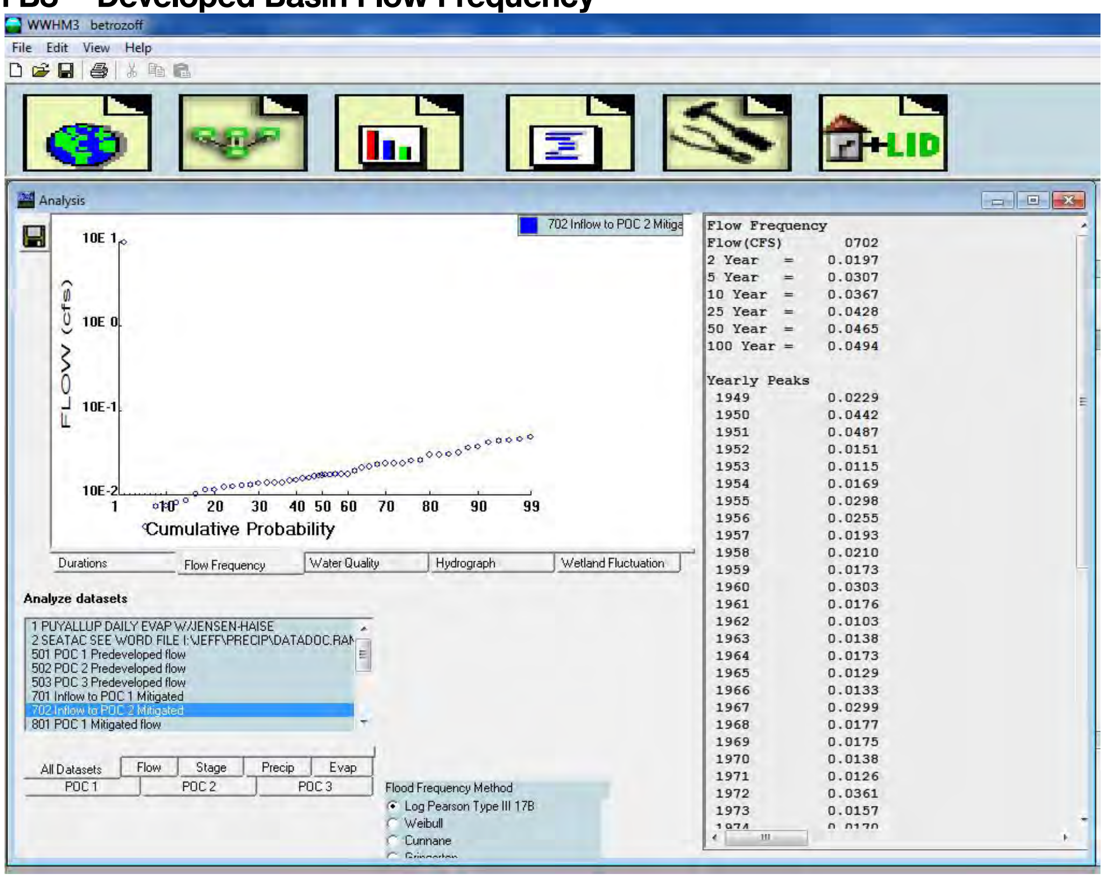
Basin B2 – Developed Basin Flow Frequency



Basin B3 Developed Basin



Basin B3 – Developed Basin Flow Frequency



Project Report

Western Washington Hydrology Model
PROJECT REPORT

Project Name: Betrozoff
Site Address: 11845 Woodinville Redmond Road NE
City : Redmond
Report Date : 8/8/2013
Gage : Seatac
Data Start : 1948/10/01
Data End : 1998/09/30
Precip Scale: 1.00
WWHM3 Version:

PREDEVELOPED LAND USE

Name : Basin A
Bypass: No

GroundWater: No

Pervious Land Use	Acres
C, Forest, Mod	4.92

Impervious Land Use	Acres

Element Flows To:

Surface	Interflow	Groundwater

Name : Basin B
Bypass: No

GroundWater: No

Pervious Land Use	Acres
C, Forest, Mod	3.13

Impervious Land Use	Acres

Element Flows To:

Surface	Interflow	Groundwater

Name : Basin Overall
Bypass: No

GroundWater: No

Pervious Land Use **Acres**
C, Forest, Mod **8.04**

Impervious Land Use **Acres**

Element Flows To:
Surface **Interflow** **Groundwater**

Name : DEV-Basin Overall
Bypass: No

GroundWater: No

Pervious Land Use **Acres**
C, Forest, Mod **1.1**
C, Lawn, Mod **3.54**

Impervious Land Use **Acres**
ROADS MOD **3.4**

Element Flows To:
Surface **Interflow** **Groundwater**
Vault 1, Vault 1,

Name : Vault 1
Width : 50 ft.
Length : 148.3 ft.
Depth: 11ft.
Discharge Structure
Riser Height: 10 ft.
Riser Diameter: 12 in.
Orifice 1 Diameter: 1.25 in. **Elevation:** 0 ft.
Orifice 1 Diameter: 1.96 in. **Elevation:** 6.6 ft.
Orifice 1 Diameter: 1.16 in. **Elevation:** 7.43 ft.

Element Flows To:
Outlet 1 **Outlet 2**

Vault Hydraulic Table

Stage(ft)	Area(acr)	Volume(acr-ft)	Dschrg(cfs)	Infilt(cfs)
0.000	0.170	0.000	0.000	0.000
0.122	0.170	0.021	0.014	0.000
0.244	0.170	0.042	0.020	0.000
0.367	0.170	0.062	0.025	0.000
0.489	0.170	0.083	0.029	0.000

0.611	0.170	0.104	0.032	0.000
0.733	0.170	0.125	0.035	0.000
0.856	0.170	0.146	0.038	0.000
0.978	0.170	0.166	0.041	0.000
1.100	0.170	0.187	0.043	0.000
1.222	0.170	0.208	0.045	0.000
1.344	0.170	0.229	0.048	0.000
1.467	0.170	0.250	0.050	0.000
1.589	0.170	0.270	0.052	0.000
1.711	0.170	0.291	0.054	0.000
1.833	0.170	0.312	0.056	0.000
1.956	0.170	0.333	0.057	0.000
2.078	0.170	0.354	0.059	0.000
2.200	0.170	0.374	0.061	0.000
2.322	0.170	0.395	0.063	0.000
2.444	0.170	0.416	0.064	0.000
2.567	0.170	0.437	0.066	0.000
2.689	0.170	0.458	0.067	0.000
2.811	0.170	0.479	0.069	0.000
2.933	0.170	0.499	0.070	0.000
3.056	0.170	0.520	0.072	0.000
3.178	0.170	0.541	0.073	0.000
3.300	0.170	0.562	0.075	0.000
3.422	0.170	0.583	0.076	0.000
3.544	0.170	0.603	0.077	0.000
3.667	0.170	0.624	0.079	0.000
3.789	0.170	0.645	0.080	0.000
3.911	0.170	0.666	0.081	0.000
4.033	0.170	0.687	0.082	0.000
4.156	0.170	0.707	0.084	0.000
4.278	0.170	0.728	0.085	0.000
4.400	0.170	0.749	0.086	0.000
4.522	0.170	0.770	0.087	0.000
4.644	0.170	0.791	0.088	0.000
4.767	0.170	0.811	0.090	0.000
4.889	0.170	0.832	0.091	0.000
5.011	0.170	0.853	0.092	0.000
5.133	0.170	0.874	0.093	0.000
5.256	0.170	0.895	0.094	0.000
5.378	0.170	0.915	0.095	0.000
5.500	0.170	0.936	0.096	0.000
5.622	0.170	0.957	0.097	0.000
5.744	0.170	0.978	0.098	0.000
5.867	0.170	0.999	0.099	0.000
5.989	0.170	1.019	0.100	0.000
6.111	0.170	1.040	0.101	0.000
6.233	0.170	1.061	0.102	0.000
6.356	0.170	1.082	0.103	0.000
6.478	0.170	1.103	0.104	0.000
6.600	0.170	1.123	0.105	0.000
6.722	0.170	1.144	0.142	0.000
6.844	0.170	1.165	0.157	0.000
6.967	0.170	1.186	0.169	0.000
7.089	0.170	1.207	0.180	0.000
7.211	0.170	1.228	0.189	0.000
7.333	0.170	1.248	0.198	0.000
7.456	0.170	1.269	0.211	0.000
7.578	0.170	1.290	0.226	0.000
7.700	0.170	1.311	0.238	0.000

7.822	0.170	1.332	0.248	0.000
7.944	0.170	1.352	0.258	0.000
8.067	0.170	1.373	0.267	0.000
8.189	0.170	1.394	0.275	0.000
8.311	0.170	1.415	0.283	0.000
8.433	0.170	1.436	0.291	0.000
8.556	0.170	1.456	0.299	0.000
8.678	0.170	1.477	0.306	0.000
8.800	0.170	1.498	0.313	0.000
8.922	0.170	1.519	0.320	0.000
9.044	0.170	1.540	0.326	0.000
9.167	0.170	1.560	0.332	0.000
9.289	0.170	1.581	0.339	0.000
9.411	0.170	1.602	0.345	0.000
9.533	0.170	1.623	0.351	0.000
9.656	0.170	1.644	0.357	0.000
9.778	0.170	1.664	0.362	0.000
9.900	0.170	1.685	0.368	0.000
10.02	0.170	1.706	0.406	0.000
10.14	0.170	1.727	0.914	0.000
10.27	0.170	1.748	1.725	0.000
10.39	0.170	1.768	2.751	0.000
10.51	0.170	1.789	3.953	0.000
10.63	0.170	1.810	5.308	0.000
10.76	0.170	1.831	6.801	0.000
10.88	0.170	1.852	8.419	0.000
11.00	0.170	1.872	10.15	0.000
11.12	0.170	1.893	12.00	0.000
11.24	0.000	0.000	13.94	0.000

Name : Basin A
Bypass: No

GroundWater: No

Pervious Land Use	**Acres**
C, Lawn, Mod	**.7**

Impervious Land Use **Acres**

Element Flows To:
Surface **Interflow** **Groundwater**

Name : Basin B1
Bypass: No

GroundWater: No

Pervious Land Use	**Acres**
C, Forest, Mod	**.33**

Impervious Land Use **Acres**

Element Flows To:
Surface **Interflow** **Groundwater**

Name : Basin B2
Bypass: No

GroundWater: No

Pervious Land Use	Acres
C, Forest, Mod	.77

Impervious Land Use	Acres

Element Flows To:
Surface **Interflow** **Groundwater**

MITIGATED LAND USE

ANALYSIS RESULTS

Flow Frequency Return Periods for Predeveloped. POC #1

Return Period	Flow(cfs)
2 year	0.205609
5 year	0.320673
10 year	0.3833
25 year	0.447103
50 year	0.48505
100 year	0.516267

Flow Frequency Return Periods for Mitigated. POC #1

Return Period	Flow(cfs)
2 year	0.135044
5 year	0.230771
10 year	0.316314
25 year	0.455139
50 year	0.584732
100 year	0.740324

Yearly Peaks for Predeveloped and Mitigated. POC #1

Year	Predeveloped	Mitigated
1950	0.239	0.092
1951	0.461	0.197
1952	0.508	0.768
1953	0.157	0.082
1954	0.120	0.100
1955	0.177	0.103

1956	0.311	0.100
1957	0.266	0.264
1958	0.201	0.100
1959	0.219	0.146
1960	0.181	0.091
1961	0.317	0.329
1962	0.183	0.177
1963	0.107	0.077
1964	0.144	0.103
1965	0.181	0.105
1966	0.135	0.212
1967	0.138	0.097
1968	0.312	0.162
1969	0.184	0.098
1970	0.182	0.098
1971	0.144	0.103
1972	0.131	0.102
1973	0.377	0.322
1974	0.164	0.200
1975	0.177	0.146
1976	0.271	0.094
1977	0.167	0.101
1978	0.017	0.074
1979	0.145	0.169
1980	0.084	0.074
1981	0.245	0.315
1982	0.129	0.102
1983	0.247	0.256
1984	0.224	0.143
1985	0.143	0.082
1986	0.077	0.081
1987	0.391	0.198
1988	0.330	0.297
1989	0.120	0.083
1990	0.076	0.080
1991	0.535	0.357
1992	0.467	0.320
1993	0.154	0.174
1994	0.172	0.083
1995	0.044	0.070
1996	0.245	0.182
1997	0.479	0.636
1998	0.440	0.358
1999	0.089	0.080

Ranked Yearly Peaks for Predeveloped and Mitigated. POC #1

Rank	**Predeveloped**	**Mitigated**
1	0.5346	0.7681
2	0.5080	0.6360
3	0.4792	0.3584
4	0.4668	0.3572
5	0.4611	0.3290
6	0.4403	0.3218
7	0.3906	0.3202
8	0.3768	0.3154
9	0.3304	0.2971
10	0.3168	0.2638
11	0.3117	0.2560

12	0.3113	0.2115
13	0.2707	0.1997
14	0.2664	0.1983
15	0.2475	0.1973
16	0.2448	0.1816
17	0.2447	0.1772
18	0.2386	0.1743
19	0.2244	0.1687
20	0.2188	0.1625
21	0.2011	0.1460
22	0.1843	0.1455
23	0.1834	0.1430
24	0.1823	0.1048
25	0.1808	0.1033
26	0.1808	0.1029
27	0.1772	0.1027
28	0.1766	0.1024
29	0.1722	0.1015
30	0.1668	0.1014
31	0.1642	0.1003
32	0.1572	0.1001
33	0.1542	0.1000
34	0.1455	0.0979
35	0.1444	0.0978
36	0.1441	0.0968
37	0.1428	0.0944
38	0.1384	0.0916
39	0.1347	0.0911
40	0.1314	0.0832
41	0.1293	0.0830
42	0.1204	0.0822
43	0.1196	0.0818
44	0.1070	0.0811
45	0.0894	0.0799
46	0.0844	0.0799
47	0.0773	0.0767
48	0.0757	0.0740
49	0.0435	0.0738
50	0.0165	0.0700

POC #1
The Facility PASSED

The Facility PASSED.

Flow(CFS)	Predev	Dev	Percentage	Pass/Fail
0.1028	3834	2977	77	Pass
0.1067	3535	1817	51	Pass
0.1105	3278	1751	53	Pass
0.1144	3052	1696	55	Pass
0.1182	2846	1653	58	Pass
0.1221	2635	1627	61	Pass
0.1260	2491	1600	64	Pass
0.1298	2310	1571	68	Pass
0.1337	2157	1527	70	Pass
0.1376	2024	1500	74	Pass
0.1414	1901	1466	77	Pass
0.1453	1779	1421	79	Pass

0.1491	1679	1371	81	Pass
0.1530	1606	1325	82	Pass
0.1569	1508	1280	84	Pass
0.1607	1416	1230	86	Pass
0.1646	1322	1173	88	Pass
0.1684	1250	1120	89	Pass
0.1723	1190	1072	90	Pass
0.1762	1114	1013	90	Pass
0.1800	1056	951	90	Pass
0.1839	1019	910	89	Pass
0.1877	959	852	88	Pass
0.1916	907	804	88	Pass
0.1955	864	755	87	Pass
0.1993	821	708	86	Pass
0.2032	775	678	87	Pass
0.2071	738	658	89	Pass
0.2109	714	644	90	Pass
0.2148	678	626	92	Pass
0.2186	648	605	93	Pass
0.2225	619	586	94	Pass
0.2264	588	574	97	Pass
0.2302	567	554	97	Pass
0.2341	533	538	100	Pass
0.2379	515	524	101	Pass
0.2418	479	503	105	Pass
0.2457	455	474	104	Pass
0.2495	434	454	104	Pass
0.2534	418	432	103	Pass
0.2572	391	408	104	Pass
0.2611	372	389	104	Pass
0.2650	357	374	104	Pass
0.2688	344	362	105	Pass
0.2727	326	347	106	Pass
0.2766	307	332	108	Pass
0.2804	283	312	110	Pass
0.2843	274	298	108	Pass
0.2881	260	279	107	Pass
0.2920	248	264	106	Pass
0.2959	233	250	107	Pass
0.2997	226	230	101	Pass
0.3036	215	215	100	Pass
0.3074	206	199	96	Pass
0.3113	199	182	91	Pass
0.3152	189	157	83	Pass
0.3190	182	138	75	Pass
0.3229	175	127	72	Pass
0.3267	171	117	68	Pass
0.3306	163	98	60	Pass
0.3345	155	90	58	Pass
0.3383	151	79	52	Pass
0.3422	145	71	48	Pass
0.3461	143	60	41	Pass
0.3499	136	52	38	Pass
0.3538	130	42	32	Pass
0.3576	123	29	23	Pass
0.3615	113	21	18	Pass
0.3654	109	18	16	Pass
0.3692	107	16	14	Pass
0.3731	96	14	14	Pass

0.3769	91	13	14	Pass
0.3808	86	12	13	Pass
0.3847	75	12	16	Pass
0.3885	73	11	15	Pass
0.3924	64	11	17	Pass
0.3962	61	9	14	Pass
0.4001	58	9	15	Pass
0.4040	56	9	16	Pass
0.4078	49	9	18	Pass
0.4117	47	8	17	Pass
0.4156	43	8	18	Pass
0.4194	39	8	20	Pass
0.4233	34	8	23	Pass
0.4271	31	8	25	Pass
0.4310	30	8	26	Pass
0.4349	26	7	26	Pass
0.4387	25	7	28	Pass
0.4426	23	7	30	Pass
0.4464	21	7	33	Pass
0.4503	21	7	33	Pass
0.4542	20	7	35	Pass
0.4580	18	7	38	Pass
0.4619	18	7	38	Pass
0.4657	15	7	46	Pass
0.4696	13	7	53	Pass
0.4735	12	6	50	Pass
0.4773	10	5	50	Pass
0.4812	7	5	71	Pass
0.4851	7	5	71	Pass

Water Quality BMP Flow and Volume for POC 1.
On-line facility volume: 0.5425 acre-feet
On-line facility target flow: 0.01 **cfs.**
Adjusted for 15 min: 0.5759 **cfs.**
Off-line facility target flow: 0.3031 **cfs.**
Adjusted for 15 min: 0.3197 **cfs.**

Flow Frequency Return Periods for Predeveloped. POC #2

Return Period	Flow(cfs)
2 year	0.080044
5 year	0.124839
10 year	0.14922
25 year	0.174059
50 year	0.188832
100 year	0.200985

Flow Frequency Return Periods for Mitigated. POC #2

Return Period	Flow(cfs)
2 year	0
5 year	0
10 year	0
25 year	0
50 year	0
100 year	0

Yearly Peaks for Predeveloped and Mitigated. POC #2

Year	Predeveloped	Mitigated
1950	0.093	0.000
1951	0.180	0.000
1952	0.198	0.000
1953	0.061	0.000

1954	0.047	0.000
1955	0.069	0.000
1956	0.121	0.000
1957	0.104	0.000
1958	0.078	0.000
1959	0.085	0.000
1960	0.070	0.000
1961	0.123	0.000
1962	0.071	0.000
1963	0.042	0.000
1964	0.056	0.000
1965	0.070	0.000
1966	0.052	0.000
1967	0.054	0.000
1968	0.121	0.000
1969	0.072	0.000
1970	0.071	0.000
1971	0.056	0.000
1972	0.051	0.000
1973	0.147	0.000
1974	0.064	0.000
1975	0.069	0.000
1976	0.105	0.000
1977	0.065	0.000
1978	0.006	0.000
1979	0.057	0.000
1980	0.033	0.000
1981	0.095	0.000
1982	0.050	0.000
1983	0.096	0.000
1984	0.087	0.000
1985	0.056	0.000
1986	0.030	0.000
1987	0.152	0.000
1988	0.129	0.000
1989	0.047	0.000
1990	0.029	0.000
1991	0.208	0.000
1992	0.182	0.000
1993	0.060	0.000
1994	0.067	0.000
1995	0.017	0.000
1996	0.095	0.000
1997	0.187	0.000
1998	0.171	0.000
1999	0.035	0.000

Ranked Yearly Peaks for Predeveloped and Mitigated. POC #2

Rank	**Predeveloped**	**Mitigated**
1	0.2081	0.0000
2	0.1978	0.0000
3	0.1866	0.0000
4	0.1817	0.0000
5	0.1795	0.0000
6	0.1714	0.0000
7	0.1520	0.0000
8	0.1467	0.0000
9	0.1286	0.0000
10	0.1233	0.0000
11	0.1214	0.0000
12	0.1212	0.0000
13	0.1054	0.0000
14	0.1037	0.0000
15	0.0963	0.0000
16	0.0953	0.0000
17	0.0953	0.0000
18	0.0929	0.0000
19	0.0874	0.0000
20	0.0852	0.0000
21	0.0783	0.0000
22	0.0718	0.0000
23	0.0714	0.0000

24	0.0710	0.0000
25	0.0704	0.0000
26	0.0704	0.0000
27	0.0690	0.0000
28	0.0687	0.0000
29	0.0670	0.0000
30	0.0649	0.0000
31	0.0639	0.0000
32	0.0612	0.0000
33	0.0600	0.0000
34	0.0566	0.0000
35	0.0562	0.0000
36	0.0561	0.0000
37	0.0556	0.0000
38	0.0539	0.0000
39	0.0525	0.0000
40	0.0511	0.0000
41	0.0503	0.0000
42	0.0469	0.0000
43	0.0466	0.0000
44	0.0417	0.0000
45	0.0348	0.0000
46	0.0329	0.0000
47	0.0301	0.0000
48	0.0295	0.0000
49	0.0169	0.0000
50	0.0064	0.0000

POC #2
The Facility PASSED

The Facility PASSED.

Flow(CFS)	Predev	Dev	Percentage	Pass/Fail
0.0400	3772	0	0	Pass
0.0415	3490	0	0	Pass
0.0430	3242	0	0	Pass
0.0445	3036	0	0	Pass
0.0460	2832	0	0	Pass
0.0475	2633	0	0	Pass
0.0490	2450	0	0	Pass
0.0505	2274	0	0	Pass
0.0520	2129	0	0	Pass
0.0536	1998	0	0	Pass
0.0551	1884	0	0	Pass
0.0566	1771	0	0	Pass
0.0581	1678	0	0	Pass
0.0596	1584	0	0	Pass
0.0611	1490	0	0	Pass
0.0626	1398	0	0	Pass
0.0641	1315	0	0	Pass
0.0656	1238	0	0	Pass
0.0671	1187	0	0	Pass
0.0686	1114	0	0	Pass
0.0701	1056	0	0	Pass
0.0716	1004	0	0	Pass
0.0731	954	0	0	Pass
0.0746	902	0	0	Pass
0.0761	861	0	0	Pass
0.0776	818	0	0	Pass
0.0791	774	0	0	Pass
0.0806	738	0	0	Pass
0.0821	710	0	0	Pass
0.0836	669	0	0	Pass

0.0851	643	0	0	Pass
0.0866	616	0	0	Pass
0.0881	586	0	0	Pass
0.0896	566	0	0	Pass
0.0911	533	0	0	Pass
0.0926	508	0	0	Pass
0.0941	474	0	0	Pass
0.0956	452	0	0	Pass
0.0971	432	0	0	Pass
0.0986	417	0	0	Pass
0.1001	391	0	0	Pass
0.1017	372	0	0	Pass
0.1032	354	0	0	Pass
0.1047	342	0	0	Pass
0.1062	322	0	0	Pass
0.1077	305	0	0	Pass
0.1092	282	0	0	Pass
0.1107	273	0	0	Pass
0.1122	260	0	0	Pass
0.1137	245	0	0	Pass
0.1152	233	0	0	Pass
0.1167	224	0	0	Pass
0.1182	212	0	0	Pass
0.1197	206	0	0	Pass
0.1212	198	0	0	Pass
0.1227	189	0	0	Pass
0.1242	184	0	0	Pass
0.1257	174	0	0	Pass
0.1272	168	0	0	Pass
0.1287	159	0	0	Pass
0.1302	155	0	0	Pass
0.1317	151	0	0	Pass
0.1332	145	0	0	Pass
0.1347	143	0	0	Pass
0.1362	135	0	0	Pass
0.1377	127	0	0	Pass
0.1392	120	0	0	Pass
0.1407	113	0	0	Pass
0.1422	109	0	0	Pass
0.1437	107	0	0	Pass
0.1452	96	0	0	Pass
0.1467	89	0	0	Pass
0.1482	84	0	0	Pass
0.1498	75	0	0	Pass
0.1513	72	0	0	Pass
0.1528	63	0	0	Pass
0.1543	61	0	0	Pass
0.1558	58	0	0	Pass
0.1573	52	0	0	Pass
0.1588	49	0	0	Pass
0.1603	47	0	0	Pass
0.1618	43	0	0	Pass
0.1633	39	0	0	Pass
0.1648	34	0	0	Pass
0.1663	31	0	0	Pass
0.1678	29	0	0	Pass
0.1693	26	0	0	Pass
0.1708	25	0	0	Pass
0.1723	23	0	0	Pass

0.1738	21	0	0	Pass
0.1753	21	0	0	Pass
0.1768	20	0	0	Pass
0.1783	18	0	0	Pass
0.1798	17	0	0	Pass
0.1813	14	0	0	Pass
0.1828	13	0	0	Pass
0.1843	12	0	0	Pass
0.1858	10	0	0	Pass
0.1873	7	0	0	Pass
0.1888	7	0	0	Pass

Water Quality BMP Flow and Volume for POC 2.
On-line facility volume: 0 acre-feet
On-line facility target flow: 0 **cfs.**
Adjusted for 15 min: 0 **cfs.**
Off-line facility target flow: 0 **cfs.**
Adjusted for 15 min: 0 **cfs.**

Flow Frequency Return Periods for Predeveloped. POC #3

Return Period	Flow(cfs)
2 year	0.205609
5 year	0.320673
10 year	0.3833
25 year	0.447103
50 year	0.48505
100 year	0.516267

Flow Frequency Return Periods for Mitigated. POC #3

Return Period	Flow(cfs)
2 year	0
5 year	0
10 year	0
25 year	0
50 year	0
100 year	0

Yearly Peaks for Predeveloped and Mitigated. POC #3

Year	Predeveloped	Mitigated
1950	0.239	0.000
1951	0.461	0.000
1952	0.508	0.000
1953	0.157	0.000
1954	0.120	0.000
1955	0.177	0.000
1956	0.311	0.000
1957	0.266	0.000
1958	0.201	0.000
1959	0.219	0.000
1960	0.181	0.000
1961	0.317	0.000
1962	0.183	0.000
1963	0.107	0.000
1964	0.144	0.000
1965	0.181	0.000
1966	0.135	0.000
1967	0.138	0.000
1968	0.312	0.000
1969	0.184	0.000
1970	0.182	0.000
1971	0.144	0.000
1972	0.131	0.000
1973	0.377	0.000
1974	0.164	0.000
1975	0.177	0.000

1976	0.271	0.000
1977	0.167	0.000
1978	0.017	0.000
1979	0.145	0.000
1980	0.084	0.000
1981	0.245	0.000
1982	0.129	0.000
1983	0.247	0.000
1984	0.224	0.000
1985	0.143	0.000
1986	0.077	0.000
1987	0.391	0.000
1988	0.330	0.000
1989	0.120	0.000
1990	0.076	0.000
1991	0.535	0.000
1992	0.467	0.000
1993	0.154	0.000
1994	0.172	0.000
1995	0.044	0.000
1996	0.245	0.000
1997	0.479	0.000
1998	0.440	0.000
1999	0.089	0.000

Ranked Yearly Peaks for Predeveloped and Mitigated. POC #3

Rank	Predeveloped	Mitigated
1	0.5346	0.0000
2	0.5080	0.0000
3	0.4792	0.0000
4	0.4668	0.0000
5	0.4611	0.0000
6	0.4403	0.0000
7	0.3906	0.0000
8	0.3768	0.0000
9	0.3304	0.0000
10	0.3168	0.0000
11	0.3117	0.0000
12	0.3113	0.0000
13	0.2707	0.0000
14	0.2664	0.0000
15	0.2475	0.0000
16	0.2448	0.0000
17	0.2447	0.0000
18	0.2386	0.0000
19	0.2244	0.0000
20	0.2188	0.0000
21	0.2011	0.0000
22	0.1843	0.0000
23	0.1834	0.0000
24	0.1823	0.0000
25	0.1808	0.0000
26	0.1808	0.0000
27	0.1772	0.0000
28	0.1766	0.0000
29	0.1722	0.0000
30	0.1668	0.0000
31	0.1642	0.0000
32	0.1572	0.0000
33	0.1542	0.0000
34	0.1455	0.0000
35	0.1444	0.0000
36	0.1441	0.0000
37	0.1428	0.0000
38	0.1384	0.0000
39	0.1347	0.0000
40	0.1314	0.0000
41	0.1293	0.0000
42	0.1204	0.0000
43	0.1196	0.0000
44	0.1070	0.0000
45	0.0894	0.0000

46	0.0844	0.0000
47	0.0773	0.0000
48	0.0757	0.0000
49	0.0435	0.0000
50	0.0165	0.0000

POC #3

The Facility PASSED

The Facility PASSED.

Flow(CFS)	Predev	Dev	Percentage	Pass/Fail
0.1028	3834	0	0	Pass
0.1067	3535	0	0	Pass
0.1105	3278	0	0	Pass
0.1144	3052	0	0	Pass
0.1182	2846	0	0	Pass
0.1221	2635	0	0	Pass
0.1260	2491	0	0	Pass
0.1298	2310	0	0	Pass
0.1337	2157	0	0	Pass
0.1376	2024	0	0	Pass
0.1414	1901	0	0	Pass
0.1453	1779	0	0	Pass
0.1491	1679	0	0	Pass
0.1530	1606	0	0	Pass
0.1569	1508	0	0	Pass
0.1607	1416	0	0	Pass
0.1646	1322	0	0	Pass
0.1684	1250	0	0	Pass
0.1723	1190	0	0	Pass
0.1762	1114	0	0	Pass
0.1800	1056	0	0	Pass
0.1839	1019	0	0	Pass
0.1877	959	0	0	Pass
0.1916	907	0	0	Pass
0.1955	864	0	0	Pass
0.1993	821	0	0	Pass
0.2032	775	0	0	Pass
0.2071	738	0	0	Pass
0.2109	714	0	0	Pass
0.2148	678	0	0	Pass
0.2186	648	0	0	Pass
0.2225	619	0	0	Pass
0.2264	588	0	0	Pass
0.2302	567	0	0	Pass
0.2341	533	0	0	Pass
0.2379	515	0	0	Pass
0.2418	479	0	0	Pass
0.2457	455	0	0	Pass
0.2495	434	0	0	Pass
0.2534	418	0	0	Pass
0.2572	391	0	0	Pass
0.2611	372	0	0	Pass
0.2650	357	0	0	Pass
0.2688	344	0	0	Pass
0.2727	326	0	0	Pass
0.2766	307	0	0	Pass
0.2804	283	0	0	Pass

0.2843	274	0	0	Pass
0.2881	260	0	0	Pass
0.2920	248	0	0	Pass
0.2959	233	0	0	Pass
0.2997	226	0	0	Pass
0.3036	215	0	0	Pass
0.3074	206	0	0	Pass
0.3113	199	0	0	Pass
0.3152	189	0	0	Pass
0.3190	182	0	0	Pass
0.3229	175	0	0	Pass
0.3267	171	0	0	Pass
0.3306	163	0	0	Pass
0.3345	155	0	0	Pass
0.3383	151	0	0	Pass
0.3422	145	0	0	Pass
0.3461	143	0	0	Pass
0.3499	136	0	0	Pass
0.3538	130	0	0	Pass
0.3576	123	0	0	Pass
0.3615	113	0	0	Pass
0.3654	109	0	0	Pass
0.3692	107	0	0	Pass
0.3731	96	0	0	Pass
0.3769	91	0	0	Pass
0.3808	86	0	0	Pass
0.3847	75	0	0	Pass
0.3885	73	0	0	Pass
0.3924	64	0	0	Pass
0.3962	61	0	0	Pass
0.4001	58	0	0	Pass
0.4040	56	0	0	Pass
0.4078	49	0	0	Pass
0.4117	47	0	0	Pass
0.4156	43	0	0	Pass
0.4194	39	0	0	Pass
0.4233	34	0	0	Pass
0.4271	31	0	0	Pass
0.4310	30	0	0	Pass
0.4349	26	0	0	Pass
0.4387	25	0	0	Pass
0.4426	23	0	0	Pass
0.4464	21	0	0	Pass
0.4503	21	0	0	Pass
0.4542	20	0	0	Pass
0.4580	18	0	0	Pass
0.4619	18	0	0	Pass
0.4657	15	0	0	Pass
0.4696	13	0	0	Pass
0.4735	12	0	0	Pass
0.4773	10	0	0	Pass
0.4812	7	0	0	Pass
0.4851	7	0	0	Pass

Water Quality BMP Flow and Volume for POC 3.
On-line facility volume: 0 acre-feet
On-line facility target flow: 0 **cfs.**

Adjusted for 15 min: 0 cfs.
Off-line facility target flow: 0 cfs.
Adjusted for 15 min: 0 cfs.

Perlnd and Implnd Changes
No changes have been made.

***Mitigated flow values are zeros on POC #2 and 3 where the flow control facility is designed under POC #1 which includes the overall site.**