PRELIMINARY TIR

Rio Vista

Vicinity NE 143rd Place and 272nd Place NE
Duvall, WA
Parcel Nos. **782580-0150 thru -0180**

Prepared for:
Rio Vista Investments, LLC
PO Box 1282
Bellevue, WA 98009

Prepared by:

D A
Land Development Advisors, LLC • 12865 SE 47th Place • Bellevue, WA 98006
(425) 466-5203

April, 2015 **REID-004**

TABLE OF CONTENTS

TABLE OF CONTENTS ..... *i*
TIR WORKSHEET I
SECTION 1 PROJECT OVERVIEW 1
EXISTING DRAINAGE SYSTEM 1
PROPOSED DRAINAGE SYSTEM 1
SECTION 2 CONDITIONS AND REQUIREMENTS SUMMARY 2
CORE REQUIREMENTS 2
Core Requirement #1: Discharge at the Natural Location 2
Core Requirement #2: Offsite Analysis 2
Core Requirement #3: Flow Control 2
Core Requirement #4: Conveyance System 2
Core Requirement #5: Erosion and Sediment Control 2
Core Requirement #6: Maintenance and Operations 2
Core Requirement #7: Financial Guarantees and Liability 2
Core Requirement #8: Water Quality 2
SPECIAL REQUIREMENTS 3
Special Requirement #1: Other Adopted Area-Specific Requirements 3
Special Requirement #2: Flood Hazard Delineation 3
Special Requirement #3: Flood Protection Facilities 3
Special Requirement #4: Source Controls 3
Special Requirement #5: Oil Control 3
SECTION 3 OFF-SITE ANALYSIS 4
TASK 1 - STUDY AREA DEFINITION AND MAPS 4
TASK 2 - RESOURCE REVIEW 4
Adopted Basin Plans 4
FEMA Maps 4
Other Off-Site Analyses 5
Sensitive Areas Folio 5
Drainage Complaints and Studies 5
Road Drainage Problems .....
USDA - NRSC Soil Survey Maps 5
Wetlands Inventory Maps 5
Migrating River Studies 5
Section 303d Listing 5
King County Designated Water Quality Problems 5
Adopted Stormwater Compliance Plans 5

TASK 3 – FIELD INSPECTION **6**
TASK 4 – DRAINAGE SYSTEM DESCRIPTION AND PROBLEM DESCRIPTIONS **6**
Downstream Drainage System Description *6*
Downstream Drainage Problems Requiring Special Attention *6*
Downstream Water Quality Problems Requiring Special Attention *7*
TASK 5 – MITIGATION OF EXISTING OR POTENTIAL PROBLEMS **8**
SECTION 4 FLOW CONTROL/WATER QUALITY ANALYSIS AND DESIGN 18
SITE HYDROLOGY **18**
Part A - Existing Site Hydrology *18*
Part B - Developed Site Hydrology *19*
Part C – Performance Standards *22*
Part D – Flow Control System *22*
Part E – Water Quality System *29*
SECTION 5 - CONVEYANCE SYSTEM ANALYSIS AND DESIGN 36
Pipe Sizing *36*
Dispersion Trenches *36*
RECTANGULAR CONTRACTED WEIR **38**
The Equation *38*
APPENDIX **39**
Drainage Complaint *40*
Offsite Analysis and Drainage System Table *41*
Photographs *42*
Calculations *53*
PacLand Downstream Analysis *54*
Associated Earth Sciences, Inc. Reports *55*
Wetland Resources, Inc. Reports *56*

FIGURES

Figure 3.0 - Vicinity Map 9
Figure 3.1 – Site and Upstream Basin Map 10
Figure 3.2 - Downstream Conveyance System 11
Figure 3.3 - NRCS Soils Map 12
Figure 3.4 – Critical Aquifer Recharge Areas (CARA's) 13
Figure 3.5 – Steep Slopes 14
Figure 3.6 – Wetlands 15
Figure 3.7 – Drainage Complaints 16
Figure 3.8 – City Basin Map 17
Figure 4.1 – Basin Areas 31

Figure 4.2 – Rainfall Regions and Regional Scale Factors 32
Figure 4.3 – Precipitation for Mean Annual Storm..... 33
Table 4.3 – Basin Areas 34

TIR WORKSHEET

TECHNICAL INFORMATION REPORT (TIR) WORKSHEET

Part 1 PROJECT OWNER AND PROJECT ENGINEER

Project Owner: Rio Vista Investments, LLC
Phone: 425-785-9651
Address: PO Box 1282
Bellevue, WA 98009
Project Engineer: Jon Nelson
Company: Land Development Advisors, LLC
Phone: 425 466 5203

Part 2 PROJECT LOCATION AND DESCRIPTION

Project Name: Rio Vista
DDES Permit #: _____
Location Township: 26N
Range: 6E
Section: 24
Site Address: _____

Part 3 TYPE OF PERMIT APPLICATION

- [x] Landuse Services
Subdivison / Short Subd. / UPD
- [ ] Building Services
M/F / Commerical / SFR
- [ ] Clearing and Grading
- [ ] Right-of-Way Use
- [ ] Other _____

Part 4 OTHER REVIEWS AND PERMITS

- [ ] DFW HPA
- [ ] COE 404
- [ ] DOE Dam Safety
- [ ] FEMA Floodplain
- [ ] COE Wetlands
- [ ] Other _____
- [ ] Shoreline Management
- [ ] Structural Rockery/Vault/_____
- [ ] ESA Section 7

Part 5 PLAN AND REPORT INFORMATION

Technical Information Report

Type of Drainage Review (circle): Full / Targeted / Large Site
Date (include revision dates): April, 2015
Date of Final: _____

Site Improvement Plan (Engr. Plans)

Type (circle one): Full / Modified / Small Site
Date (include revision dates): _____
Date of Final: _____

Part 6 ADJUSTMENT APPROVALS

Type (circle one): Standard / Complex / Preapplication / Experimental / Blanket
Description: (include conditions in TIR Section 2)
_____
Date of Approval: _____

TECHNICAL INFORMATION REPORT (TIR) WORKSHEET

Part 7 MONITORING REQUIREMENTS

Monitoring Required: (Yes) / No

Start Date: _____

Completion Date: _____

Describe: Wetland enhancement/restoration

Part 8 SITE COMMUNITY AND DRAINAGE BASIN

Community Plan : N/A

Special District Overlays: None

Drainage Basin: Thayer Creek & Coe Clemons

Stormwater Requirements: Stream protection flow control, basic water quality

Part 9 ONSITE AND ADJACENT SENSITIVE AREAS

- ☐ River/Stream _____
- ☐ Lake _____
- ☒ Wetlands _____
- ☐ Closed Depression _____
- ☐ Floodplain _____
- ☐ Other _____
- ☐ Steep Slope _____
- ☐ Erosion Hazard _____
- ☐ Landslide Hazard _____
- ☐ Coal Mine Hazard _____
- ☐ Seismic Hazard _____
- ☐ Habitat Protection _____
- ☐ _____

Part 10 SOILS

Soil Type	Slopes	Erosion Potential
Vashon Lodgement Till	0%-15%	Moderate

- ☐ High Groundwater Table (within 5 feet)
- ☐ Other _____
- ☐ Sole Source Aquifer
- ☐ Seeps/Springs

☐ Additional Sheets Attached

TECHNICAL INFORMATION REPORT (TIR) WORKSHEET

Part 11 DRAINAGE DESIGN LIMITATIONS	
REFERENCE	LIMITATION / SITE CONSTRAINT
☒ Core 2 – Offsite Analysis	Downstream channels and pipes
☒ Sensitive/Critical Areas	Onsite wetland
☐ SEPA	
☐ Other	
☐	
☐ Additional Sheets Attached	

Part 12 TIR SUMMARY SHEET (provide one TIR Summary Sheet per Threshold Discharge Area)	
Threshold Discharge Area: All (name or description)	
Core Requirements (all 8 apply)	
Discharge at Natural Location	Number of Natural Discharge Locations: 3
Offsite Analysis	Level: (1) / 2 / 3 dated: This report
Flow Control (incl. facility summary sheet)	Level: 1 / (2) / 3 or Exemption Number _____ Small Site BMPs _____
Conveyance System	Spill containment located at: Vault
Erosion and Sediment Control	ESC Site Supervisor: To be determined Contact Phone: After Hours Phone:
Maintenance and Operation	Responsibility: Private / (Public) If Private, Maintenance Log Required: Yes / No
Financial Guarantees and Liability	Provided: Yes / (No)
Water Quality (include facility summary sheet)	Type: (Basic) / Sens. Lake / Enhanced Basicm / Bog or Exemption No. _____ Landscape Management Plan: Yes / (No)
Special Requirements (as applicable)	
Area Specific Drainage Requirements	Type: CDA / SDO / MDP / BP / LMP / Shared Fac. / None Name: _____
Floodplain/Floodway Delineation	Type: Major / Minor / Exemption / None 100-year Base Flood Elevation (or range): _____ Datum:
Flood Protection Facilities	Describe:
Source Control (comm./industrial landuse)	Describe landuse: Describe any structural controls:

TECHNICAL INFORMATION REPORT (TIR) WORKSHEET

Oil Control	High-use Site: Yes / (No) Treatment BMP: _____ Maintenance Agreement: Yes / No with whom? _____
Other Drainage Structures	
Describe:	

Part 13 EROSION AND SEDIMENT CONTROL REQUIREMENTS Provided with Final plans		
MINIMUM ESC REQUIREMENTS DURING CONSTRUCTION		MINIMUM ESC REQUIREMENTS AFTER CONSTRUCTION
☐ Clearing Limits		☐ Stabilize Exposed Surfaces
☐ Cover Measures		☐ Remove and Restore Temporary ESC Facilities
☐ Perimeter Protection		☐ Clean and Remove All Silt and Debris, Ensure Operation of Permanent Facilities
☐ Traffic Area Stabilization		☐ Flag Limits of SAO and open space preservation areas
☐ Sediment Retention		☐ Other _____
☐ Surface Water Collection		
☐ Dewatering Control		
☐ Dust Control		
☐ Flow Control		

Part 14 STORMWATER FACILITY DESCRIPTIONS (Note: Include Facility Summary and Sketch)				
Flow Control	Type/Description		Water Quality	Type/Description
☒ Detention	Vault		☐ Biofiltration	_____
☐ Infiltration	_____		☒ Wetpool	_____
☐ Regional Facility	_____		☐ Media Filtration	_____
☐ Shared Facility	_____		☐ Oil Control	_____
☐ Flow Control BMPs	_____		☐ Spill Control	_____
☐ Other	_____		☐ Flow Control BMPs	_____
			☐ Other	_____

TECHNICAL INFORMATION REPORT (TIR) WORKSHEET

Part 15 EASEMENTS/TRACTS		Part 16 STRUCTURAL ANALYSIS
☒ Drainage Easement		☒ Cast in Place Vault
☐ Covenant		☐ Retaining Wall
☐ Native Growth Protection Covenant		☒ Rockery > 4' High
☒ Tract		☐ Structural on Steep Slope
☐ Other		☐ Other

Part 17 SIGNATURE OF PROFESSIONAL ENGINEER
I, or a civil engineer under my supervision, have visited the site. Actual site conditions as observed were incorporated into this worksheet and the attached Technical Information Report. To the best of my knowledge the information provided here is accurate.
Signed/Date

SECTION 1
PROJECT OVERVIEW

This section has been prepared in support of a Preliminary Plat application for the subject property. The 12.72 acre site is located in the south end of the City and is part of the Snoqualmie River Watershed and Snohomish WRIA No. 7. See Figure 3.0 – Vicinity Map. The portion of the site lying southerly of NE 143rd Place is within the Thayer Creek Subbasin with the remainder in the Coe Clemons Sub-basin. This report identifies the tributary basin areas upstream of the Project site, and evaluates downstream drainage systems. The intent of this section is to demonstrate that the proposed Project will not aggravate existing problems nor create new drainage problems.

EXISTING DRAINAGE SYSTEM

There are two separate flow paths for runoff from the project site. Refer to Figure 3.2 – Downstream Conveyance System. Parcel 0150 flows westerly into an offsite wetland. Flows continue westerly towards a culvert crossing NE 143rd Place that discharges into the storm system running westerly in NE 143rd Place. This flow, together with runoff from NE 143rd, continues westerly through a series of ditches, pipes and catchbasins and discharges into Thayer Creek near Main Street, approximately 2,000 feet west. Details of the various components can be found in the Offsite Analysis Drainage System Table. There were no existing problems or deficiencies observed.

The remaining parcels flow into a Class III wetland located in the central portion of the three lots. Flows continue in a northerly direction through various open and closed conveyance elements and ultimately discharge into a City detention facility approximately 1,400 feet to the north. Details of the various components can be found in the Offsite Analysis Drainage System Table.

PROPOSED DRAINAGE SYSTEM

The proposed development will provide appropriate storm drainage conveyance, detention and water quality components. All necessary utilities and stormwater facilities will be constructed to serve the development. NE 143rd Place will be improved and an appropriately sized conveyance system will be designed and constructed to collect the developed on-site runoff from the proposed project. Runoff from the southeast and northeast basins will drain into a detention system on the east side of the central wetland. The outlet from this vault will split flows to the north and west so that existing flows are maintained to each existing flow path. The westerly basin will be captured in a second vault near the northwest corner of the parcel. Both vaults will include basic water quality enhancement by the use of dead storage and Stream Protection outlet controls (aka "Level 2 Flow Control").

The following pages include various exhibits and calculations supporting the design.

SECTION 2
CONDITIONS AND REQUIREMENTS SUMMARY

The City has determined this project is subject to the 2009 King County Surface Water Design Manual ("KCSWDM") including Conservation Flow Control ("Level 2") and Basic water quality requirements. Existing runoff will be computed assuming the site is "forested".

CORE REQUIREMENTS

Core Requirement #1: Discharge at the Natural Location

Site runoff from the SE portion of the site (tax lot 0150) currently discharges to NE 143rd Place and then easterly after flowing through a wetland directly west of the parcel. The remaining parcels on the north side of 143rd flow to the north. Tax lot 0150 will be combined in the detention vault for the northeasterly section of the site (tax lot 0160). The outlet from the vault will split flows to the north and west so that existing flows are maintained in each basin. Please refer to Figure 3.1 and the Level I Downstream Analysis in Section 3 for more information about the downstream system.

Core Requirement #2: Offsite Analysis

A downstream analysis has been completed for the project. Please refer to Section 3 - Offsite Analysis for a complete description.

Core Requirement #3: Flow Control

The project site is required to provide Level 2 flow control.

Core Requirement #4: Conveyance System

Conveyance system calculations will be provided within Section 5 of this report in the final TIR that will accompany the final design to demonstrate the proposed conveyance system will have adequate capacity to convey the 25 year storm without overtopping.

Core Requirement #5: Erosion and Sediment Control

A Temporary Erosion and Sediment Control plan will be prepared with the final construction documents in order to prevent, to the maximum extent possible, the transport of sediment to downstream drainage systems, water resources and adjacent properties. BMP measures anticipated include clearing limit delineation, cover measures (straw, plastic, etc.), traffic area stabilization (rock construction entrance) and perimeter protection (silt fencing).

Core Requirement #6: Maintenance and Operations

Operations and maintenance for the onsite storm system will be provided by the City and they will follow their standard procedures.

Core Requirement #7: Financial Guarantees and Liability

A bond quantity estimate for this project will be provided in Section 9 of this report with the final design.

Core Requirement #8: Water Quality

Basic water quality will be provided in the vaults with "dead" storage.

SPECIAL REQUIREMENTS

Special Requirement #1: Other Adopted Area-Specific Requirements

Critical Drainage Areas
No CDA's are designated on the site.

Master Drainage Plan
Not applicable.

Basin Plans
No plans are in effect.

Lake Management Plans
Not applicable.

Shared Facility Drainage Plans
Not applicable.

Special Requirement #2: Flood Hazard Delineation
The project area does not fall within a flood hazard area.

Special Requirement #3: Flood Protection Facilities
There are currently no flood protection facilities onsite or downstream of the site. Flood protection facilities will not need to be implemented as a result of the proposed development.

Special Requirement #4: Source Controls
Source controls are not required.

Special Requirement #5: Oil Control
Anticipated site uses are not expected to trigger oil control requirements.

SECTION 3
OFF-SITE ANALYSIS

This section has been prepared in support of a Preliminary Plat application for the subject property. The 12.72 acre site is located in the south end of the City and is part of the Snoqualmie River Watershed and Snohomish WRIA No. 7. See Figure 3.0 – Vicinity Map. The portion of the site lying southerly of NE 143rd Place is within the Thayer Creek Subbasin with the remainder in the Coe Clemons Sub-basin. The report identifies the tributary basin areas upstream of the Project site, and evaluates downstream drainage systems. The intent of this section is to demonstrate that the proposed Project will not aggravate existing problems nor create new drainage problems.

TASK 1 - STUDY AREA DEFINITION AND MAPS

The area under consideration is shown on Figure 3.1 – Site and Upstream Basin Map and Figure 3.2 – Downstream Conveyance System. The Study Area extends 1 mile downstream for resource review tasks and ¼ mile for all other tasks.

Rio Vista is located on 4 tax parcels (7325800150, -0160, -0170 & -0180) along NE 143rd Place lying westerly of 272nd Place NE. All four parcels are currently used as single family residences. Existing slopes in the developed portion are 0%-10% and between 10% and 20% on the undeveloped portion. Site soils are mapped as Tokul (255) gravelly sandy loam by the NRCS Web Soil Service as shown on Figure 3.3. The geotechnical evaluations completed in 2008 by Associated Earth Sciences, Inc. found topsoil underlain by Vashon Lodgement till which are classified as Hydrologic Group C. Their reports are attached in the Appendix.

Upstream Basin Description

From available topographic information augmented by visual observations of the upstream area, runoff from the Glencairn subdivision flows across the southeast corner of parcel 0150 and into the wetland adjacent to the parcel. This flow will not affect the property development. Half of 272nd Place fronting the property is contributing a minor amount of runoff to the property. All other areas lying up-gradient from the property are captured in drainage conveyance elements before entering the site.

Due to the negligible upstream area, this site development will not be affected by the existing project or proposed improvements.

TASK 2 - RESOURCE REVIEW

Documents reviewed for existing and/or potential drainage problems, and the findings of each, are listed below:

Adopted Basin Plans

No adopted basin plans exist for the study area.

FEMA Maps

FEMA maps were consulted and the site was not located within a designated flood plain.

Other Off-Site Analyses

The downstream analysis prepared for *Duvall 143* by Pacland dated May 24, 2013 was reviewed as part of this analysis ("Pacland").

Sensitive Areas Folio

Duvall sensitive area mapping augmented with IMAP mapping was utilized to determine if known sensitive areas are within or near the project boundary. The sensitive area maps from the City are included as Figures 3.3 through 3.6. There are no sensitive areas shown on the property; however, the applicant's critical area study (see appendix) indicates there are Class III wetlands within the project site.

Drainage Complaints and Studies

The County was consulted regarding reports of downstream conveyance and capacity issues which may have been reported by adjoining or downstream property owners. Refer to Figure 3.7 – Drainage Complaints. All complaints were more than 10 years old and do not require further investigation.

Road Drainage Problems

None known.

USDA - NRSC Soil Survey Maps

The NRSC Soil Survey Map indicates onsite soils to be of the Tokul series. Please refer to Figure 3.3. Within KCRTS, the soils will be modeled as till type soils.

Wetlands Inventory Maps

There are no inventoried wetlands on or near the site. Refer to Figure 3.6.

Migrating River Studies

There are no studies on or near the site.

Section 303d Listing

None.

King County Designated Water Quality Problems

None.

Adopted Stormwater Compliance Plans

None.

TASK 3 – FIELD INSPECTION

The field inspection was completed May 31, 2013. Weather conditions were 50 degrees and partly cloudy. Refer to the Appendix for the Offsite Analysis Drainage System Table and photographs.

TASK 4 – DRAINAGE SYSTEM DESCRIPTION AND PROBLEM DESCRIPTIONS

Downstream Drainage System Description

There are two separate flow paths for runoff from the project site. Refer to Figure 3.2 – Downstream Conveyance System. Parcel 0150 flows westerly into an offsite wetland. Flows continue westerly towards a culvert crossing NE 143rd Place that discharges into the storm system running westerly in NE 143rd Place. This flow, together with runoff from NE 143rd, continues westerly through a series of ditches, pipes and catchbasins and discharges into Thayer Creek near Main Street, approximately 2,000 feet west. Details of the various components can be found in the Offsite Analysis Drainage System Table. There were no existing problems or deficiencies observed.

The remaining parcels flow into a Class III wetland located in the central portion of the three lots. Flows continue in a northerly direction through various open and closed conveyance elements and ultimately discharge into a City detention facility approximately 1,400 feet to the north. Details of the various components can be found in the Offsite Analysis Drainage System Table. Photographs of the several conveyance components can be found in the Appendix.

Downstream Drainage Problems Requiring Special Attention

Three problem areas are identified for investigation. The definition of each has been excerpted from the 2009 Drainage Manual.

Type 1 problems are:

Conveyance system nuisance problem means a flooding or erosion problem that does not constitute a ***severe flooding problem*** or ***severe erosion problem*** and that results from the overflow of a constructed conveyance system for runoff events less than or equal to a 10-year event. Examples include inundation of a shoulder or lane of a roadway, overflows collecting in yards or pastures, shallow flows across driveways, minor flooding of crawl spaces or unheated garages/outbuildings, and minor erosion.

Type 2 problems are:

Severe erosion problem means there is an open drainage feature with evidence of or potential for erosion/incision sufficient to pose a sedimentation hazard to downstream conveyance systems or pose a landslide hazard by undercutting adjacent slopes. Severe erosion problems do not include roadway shoulder rilling or minor ditch erosion.

Finally, there are 2 elements of Type 3, Severe Flooding Problems:

Severe building flooding problem means there is flooding of the *finished floor area*[6] of a *habitable building*,[7] or the electrical/heating system of a habitable building for runoff events less than or equal to a 100-year event. Examples include flooding of finished floors of homes and commercial or industrial buildings, or flooding of electrical/heating system components in the crawl space or garage of a home.

Severe roadway flooding problem means there is flooding over all lanes of a *roadway*,[8] or a *sole access driveway*[9] is severely impacted, for runoff events less than or equal to the 100-year event. A severely impacted sole access driveway is one in which flooding overtops a culverted section of the driveway, posing a threat of washout or unsafe access conditions due to indiscernible driveway edges, or flooding is deeper than 6 inches on the driveway, posing a severe impediment to emergency access.

The northerly downstream drainage system may contain Type 1 and Type 3 problems. The potential for Type 1 problems seems most likely in the ditch section at N1. Potential Type 1 & 3 problems exist for all road crossings on the northerly flow path.

Impacts from this project are expected to be negligible when compared to the entire Coe Clemons Creek basin. The City's basin map (see Figure 3.8) was used as a guide to approximate tributary areas at the road crossings of NE 144th Place, NE 145th Street and NE Miller Street to provide perspective. The site area is 12.72 acres.

Location	Tributary Area (Acres)	Site Contribution (%)	Estimated Flow* [100 yr/10yr] (cfs)	Estimated Site Contribution [100yr] (cfs)
NE 144th Place/NE 145th Street	61.5	20.7	20.7/10.5	4.3
NE Miller Street	210.3	6.1	70.7/36.0	4.3

*KCRTS calculation, see appendix.

There are 24" diameter culverts crossing both NE 144th Place and NE 145th Street at approximately 5% slope. Assuming full flow conditions (no blockages or head developed at the inlets), they would have a maximum capacity in the neighborhood of 54 cfs; adequate for the 100 year flows. However, blockages and/or other obstructions to flow downstream from the culverts (such as vegetation and other elements related to lack of maintenance) will affect the flow capacity of the culverts to the extent some overtopping of the road may occur.

Combined capacity at Miller Street is estimated to be 60 cfs (2 18" culverts @ 25.4 cfs each and 8.6 cfs for the 12"). The capacity is less than the 100 year flow. In our opinion, the flow reduction is due to downstream ditch conditions, generally attributable to lack of maintenance, and may reduce the flow capacity by half or more at this location. This problem exists with or without this development.

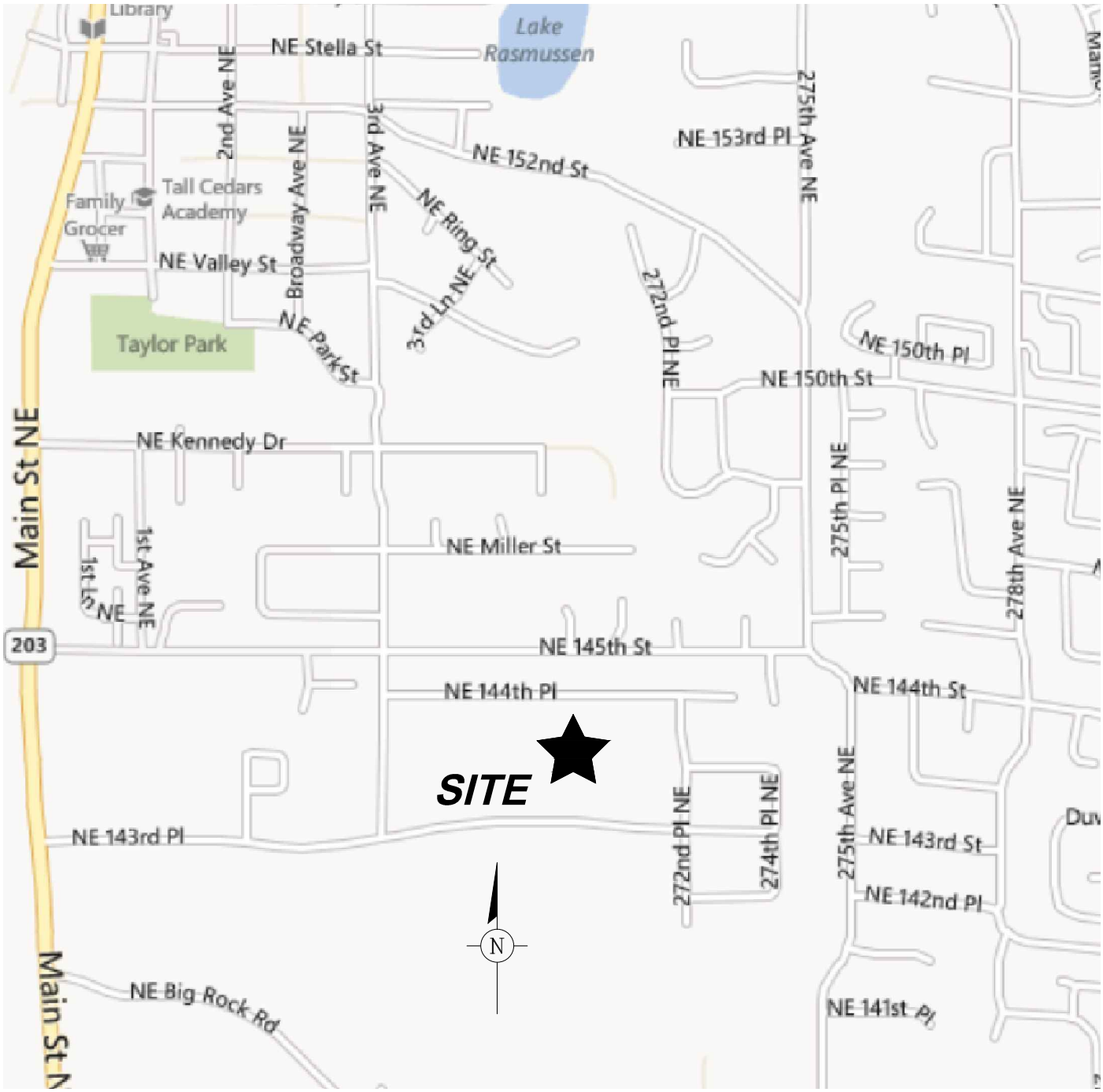
Downstream Water Quality Problems Requiring Special Attention

There are no identified downstream water quality problems.

TASK 5 – MITIGATION OF EXISTING OR POTENTIAL PROBLEMS

Onsite detention will be provided to a Conservation Flow control standard. This standard is warranted so that downstream capacity problems are not exacerbated. The City should budget and perform ditch maintenance, including vegetation removal and dredging, on a regular basis.

Figure 3.0 - Vicinity Map



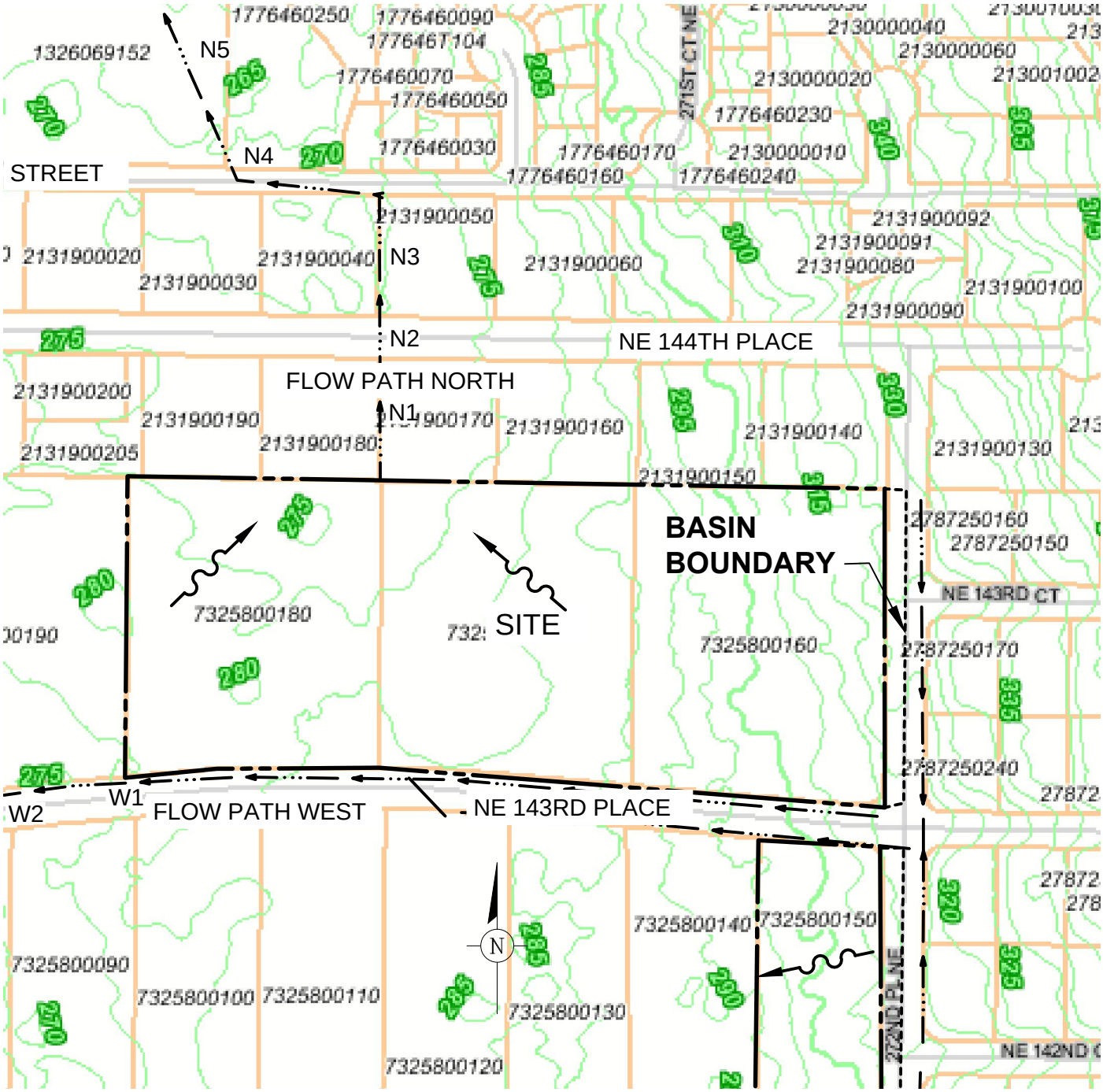
SEC. 24, TWP. 26N, RGE. 6E W.M.

VICINITY MAP
RIO VISTA

LDA Planning, Engineering, Project Management
Land Development Advisors, LLC
12865 SE 47th Place
Bellevue, WA 98006
425-466-5203

DWN. BY:	DATE:	JOB NO.
JWN	7/17/13	REID004
CHKD. BY:	SCALE:	FIG. 3.0
J. NELSON	NO SCALE	

Figure 3.1 – Site and Upstream Basin Map



SITE AND UPSTREAM BASIN MAP
RIO VISTA

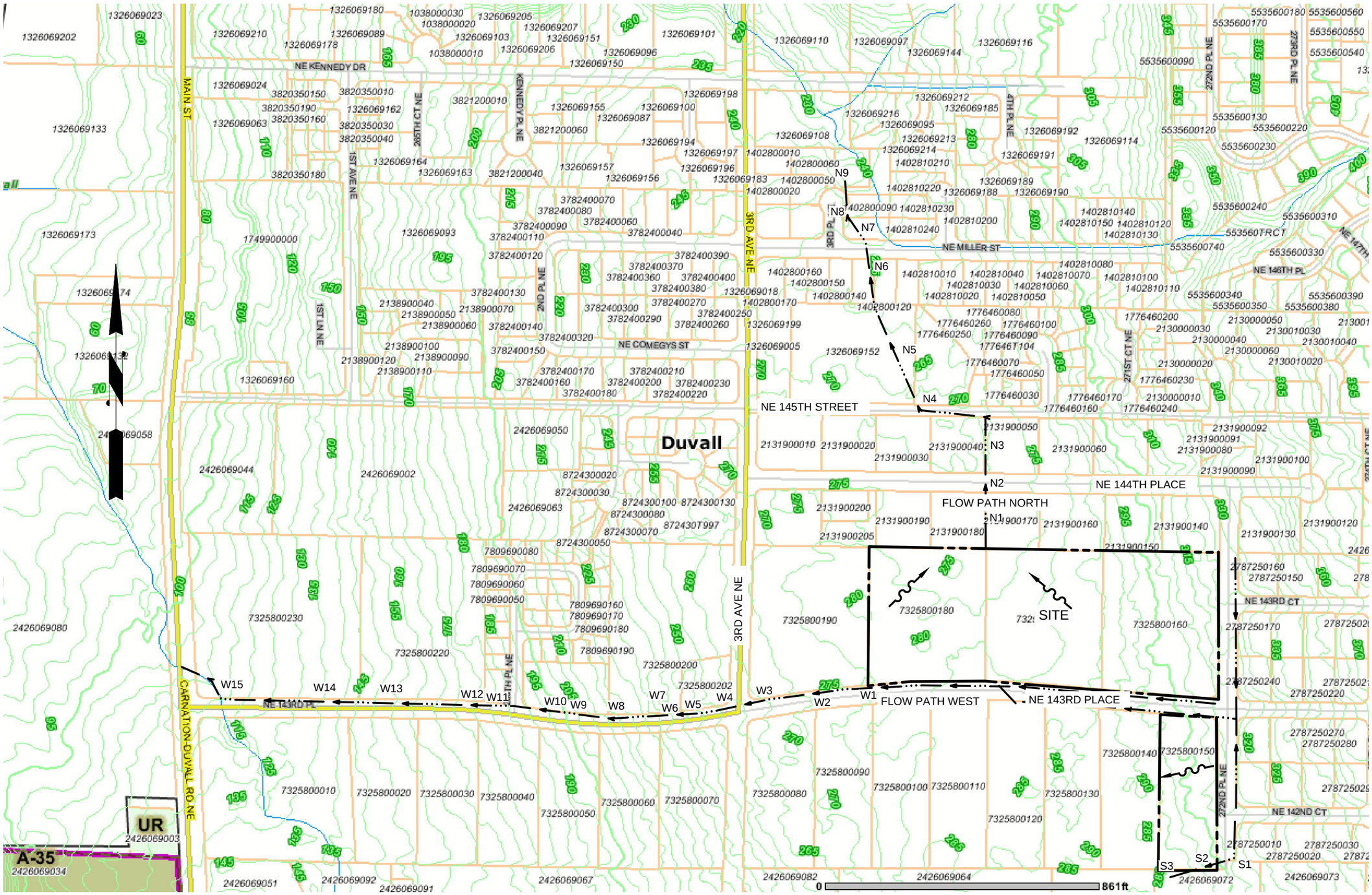
SEC. 24, TWP. 26N, RGE. 6E W.M.

LDA
Planning, Engineering, Project Management
Land Development Advisors, LLC
12865 SE 47th Place
Bellevue, WA 98006
425-466-5203

DWN. BY:	DATE:	JOB NO.
JWN	7/17/13	REID004
CHKD. BY:	SCALE:	FIG. 3.1
J. NELSON	NO SCALE	

Figure 3.2 - Downstream Conveyance System

SEC. 24N, TWP. 26N, RGE. 6E W.M.



W1 REFER TO "OFFSITE ANALYSIS DRAINAGE SYSTEM TABLE"

NO	DATE	REVISION	BY	CK

DESIGN GROUP
JON W. NELSON, P.E.
PROJECT MANAGER:
JWN
DESIGNED:
JWN
CADD:
JWN
CHECKED:
JULY, 2013
DATE:
ALL FIGS
FILE NAME:

DOWNSTREAM CONVEYANCE SYSTEM

RIO VISTA INVESTMENTS, LLC

RIO VISTA

CITY OF DUVALL WASHINGTON

LDA Planning, Engineering, Project Management
Land Development Advisors, LLC
12865 SE 47th Place
Bellevue, WA 98006
425-466-5203

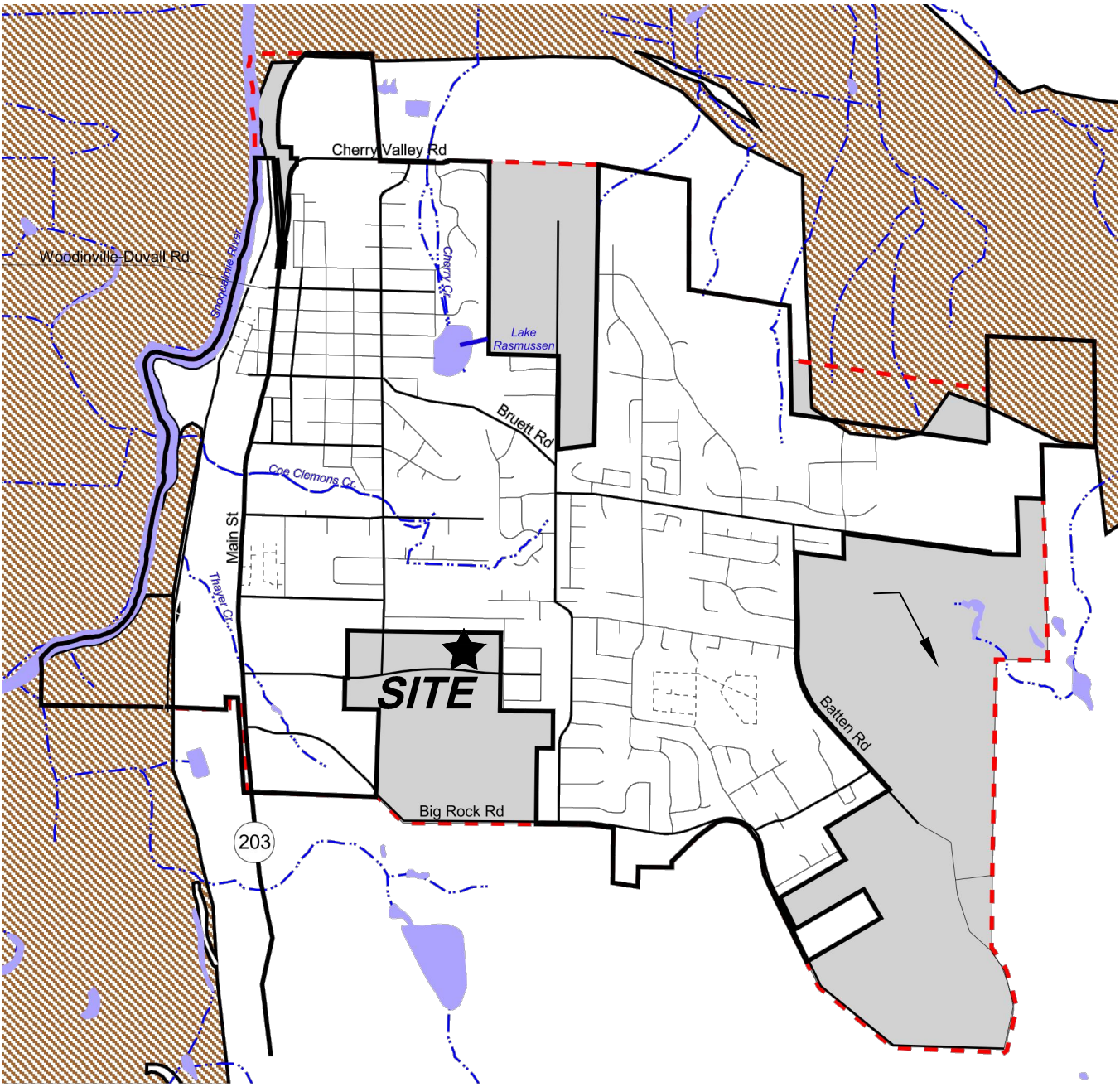
JOB NUMBER REID004

FIGURE 3.2

C:\Users\Jon Nelson\Documents\My Documents\Land Development Advisors\Reid Companies\Rio Vista\Reports\TIR\Figures\ALL FIGS.dwg <<FIG 3.2>> Plotted: 7/17/2013 2:37:11 PM By: Jon Nelson (c) LDA

Figure 3.3 - NRCS Soils Map

Figure 3.4 – Critical Aquifer Recharge Areas (CARA's)



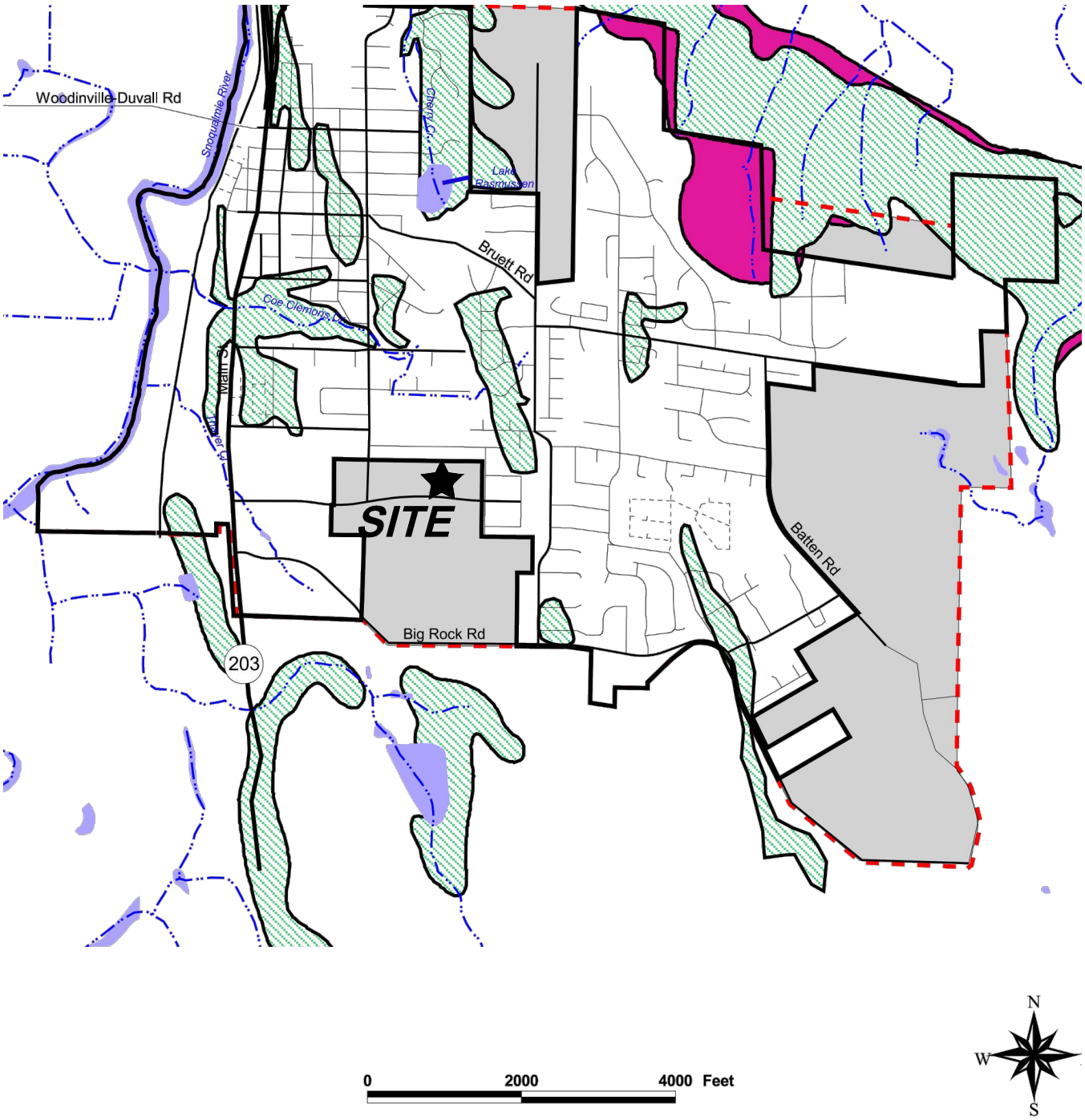
CRITICAL AQUIFER RECHARGE AREAS
RIO VISTA

SEC. 24, TWP. 26N, RGE. 6E W.M.

LDA Planning, Engineering, Project Management
Land Development Advisors, LLC
12865 SE 47th Place
Bellevue, WA 98006
425-466-5203

DWN. BY:	DATE:	JOB NO.
JWN	7/17/13	REID004
CHKD. BY:	SCALE:	FIG. 3.4
J. NELSON	AS SHOWN	

Figure 3.5 – Steep Slopes



LANDSLIDE AND EROSION HAZARD AREAS
RIO VISTA

SEC. 24, TWP. 26N, RGE. 6E W.M.

LDA Planning, Engineering, Project Management
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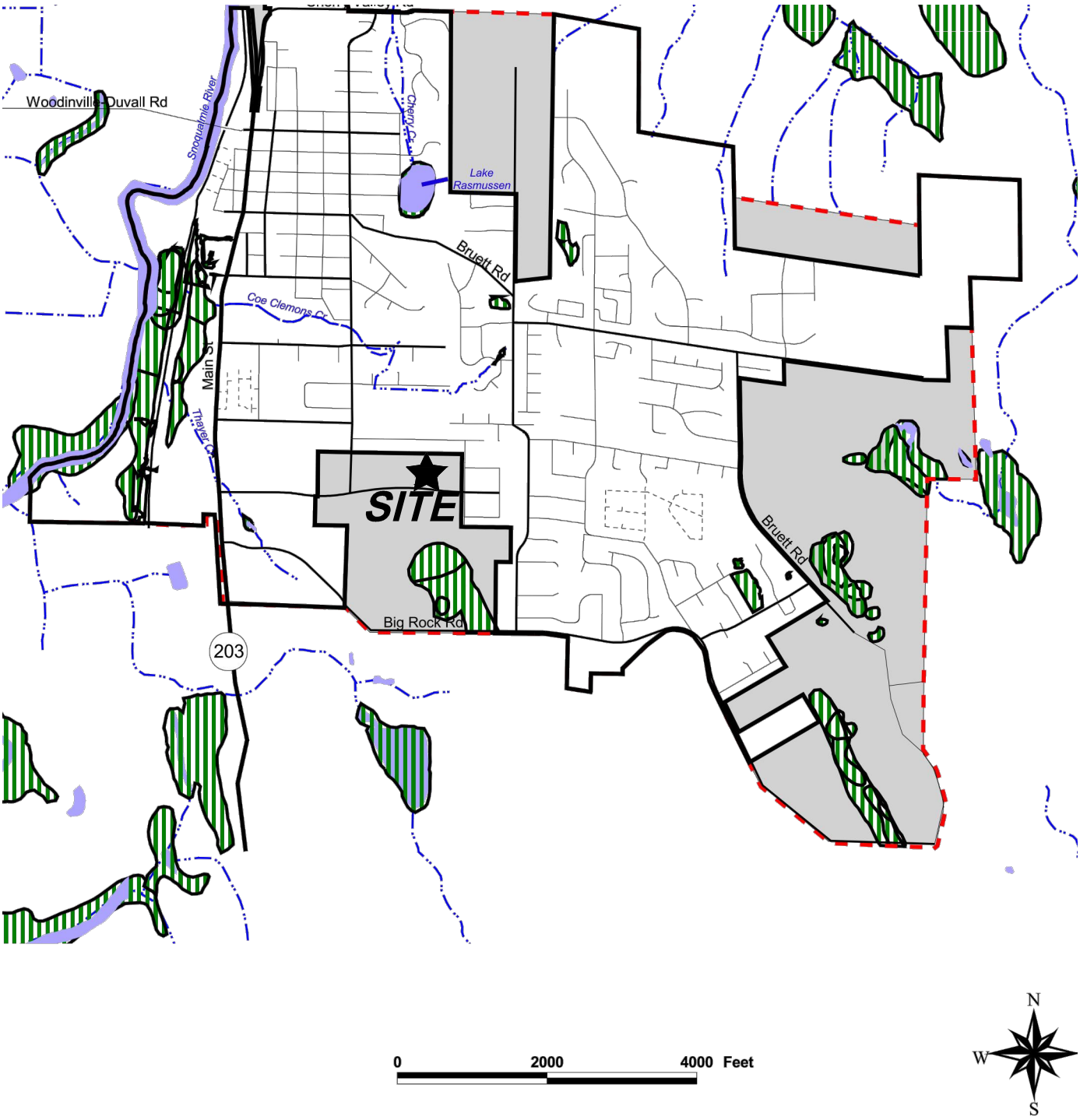
12865 SE 47th Place
Bellevue, WA 98006
425-466-5203

DWN. BY:	DATE:	JOB NO.
JWN	7/17/13	REID004
CHKD. BY:	SCALE:	FIG. 3.5
J. NELSON	AS SHOWN	

Figure 3.6 – Wetlands



SEC. 24, TWP. 26N, RGE. 6E W.M.

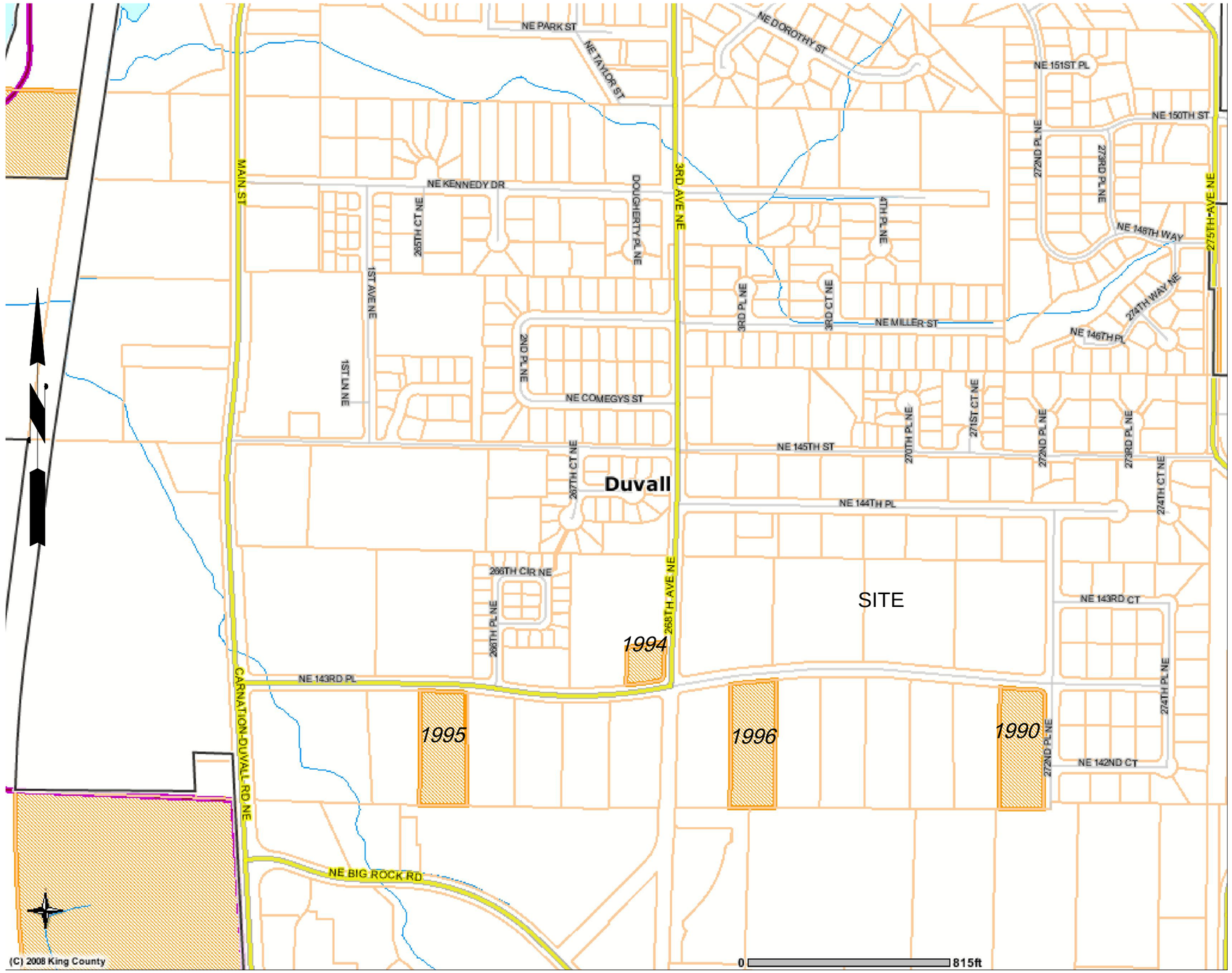
WETLANDS
RIO VISTA

LDA Planning, Engineering, Project Management
Land Development Advisors, LLC
12865 SE 47th Place
Bellevue, WA 98006
425-466-5203

DWN. BY:	DATE:	JOB NO.
JWN	7/17/13	REID004
CHKD. BY:	SCALE:	FIG. 3.6
J. NELSON	AS SHOWN	

Figure 3.7 – Drainage Complaints

SEC. 24N, TWP. 26N, RGE. 6E W.M.



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△				

DESIGN GROUP
JON W. NELSON, P.E.
PROJECT MANAGER:
JWN
DESIGNED:
JWN
CADD:
JWN
CHECKED:
JULY, 2013
DATE:
ALL FIGS
FILE NAME:

DRAINAGE COMPLAINTS

RIO VISTA INVESTMENTS, LLC

RIO VISTA

CITY OF DUVALL

WASHINGTON

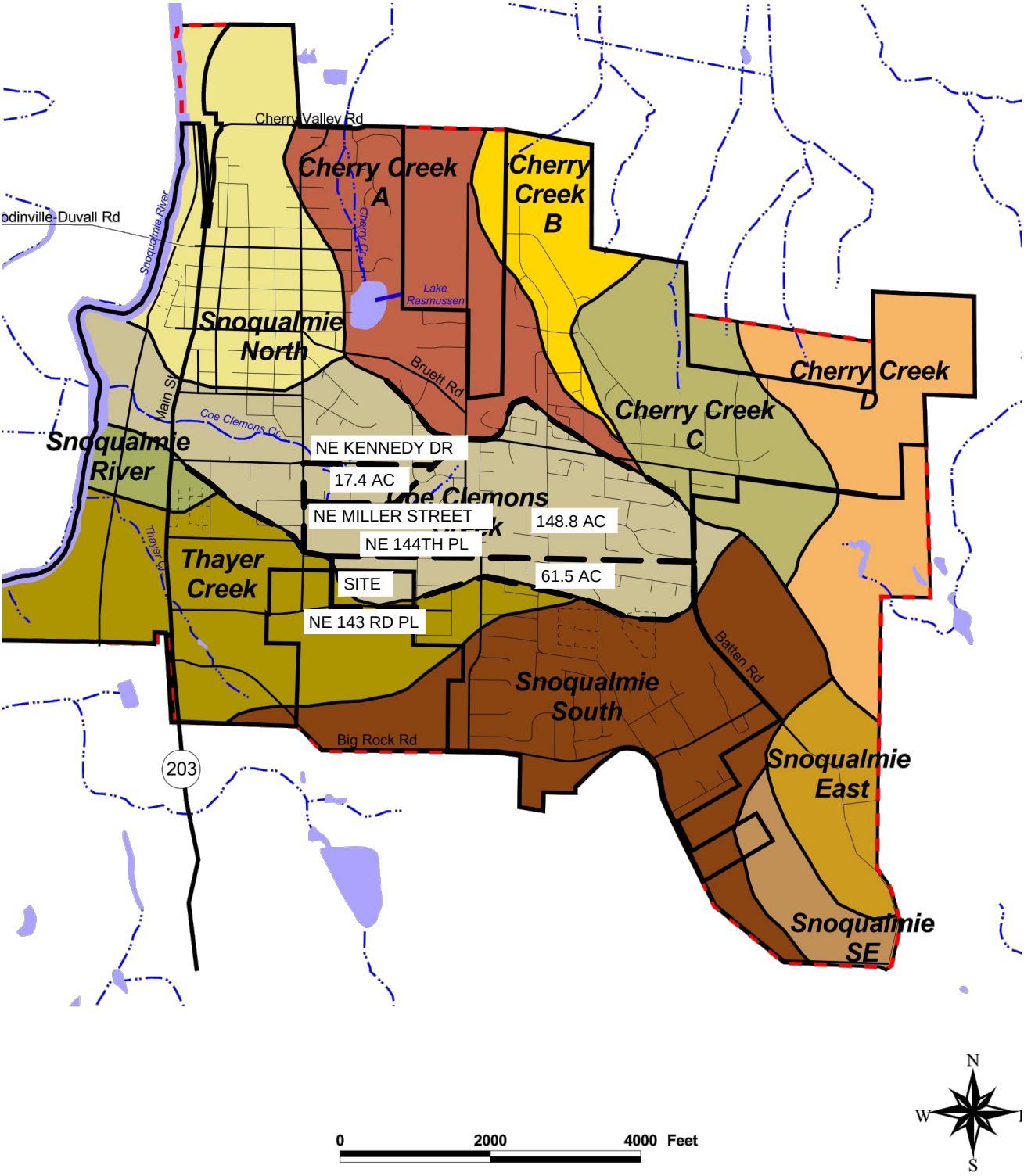
LDA Land Development Advisors, LLC
Planning, Engineering, Project Management
12865 SE 47th Place
Bellevue, WA 98006
425-466-5203

JOB NUMBER REID004

FIGURE 3.7

C:\Users\Jon Nelson\Documents\My Documents\Land Development Advisors\Reid Companies\Rio Vista\Reports\TIR\Figures\ALL FIGS.dwg <<FIG 3.7>> Plotted: 7/17/2013 2:39:17 PM By: Jon Nelson (c) LDA

Figure 3.8 – City Basin Map



CITY BASIN MAP
RIO VISTA

SEC. 24, TWP. 26N, RGE. 6E W.M.

LDA Planning, Engineering, Project Management
Land Development Advisors, LLC
12865 SE 47th Place
Bellevue, WA 98006
425-466-5203

DWN. BY:	DATE:	JOB NO.
JWN	7/17/13	REID004
CHKD. BY:	SCALE:	FIG. 3.8
J. NELSON	AS SHOWN	

SECTION 4
FLOW CONTROL/WATER QUALITY ANALYSIS AND DESIGN

The project is subject to the 2009 King County Surface Water Design Manual ("KCSWDM") including Conservation Flow Control ("Level 2") and Basic water quality requirements. Level 2 flow control includes matching durations from 50% of the 2-year to the 50-year peak. Additionally, the 2-year and 10-year developed peak flows must be below the 2-year and 10-year existing peak flows. Existing onsite runoff will be computed assuming the site is "forested" and till soils (refer to the geotechnical reports in the appendix). Site is within the Landsburg rainfall region with a 0.85 scale factor. Refer to Figure 4.2 for the map.

SITE HYDROLOGY

Part A - Existing Site Hydrology

The site area used to establish the existing conditions will not include the NGPE areas since they will not be modified or directed to the detention system with this project. If they are modified in the future, detention and water quality will need to be provided for that area.

Offsite areas are modeled as 100% impervious in the existing and developed conditions.

Table 4.1. Existing Basin Summaries

Total Basin Area - NE+SE	8.31 Acres (“ac”)
Total Basin Area - W	2.68 ac

Existing Flow Frequencies

NE ("targne.tsf") + SE ("targse.tsf") Basins

```
          Flow Frequency Analysis
  Time Series File:targne.tsf
  Project Location:Landsburg

  ---Annual Peak Flow Rates---      -----Flow Frequency Analysis-------
 Flow Rate  Rank  Time of Peak       - - Peaks - -  Rank  Return  Prob
   (CFS)                                (CFS)             Period
   0.586      2   2/09/01 15:00        0.841         1  100.00   0.990
   0.168      8   1/05/02 16:00        0.586         2   25.00   0.960
   0.410      4   2/28/03 16:00        0.520         3   10.00   0.900
   0.224      7   8/26/04  1:00        0.410         4    5.00   0.800
   0.379      5   1/05/05 10:00        0.379         5    3.00   0.667
   0.306      6   1/18/06 16:00        0.306         6    2.00   0.500
   0.520      3  11/24/06  5:00        0.224         7    1.30   0.231
   0.841      1   1/09/08  7:00        0.168         8    1.10   0.091
Computed Peaks                         0.756             50.00   0.980
```

```
          Flow Frequency Analysis
  Time Series File:targse.tsf
  Project Location:Landsburg

  ---Annual Peak Flow Rates---      -----Flow Frequency Analysis-------
 Flow Rate  Rank  Time of Peak       - - Peaks - -  Rank  Return  Prob
   (CFS)                                (CFS)              Period
   0.169      2   2/09/01 15:00        0.254         1  100.00   0.990
   0.054      8   1/05/02 16:00        0.169         2   25.00   0.960
   0.121      4   2/28/03 16:00        0.153         3   10.00   0.900
   0.083      7   8/26/04  1:00        0.121         4    5.00   0.800
   0.112      5   1/05/05 10:00        0.112         5    3.00   0.667
   0.092      6   1/18/06 16:00        0.092         6    2.00   0.500
   0.153      3  11/24/06  5:00        0.083         7    1.30   0.231
   0.254      1   1/09/08  7:00        0.054         8    1.10   0.091
Computed Peaks                         0.225             50.00   0.980
```

W Basin

```
          Flow Frequency Analysis
  Time Series File:targw.tsf
  Project Location:Landsburg

  ---Annual Peak Flow Rates---      -----Flow Frequency Analysis-------
 Flow Rate  Rank  Time of Peak       - - Peaks - -  Rank  Return  Prob
   (CFS)                                (CFS)              Period
   0.210      2   2/09/01 18:00        0.248         1  100.00   0.990
   0.034      7   1/05/02 16:00        0.210         2   25.00   0.960
   0.133      4   2/28/03 16:00        0.171         3   10.00   0.900
   0.024      8   3/03/04  3:00        0.133         4    5.00   0.800
   0.122      5   1/05/05 10:00        0.122         5    3.00   0.667
   0.099      6   1/18/06 21:00        0.099         6    2.00   0.500
   0.171      3  11/24/06  5:00        0.034         7    1.30   0.231
   0.248      1   1/09/08  7:00        0.024         8    1.10   0.091
Computed Peaks                         0.235             50.00   0.980
```

Part B - Developed Site Hydrology

The vaults will be designed to the extent that the feasibility of the vault together with the approximate nature and extent of the detention/water quality system can be identified. The designs will be fine-tuned and construction details added during preparation of construction documents. Developed basin areas and land covers are summarized below. The on-lot impervious surface area is 75% and 50% was used for open space areas. Pervious areas are modeled as grass. Refer to Figure 4.1 and Table 4.3 – Basin Areas.

A portion of the site will bypass the detention system. The area bypassed is in the west basin and will be 0.15 ac., all impervious.

Section 1.2.3.E (Page 1-47) of the KCSWDM provides guidance to deal with onsite bypass areas. We will address items 1 through 5 from section E below:

E. MITIGATION OF TARGET SURFACES THAT BYPASS FACILITY

*On some **sites**, topography may make it difficult or costly to collect all target surface runoff for discharge to the onsite flow control facility. Therefore, some project runoff subject to flow control*

may bypass required onsite flow control facilities provided that all of the following conditions are met:

*1. The **point of convergence** for runoff discharged from the bypassed target surfaces and from the project's flow control facility must be within **a quarter-mile downstream** of the facility's **project site** discharge point, AND*

The flow from the bypassed area does not diverge from the drainage course.

*2. The increase in the **existing site conditions 100-year peak discharge** from the area of bypassed target surfaces must not exceed 0.4 cfs, AND*

The increase in the 100 year developed flow from the bypassed area is 0.1 cfs. Refer to the output in the Appendix under file names "WBYPEX" and "WBYPDEV". The area was modeled as 50% impervious in the existing condition and 100% impervious in the developed condition. The existing 100 year flow is 0.056 cfs and the developed flow is 0.069 cfs, an increase of 0.013 cfs.

*3. Runoff from the bypassed target surfaces **must not create a significant adverse impact** to downstream drainage systems, salmonid habitat, or properties as determined by DDES, AND*

The small size of the area and miniscule runoff will not create an adverse impact.

*4. **Water quality requirements** applicable to the bypassed target surfaces must be met, AND*

New PGIS in the bypassed area is less than 5,000 square feet; therefore, water quality is not required.

*5. **Compensatory mitigation by a flow control facility** must be provided so that the net effect at the point of convergence downstream is the same with or without the bypass. This mitigation may be waived if the **existing site conditions** 100-year peak discharge from the area of bypassed target surfaces is increased by no more than 0.1 cfs and **flow control BMPs** as detailed in Appendix C are applied to all impervious surfaces within the area of bypassed target surfaces. One or combination of the following methods may be used to provide compensatory mitigation by a flow control facility subject to permission/approvals from other parties as deemed necessary by DDES:*

a) Design the project's flow control facility or retrofit an existing offsite flow control facility as needed to achieve the desired effect at the point of convergence, OR

b) Design the project's flow control facility or provide/retrofit an offsite flow control facility to mitigate an existing developed area (either onsite or offsite) that has runoff characteristics (i.e., peak flow and volume) equivalent to those of the bypassed target surfaces but is currently not mitigated or required to be mitigated to the same flow control performance requirement as the bypassed target surfaces.

The detention facility for the NE basin will compensate for the bypassed area.

Table 4.2. Developed/Detained Basin Summaries

NE+SE Basin	8.31 Acres("ac")
Impervious Area	6.59 ac
Pervious Area	1.72 ac

Bypassed Impervious Area	0.15 ac
W Basin	2.68 ac
Impervious Area	2.08 ac
Pervious Area	0.60 ac

Developed Basin Flow Frequencies

NE ("devne.tsf") + SE ("devse.tsf") Basins

```
         Flow Frequency Analysis
  Time Series File:devne.tsf
  Project Location:Landsburg

  ---Annual Peak Flow Rates---      -----Flow Frequency Analysis-------
 Flow Rate  Rank  Time of Peak       - - Peaks - -  Rank  Return  Prob
   (CFS)                                (CFS)              Period
    1.61      7   2/09/01  2:00        2.81          1  100.00   0.990
    1.06      8  12/03/01 17:00        2.03          2   25.00   0.960
    1.85      4   9/10/03 15:00        1.99          3   10.00   0.900
    2.03      2   8/26/04  1:00        1.85          4    5.00   0.800
    1.69      6  10/28/04 18:00        1.76          5    3.00   0.667
    1.76      5  10/22/05 17:00        1.69          6    2.00   0.500
    1.99      3  10/26/06  3:00        1.61          7    1.30   0.231
    2.81      1   1/09/08  7:00        1.06          8    1.10   0.091
Computed Peaks                          2.55              50.00   0.980

          Flow Frequency Analysis
  Time Series File:devse.tsf
  Project Location:Landsburg

  ---Annual Peak Flow Rates---      -----Flow Frequency Analysis-------
 Flow Rate  Rank  Time of Peak       - - Peaks - -  Rank  Return  Prob
   (CFS)                                (CFS)              Period
   0.399      7   2/09/01  2:00       0.702          1  100.00   0.990
   0.258      8  12/03/01 17:00       0.496          2   25.00   0.960
   0.450      4   9/10/03 15:00       0.486          3   10.00   0.900
   0.496      2   8/26/04  1:00       0.450          4    5.00   0.800
   0.412      6  10/28/04 18:00       0.429          5    3.00   0.667
   0.429      5  10/22/05 17:00       0.412          6    2.00   0.500
   0.486      3  10/26/06  3:00       0.399          7    1.30   0.231
   0.702      1   1/09/08  7:00       0.258          8    1.10   0.091
Computed Peaks                         0.633              50.00   0.980
```

West Basin

```
          Flow Frequency Analysis
  Time Series File:w dev.tsf
  Project Location:Landsburg

  ---Annual Peak Flow Rates---      -----Flow Frequency Analysis-------
 Flow Rate  Rank  Time of Peak       - - Peaks - -  Rank  Return  Prob
   (CFS)                                (CFS)              Period
   0.756      7   2/09/01  2:00        1.32          1  100.00   0.990
   0.491      8  12/03/01 17:00       0.943          2   25.00   0.960
   0.858      4   9/10/03 15:00       0.924          3   10.00   0.900
   0.943      2   8/26/04  1:00       0.858          4    5.00   0.800
   0.784      6  10/28/04 18:00       0.816          5    3.00   0.667
   0.816      5  10/22/05 17:00       0.784          6    2.00   0.500
   0.924      3  10/26/06  3:00       0.756          7    1.30   0.231
```

```
    1.32      1   1/09/08  7:00         0.491         8    1.10   0.091
Computed Peaks                           1.20             50.00   0.980
```

Part C – Performance Standards

The 2009 King County Surface Water Design Manual (KCSWDM) applies to this project as the regulatory guide for the analysis and subsequent design of storm water drainage systems (i.e. detention facilities, pipe networks, etc.). The detention facility has been modeled using the King County Runoff Time Series (KCRTS) method, using the Landsburg rainfall station data with a 0.85 scale factor. Please see the full KCRTS printouts in the Appendix.

The flow control applied to the site includes matching durations from 50% of the 2-year to the 50-year peak. Additionally, the 2-year and 10-year developed peak flows must be below the 2-year and 10-year existing peak flows.

Basic water quality requirements are required for this project. Water quality treatment will be achieved by the use of dead storage in the vaults.

Part D – Flow Control System

Detention Vault Design Calculations

NE+SE Basins

One vault will be used with two outlets, one discharging to the edge of the buffer in the northeast basin and one discharging to the storm system along NE 143rd Place in the southeast basin. In designing the stormwater vault, target duration curves were developed for each discharge location and were matched under developed conditions. Both basins have offsite tributary areas of impervious that will be collected and conveyed to the stormwater vault under developed conditions. Drainage from these offsite non-project areas were treated as “flow-through” in the vault design. This was accomplished by modeling runoff from these tributary areas as impervious in both the target and developed time series. In addition, the southeast basin includes 0.15 acre of impervious in the vault design.

The configuration of the outlet structure for each discharge location was done by trial-and-error. The vault was modeled as a two-outlet reservoir in KCRTS with the vault volume and control structures orifice being adjusted until the flow duration curves at each outlet matched the target durations.

The detention portion of the combined stormwater facility is required to have a storage volume of 119,680 cubic feet ("cf"). The actual volume is 120,064 cf based on a footprint of 15,006 sf and a maximum depth of 8'. Detailed calculations are provided in the Appendix. Vault details will be included in the final plans.

Dimensions:

Bot Area	14960	SF	**Revised to 15,008 SF**
Length	170	FT	**Revised to 268'**
Width	88	FT	**Revised to 56'**
Depth	8	FT	

Outlet Structures:

NE Outlet Structure

Orifice	Dia. (IN)	Height (FT)	Area (SF)
O1	1.69	-	0.015577596
O2	0.88	4.40	0.004223693
O3	1.70	4.80	0.015762491
Riser	18.00	8.00	

SE Outlet Structure

Orifice	Dia. (IN)	Height (FT)	Area (SF)
O1	0.87	-	0.004128
O2	1.50	5.80	0.012272
O3	-	-	0
Riser	12.00	8.00	

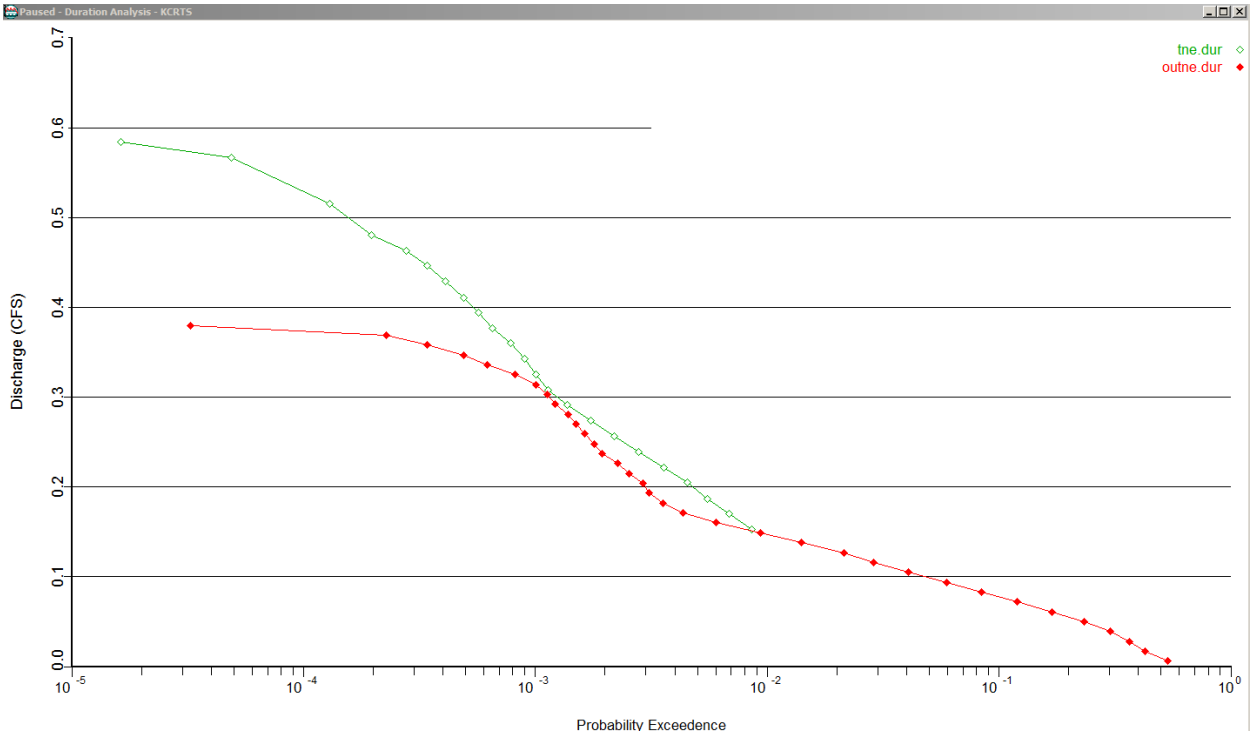
```
          Flow Frequency Analysis
  Time Series File:outne.tsf
  Project Location:Landsburg

  ---Annual Peak Flow Rates---      -----Flow Frequency Analysis-------
 Flow Rate  Rank  Time of Peak       - - Peaks - -  Rank  Return  Prob
   (CFS)                             (CFS)    (ft)        Period
   0.380      3   2/09/01 20:00      2.18    8.05    1  100.00   0.990
   0.113      8  11/05/01  3:00     0.381    7.56    2   25.00   0.960
   0.236      5   3/01/03  7:00     0.380    7.56    3   10.00   0.900 10 YR
DEVELOPED OK
   0.136      7   8/26/04  6:00     0.259    5.36    4    5.00   0.800
   0.259      4   1/08/05  6:00     0.236    5.11    5    3.00   0.667
   0.152      6   1/19/06  1:00     0.152    3.85    6    2.00   0.500 2 YR DEVELOPED
OK
   0.381      2  11/24/06  8:00     0.136    3.09    7    1.30   0.231
    2.18      1   1/09/08  8:00     0.113    2.10    8    1.10   0.091
Computed Peaks                       1.58    8.04        50.00   0.980
```

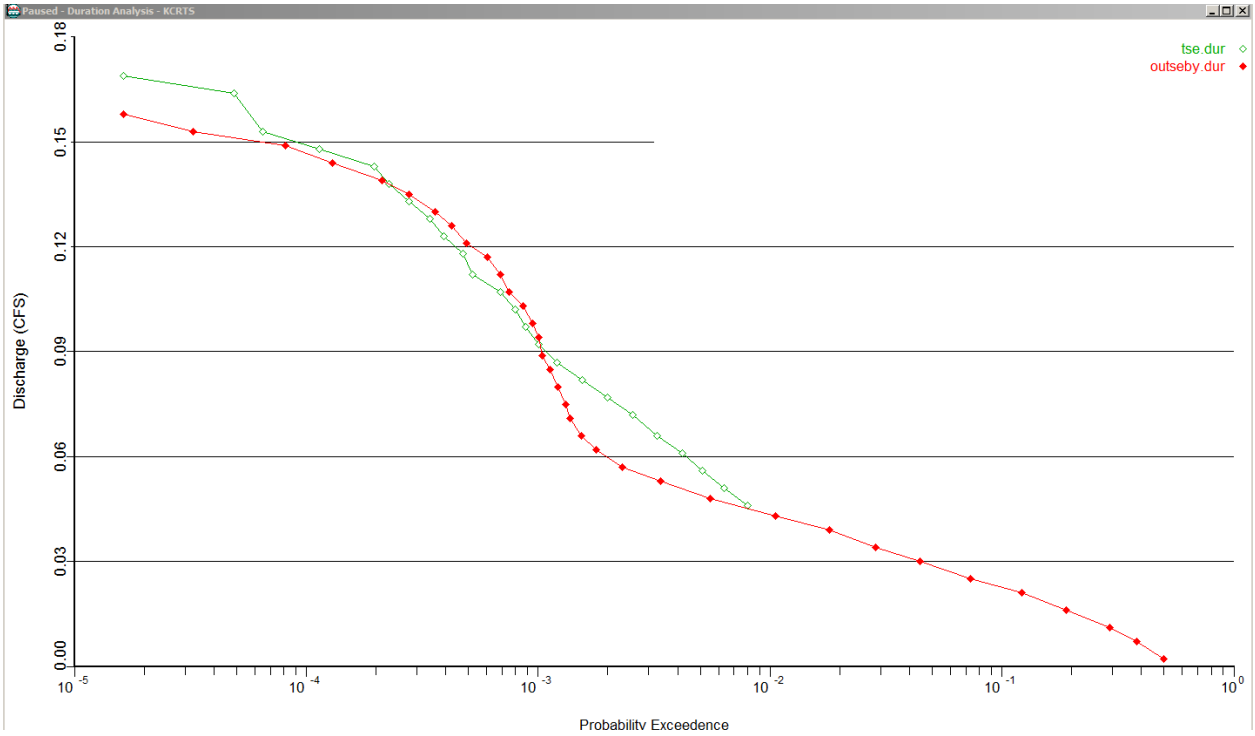
```
          Flow Frequency Analysis
  Time Series File:outse.tsf
  Project Location:Landsburg

  ---Annual Peak Flow Rates---      -----Flow Frequency Analysis-------
 Flow Rate  Rank  Time of Peak       - - Peaks - -  Rank  Return  Prob
   (CFS)                             (CFS)    (ft)         Period
   0.138      3   2/09/01 20:00      1.34    8.05    1  100.00   0.990
   0.030      8  11/05/01  3:00     0.138    7.57    2   25.00   0.960
   0.047      5   3/01/03  7:00     0.138    7.56    3   10.00   0.900 10 YR
DEVELOPED OK
   0.036      7   8/26/04  6:00     0.048    5.36    4    5.00   0.800
   0.048      4   1/08/05  6:00     0.047    5.11    5    3.00   0.667
   0.040      6   1/19/06  1:00     0.040    3.85    6    2.00   0.500 2 YR DEVELOPED
OK
   0.138      2  11/24/06  8:00     0.036    3.09    7    1.30   0.231
    1.34      1   1/09/08  8:00     0.030    2.10    8    1.10   0.091
Computed Peaks                      0.942    8.04        50.00   0.980
```

NE Discharge – at Facility Outlet



SE Discharge – at Downstream Point-of-Compliance



W Basin

The detention portion of the combined stormwater facility is required to have a storage volume of 42,720 cf. The actual volume is 43,520 cf based on a footprint of 10,880 sf and a maximum depth of 4'. Detailed calculations are provided in the Appendix. Vault details will be included in the final plans.

The vault was modeled as summarized below:

```
Retention/Detention Facility

         Type of Facility: Detention Vault
         Facility Length:      120.00  ft   Revised to 116'
          Facility Width:       89.00  ft   Revised to 80' & added 40'x40' area to achieve
required surface area
           Facility Area:    10680.    sq. ft
 Effective Storage Depth:        4.00  ft
       Stage 0 Elevation:        0.00  ft
          Storage Volume:    42720.    cu. ft
              Riser Head:        4.00  ft
          Riser Diameter:       12.00  inches
      Number of orifices:        2
                                     Full Head    Pipe
    Orifice #      Height  Diameter  Discharge  Diameter
                    (ft)     (in)      (CFS)      (in)
       1            0.00      1.07        0.062
       2            2.20      1.40        0.071    4.0
          Top Notch Weir: None
    Outflow Rating Curve: None
```

```
          Flow Frequency Analysis
  Time Series File:rdout.tsf
  Project Location:Landsburg

  ---Annual Peak Flow Rates---      -----Flow Frequency Analysis-------
 Flow Rate  Rank  Time of Peak       - - Peaks - -  Rank  Return  Prob
   (CFS)                             (CFS)    (ft)         Period
   0.132      3   2/09/01 21:00     0.825    4.17    1  100.00   0.990
   0.033      8   1/06/02 22:00     0.214    4.03    2   25.00   0.960
   0.088      5   3/06/03 23:00     0.132    3.94    3   10.00   0.900 10 YR DEVELOPED
OK
   0.039      7   8/26/04  7:00     0.089    2.72    4    5.00   0.800
   0.089      4   1/08/05  6:00     0.088    2.70    5    3.00   0.667
   0.053      6  10/28/05  2:00     0.053    2.26    6    2.00   0.500 2 YR DEVELOPED
OK
   0.214      2  11/24/06  8:00     0.039    1.64    7    1.30   0.231
   0.825      1   1/09/08  9:00     0.033    1.14    8    1.10   0.091
Computed Peaks                      0.622    4.13        50.00   0.980
```

Duration Performance

The following tables compare the duration of outflow from the proposed detention facility to the existing flow duration. The following conditions must be met:

- *First cutoff must have a percentage change (last column) equal to or less than 0.0.*
- *The remaining cutoffs must have a value less than positive 10.0 percent.*
- *At least on-half of the cutoffs must have zero or negative percent change.*
- *The maximum positive excursion must be less than positive 10.0 percent.*

NE Discharge Location - Duration Comparison Analysis

```
   Base File: targne.tsf
    New File: outne.tsf
Cutoff Units: Discharge in CFS

            -----Fraction of Time----- ---------Check of Tolerance-------
  Cutoff    Base      New      %Change Probability   Base     New %Change
   0.153 |  0.86E-02  0.79E-02   -8.5 |  0.86E-02   0.153   0.151    -1.6
   0.186 |  0.57E-02  0.33E-02  -41.5 |  0.57E-02   0.186   0.160   -14.0
   0.220 |  0.37E-02  0.24E-02  -34.8 |  0.37E-02   0.220   0.180   -17.9
   0.253 |  0.23E-02  0.17E-02  -24.1 |  0.23E-02   0.253   0.223   -11.7
   0.286 |  0.15E-02  0.13E-02  -15.8 |  0.15E-02   0.286   0.264    -7.8
   0.320 |  0.11E-02  0.91E-03  -13.8 |  0.11E-02   0.320   0.309    -3.3
   0.353 |  0.83E-03  0.41E-03  -51.0 |  0.83E-03   0.353   0.324    -8.1
   0.386 |  0.62E-03  0.00E+00 -100.0 |  0.62E-03   0.386   0.335   -13.2
   0.419 |  0.44E-03  0.00E+00 -100.0 |  0.44E-03   0.419   0.350   -16.5
   0.453 |  0.31E-03  0.00E+00 -100.0 |  0.31E-03   0.453   0.362   -20.1
   0.486 |  0.16E-03  0.00E+00 -100.0 |  0.16E-03   0.486   0.374   -23.1
   0.519 |  0.98E-04  0.00E+00 -100.0 |  0.98E-04   0.519   0.377   -27.4
   0.553 |  0.49E-04  0.00E+00 -100.0 |  0.49E-04   0.553   0.379   -31.4

There is no positive excursion

Maximum negative excursion = 0.205 cfs (-35.0%)
occurring at 0.585 cfs on the Base Data:targne.tsf
and at 0.380 cfs on the  New Data:outne.tsf
```

SE Discharge Location - Duration Comparison Analysis

```
   Base File: targse.tsf
    New File: outseby.tsf
Cutoff Units: Discharge in CFS
```

Cutoff	Fraction of Time: Base	New	%Change	Check of Tolerance: Probability	Base	New	%Change
0.046	0.80E-02	0.78E-02	-2.4	0.80E-02	0.046	0.045	-0.8
0.055	0.53E-02	0.27E-02	-50.0	0.53E-02	0.055	0.049	-12.0
0.065	0.35E-02	0.16E-02	-54.9	0.35E-02	0.065	0.052	-19.0
0.074	0.23E-02	0.13E-02	-41.7	0.23E-02	0.074	0.058	-22.5
0.084	0.14E-02	0.12E-02	-15.5	0.14E-02	0.084	0.071	-15.2
0.093	0.98E-03	0.10E-02	5.0	0.98E-03	0.093	0.097	4.4
0.103	0.78E-03	0.86E-03	10.4	0.78E-03	0.103	0.107	4.0
0.112	0.54E-03	0.68E-03	27.3	0.54E-03	0.112	0.119	6.1
0.122	0.41E-03	0.49E-03	20.0	0.41E-03	0.122	0.127	4.1
0.131	0.28E-03	0.34E-03	23.5	0.28E-03	0.131	0.135	2.9
0.140	0.20E-03	0.20E-03	0.0	0.20E-03	0.140	0.141	0.6
0.150	0.98E-04	0.82E-04	-16.7	0.98E-04	0.150	0.146	-2.4
0.159	0.49E-04	0.00E+00	-100.0	0.49E-04	0.159	0.153	-4.1
0.169	0.16E-04	0.00E+00	-100.0	0.16E-04	0.169	0.158	-6.1

```
Maximum positive excursion = 0.007 cfs (  6.3%)
occurring at 0.112 cfs on the Base Data:targse.tsf
and at 0.119 cfs on the  New Data:outseby.tsf

Maximum negative excursion = 0.017 cfs (-23.2%)
occurring at 0.073 cfs on the Base Data:targse.tsf
and at 0.056 cfs on the  New Data:outseby.tsf
```

West Basin - Duration Comparison Analysis

```
Duration Comparison Analysis
   Base File: targw.tsf
    New File: rdout.tsf
Cutoff Units: Discharge in CFS
```

Cutoff	Fraction of Time: Base	New	%Change	Check of Tolerance: Probability	Base	New	%Change
0.050	0.10E-01	0.78E-02	-22.0	0.10E-01	0.050	0.045	-9.1
0.062	0.68E-02	0.66E-02	-2.9	0.68E-02	0.062	0.060	-3.1
0.074	0.46E-02	0.48E-02	3.5	0.46E-02	0.074	0.075	1.2
0.087	0.30E-02	0.25E-02	-17.3	0.30E-02	0.087	0.082	-5.3
0.099	0.20E-02	0.19E-02	-5.7	0.20E-02	0.099	0.097	-1.9
0.111	0.15E-02	0.13E-02	-13.2	0.15E-02	0.111	0.107	-3.4
0.124	0.10E-02	0.54E-03	-47.6	0.10E-02	0.124	0.117	-5.0
0.136	0.75E-03	0.49E-04	-93.5	0.75E-03	0.136	0.121	-10.7
0.148	0.57E-03	0.49E-04	-91.4	0.57E-03	0.148	0.123	-16.8
0.160	0.33E-03	0.33E-04	-90.0	0.33E-03	0.160	0.128	-20.3
0.173	0.20E-03	0.33E-04	-83.3	0.20E-03	0.173	0.130	-24.4
0.185	0.15E-03	0.16E-04	-88.9	0.15E-03	0.185	0.131	-29.0
0.197	0.98E-04	0.16E-04	-83.3	0.98E-04	0.197	0.132	-33.3
0.209	0.16E-04	0.16E-04	0.0	0.16E-04	0.209	0.214	2.4

```
Maximum positive excursion = 0.008 cfs (  4.1%)
occurring at 0.206 cfs on the Base Data:targw.tsf
and at 0.214 cfs on the  New Data:rdout.tsf

Maximum negative excursion = 0.069 cfs (-34.4%)
occurring at 0.202 cfs on the Base Data:targw.tsf
and at 0.132 cfs on the  New Data:rdout.tsf
```

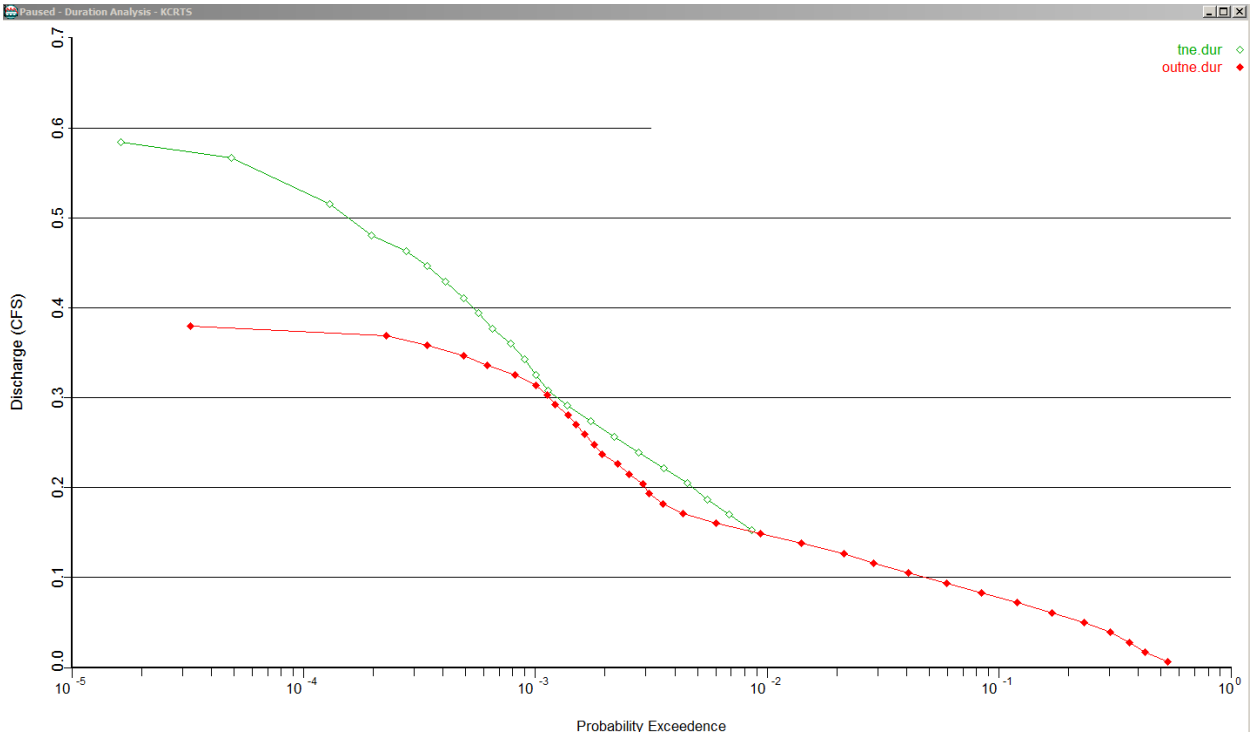
```
----------------------------------
Route Time Series through Facility
  Inflow Time Series File:devw.tsf
 Outflow Time Series File:rdout
```

Performance Graph

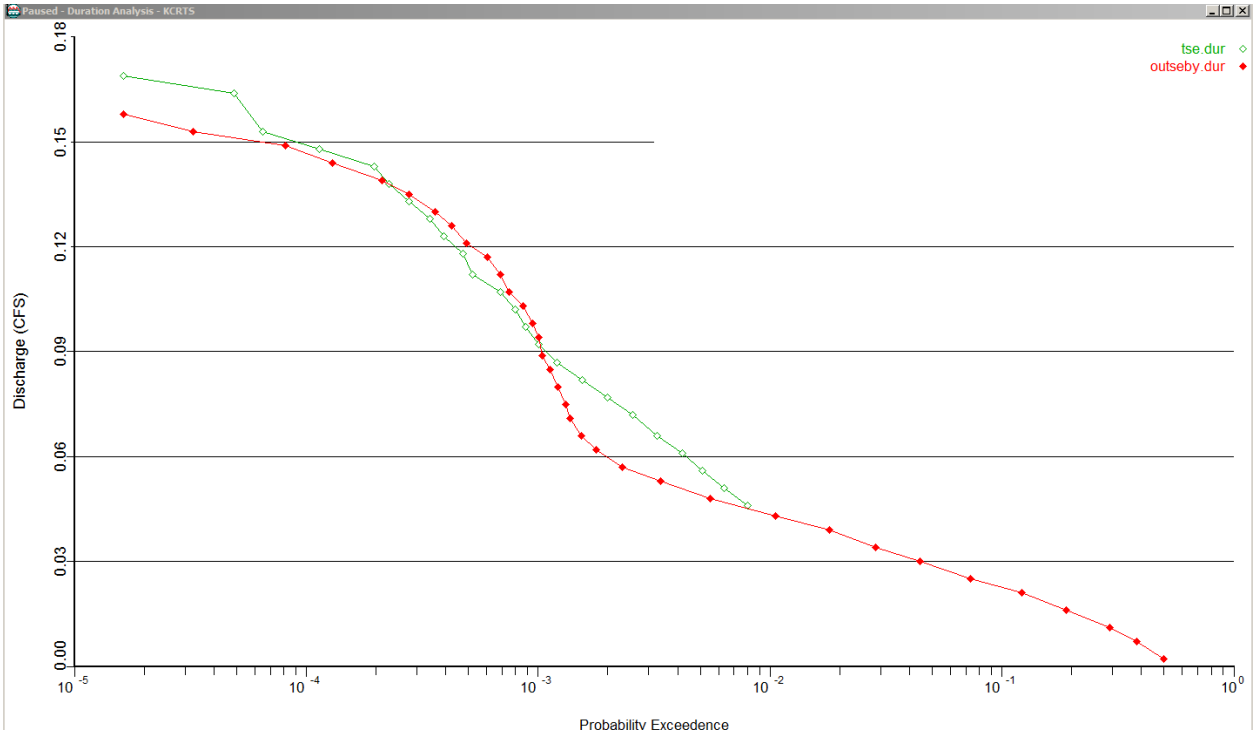
The following figure is a graph of probability vs. discharge at the point of compliance. The graph shows that the outflow duration curve is within 10% of the target duration (corresponding to 50% of the existing 2-year flow rate). A correction factor was not implemented in the design of the facility.

NE+SE Basins

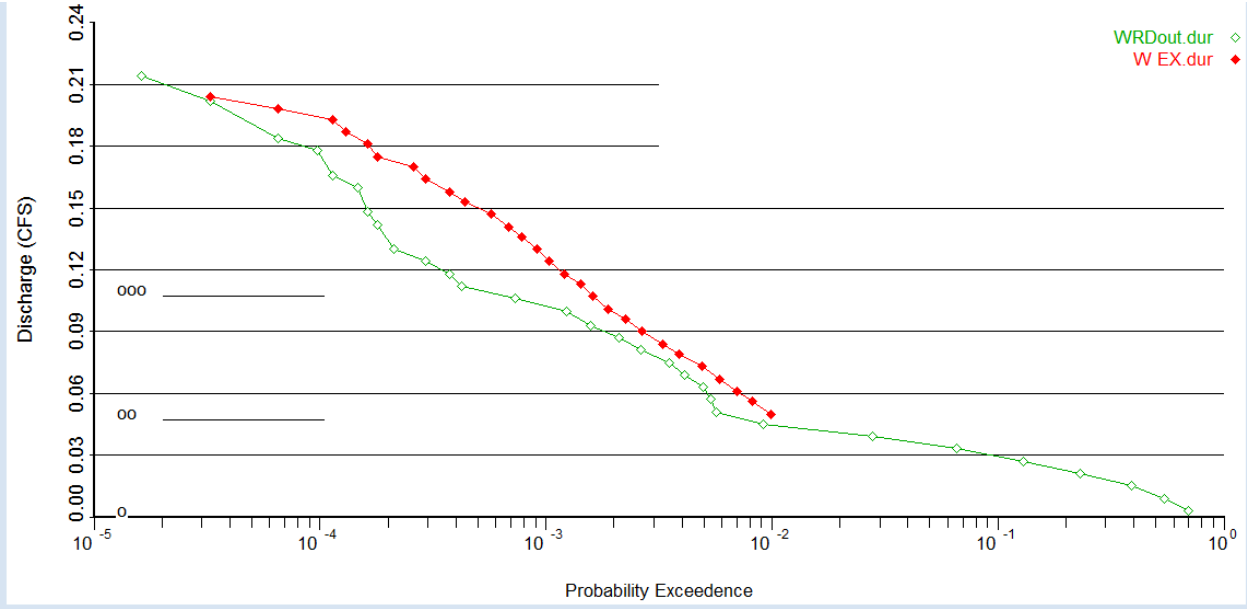
NE Discharge – at Facility Outlet



SE Discharge – at Downstream Point-of-Compliance



W Basin



Part E – Water Quality System

Basic water quality criteria are applicable to this project. The site will provide dead storage within the vaults.

Wetpool Volumes

The KCSWDM specifies that the following equation be used when sizing wet facilities. A summary of the water quality facility sizing calculation is provided below.

The wetpond volume (V_b) is calculated from the volume of runoff (V_r) from the mean annual storm and multiplying it by a volume factor (f) of 3.0.

NE+SE Basins

$V_r = (0.9A_i + 0.25A_{tg} + 0.10A_{tf} + 0.01A_o)* R/12$

Where V_r = volume of runoff from mean annual storm (cf)

A_i = area of impervious surface [(1.28+5.31)*43,560=287,060 sf]

A_{tg} = area of till soil covered with grass [(0.4+1.32)*43,560=74,923 sf]

A_{tf} area of till soil covered with forest (0)

A_o = area of outwash soil covered with grass or forest (0)

R = rainfall from mean annual storm (0.50") [Figure 6.4.1.A]

V_r = (0.9(287,060)+0.25(74,923))*0.50/12 = 11,545 cubic feet ("cf")

$V_b = V_r * f$ = 11,545 *3 = 34,635 cf

Required Water Quality Volume = 34,635 cf. Volume provided will be 4' minimum depth over the floor area of 15,008 sf or 60,032 cf.

W Basin

$V_r = (0.9A_i + 0.25A_{tg} + 0.10A_{tf} + 0.01A_o)* R/12$

Where V_r = volume of runoff from mean annual storm (cf)

A_i = area of impervious surface (2.08*43,560=90,605 sf)

A_{tg} = area of till soil covered with grass (0.60*43,560=26,136 sf)

A_{tf} area of till soil covered with forest (0)

A_o = area of outwash soil covered with grass or forest (0)

R = rainfall from mean annual storm (0.50") [Figure 6.4.1.A]

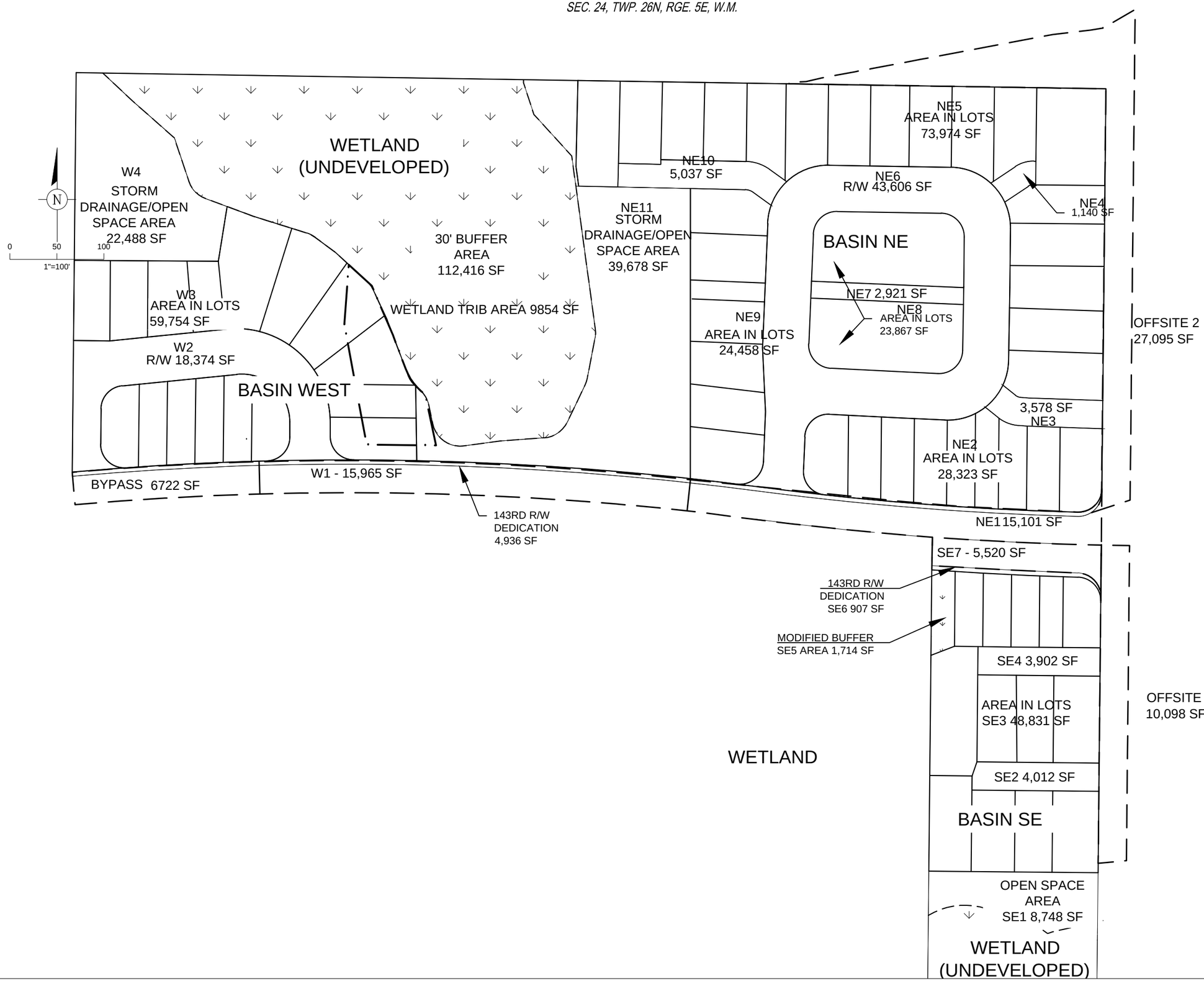
V_r = (0.9(90,605)+0.25(26,136))*0.50/12 = 3,670 cubic feet ("cf")

$V_b = V_r * f$ = 3,670 *3 = 11,010 cf

Required Water Quality Volume = 11,010 cf, Volume provided will be 4' minimum over the 116'x80' portion of the vault. Minimum volume would be 37,120 cf

Figure 4.1 – Basin Areas

SEC. 24, TWP. 26N, RGE. 5E, W.M.



NO	DATE	REVISION	BY	CK

DESIGN GROUP
JON W. NELSON, P.E.
PROJECT MANAGER:
JWN
DESIGNED:
JWN
CADD:
JWN
CHECKED:
APRIL 2015
DATE:
DBT REID0004
FILE NAME:

DEVELOPED BASIN MAP

RIO VISTA INVESTMENTS, LLC

RIO VISTA

CITY OF DUVALL

WASHINGTON

LDA
Land Development Advisors, LLC
Planning, Engineering, Project Management
12865 SE 47th Place
Bellevue, WA 98006
425-466-5203

JOB NUMBER REID004

FIG. 4.1

C:\Users\Admin\Documents\Land Development Advisors\Reid Companies\Rio Vista\Reports\TIR\Figures\DB1 REID0004.dwg <<DEV BASINS FIG 4.1>> Plotted: 4/20/2015 10:42:20 AM By: Admin (c) LDA

Figure 4.2 – Rainfall Regions and Regional Scale Factors

FIGURE 3.2.2.A RAINFALL REGIONS AND REGIONAL SCALE FACTORS

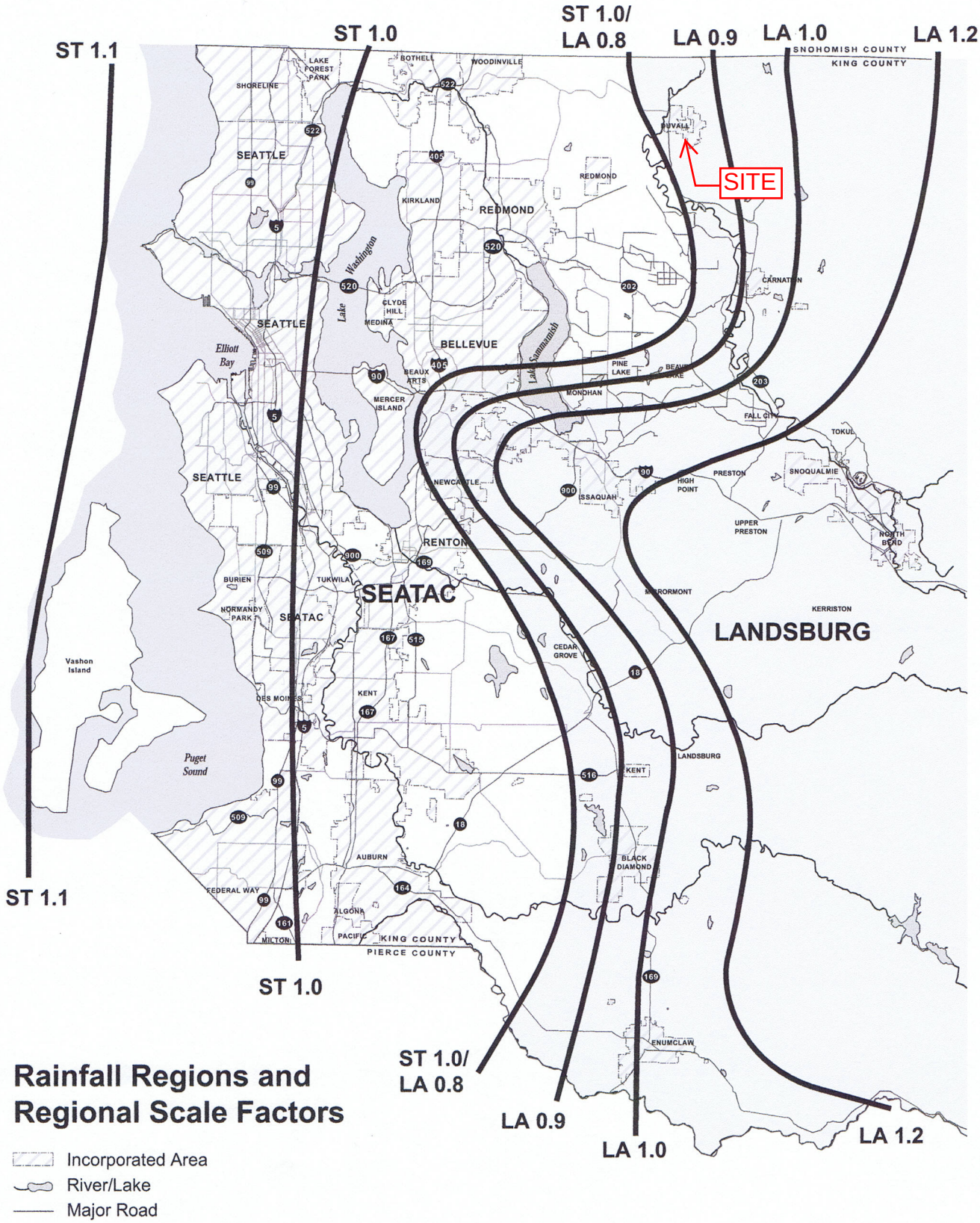
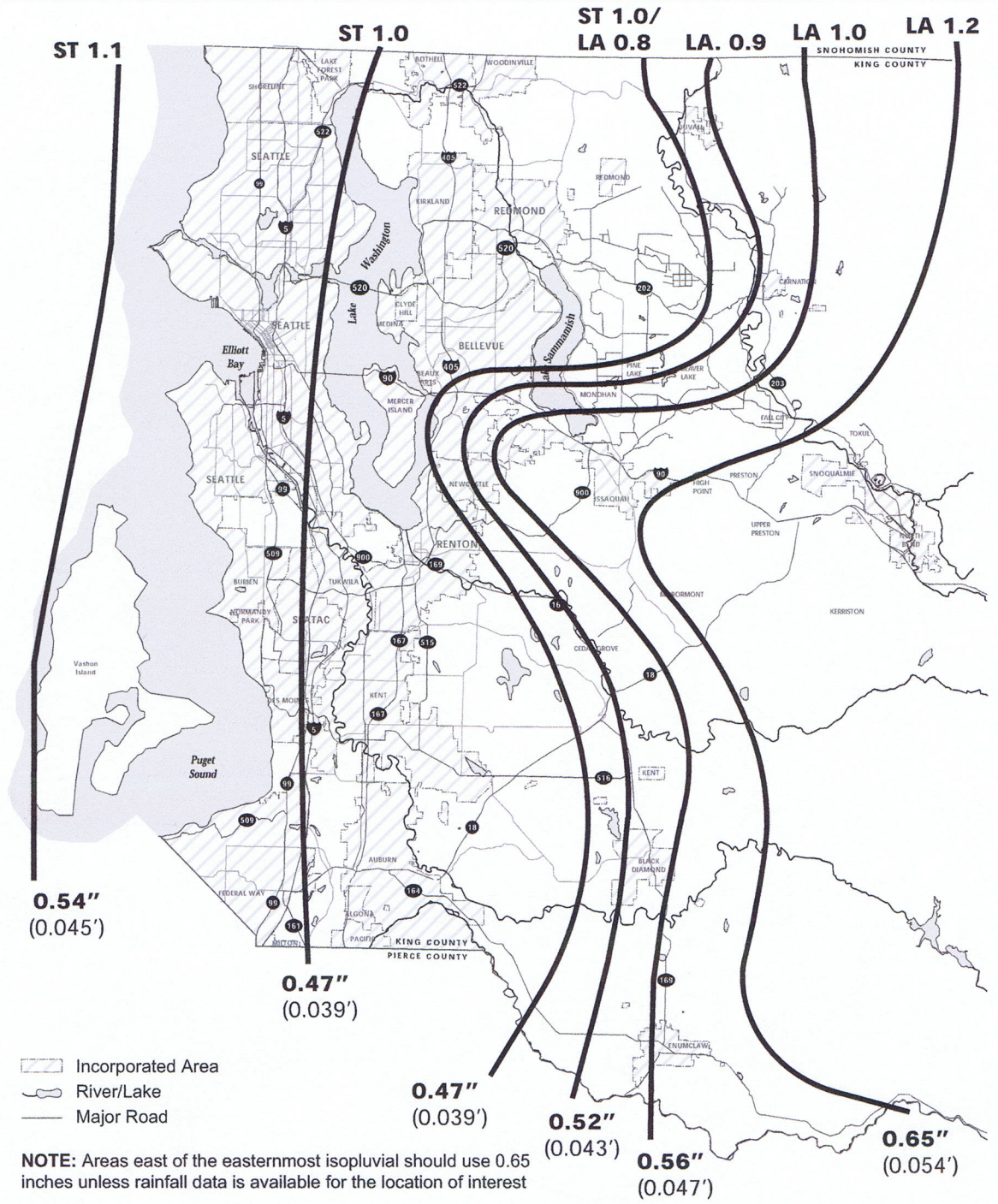


Figure 4.3 – Precipitation for Mean Annual Storm

FIGURE 6.4.1.A PRECIPITATION FOR MEAN ANNUAL STORM IN INCHES (FEET)



result, generates large amounts of runoff. For this application, till soil types include Buckley and bedrock soils, and alluvial and outwash soils that have a seasonally high water table or are underlain at a shallow depth (less than 5 feet) by glacial till. U.S. Soil Conservation Service (SCS) hydrologic soil groups that are classified as till soils include a few B, most C, and all D soils. See Chapter 3 for classification of specific SCS soil types.

Table 4.3 – Basin Areas

DEVELOPED				TOTALS					
AREA	LOT AREA	ROAD AREA	O.S AREA	SF	ACRES			IMPERVIOUS	PERVIOUS (25% OF LOT AREA, 50% OF O.S AREA)
SE1			8,748						
SE2		4,012		63,172	1.45	**SE BASIN**			
SE3	48,831			48,831	1.12	**LOTS**		0.84	0.28
SE4		3,902		14,341	0.09	**ROADS**		0.09	
SE5			1,714	10,462	0.24	**OPEN SPACE**		0.12	0.12
SE6		907				**OFFSITE**	**10,098**	**0.23**	
SE7		5,520					**TOTALS**	**1.28**	**0.40**
NE1		15,101		261,683	6.01	**NE BASIN**			
NE2	28,323			150,622	3.46	**LOTS**		2.59	0.86
NE3		3,578		71,383	1.64	**ROADS**		1.64	
NE4		1,140		39,678	0.91	**OPEN SPACE**		0.46	0.46
NE5	73,974			27,095	0.62	**OFFSITE**		0.62	
NE6		43,606					**TOTALS**	**5.31**	**1.32**
NE7		2,921							

NE8	23,867								
NE9	24,458								
NE10		5,037							
NE11			39,678						
W1		15,965		123,303	2.83	**W BASIN**			
W2		18,374		59,754	1.37	**LOTS**		1.03	0.34
W3	59,754			34,339	0.79	**ROADS**		0.79	
W4			22,488	22,488	0.52	**OPEN SPACE**		0.26	0.26
BYPASS		6,722					**TOTALS**	**2.08**	**0.60**
				6,722	0.15	**BYPASS**		**0.15**	
NE+SE								**6.59**	**1.72**

SECTION 5 - CONVEYANCE SYSTEM ANALYSIS AND DESIGN

Pipe Sizing

For development projects, new conveyance systems shall be designed with sufficient capacity to convey and contain the 25-year peak flow. Pipe systems may overtop for the 100-year peak provided there is no creation or aggravation of a "severe flooding problem" or "severe erosion problem". There are no such areas within or ¼ mile downstream of the property.

The analysis will be completed with the final construction plans using the Storm Sewers computer model included with AutoCAD 2013. This model calculates 25 year flows in accordance with the Rational Method. The model will assume a C value of 0.9 for impervious areas and 0.25 for landscaped areas. All onsite areas will be assigned a weighted C value of 0.5. This value was calculated from Table 4.1 using the NE+SE Basin, ([6.59*0.9+1.72*0.25]/8.31=0.77) Additionally, a T_c of 6.3 minutes was used and pipe travel times were suppressed to generate a conservative analysis. The offsite areas contributory to the site were assigned C values of 0.9.

The conveyance modeling will demonstrate the system is capable of meeting the designated conveyance criteria.

Dispersion Trenches

The outlet from the NE and West basins will be to a dispersion facility. The purpose will be to spread flows in a linear fashion to mimic existing conditions. Maximum flow depths should be limited to 3" during a 100 year overflow event as calculated by KCRTS using 15 min time steps. Design was obtained from http://irrigation.wsu.edu/Content/Calculators/Water-Measurements/Rectangular-Contracted-Weir.php.

NE Basin

```
          Flow Frequency Analysis
  Time Series File:devne15.tsf
  Project Location:Landsburg

  ---Annual Peak Flow Rates---      -----Flow Frequency Analysis-------
 Flow Rate  Rank  Time of Peak       - - Peaks - -  Rank  Return  Prob
   (CFS)                                (CFS)              Period
    2.52      7  10/04/00 20:15        7.52          1  100.00   0.990
    2.49      8   5/06/02  7:15        5.21          2   25.00   0.960
    5.21      2   9/10/03 13:45        5.16          3   10.00   0.900
    4.63      4   8/25/04 23:30        4.63          4    5.00   0.800
    3.35      6   9/10/05 16:45        3.46          5    3.00   0.667
    5.16      3  10/22/05 16:15        3.35          6    2.00   0.500
    3.46      5  11/21/06  8:00        2.52          7    1.30   0.231
    7.52      1   1/09/08  7:30        2.49          8    1.10   0.091
Computed Peaks                         6.75              50.00   0.980
```

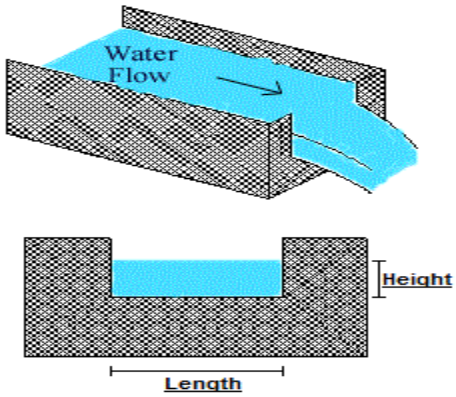
West Basin

```
          Flow Frequency Analysis
  Time Series File:devw15.tsf
  Project Location:Landsburg

  ---Annual Peak Flow Rates---      -----Flow Frequency Analysis-------
 Flow Rate  Rank  Time of Peak       - - Peaks - -  Rank  Return  Prob
   (CFS)                                (CFS)             Period
   0.958      7  10/04/00 20:15          2.65         1  100.00   0.990
   0.944      8   5/06/02  7:15          1.98         2   25.00   0.960
    1.98      2   9/10/03 13:45          1.87         3   10.00   0.900
    1.58      4   8/25/04 23:30          1.58         4    5.00   0.800
    1.24      5   9/10/05 16:45          1.24         5    3.00   0.667
    1.87      3  10/22/05 16:15          1.24         6    2.00   0.500
    1.24      6  11/21/06  8:00         0.958         7    1.30   0.231
    2.65      1   1/09/08  7:30         0.944         8    1.10   0.091
Computed Peaks                           2.43             50.00   0.980
```

A 50' section will be used.

Rectangular Contracted Weir



Length:

Height:

Flow Rate:

The Equation

The Equation used to determine the flow rate (Q) of a Rectangular Contracted Weir is:

$$Q = \frac{3.247 \cdot L \cdot H^{1.48} - 0.566 \cdot L^{1.9}}{1 + 2L^{1.87}} \cdot H^{1.9}$$

Where:

Q = Flow Rate in cfs.

L = Bottom width of the weir in feet.

H = Height of the upstream water above the weir crest in feet.

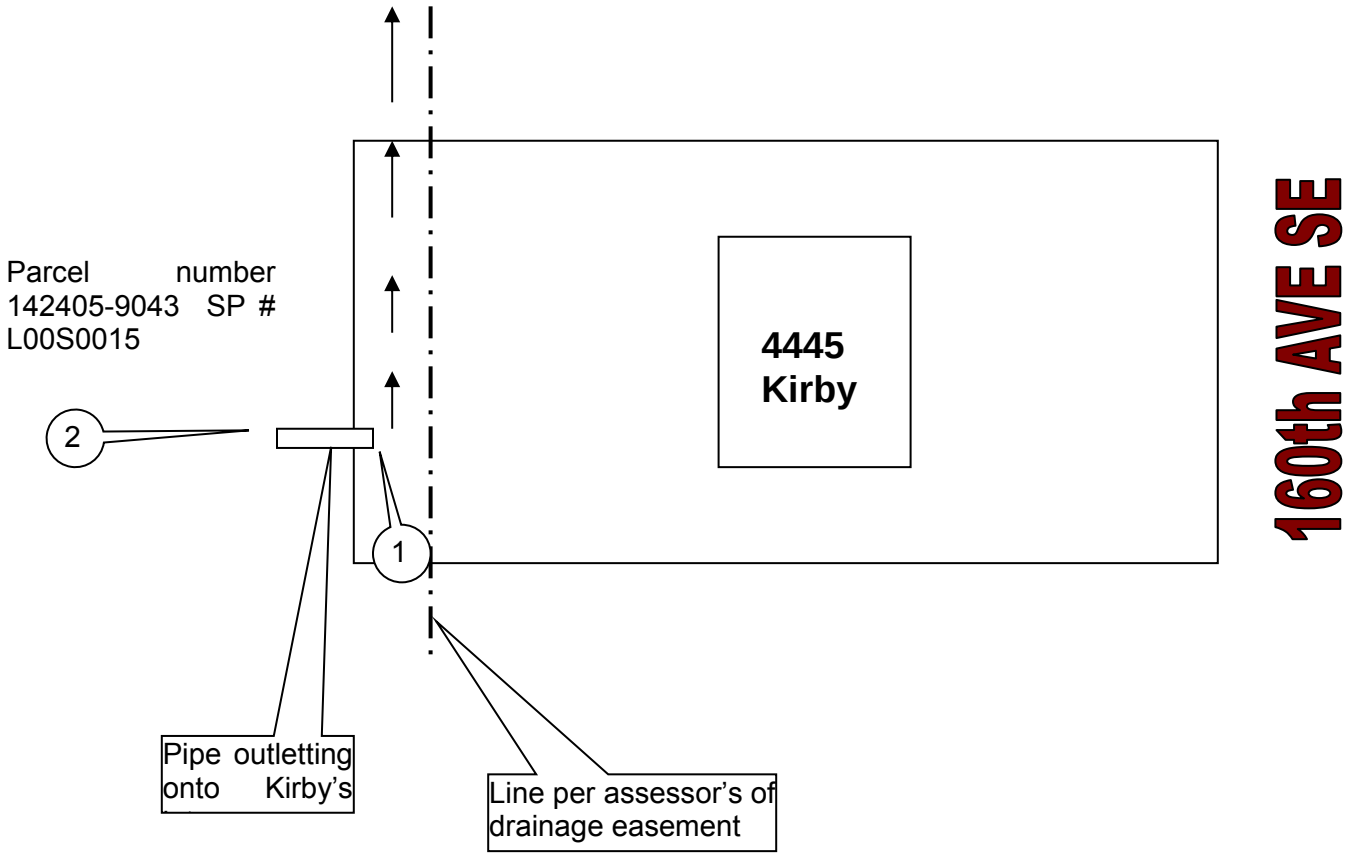
50' weir with a height of 0.25' (3") provides for over 20 cfs of flow.

APPENDIX

Drainage Complaint

Complaint Number 2004-0731 Kirby
Investigated by Andrew McDonald on October 4, 2004

Visited site and took photographs after noting pipe identified by complainant. Construction is taking place on the adjacent lot to the west of the complainant's property. Upon returning to the office I found the activity under L00S0015, a four lot short plat under Herb Mull/New Concept Homes. It appears that this short plat is still under review but allowed to begin grading and development via an "early start" and is under inspection by Jay Young at DDES. The development in which the complainant lives, Baker View, is still under a "Punchlist" status with DDES. The pipe outletting into Ms. Kirby's back yard appears to be an overflow and/or outlet to a tank or vault system by just a visual inspection. I've tried contacting Ms. Kirby twice to no avail. I am attempting to steer her towards DDES where the plans are in for review and where both developments are still under review/inspection.



Offsite Analysis and Drainage System Table

Photographs

Photo 1: The swale at point W1 is shallow and stable. This view is looking west.

Photo 2: This view at point W4 looking towards W5 shows a stable ditch.

Photo 3: The ditch near W9 is rock lined with check dams to help dissipate energy. It is stable and performing well.

Photo 4 – This view looking downstream from point W12 shows a heavily vegetated ditch. Channel maintenance is necessary.

Photo 5 – The alignment of Thayer Creek can be seen traversing the pasture. Photograph taken at point W15 looking northwesterly.

Photo 6 – This view looking west at N2 shows the inlet to the 24" pipe crossing NE 144th Place.

Photo 7 – This view, looking north at point N3, shows the ditch downstream from NE 144th Place.

Photo 8 – This is the outlet of the 24" CPP at N4.

Photo 9 – This view at N6 shows the inlets of the three culverts crossing NE Miller Street.

Photo 10 – This view shows the outlet end of the culverts crossing NE Miller Street. Note standing water in the ditch that could be alleviated by dredging and cleaning the downstream section of the channel.

Photo 11 – This view from N8 shows the overgrown ditch heading north towards the City's detention pond.

Calculations

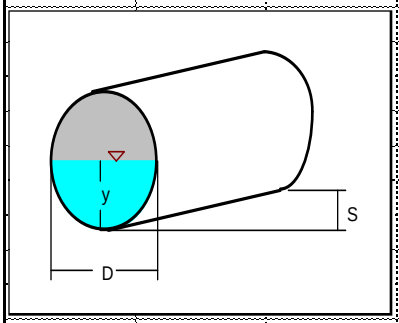
```
          Flow Frequency Analysis
  Time Series File:wbypex.tsf
  Project Location:Landsburg

  ---Annual Peak Flow Rates---      -----Flow Frequency Analysis-------
 Flow Rate  Rank  Time of Peak       - - Peaks - -  Rank  Return  Prob
   (CFS)                                (CFS)             Period
   0.030      4   2/09/01  2:00        0.056         1  100.00   0.990
   0.017      8  12/03/01 17:00        0.036         2   25.00   0.960
   0.028      6   9/10/03 15:00        0.034         3   10.00   0.900
   0.034      3   8/26/04  1:00        0.030         4    5.00   0.800
   0.028      7  10/28/04 18:00        0.029         5    3.00   0.667
   0.029      5  10/22/05 17:00        0.028         6    2.00   0.500
   0.036      2  11/21/06  9:00        0.028         7    1.30   0.231
   0.056      1   1/09/08  7:00        0.017         8    1.10   0.091
Computed Peaks                         0.050             50.00   0.980

          Flow Frequency Analysis
  Time Series File:wbypdev.tsf
  Project Location:Landsburg

  ---Annual Peak Flow Rates---      -----Flow Frequency Analysis-------
 Flow Rate  Rank  Time of Peak       - - Peaks - -  Rank  Return  Prob
   (CFS)                                (CFS)             Period
   0.040      7   2/09/01  2:00        0.069         1  100.00   0.990
   0.028      8  12/03/01 17:00        0.054         2   25.00   0.960
   0.051      4   9/10/03 15:00        0.053         3   10.00   0.900
   0.054      2   8/26/04  1:00        0.051         4    5.00   0.800
   0.045      6  10/28/04 18:00        0.047         5    3.00   0.667
   0.047      5  10/22/05 17:00        0.045         6    2.00   0.500
   0.053      3  10/26/06  3:00        0.040         7    1.30   0.231
   0.069      1   1/09/08  7:00        0.028         8    1.10   0.091
Computed Peaks                         0.064             50.00   0.980
```

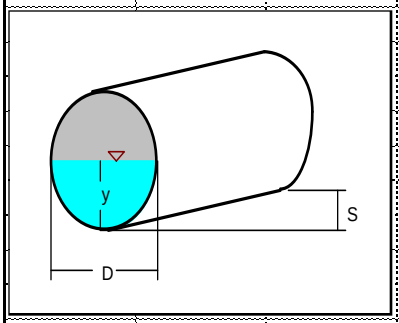
	Input	Output							
Q (cfs)	0.00	8.63							
n	0.012	0.012							
d (ft)	1.00	1.00		1	0	/16	inches		
y (ft)	1.00	1.00							
S (ft/ft)	0.050	0.050							



A (sf)	0.785		
Pw (ft)	3.142	**V (ft/s)**	10.988
R (ft)	0.250		
Critical y (ft) =			
Qmax @ y (ft) =	0.9382		
Vmax @ y (ft) =	0.8128		

Job:	Rio Vista	**Description:**	Miller Street
By:	JWN	**Date:**	7/17/2013

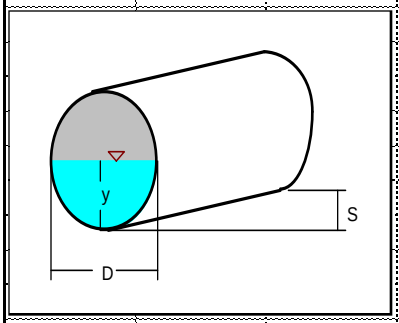
	Input	Output							
Q (cfs)	0.00	25.44							
n	0.012	0.012							
d (ft)	1.50	1.50		1	8	/16	inches		
y (ft)	1.50	1.50							
S (ft/ft)	0.050	0.050							



A (sf)	1.767		
Pw (ft)	4.712	**V (ft/s)**	14.399
R (ft)	0.375		
Critical y (ft) =			
Qmax @ y (ft) =	1.4073		
Vmax @ y (ft) =	1.2192		

Job:	Job Name	**Description:**	Description
By:	Designer	**Date:**	Date

	Input	Output							
Q (cfs)	0.00	2.73							
n	0.012	0.012							
d (ft)	1.00	1.00		1	0	/16	inches		
y (ft)	1.00	1.00							
S (ft/ft)	0.005	0.005							



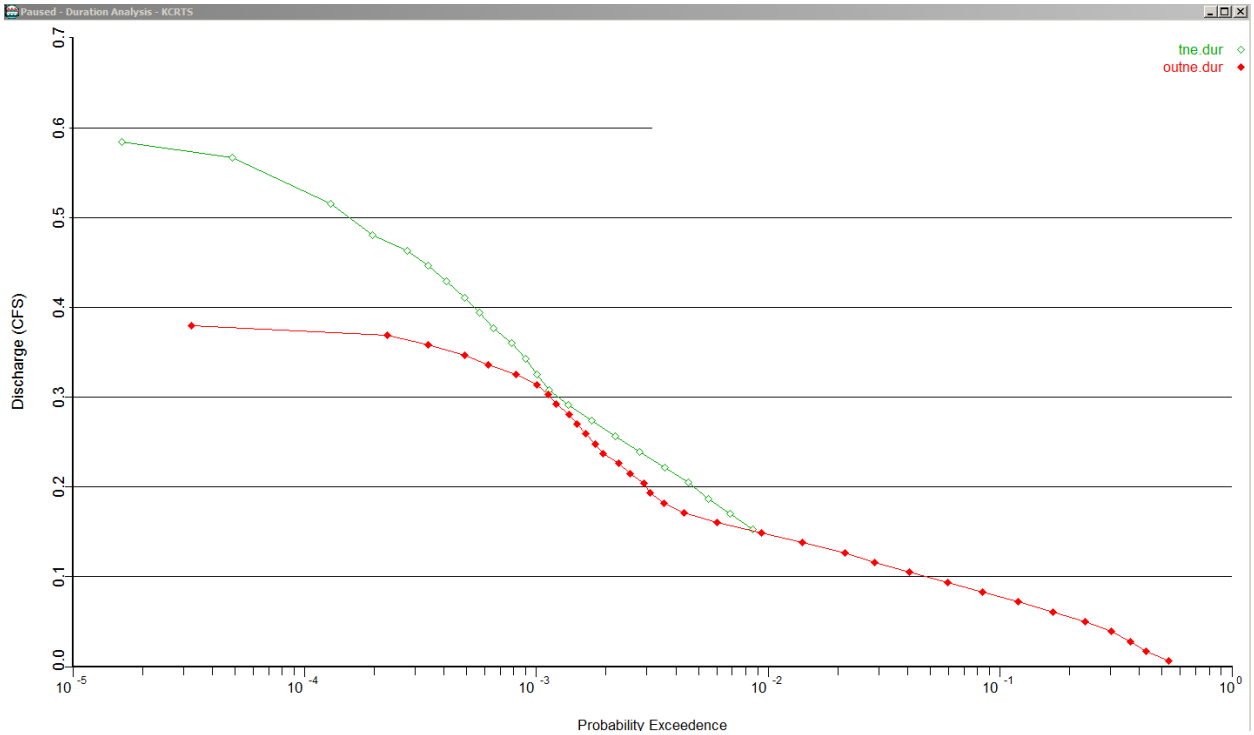
A (sf)	0.785		
Pw (ft)	3.142	**V (ft/s)**	3.475
R (ft)	0.250		
Critical y (ft) =			
Qmax @ y (ft) =	0.9382		
Vmax @ y (ft) =	0.8128		

Job:	Job Name	**Description:**	Description
By:	Designer	**Date:**	Date

NE Discharge – at Facility Outlet

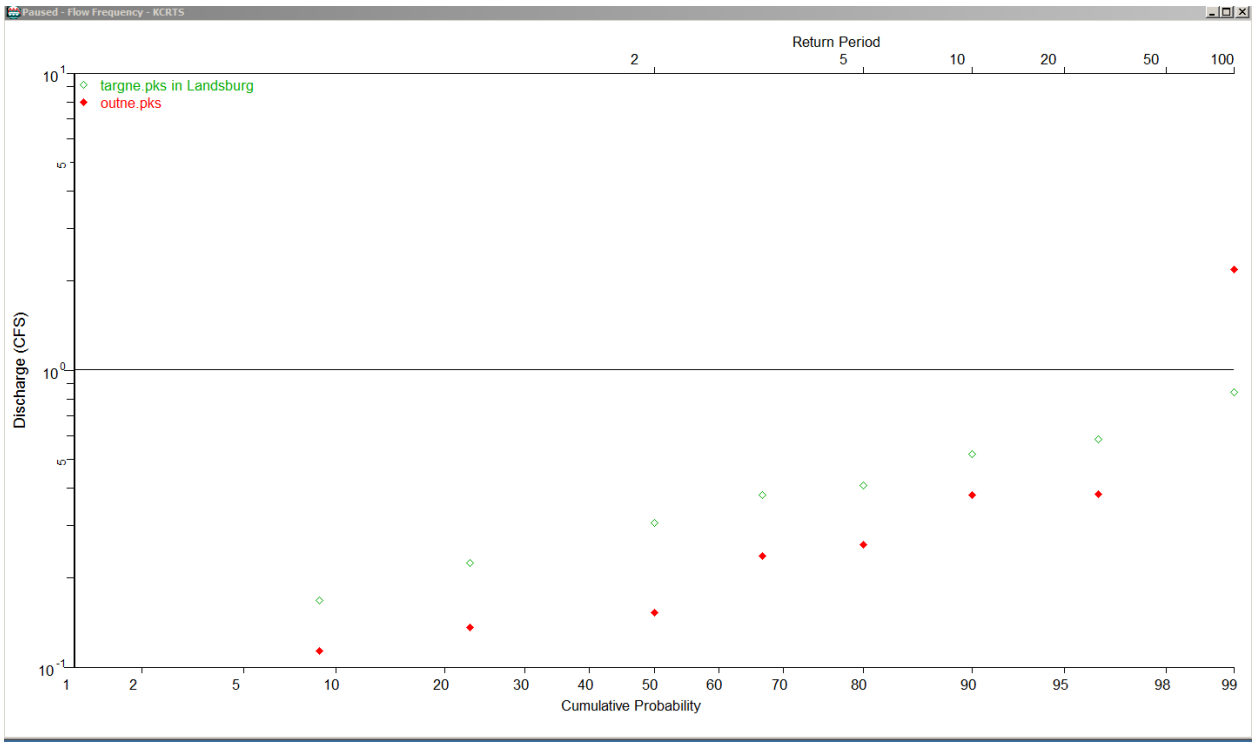
Flow Durations

```
tne: target conditions
outne: vault discharge to NE location
```



Peak Flow Rates

```
targne: target conditions
outne: vault discharge to NE location
```



NE Discharge Location - Duration Comparison Analysis

Base File: targne.tsf
New File: outne.tsf
Cutoff Units: Discharge in CFS

	-----Fraction of Time-----			---------Check of Tolerance-------			
Cutoff	Base	New	%Change	Probability	Base	New	%Change
0.153	0.86E-02	0.79E-02	-8.5	0.86E-02	0.153	0.151	-1.6
0.186	0.57E-02	0.33E-02	-41.5	0.57E-02	0.186	0.160	-14.0
0.220	0.37E-02	0.24E-02	-34.8	0.37E-02	0.220	0.180	-17.9
0.253	0.23E-02	0.17E-02	-24.1	0.23E-02	0.253	0.223	-11.7
0.286	0.15E-02	0.13E-02	-15.8	0.15E-02	0.286	0.264	-7.8
0.320	0.11E-02	0.91E-03	-13.8	0.11E-02	0.320	0.309	-3.3
0.353	0.83E-03	0.41E-03	-51.0	0.83E-03	0.353	0.324	-8.1
0.386	0.62E-03	0.00E+00	-100.0	0.62E-03	0.386	0.335	-13.2
0.419	0.44E-03	0.00E+00	-100.0	0.44E-03	0.419	0.350	-16.5
0.453	0.31E-03	0.00E+00	-100.0	0.31E-03	0.453	0.362	-20.1
0.486	0.16E-03	0.00E+00	-100.0	0.16E-03	0.486	0.374	-23.1
0.519	0.98E-04	0.00E+00	-100.0	0.98E-04	0.519	0.377	-27.4
0.553	0.49E-04	0.00E+00	-100.0	0.49E-04	0.553	0.379	-31.4

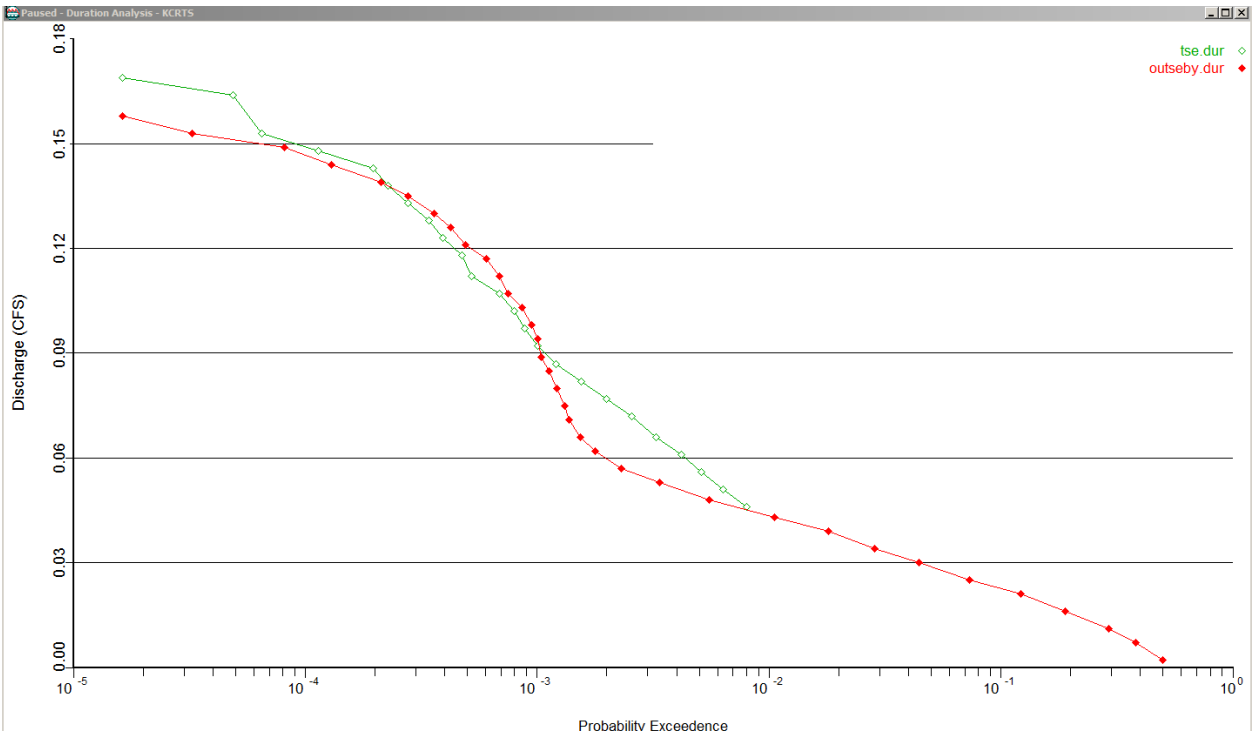
There is no positive excursion

Maximum negative excursion = 0.205 cfs (-35.0%)
occurring at 0.585 cfs on the Base Data:targne.tsf
and at 0.380 cfs on the New Data:outne.tsf

SE Discharge – at Downstream Point-of-Compliance

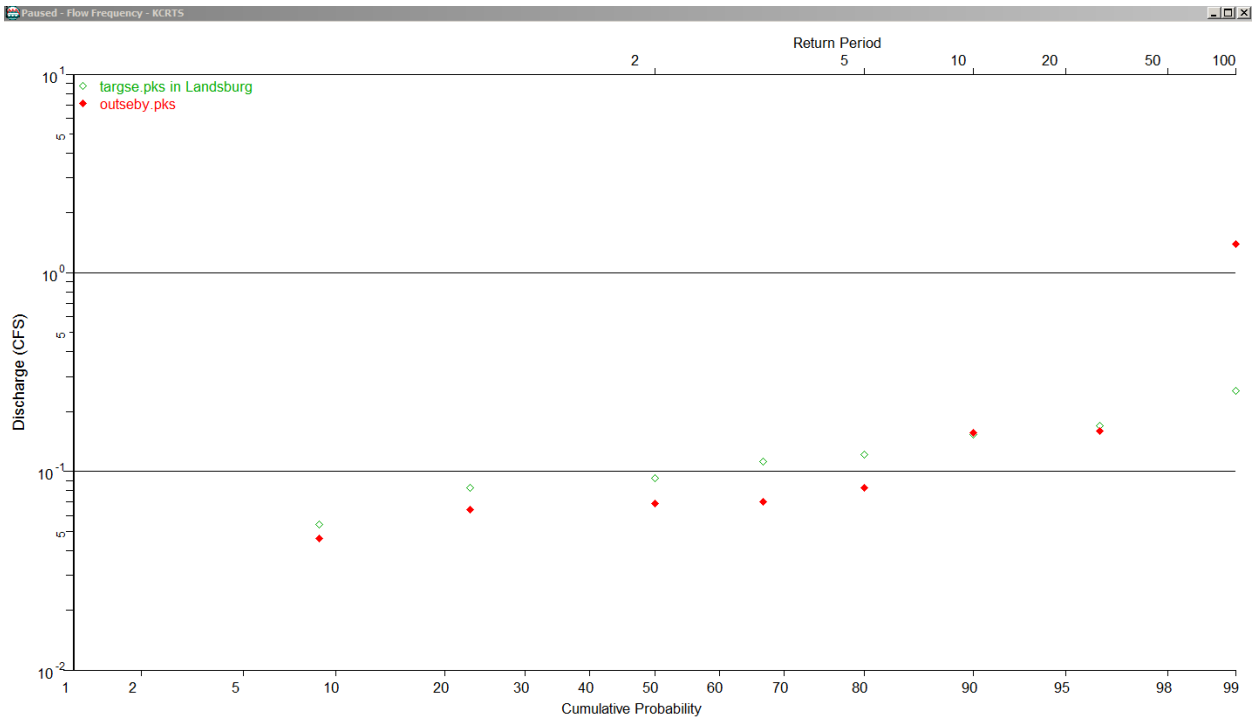
Flow Durations

```
tse: target conditions
outseby: vault discharge to SE point-of-compliance (includes bypass)
```



Peak Flow Rates

```
targse: target conditions
outseby: vault discharge to SE point-of-compliance (includes bypass)
```



SE Discharge Location - Duration Comparison Analysis

Base File: targse.tsf
New File: outseby.tsf
Cutoff Units: Discharge in CFS

	-----Fraction of Time-----			---------Check of Tolerance-------			
Cutoff	Base	New	%Change	Probability	Base	New	%Change
0.046	0.80E-02	0.78E-02	-2.4	0.80E-02	0.046	0.045	-0.8
0.055	0.53E-02	0.27E-02	-50.0	0.53E-02	0.055	0.049	-12.0
0.065	0.35E-02	0.16E-02	-54.9	0.35E-02	0.065	0.052	-19.0
0.074	0.23E-02	0.13E-02	-41.7	0.23E-02	0.074	0.058	-22.5
0.084	0.14E-02	0.12E-02	-15.5	0.14E-02	0.084	0.071	-15.2
0.093	0.98E-03	0.10E-02	5.0	0.98E-03	0.093	0.097	4.4
0.103	0.78E-03	0.86E-03	10.4	0.78E-03	0.103	0.107	4.0
0.112	0.54E-03	0.68E-03	27.3	0.54E-03	0.112	0.119	6.1
0.122	0.41E-03	0.49E-03	20.0	0.41E-03	0.122	0.127	4.1
0.131	0.28E-03	0.34E-03	23.5	0.28E-03	0.131	0.135	2.9
0.140	0.20E-03	0.20E-03	0.0	0.20E-03	0.140	0.141	0.6
0.150	0.98E-04	0.82E-04	-16.7	0.98E-04	0.150	0.146	-2.4
0.159	0.49E-04	0.00E+00	-100.0	0.49E-04	0.159	0.153	-4.1
0.169	0.16E-04	0.00E+00	-100.0	0.16E-04	0.169	0.158	-6.1

Maximum positive excursion = 0.007 cfs (6.3%)
occurring at 0.112 cfs on the Base Data:targse.tsf
and at 0.119 cfs on the New Data:outseby.tsf

Maximum negative excursion = 0.017 cfs (-23.2%)
occurring at 0.073 cfs on the Base Data:targse.tsf
and at 0.056 cfs on the New Data:outseby.tsf

Vault Design

Dimensions:

Bot Area	14960	SF **Revised to 15,008 SF**
Length	170	FT **Revised to 268'**
Width	88	FT **Revised to 56'**
Depth	8	FT

Outlet Structures:

NE Outlet Structure

Orifice	Dia. (IN)	Height (FT)	Area (SF)
O1	1.69	-	0.015577596
O2	0.88	4.40	0.004223693
O3	1.70	4.80	0.015762491
Riser	18.00	8.00	

SE Outlet Structure

Orifice	Dia. (IN)	Height (FT)	Area (SF)
O1	0.87	-	0.004128
O2	1.50	5.80	0.012272
O3	-	-	0
Riser	12.00	8.00	

Stage-Storage-Discharge

Discharge A = NE Outlet
Discharge B = SE Outlet

Stage (Ft)	Discharge (CFS) A	Discharge (CFS) B	Storage (Cu-Ft)	Perm-Area (Sq-Ft)
0.00	0.000	0.000	0.00	0.0
0.20	0.035	0.009	2992.00	0.0
0.40	0.049	0.013	5984.00	0.0
0.60	0.060	0.016	8976.00	0.0
0.80	0.069	0.018	11968.00	0.0
1.00	0.078	0.021	14960.00	0.0
1.20	0.085	0.023	17952.00	0.0
1.40	0.092	0.024	20944.00	0.0
1.60	0.098	0.026	23936.00	0.0
1.80	0.104	0.028	26928.00	0.0
2.00	0.110	0.029	29920.00	0.0
2.20	0.115	0.030	32912.00	0.0
2.40	0.120	0.032	35904.00	0.0
2.60	0.125	0.033	38896.00	0.0
2.80	0.130	0.034	41888.00	0.0
3.00	0.134	0.036	44880.00	0.0
3.20	0.139	0.037	47872.00	0.0
3.40	0.143	0.038	50864.00	0.0
3.60	0.147	0.039	53856.00	0.0
3.80	0.151	0.040	56848.00	0.0

4.00	0.155	0.041	59840.00	0.0
4.20	0.159	0.042	62832.00	0.0
4.40	0.163	0.043	65824.00	0.0
4.60	0.176	0.044	68816.00	0.0
4.80	0.183	0.045	71808.00	0.0
5.00	0.225	0.046	74800.00	0.0
5.20	0.245	0.047	77792.00	0.0
5.40	0.262	0.048	80784.00	0.0
5.60	0.277	0.049	83776.00	0.0
5.80	0.290	0.049	86768.00	0.0
6.00	0.302	0.078	89760.00	0.0
6.20	0.314	0.090	92752.00	0.0
6.40	0.325	0.099	95744.00	0.0
6.60	0.336	0.107	98736.00	0.0
6.80	0.346	0.115	101728.00	0.0
7.00	0.355	0.121	104720.00	0.0
7.20	0.365	0.127	107712.00	0.0
7.40	0.374	0.133	110704.00	0.0
7.60	0.382	0.139	113696.00	0.0
7.80	0.391	0.144	116688.00	0.0
8.00	0.399	0.149	119680.00	0.0
8.20	6.941	4.509	122672.00	0.0
8.40	9.655	6.317	125664.00	0.0
8.60	11.739	7.706	128656.00	0.0
8.80	13.497	8.878	131648.00	0.0
9.00	15.047	9.910	134640.00	0.0

KCRTS Basins

```
Description of Basins:
targne: NE basin target conditions
targse: SE basin target conditions
devne: NE basin developed conditions
devse: SE basin developed conditions
byse: SE basin bypass

                                KCRTS Command
                                -------------
                           CREATE a new Time Series
                           ------------------------
                       Production of Runoff Time Series
    Project Location : Landsburg
     Computing Series : targne.tsf
Regional Scale Factor :    0.85
            Data Type : Reduced
Creating Hourly Time Series File
                    Loading Time Series File:C:\KC_SWDM\KC_DATA\LATF60R.rnf   :
Till Forest             6.01 acres
                    Loading Time Series File:C:\KC_SWDM\KC_DATA\LAEI60R.rnf   :
Impervious              0.62 acres
                     --------------
         Total Area :    6.63 acres
Peak Discharge:  0.841 CFS at  7:00 on Jan  9 in Year 8
                                         Storing Time Series File:targne.tsf   :
                             Time Series Computed

                                KCRTS Command
                                -------------
                        Enter the Analysis TOOLS Module
                        -------------------------------

                            Analysis Tools Command
                            ----------------------
                      Compute PEAKS and Flow Frequencies
                      ----------------------------------
                                   Loading Stage/Discharge curve:targne.tsf   :
          Flow Frequency Analysis
          -------------------------------------------------------
  Time Series File:targne.tsf
  Project Location:Landsburg
                               Frequencies & Peaks saved to File:targne.pks   :

                            Analysis Tools Command
                            ----------------------
                     Compute Flow DURATION and Exceedence
                     ------------------------------------
                                         Loading Time Series File:targne.tsf   :
Computing Interval Locations
Computing Flow Durations
                    Durations & Exceedence Probabilities to File:targne.dur   :
```

```
                          Analysis Tools Command
                          ----------------------
                          RETURN to Previous Menu
                          -----------------------

                               KCRTS Command
                               -------------
                         CREATE a new Time Series
                         ------------------------
                      Production of Runoff Time Series
    Project Location : Landsburg
     Computing Series : devne.tsf
Regional Scale Factor :    0.85
            Data Type : Reduced
Creating Hourly Time Series File
                    Loading Time Series File:C:\KC_SWDM\KC_DATA\LATG60R.rnf   :
Till Grass              1.32 acres
                    Loading Time Series File:C:\KC_SWDM\KC_DATA\LAEI60R.rnf   :
Impervious              5.31 acres
                     --------------
         Total Area :    6.63 acres
Peak Discharge:   2.81 CFS at  7:00 on Jan  9 in Year 8
                                         Storing Time Series File:devne.tsf   :
                           Time Series Computed

                               KCRTS Command
                               -------------
                       Enter the Analysis TOOLS Module
                       -------------------------------

                          Analysis Tools Command
                          ----------------------
                     Compute PEAKS and Flow Frequencies
                     ----------------------------------
                                   Loading Stage/Discharge curve:devne.tsf   :
          Flow Frequency Analysis
          -------------------------------------------------------
  Time Series File:devne.tsf
  Project Location:Landsburg
                                Frequencies & Peaks saved to File:devne.pks   :

                          Analysis Tools Command
                          ----------------------
                    Compute Flow DURATION and Exceedence
                    ------------------------------------
                                         Loading Time Series File:devne.tsf   :
Computing Interval Locations
Computing Flow Durations
                    Durations & Exceedence Probabilities to File:devne.dur   :

                          Analysis Tools Command
                          ----------------------
                          RETURN to Previous Menu
                          -----------------------
```

```
                                KCRTS Command
                                -------------
                            CREATE a new Time Series
                            ------------------------
                       Production of Runoff Time Series
    Project Location : Landsburg
     Computing Series : targse.tsf
Regional Scale Factor :    0.85
            Data Type : Reduced
Creating Hourly Time Series File
                    Loading Time Series File:C:\KC_SWDM\KC_DATA\LATF60R.rnf   :
Till Forest             1.60 acres
                    Loading Time Series File:C:\KC_SWDM\KC_DATA\LAEI60R.rnf   :
Impervious              0.23 acres
                     --------------
         Total Area :    1.83 acres
Peak Discharge:  0.254 CFS at  7:00 on Jan  9 in Year 8
                                         Storing Time Series File:targse.tsf   :
                             Time Series Computed

                                KCRTS Command
                                -------------
                       Enter the Analysis TOOLS Module
                       -------------------------------

                           Analysis Tools Command
                           ----------------------
                      Compute PEAKS and Flow Frequencies
                      ----------------------------------
                                   Loading Stage/Discharge curve:targse.tsf   :
          Flow Frequency Analysis
          -------------------------------------------------------
  Time Series File:targse.tsf
  Project Location:Landsburg
                               Frequencies & Peaks saved to File:targse.pks   :

                           Analysis Tools Command
                           ----------------------
                     Compute Flow DURATION and Exceedence
                     ------------------------------------
                                         Loading Time Series File:targse.tsf   :
Computing Interval Locations
Computing Flow Durations
                    Durations & Exceedence Probabilities to File:targse.dur   :

                           Analysis Tools Command
                           ----------------------
                           RETURN to Previous Menu
                           -----------------------

                                KCRTS Command
                                -------------
                            CREATE a new Time Series
```

```
                         ------------------------
                      Production of Runoff Time Series
    Project Location : Landsburg
     Computing Series : devse.tsf
Regional Scale Factor :    0.85
            Data Type : Reduced
Creating Hourly Time Series File
                    Loading Time Series File:C:\KC_SWDM\KC_DATA\LATG60R.rnf   :
Till Grass              0.40 acres
                    Loading Time Series File:C:\KC_SWDM\KC_DATA\LAEI60R.rnf   :
Impervious              1.28 acres
                     --------------
         Total Area :    1.68 acres
Peak Discharge:  0.702 CFS at  7:00 on Jan  9 in Year 8
                                         Storing Time Series File:devse.tsf   :
                           Time Series Computed

                               KCRTS Command
                               -------------
                      Enter the Analysis TOOLS Module
                      -------------------------------

                          Analysis Tools Command
                          ----------------------
                     Compute PEAKS and Flow Frequencies
                     ----------------------------------
                                   Loading Stage/Discharge curve:devse.tsf   :
          Flow Frequency Analysis
          -------------------------------------------------------
  Time Series File:devse.tsf
  Project Location:Landsburg
                                Frequencies & Peaks saved to File:devse.pks   :

                          Analysis Tools Command
                          ----------------------
                    Compute Flow DURATION and Exceedence
                    ------------------------------------
                                         Loading Time Series File:devse.tsf   :
Computing Interval Locations
Computing Flow Durations
                    Durations & Exceedence Probabilities to File:devse.dur   :

                          Analysis Tools Command
                          ----------------------
                          RETURN to Previous Menu
                          -----------------------

                               KCRTS Command
                               -------------
                           CREATE a new Time Series
                           ------------------------
                      Production of Runoff Time Series
    Project Location : Landsburg
     Computing Series : byse.tsf
```

```
Regional Scale Factor :    0.85
            Data Type : Reduced
Creating Hourly Time Series File
                    Loading Time Series File:C:\KC_SWDM\KC_DATA\LAEI60R.rnf   :
Impervious              0.15 acres
                     --------------
         Total Area :    0.15 acres
Peak Discharge:  0.069 CFS at  7:00 on Jan  9 in Year 8
                                          Storing Time Series File:byse.tsf   :
                             Time Series Computed

                                 KCRTS Command
                                 -------------
                        Enter the Analysis TOOLS Module
                        -------------------------------

                            Analysis Tools Command
                            ----------------------
                      Compute PEAKS and Flow Frequencies
                      ----------------------------------
                                     Loading Stage/Discharge curve:byse.tsf   :
          Flow Frequency Analysis
          -------------------------------------------------------
  Time Series File:byse.tsf
  Project Location:Landsburg
                                 Frequencies & Peaks saved to File:byse.pks   :

                            Analysis Tools Command
                            ----------------------
                     Compute Flow DURATION and Exceedence
                     ------------------------------------
                                          Loading Time Series File:byse.tsf   :
Computing Interval Locations
Computing Flow Durations
                      Durations & Exceedence Probabilities to File:byse.dur   :

                            Analysis Tools Command
                            ----------------------
                            RETURN to Previous Menu
                            -----------------------

                                 KCRTS Command
                                 -------------
                              eXit KCRTS Program
                              ------------------
```

Retention/Detention Facility

Type of Facility: Detention Vault
Facility Length: 120.00 ft
Facility Width: 89.00 ft
Facility Area: 10680. sq. ft
Effective Storage Depth: 4.00 ft
Stage 0 Elevation: 0.00 ft
Storage Volume: 42720. cu. ft
Riser Head: 4.00 ft
Riser Diameter: 12.00 inches
Number of orifices: 2

Orifice #	Height (ft)	Diameter (in)	Full Head Discharge (CFS)	Pipe Diameter (in)
1	0.00	1.07	0.062	
2	2.20	1.40	0.071	4.0

Top Notch Weir: None
Outflow Rating Curve: None

Stage (ft)	Elevation (ft)	Storage (cu. ft)	(ac-ft)	Discharge (cfs)	Percolation (cfs)
0.00	0.00	0.	0.000	0.000	0.00
0.01	0.01	107.	0.002	0.003	0.00
0.02	0.02	214.	0.005	0.005	0.00
0.03	0.03	320.	0.007	0.006	0.00
0.04	0.04	427.	0.010	0.006	0.00
0.06	0.06	641.	0.015	0.007	0.00
0.07	0.07	748.	0.017	0.008	0.00
0.08	0.08	854.	0.020	0.009	0.00
0.09	0.09	961.	0.022	0.009	0.00
0.19	0.19	2029.	0.047	0.013	0.00
0.29	0.29	3097.	0.071	0.017	0.00
0.39	0.39	4165.	0.096	0.019	0.00
0.49	0.49	5233.	0.120	0.022	0.00
0.59	0.59	6301.	0.145	0.024	0.00
0.69	0.69	7369.	0.169	0.026	0.00
0.79	0.79	8437.	0.194	0.027	0.00
0.89	0.89	9505.	0.218	0.029	0.00
0.99	0.99	10573.	0.243	0.031	0.00
1.09	1.09	11641.	0.267	0.032	0.00
1.19	1.19	12709.	0.292	0.034	0.00
1.29	1.29	13777.	0.316	0.035	0.00
1.39	1.39	14845.	0.341	0.036	0.00
1.49	1.49	15913.	0.365	0.038	0.00
1.59	1.59	16981.	0.390	0.039	0.00
1.69	1.69	18049.	0.414	0.040	0.00
1.79	1.79	19117.	0.439	0.041	0.00
1.89	1.89	20185.	0.463	0.042	0.00
1.99	1.99	21253.	0.488	0.043	0.00
2.09	2.09	22321.	0.512	0.044	0.00

2.19	2.19	23389.	0.537	0.046	0.00
2.20	2.20	23496.	0.539	0.046	0.00
2.21	2.21	23603.	0.542	0.046	0.00
2.23	2.23	23816.	0.547	0.047	0.00
2.24	2.24	23923.	0.549	0.050	0.00
2.26	2.26	24137.	0.554	0.053	0.00
2.27	2.27	24244.	0.557	0.056	0.00
2.29	2.29	24457.	0.561	0.061	0.00
2.30	2.30	24564.	0.564	0.064	0.00
2.32	2.32	24778.	0.569	0.065	0.00
2.42	2.42	25846.	0.593	0.073	0.00
2.52	2.52	26914.	0.618	0.079	0.00
2.62	2.62	27982.	0.642	0.084	0.00
2.72	2.72	29050.	0.667	0.089	0.00
2.82	2.82	30118.	0.691	0.093	0.00
2.92	2.92	31186.	0.716	0.098	0.00
3.02	3.02	32254.	0.740	0.101	0.00
3.12	3.12	33322.	0.765	0.105	0.00
3.22	3.22	34390.	0.789	0.109	0.00
3.32	3.32	35458.	0.814	0.112	0.00
3.42	3.42	36526.	0.839	0.116	0.00
3.52	3.52	37594.	0.863	0.119	0.00
3.62	3.62	38662.	0.888	0.122	0.00
3.72	3.72	39730.	0.912	0.125	0.00
3.82	3.82	40798.	0.937	0.128	0.00
3.92	3.92	41866.	0.961	0.131	0.00
4.00	4.00	42720.	0.981	0.133	0.00
4.10	4.10	43788.	1.005	0.444	0.00
4.20	4.20	44856.	1.030	1.010	0.00
4.30	4.30	45924.	1.054	1.740	0.00
4.40	4.40	46992.	1.079	2.540	0.00
4.50	4.50	48060.	1.103	2.820	0.00
4.60	4.60	49128.	1.128	3.080	0.00
4.70	4.70	50196.	1.152	3.320	0.00
4.80	4.80	51264.	1.177	3.540	0.00
4.90	4.90	52332.	1.201	3.740	0.00
5.00	5.00	53400.	1.226	3.940	0.00
5.10	5.10	54468.	1.250	4.130	0.00
5.20	5.20	55536.	1.275	4.310	0.00
5.30	5.30	56604.	1.299	4.480	0.00
5.40	5.40	57672.	1.324	4.640	0.00
5.50	5.50	58740.	1.348	4.800	0.00
5.60	5.60	59808.	1.373	4.950	0.00
5.70	5.70	60876.	1.398	5.100	0.00
5.80	5.80	61944.	1.422	5.250	0.00
5.90	5.90	63012.	1.447	5.390	0.00
6.00	6.00	64080.	1.471	5.530	0.00

Hyd	Inflow	Outflow		Peak		Storage	
		Target	Calc	Stage	Elev	(Cu-Ft)	(Ac-Ft)
1	1.13	*******	0.83	4.17	4.17	44508.	1.022
2	0.77	*******	0.21	4.03	4.03	42995.	0.987
3	0.64	*******	0.13	3.88	3.88	41470.	0.952
4	0.39	*******	0.09	2.72	2.72	29058.	0.667

5	0.46	*******	0.09	2.69	2.69	28708.	0.659
6	0.69	*******	0.05	2.26	2.26	24154.	0.554
7	0.80	*******	0.04	1.64	1.64	17478.	0.401
8	0.33	*******	0.03	1.13	1.13	12112.	0.278

Route Time Series through Facility
Inflow Time Series File:devw.tsf
Outflow Time Series File:rdout

Inflow/Outflow Analysis

Peak Inflow Discharge:	1.13	CFS at 7:00 on Jan 9 in Year 8
Peak Outflow Discharge:	0.825	CFS at 9:00 on Jan 9 in Year 8
Peak Reservoir Stage:	4.17	Ft
Peak Reservoir Elev:	4.17	Ft
Peak Reservoir Storage:	44508.	Cu-Ft
:	1.022	Ac-Ft

Flow Frequency Analysis
Time Series File:rdout.tsf
Project Location:Landsburg

---Annual Peak Flow Rates---			-----Flow Frequency Analysis-------				
Flow Rate (CFS)	Rank	Time of Peak	- - Peaks - - (CFS)	(ft)	Rank	Return Period	Prob
0.132	3	2/09/01 21:00	0.825	4.17	1	100.00	0.990
0.033	8	1/06/02 22:00	0.214	4.03	2	25.00	0.960
0.088	5	3/06/03 23:00	0.132	3.94	3	10.00	0.900
0.039	7	8/26/04 7:00	0.089	2.72	4	5.00	0.800
0.089	4	1/08/05 6:00	0.088	2.70	5	3.00	0.667
0.053	6	10/28/05 2:00	0.053	2.26	6	2.00	0.500
0.214	2	11/24/06 8:00	0.039	1.64	7	1.30	0.231
0.825	1	1/09/08 9:00	0.033	1.14	8	1.10	0.091
Computed Peaks			0.622	4.13		50.00	0.980

Flow Duration from Time Series File:rdout.tsf

Cutoff CFS	Count	Frequency %	CDF %	Exceedence_Probability %	
0.003	24431	39.842	39.842	60.158	0.602E+00
0.009	8273	13.492	53.333	46.667	0.467E+00
0.015	9194	14.993	68.327	31.673	0.317E+00
0.021	7851	12.803	81.130	18.870	0.189E+00
0.027	5544	9.041	90.171	9.829	0.983E-01
0.033	2637	4.300	94.472	5.528	0.553E-01
0.039	1667	2.719	97.190	2.810	0.281E-01
0.045	1093	1.782	98.973	1.027	0.103E-01
0.051	157	0.256	99.229	0.771	0.771E-02
0.057	40	0.065	99.294	0.706	0.706E-02
0.063	28	0.046	99.340	0.660	0.660E-02
0.069	68	0.111	99.450	0.550	0.550E-02
0.075	53	0.086	99.537	0.463	0.463E-02
0.081	83	0.135	99.672	0.328	0.328E-02
0.087	50	0.082	99.754	0.246	0.246E-02
0.093	22	0.036	99.790	0.210	0.210E-02

0.099	17	0.028	99.817	0.183	0.183E-02
0.105	16	0.026	99.843	0.157	0.157E-02
0.111	17	0.028	99.871	0.129	0.129E-02
0.117	16	0.026	99.897	0.103	0.103E-02
0.123	28	0.046	99.943	0.057	0.571E-03
0.129	20	0.033	99.976	0.024	0.245E-03
0.135	12	0.020	99.995	0.005	0.489E-04
0.141	0	0.000	99.995	0.005	0.489E-04
0.147	0	0.000	99.995	0.005	0.489E-04
0.153	1	0.002	99.997	0.003	0.326E-04
0.159	0	0.000	99.997	0.003	0.326E-04
0.165	0	0.000	99.997	0.003	0.326E-04
0.171	0	0.000	99.997	0.003	0.326E-04
0.177	0	0.000	99.997	0.003	0.326E-04
0.183	1	0.002	99.998	0.002	0.163E-04
0.189	0	0.000	99.998	0.002	0.163E-04
0.195	0	0.000	99.998	0.002	0.163E-04
0.201	0	0.000	99.998	0.002	0.163E-04
0.207	0	0.000	99.998	0.002	0.163E-04
0.213	0	0.000	99.998	0.002	0.163E-04

Duration Comparison Anaylsis
Base File: targw.tsf
New File: rdout.tsf
Cutoff Units: Discharge in CFS

	-----Fraction of Time-----			---------Check of Tolerance-------				
Cutoff	Base	New	%Change	Probability	Base	New	%Change
0.050	0.10E-01	0.78E-02	-22.0	0.10E-01	0.050	0.045	-9.1
0.062	0.68E-02	0.66E-02	-2.9	0.68E-02	0.062	0.060	-3.1
0.074	0.46E-02	0.48E-02	3.5	0.46E-02	0.074	0.075	1.2
0.087	0.30E-02	0.25E-02	-17.3	0.30E-02	0.087	0.082	-5.3
0.099	0.20E-02	0.19E-02	-5.7	0.20E-02	0.099	0.097	-1.9
0.111	0.15E-02	0.13E-02	-13.2	0.15E-02	0.111	0.107	-3.4
0.124	0.10E-02	0.54E-03	-47.6	0.10E-02	0.124	0.117	-5.0
0.136	0.75E-03	0.49E-04	-93.5	0.75E-03	0.136	0.121	-10.7
0.148	0.57E-03	0.49E-04	-91.4	0.57E-03	0.148	0.123	-16.8
0.160	0.33E-03	0.33E-04	-90.0	0.33E-03	0.160	0.128	-20.3
0.173	0.20E-03	0.33E-04	-83.3	0.20E-03	0.173	0.130	-24.4
0.185	0.15E-03	0.16E-04	-88.9	0.15E-03	0.185	0.131	-29.0
0.197	0.98E-04	0.16E-04	-83.3	0.98E-04	0.197	0.132	-33.3
0.209	0.16E-04	0.16E-04	0.0	0.16E-04	0.209	0.214	2.4

Maximum positive excursion = 0.008 cfs (4.1%)
occurring at 0.206 cfs on the Base Data:targw.tsf
and at 0.214 cfs on the New Data:rdout.tsf

Maximum negative excursion = 0.069 cfs (-34.4%)
occurring at 0.202 cfs on the Base Data:targw.tsf
and at 0.132 cfs on the New Data:rdout.tsf

Route Time Series through Facility
Inflow Time Series File:devw.tsf
Outflow Time Series File:rdout

Inflow/Outflow Analysis

Peak Inflow Discharge:	1.13 CFS at 7:00 on Jan 9 in Year 8	
Peak Outflow Discharge:	0.825 CFS at 9:00 on Jan 9 in Year 8	
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Peak Reservoir Elev:	4.17	Ft
Peak Reservoir Storage:	44508.	Cu-Ft
:	1.022	Ac-Ft

Flow Frequency Analysis

Time Series File:rdout.tsf

Project Location:Landsburg

---Annual Peak Flow Rates---			-----Flow Frequency Analysis-------				
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0.039	7	8/26/04 7:00	0.089	2.72	4	5.00	0.800
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0.027	5544	9.041	90.171	9.829	0.983E-01
0.033	2637	4.300	94.472	5.528	0.553E-01
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0.045	1093	1.782	98.973	1.027	0.103E-01
0.051	157	0.256	99.229	0.771	0.771E-02
0.057	40	0.065	99.294	0.706	0.706E-02
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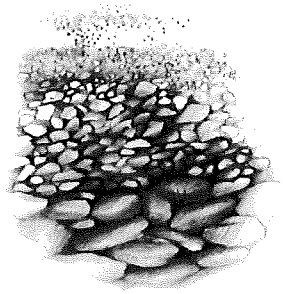
0.117	16	0.026	99.897	0.103	0.103E-02
0.123	28	0.046	99.943	0.057	0.571E-03
0.129	20	0.033	99.976	0.024	0.245E-03
0.135	12	0.020	99.995	0.005	0.489E-04
0.141	0	0.000	99.995	0.005	0.489E-04
0.147	0	0.000	99.995	0.005	0.489E-04

0.153	1	0.002	99.997	0.003	0.326E-04
0.159	0	0.000	99.997	0.003	0.326E-04
0.165	0	0.000	99.997	0.003	0.326E-04
0.171	0	0.000	99.997	0.003	0.326E-04
0.177	0	0.000	99.997	0.003	0.326E-04
0.183	1	0.002	99.998	0.002	0.163E-04
0.189	0	0.000	99.998	0.002	0.163E-04
0.195	0	0.000	99.998	0.002	0.163E-04
0.201	0	0.000	99.998	0.002	0.163E-04
0.207	0	0.000	99.998	0.002	0.163E-04
0.213	0	0.000	99.998	0.002	0.163E-04

PacLand Downstream Analysis

Associated Earth Sciences, Inc. Reports



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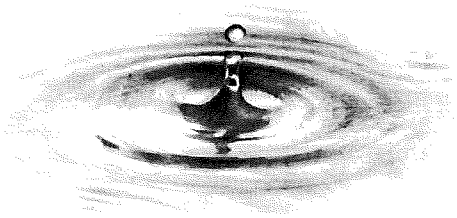

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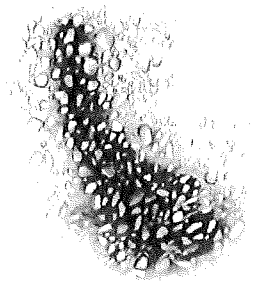
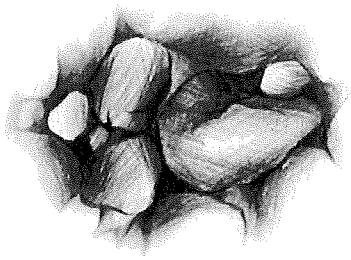


Environmental Assessments and Remediation

Sustainable Development Services



Geologic Assessments

Associated Earth Sciences, Inc.

Celebrating 25 Years of Service

Subsurface Exploration, Geologic Hazard, and Preliminary Geotechnical Engineering Report

RIO VISTA – LOT 15

Duvall, Washington

Prepared for

Rio Vista, LLC

Project No. KE060691A
May 23, 2008

Associated Earth Sciences, Inc.

Celebrating Over 25 Years of Service

May 23, 2008
Project No. KE060691A

Rio Vista, LLC
P.O. Box 1282
Bellevue, Washington 98009-1282

Attention: Mr. Mike Reid

Subject: Subsurface Exploration, Geologic Hazard, and Preliminary Geotechnical Engineering Report
Rio Vista - Lot 15
27065 NE 143rd Place
Duvall, Washington

Dear Mr. Reid:

We are pleased to present the enclosed copies of the referenced report. This report summarizes the results of our subsurface exploration, geologic hazard, and preliminary geotechnical engineering studies and offers recommendations for the preliminary design and development of the proposed project. Our recommendations are preliminary in that project plans are still under development at the time of this report.

We have enjoyed working with you on this study and are confident that the recommendations presented in this report will aid in the successful completion of your project. If you should have any questions or if we can be of additional help to you, please do not hesitate to call.

Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

Jesse P. Overton, P.G.
Project Geologist

JPO/ld
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Projects\20060691\KE\WP

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www.aesgeo.com

SUBSURFACE EXPLORATION, GEOLOGIC HAZARD, AND PRELIMINARY GEOTECHNICAL ENGINEERING REPORT

RIO VISTA – LOT 15

Duvall, Washington

Prepared for:
Rio Vista, LLC
P.O. Box 1282
Bellevue, Washington 98009-1282

Prepared by:
Associated Earth Sciences, Inc.
911 5th Avenue, Suite 100
Kirkland, Washington 98033
425-827-7701
Fax: 425-827-5424

May 23, 2008
Project No. KE060691A

I. PROJECT AND SITE CONDITIONS

1.0 INTRODUCTION

This report presents the results of our subsurface exploration, geologic hazard, and preliminary geotechnical engineering study for the proposed short-plat development. Our recommendations are preliminary in that project plans are still under development at the time of our exploration and preparation of this report. The location of the project site is shown on the "Vicinity Map," Figure 1. The existing site conditions and approximate locations of the explorations accomplished for this study are presented on the "Site and Exploration Plan," Figure 2. If there are any substantial changes in the nature, design, or location of the proposed development, the conclusions and recommendations contained in this report should be reviewed and modified, or verified, as necessary.

1.1 Purpose and Scope

The purpose of this study was to provide subsurface data to be used in the preliminary design of the project. Our study included a review of selected geologic literature, excavating exploration pits, and performing geologic studies to assess the type, thickness, distribution, and physical properties of the subsurface sediments and shallow ground water conditions. Preliminary geotechnical engineering studies were completed to formulate our recommendations for site preparation, site grading, home construction, drainage, and paving. This report summarizes our current fieldwork and offers development recommendations based on our present understanding of the project. We recommend that we be allowed to review project plans prior to construction to verify that our geotechnical recommendations have been correctly interpreted and incorporated into the design. Additional exploration or design modifications/review may be required to finalize project documentation.

1.2 Authorization

Verbal authorization to proceed with this study was granted by Mr. Mike Reid of Rio Vista, LLC during our office meeting on August 16, 2007. Our study was accomplished in general accordance with our written proposal and scope of work for geotechnical services dated July 23, 2007. This report has been prepared for the exclusive use of Rio Vista, LLC and their agents for specific application to this project.

Within the limitations of scope, schedule, and budget, our services have been performed in accordance with generally accepted geotechnical engineering and engineering geology practices in effect in this area at the time our report was prepared. No other warranty, express or implied, is made.

2.0 PROJECT AND SITE DESCRIPTION

This report was completed with an understanding of the project based on the boundary and topographic survey prepared by ESM Consulting Engineers, LLC (ESM), dated August 10, 2006, and our conversations with Mr. Mike Reid. It is our understanding that the existing residence and outbuildings will be demolished and replaced with a short-plat development consisting of an undetermined number of residential units.

The project site is located over 1.96 acres within Township 26N, Range 6E, Section 13 at 27065 NE 143rd Place in Duvall, King County, Washington. The site is bordered to the west by a private, residential lot, to the north by NE 143rd Place, to the east by 272nd Place NE, and to the south by undeveloped forested land. Site topography exhibits a gentle slope on the order of 10 percent from east to west across the property with a steep 6- to 10-foot slope down from 272nd Place NE.

At present, the site supports one single-family residence with a detached garage and outbuilding across the north half of the property with driveway access from NE 143rd Place. Vegetation across the south, undeveloped half of the property consists of a medium dense canopy of Douglas fir, western red cedar, and alder trees with a thick undergrowth of blackberries, salmonberries, and other native shrubs.

3.0 SUBSURFACE EXPLORATION

Our field study included the excavation of two exploration pits to gain subsurface information about the site. The various types of sediments, as well as the depths where characteristics of the sediments changed, are indicated on the exploration logs presented in the Appendix. The depths indicated on the logs where conditions changed may represent gradational variations between sediment types in the field.

The conclusions and recommendations presented in this report are based on the two exploration pits completed for this study. The number, locations, and depths of the explorations were completed within site and budgetary constraints. Because of the nature of exploratory work below ground, extrapolation of subsurface conditions between field explorations is necessary. It should be noted that differing subsurface conditions may sometimes be present due to the random nature of deposition and the alteration of topography by past grading and/or filling. The nature and extent of any variations between the field explorations may not become fully evident until construction. If variations are observed at that time, it may be necessary to re-evaluate specific recommendations in this report and make appropriate changes.

3.1 Exploration Pits

The exploration pits were excavated using a rubber-tired backhoe under subcontract to our firm. The pits permitted direct, visual observation of subsurface conditions. Materials encountered in the exploration pits were studied and classified in the field by an engineering geologist from our firm. All exploration pits were backfilled immediately after examination and logging. Selected samples were then transported to our laboratory for further visual classification and testing, as necessary.

4.0 SUBSURFACE CONDITIONS

Subsurface conditions at the project site were inferred from the field explorations accomplished for this study, visual reconnaissance of the site, and review of selected geologic literature. As shown on the field logs, the explorations encountered a surficial layer of topsoil over dense till soils.

4.1 Stratigraphy

Topsoil

We encountered a thin, surficial layer of topsoil within each of our exploration pits (EP-1 and EP-2). The topsoil generally consisted of loose, moist, dark brown, silty sand rich with organic material. We observed the topsoil to a maximum depth of 8 inches. This unit is not considered suitable for the support of foundations and should be removed from the foundation areas.

Lodgement Till

Natural sediments encountered below the topsoil consisted of medium dense to very dense, medium- to fine-grained sand with variable amounts of silt, gravel, and cobbles with occasional boulders. These sediments were interpreted to be representative of Vashon lodgement till. Vashon lodgement till consists of an unsorted mixture of silt, sand, gravel, and cobbles that was deposited by basal, debris-laden glacial ice during the Vashon Stade of the Fraser Glaciation approximately 12,500 to 15,000 years ago. Lodgement till typically possesses high-strength and low-compressibility attributes that are favorable for support of foundations, floor slabs, and paving with proper preparation. The high relative density characteristic of Vashon lodgement till is due to its consolidation by the massive weight of the glacial ice from which it was deposited. In the areas such as the subject site, where Vashon lodgement till sediments are exposed at or near the ground surface, the density of the upper several feet of the Vashon lodgement till typically becomes reduced to a loose to medium dense state by weathering. Where the weathering process rendered these soils loose, the weathered

till is not suitable for direct foundation support, but may be suitable for reuse as structural fill provided it can be properly moisture-conditioned and compacted to project specifications.

Our interpretations of subsurface conditions on-site are consistent with a published geologic map of the area (*Geologic Map of King County, Washington*, by Booth, Haugerud, and Sackett, 2002).

4.2 Hydrology

No ground water was encountered in either of our exploration pits at the time of our site exploration. It should be noted that fluctuations in the level of ground water may occur due to the time of the year and variations in rainfall, and may occur randomly from fill/disturbed soil layers.

II. GEOLOGIC HAZARDS AND MITIGATIONS

The following discussion of potential geologic hazards is based on the geologic, slope, and shallow ground water conditions as observed and discussed herein.

5.0 SEISMIC HAZARDS AND MITIGATION

Earthquakes occur in the Puget Lowland with great regularity. The vast majority of these events are small and are usually not felt. However, large earthquakes do occur, as evidenced by the 1949, 7.2-magnitude event; the 1965, 6.5-magnitude event; and the 2001, 6.8-magnitude event. The 1949 earthquake appears to have been the largest in this area during recorded history and was centered in the Olympia area. Evaluation of earthquake return rates in the Puget Sound area indicates that an earthquake of the magnitude between 5.5 and 6.0 will likely occur every 25 to 40 years in the Puget Sound area.

Generally, there are four types of potential geologic hazards associated with large seismic events: 1) surficial ground rupture, 2) seismically induced landslides, 3) liquefaction, and 4) ground motion. The potential for each of these hazards to adversely impact the proposed project is discussed below.

5.1 Surficial Ground Rupture

The nearest known fault trace to the project site is the Seattle Fault located approximately 12 miles to the south. Recent studies by the U.S. Geological Survey (USGS) (e.g., Johnson et al., 1994, *Origin and Evolution of the Seattle Fault and Seattle Basin, Washington,* Geology, v. 22, pp. 71-74; and Johnson et al., 1999, *Active Tectonics of the Seattle Fault and Central Puget Sound Washington - Implications for Earthquake Hazards*, Geological Society of America Bulletin, July 1999, v. 111, n. 7, pp. 1042-1053) have provided evidence of surficial ground rupture along a northern splay of the Seattle Fault. The recognition of this fault splay is relatively new, and data pertaining to it are limited, with the studies still ongoing. According to the USGS studies, the latest movement of this fault was about 1,100 years ago when about 20 feet of surficial displacement took place. This displacement can presently be seen in the form of raised, wave-cut beach terraces along Alki Point in West Seattle and Restoration Point at the south end of Bainbridge Island. The recurrence interval of movement along this fault system is still unknown, although it is hypothesized to be in excess of several thousand years. Due to the suspected long recurrence interval, the potential for surficial ground rupture is considered to be low during the expected life of the proposed development.

5.2 Seismically Induced Landslides

Based upon the relatively flat topography across the site, it is our opinion that the potential risk of damage to the proposed development by seismically induced landsliding is low provided the recommendations contained herein are properly followed.

5.3 Liquefaction

Liquefaction is a condition that occurs when loose, saturated sand is subjected to a high intensity, cyclic loading (such as an earthquake) and it loses its shear strength. The encountered stratigraphy has a low potential for liquefaction due to its relatively dense, consolidated state and lack of adverse ground water conditions.

5.4 Ground Motion

Based on the encountered stratigraphy and our visual reconnaissance of the site, it is our opinion that earthquake damage to the proposed development, when founded on suitable bearing strata in accordance with the recommendations presented in this report, would likely be caused by the intensity and acceleration associated with the event.

Guidelines presented in the 2006 *International Building Code* (IBC) standards using a Site Class "C" designation, as defined in Table 1613.5.2, were used. The 2006 IBC seismic design parameters for short period (S_S) and 1-second period (S_1) spectral acceleration values were determined by the latitude and longitude of the project site using the USGS National Seismic Hazard Mapping Project website (http://earthquake.usgs.gov/hazmaps/). Based on the 2002 data, the USGS website interpolated ground motions at the project site for the period of 0.2 seconds at 1.018 and for a 1-second period at 0.33, with a 2 percent chance of exceedence in 50 years.

6.0 EROSION HAZARDS AND MITIGATION

As of October 1, 2006, the Washington State Department of Ecology (Ecology) Construction Storm Water General Permit (also known as the National Pollutant Discharge Elimination System [NPDES] permit) requires weekly Temporary Erosion and Sedimentation Control (TESC) inspections for all sites 1 or more acres in size that discharge storm water to surface waters of the state. The TESC inspections must be completed by a Certified Erosion and Sediment Control Lead (CESCL) for the duration of the construction. TESC reports do not need to be sent to Ecology, but should be logged into the project Storm Water Pollution Prevention Plan (SWPPP). If the project does not require a SWPPP, the TESC reports should be kept in a file on-site, or by the permit holder if there is no facility on-site. Ecology also requires weekly turbidity monitoring by a CESCL of storm water leaving a site for all sites

5 acres or greater. Ecology requires a monthly summary report of the turbidity monitoring results (if performed) signed by the NPDES permit holder. If the monitored turbidity equals or exceeds 25 nephelometric turbidity units (NTU) (Ecology benchmark standard), the project best management practices (BMPs) should be modified to decrease the turbidity of storm water leaving the site. Changes and upgrades to the BMPs should be continued until the weekly turbidity reading is 25 NTU or lower. If the monitored turbidity exceeds 250 NTU, the results must be reported to Ecology within 24 hours and corrective action taken. Daily turbidity monitoring is continued until the corrective action lowers the turbidity to below 25 NTU.

In order to meet the current Ecology requirements, a properly developed, constructed, and maintained erosion control plan consistent with the City of Duvall standards and best management erosion control practices will be required for this project. Associated Earth Sciences, Inc. (AESI) is available to assist the project civil engineer in developing site-specific erosion control plans. Based on past experience, it will be necessary to make adjustments and provide additional measures to the TESC plan in order to optimize its effectiveness. Ultimately, the success of the TESC plan depends on a proactive approach to project planning and contractor implementation and maintenance.

The erosion hazard of the site soils is high. The most effective erosion control measure is the maintenance of adequate ground cover. Maintaining cover measures atop disturbed ground provides the greatest reduction to the potential generation of turbid runoff and sediment transport. During the local wet season (October 1st through March 31st), exposed soil should not remain uncovered for more than 2 days unless it is actively being worked. Ground-cover measures can include erosion control matting, plastic sheeting, straw mulch, crushed rock or recycled concrete, or mature hydroseed.

Flow-control measures are also essential for collecting and controlling the site runoff. Flow paths across slopes should be kept to less than 50 feet in order to reduce the erosion and sediment transport potential of concentrated flow. Ditch/swale spacing will need to be shortened with increasing slope gradient. Ditches and swales that exceed a gradient of about 7 to 10 percent, depending on their flow length, should have properly constructed check dams installed to reduce the flow velocity of the runoff and reduce the erosion potential within the ditch. Flow paths that are required to be constructed on gradients between 10 to 15 percent should be placed in a riprap-lined swale with the riprap properly sized for the flow conditions. Flow paths constructed on slope gradients steeper than 15 percent should be placed in a pipe slope drain. AESI is available to assist the project civil engineer in developing a suitable erosion control plan with proper flow control.

Some fine-grained surface soils are the result of natural weathering processes that have broken down parent materials into their mineral components. These mineral components can have an inherent electrical charge. Electrically charged mineral fines will attract oppositely charged particles and can combine (flocculate) to form larger particles that will settle out of suspension.

The sediments produced during the recent glaciation of Puget Sound are, however, most commonly the suspended soils that are carried by site storm water. The fine-grained fraction of the glacially derived soil is referred to as "rock flour," which is primarily a silt-sized particle with no electrical charge. These particles, once suspended in water, may have settling times in periods of months, not hours.

Therefore, the flow length within a temporary sediment control trap or pond has virtually no effect on the water quality of the discharge since it is not going to settle out of suspension in the time it takes to flow from one end of the pond to the other. Reduction of turbidity from a construction site is almost entirely a function of cover measures and flow control. Temporary sediment traps and ponds are necessary to control the release rate of the runoff and to provide a catchment for sand-sized and larger soil particles, but are very ineffective at reducing the turbidity of the runoff.

Silt fencing should be utilized as buffer protection and not as a flow-control measure. Silt fencing is meant to be placed parallel with topographic contours to prevent sediment-laden runoff from leaving a work area or entering a sensitive area. Silt fences should not be placed to cross contour lines without having separate flow control in front of the silt fence. A swale/berm combination should be constructed to provide flow control rather than let the runoff build up behind the silt fence and utilize the silt fence as the flow-control measure. Runoff flowing in front of a silt fence will cause additional erosion and usually will cause a failure of the silt fence. Improperly installed silt fencing has the potential to cause a much larger erosion hazard than if the silt fence was not installed at all. The use of silt fencing should be limited to protect sensitive areas, and swales should be used to provide flow control.

6.1 Erosion Hazard Mitigation

To mitigate and reduce the erosion hazard and potential for off-site sediment transport, we would recommend the following:

1. Surface water should not be allowed to flow across the site over unprotected surfaces.

2. Silt fences should be placed and maintained around the perimeter of the proposed construction area throughout the entire construction phase of the project until permanent landscaping and permanent storm water collection facilities have been installed.

3. Soils that are to be reused around the site should be stored in such a manner as to reduce erosion from the stockpile. Protective measures may include, but are not necessarily limited to, covering with plastic sheeting, the use of low stockpiles in flat areas behind the residence, or the use of straw bales and/or additional silt fences around

pile perimeters. Soils should not be stockpiled on the steeply sloping portions of the lot.

4. Areas stripped of natural vegetation during construction should be replanted and mulched as soon as possible or otherwise protected.

5. All storm water from impermeable surfaces, including driveways and roofs, should be permanently tightlined to a suitable storm water collection system.

6. A rocked construction entrance should be provided for truck traffic onto and off the site.

III. PRELIMINARY DESIGN RECOMMENDATIONS

7.0 INTRODUCTION

Our exploration indicates that, from a geotechnical standpoint, the proposed project is feasible provided the recommendations contained herein are properly followed. The bearing stratum appears relatively shallow across the property; therefore, we recommend the use of standard spread footing foundations to support the new structures.

8.0 SITE PREPARATION

Any existing buildings, paving, buried utilities, and any other structures should be removed from areas where structures or paving are planned. Any existing underground heating oil storage tanks (USTs) should be emptied, inerted, and removed in accordance with Ecology requirements and the resulting excavation backfilled with structural fill. AESI can assist the owner with proper UST abandonment, sampling, contaminated soil remediation (in the unlikely event that contamination is encountered), and State-required documentation.

Adequate TESC measures should be constructed in accordance with City of Duvall requirements and the project civil engineering TESC design. We recommend that the project contractor work together with the design team and the City of Duvall to design, install, and maintain the erosion control measures. It is easier and much less costly to keep fine-grained soils in place than it is to remove suspended sediment from site storm runoff.

Site preparation of planned structural fill, building, and paving areas should include removal of all trees, brush, landscaping, debris, and any other deleterious material. Additionally, the upper, organic topsoil should be removed and the remaining roots grubbed. One should refer to topsoil thicknesses depicted on the exploration logs in the Appendix for specific observed values. Topsoil materials will "swell" some 20 to 30 percent upon excavation. Project stripping volume estimates should include this swell factor. Areas where loose surficial soils exist below finished grade due to demolition or grubbing operations should be considered as fill to the depth of disturbance and treated as subsequently recommended for structural fill placement. Topsoil should be processed and reused as topsoil, if allowed by the project plans and specifications, or should be removed from the site or used as fill in non-structural areas. If some minor settlement can be tolerated, the topsoil could be placed as fill in non-structural areas as long as it is free of stumps and roots larger than 3 inches in diameter and is compacted to a firm and unyielding condition in lifts, as described for structural fill. We should be involved in the planning of the location and thickness of all fill and should observe the placement and compaction operations.

8.1 Site Drainage and Surface Water Control

Adequate temporary and permanent control of surface water runoff will be required in order to allow site access and grading for construction of the new buildings, access driveways, installation of underground utilities, and other proposed improvements. Excavation, filling, subgrade, and grade preparation should be performed in a manner and sequence that will provide controlled drainage at all times and proper control of erosion. Surface water should be collected and pumped or drained to provide a suitable working platform. Successful drainage of wet zones due to ground water flow and accumulations of surface water runoff could be accomplished by ditching and/or the installation of cut-off trenches or "French" drains, where necessary.

The site should be graded to prevent water from ponding in construction areas and/or flowing into excavations. Exposed grades should be crowned, sloped, and smooth-drum rolled at the end of each day to facilitate drainage. Accumulated water must be removed from subgrades and work areas immediately prior to performing further work in the area. Equipment access may be limited, and the amount of soil rendered unfit for use as structural fill may be greatly increased if drainage efforts are not accomplished in a timely sequence. If an effective drainage system is not utilized, project delays and increased costs could be incurred due to the greater quantities of wet and unsuitable fill, or poor access and unstable conditions.

Final exterior grades should promote free and positive drainage away from all buildings at all times. Water must not be allowed to pond or to collect adjacent to foundations or within the immediate building areas. We recommend that a gradient of at least 3 percent for a minimum distance of 10 feet from the building perimeters be provided, except in paved locations. In paved locations, a minimum gradient of 1 percent should be provided unless provisions are included for collection and disposal of surface water adjacent to all structures.

8.2 Wet Weather Conditions

If construction proceeds during an extended wet weather construction period and the moisture-sensitive, silty site soils become wet, they will become unstable. Therefore, the bids for site grading operations should be based upon the time of year that construction will proceed. Construction during wet weather is expected to require protection of subgrades in staging and construction areas, as recommended in the "Subgrade Protection" section of this report. It is expected that in wet conditions, additional soils may need to be removed and/or other stabilization methods used, such as a coarse, crushed rock working mat to develop a stable condition if silty subgrade soils are disturbed in the presence of excess moisture. The severity of construction problems will be dependent, in part, on the precautions that are taken by the contractor to protect the moisture- and disturbance-sensitive site soils. If overexcavation is necessary, it should be confirmed through continuous observation and testing by a representative of our firm. The site contractor should provide properly surfaced access roads

to fill areas. Maintenance and reconstruction of access roads will be necessary. Daily "sealing" of fill surfaces and drainage improvements is required. It is the contractor's responsibility to keep silt-laden runoff to a minimum by protecting exposed subgrades and maintaining erosion control measures.

8.3 Subgrade Protection

The site soils that are expected below the building pad and paving subgrades contain a significant silt fraction and are considered to be moisture- and disturbance-sensitive. These soils will become unstable if disturbed by construction equipment while at elevated moisture contents, requiring additional soil removal at an increased cost. Therefore, in addition to the recommendations presented in the "Site Drainage and Surface Water Control" section of this report, site preparation and initial construction activities should be planned to minimize disturbance to the existing ground surface, particularly during extended wet weather periods and the wet season (typically October through May). Construction traffic should be restricted to specific rock-surfaced drive areas to limit the area where disturbance of the subgrade will occur. If site stripping and grading activities are performed during extended dry weather periods, we anticipate that site stabilization requirements will be much less.

If construction will proceed in the winter, we recommend the use of a temporary working surface of sand and gravel, crushed rock, or quarry spalls to protect the silty soils, particularly in areas supporting concentrated equipment traffic. In winter construction staging areas, a minimum thickness of 12 inches of quarry spalls or 18 inches of pit run sand and gravel is recommended. If subgrade conditions are soft and silty, a geotextile separation fabric, such as Mirafi 500x or approved equivalent, should be used between the subgrade and the new fill. For building pads where floor slabs and foundation construction will be completed in the winter, a similar working surface, composed of at least 12 inches of pit run sand and gravel or crushed rock, should be used. Construction of working surfaces from advancing fill pads could be used to avoid directly exposing the subgrade soils to vehicular traffic. Similar minimum (or greater) sections will be required for temporary access road construction.

Foundation subgrades may require protection from foot and equipment traffic, and ponding of runoff during wet weather conditions. Typically, compacted crushed rock or a lean-mix concrete mat placed over a properly prepared subgrade provides adequate subgrade protection. Foundation concrete should be placed and excavations backfilled as soon as possible to protect the bearing grade.

8.4 Proof-Rolling and Subgrade Compaction

Following the recommended demolition, site stripping procedures, and required excavation to grade, the stripped subgrade within the building and pavement areas should be proof-rolled with heavy, rubber-tired construction equipment, such as a fully loaded, tandem-axle dump

truck. Proof-rolling should be performed prior to structural fill placement or foundation excavation. The proof-roll should be observed by the geotechnical engineer so that any soft or yielding subgrade soils can be identified. Any soft/loose, yielding soils should be removed to a stable subgrade. The subgrade should then be scarified, adjusted in moisture content, and recompacted to the required density. Proof-rolling should only be attempted if soil moisture contents are at or near optimum moisture content. Proof-rolling of wet subgrades could result in further degradation. Low areas and excavations may then be raised to the planned finished grade with compacted structural fill. Subgrade preparation and selection, placement, and compaction of structural fill should be performed under engineering-controlled conditions in accordance with the project specifications.

8.5 Overexcavation/Stabilization

Construction during extended wet weather periods could create the need to overexcavate exposed soils if they become disturbed and cannot be recompacted due to elevated moisture content and/or weather conditions. During dry weather periods, soft/wet soils, which may need to be overexcavated, may be encountered in some portions of the site. If necessary, it should be confirmed through continuous observation and testing by AESI. Soils that have become unstable may require remedial measures in the form of one or more of the following:

1. Drying and recompaction. Selective drying may be accomplished by scarifying or windrowing surficial material during extended periods of dry and warm weather.

2. Removal of affected soils to expose a suitable bearing subgrade and replacement with compacted structural fill.

3. Mechanical stabilization with a coarse, crushed aggregate compacted into the subgrade, possibly in conjunction with a geotextile.

4. Admixture stabilization with cement powder. Admixture design and installation procedures need to be reviewed and approved by the design team and City prior to site use.

8.6 Temporary and Permanent Cut Slopes

In our opinion, stable construction slopes should be the responsibility of the contractor and should be determined during construction. For estimating purposes, however, we anticipate that temporary, unsupported cut slopes in the upper, weathered till section should not exceed a slope of 1H:1V (Horizontal:Vertical). Unsupported cut slopes within the unsaturated, very dense lodgement till at depth can be made at a maximum slope of 0.5H:1V or flatter. As is typical with earthwork operations, some sloughing and raveling may occur, and cut slopes may have to be adjusted in the field. If ground water seepage is encountered in cut slopes or if

surface water is not routed away from temporary cut slope faces, flatter slopes will be required. In addition, WISHA/OSHA regulations should be followed at all times. Permanent cut and structural fill slopes that are not intended to be exposed to surface water should be designed at inclinations of 2H:1V or flatter. Slopes that are intended to be exposed to surface water, such as bioswale side slopes, if planned, should be designed at inclinations of 3H:1V or flatter. All permanent cut or fill slopes should be compacted to at least 95 percent of the modified Proctor maximum dry density, as determined by *American Society for Testing and Materials* (ASTM):D 1557, and the slopes should be protected from erosion by sheet plastic until vegetation cover can be established during favorable weather.

8.7 Frozen Subgrades

If earthwork takes place during freezing conditions, all exposed subgrades should be allowed to thaw and then be recompacted prior to placing subsequent lifts of structural fill or foundation components. Alternatively, the frozen material could be stripped from the subgrade to reveal unfrozen soil prior to placing subsequent lifts of fill or foundation components. The frozen soil should not be reused as structural fill until allowed to thaw and adjusted to the proper moisture content, which may not be possible during winter months.

9.0 STRUCTURAL FILL

Structural fill may be necessary to establish desired grades. All references to structural fill in this report refer to subgrade preparation, fill type, placement, and compaction of materials, as discussed in this section. If a percentage of compaction is specified under another section of this report, the value given in that section should be used.

After stripping, planned excavation, and any required overexcavation have been performed to the satisfaction of the geotechnical engineer/engineering geologist, the surface should be recompacted to a firm and unyielding condition. If the subgrade contains too much moisture, adequate recompaction may be difficult or impossible to obtain, and should probably not be attempted. In lieu of recompaction, the area to receive fill should be blanketed with washed rock or quarry spalls to act as a capillary break between the new fill and the wet subgrade. Where the exposed ground remains soft and further overexcavation is impractical, placement of an engineering stabilization fabric may be necessary to prevent contamination of the free-draining layer by silt migration from below.

After recompaction of the exposed ground is approved, or a free-draining rock course is laid, structural fill may be placed to attain desired grades. Structural fill is defined as non-organic soil, acceptable to the geotechnical engineer, placed in maximum 8-inch loose lifts, with each lift being compacted to 95 percent of ASTM:D 1557. In the case of roadway and utility trench filling, the backfill should be placed and compacted in accordance with City of Duvall codes

and standards. The top of the compacted fill should extend horizontally outward a minimum distance of 3 feet beyond the locations of the perimeter footings or roadway edges before sloping down at a maximum angle of 2H:1V.

The contractor should note that any proposed fill soils must be evaluated by AESI prior to their use in fills. This would require that we have a sample of the material 72 hours in advance of filling activities to perform a Proctor test and determine its field compaction standard. Soils in which the amount of fine-grained material (smaller than the No. 200 sieve) is greater than approximately 5 percent (measured on the minus No. 4 sieve size) should be considered moisture-sensitive. Use of moisture-sensitive soil in structural fills should be limited to favorable dry weather conditions. The on-site, native till soils contain substantial amounts of silt and fine sand and are considered highly moisture-sensitive when excavated and used as fill materials. We anticipate that all excavated site soils will require aeration and drying prior to compaction in structural fill applications. Construction equipment traversing the site when the soils are wet can cause considerable disturbance.

If fill is placed during wet weather or if proper compaction cannot be obtained, a select import material consisting of a clean, free-draining gravel and/or sand should be used. Free-draining fill consists of non-organic soil with the amount of fine-grained material limited to 5 percent by weight when measured on the minus No. 4 sieve fraction and at least 25 percent retained on the No. 4 sieve.

10.0 HOUSE FOUNDATIONS

Spread footings may be used for building support when founded upon the native, medium dense till soils prepared as recommended in this report. Based on our observations, suitable foundation bearing soils are expected approximately 2 feet below the existing ground surface. If existing fill is discovered around existing structures, it should be removed and replaced with structural fill, which is also suitable for foundation support.

The footings for the proposed development may be designed for an allowable foundation soil bearing pressure of 2,500 pounds per square foot (psf), including both dead and live loads. An increase of one-third may be used for short-term wind or seismic loading. Perimeter footings should be buried at least 18 inches into the surrounding soil for frost protection. However, all foundation elements must penetrate to the prescribed bearing stratum, and no foundation elements should be constructed in or above loose, organic, or existing fill soils.

Anticipated settlement of footings founded as recommended should be on the order of ¾ inch or less, with differential settlement of ½ inch or less. However, disturbed material not removed from footing trenches prior to footing placement could result in increased settlements. All footing areas should be inspected by AESI prior to placing concrete to verify that the

foundation subgrades are undisturbed and construction conforms to the recommendations contained in this report. Such inspections may be required by City of Duvall. Perimeter footing drains should be provided as discussed under the "Drainage Considerations" section of this report.

It should be noted that the area bounded by lines extending downward at 1H:1V from any footing must not intersect another footing or intersect a filled area that has not been compacted to at least 95 percent of ASTM:D 1557. In addition, a 1.5H:1V line extending down and away from any footing must not daylight because sloughing or raveling may eventually undermine the footing. Thus, footings should not be placed near the edge of steps or cuts in the bearing soils.

11.0 FLOOR SUPPORT

We anticipate the use of concrete, slab-on-grade floors within the proposed residential units. The concrete, slab-on-grade floor should be cast atop a minimum of 4 inches of pea gravel to act as a capillary break. The floors should also be protected from dampness by covering the capillary break layer with an impervious moisture barrier at least 10 mils in thickness. Floor slabs may be supported by native soils.

12.0 DRAINAGE CONSIDERATIONS

All retaining and perimeter footing walls should be provided with a drain at the footing elevation. Drains should consist of rigid, perforated, polyvinyl chloride (PVC) pipe surrounded by washed pea gravel or drain rock. The level of the perforations in the pipe should be set approximately 2 inches below the bottom of the footing and should be constructed with sufficient gradient to allow gravity discharge away from the buildings. In addition, all retaining walls should be lined with a minimum, 12-inch-thick, washed gravel or washed crushed rock blanket provided over the full height of the wall that ties into the footing drain. Roof and surface runoff should not discharge into the footing drain system, but should be handled by a separate, rigid, tightline drain. In planning, exterior grades adjacent to walls should be sloped downward away from the structures to achieve surface drainage.

13.0 PROJECT DESIGN AND CONSTRUCTION MONITORING

At the time of this report, site grading, structural plans, and construction methods have not been finalized, and the recommendations presented herein are preliminary. We are available to provide additional geotechnical consultation as the project design develops and possibly changes from that upon which this report is based. We recommend that AESI perform a

geotechnical review of the plans prior to final design completion. In this way, our earthwork and foundation recommendations may be properly interpreted and implemented in the design.

We are also available to provide geotechnical engineering and monitoring services during construction. The integrity of the foundations for buildings and of new pavement depends on proper site preparation and construction procedures. In addition, engineering decisions may have to be made in the field in the event that variations in subsurface conditions become apparent. Construction monitoring services are not part of the current scope of work. If these services are desired, please let us know, and we will prepare a cost proposal.

We have enjoyed working with you on this study and are confident that these recommendations will aid in the successful completion of your project. If you should have any questions, or require further assistance, please do not hesitate to call.

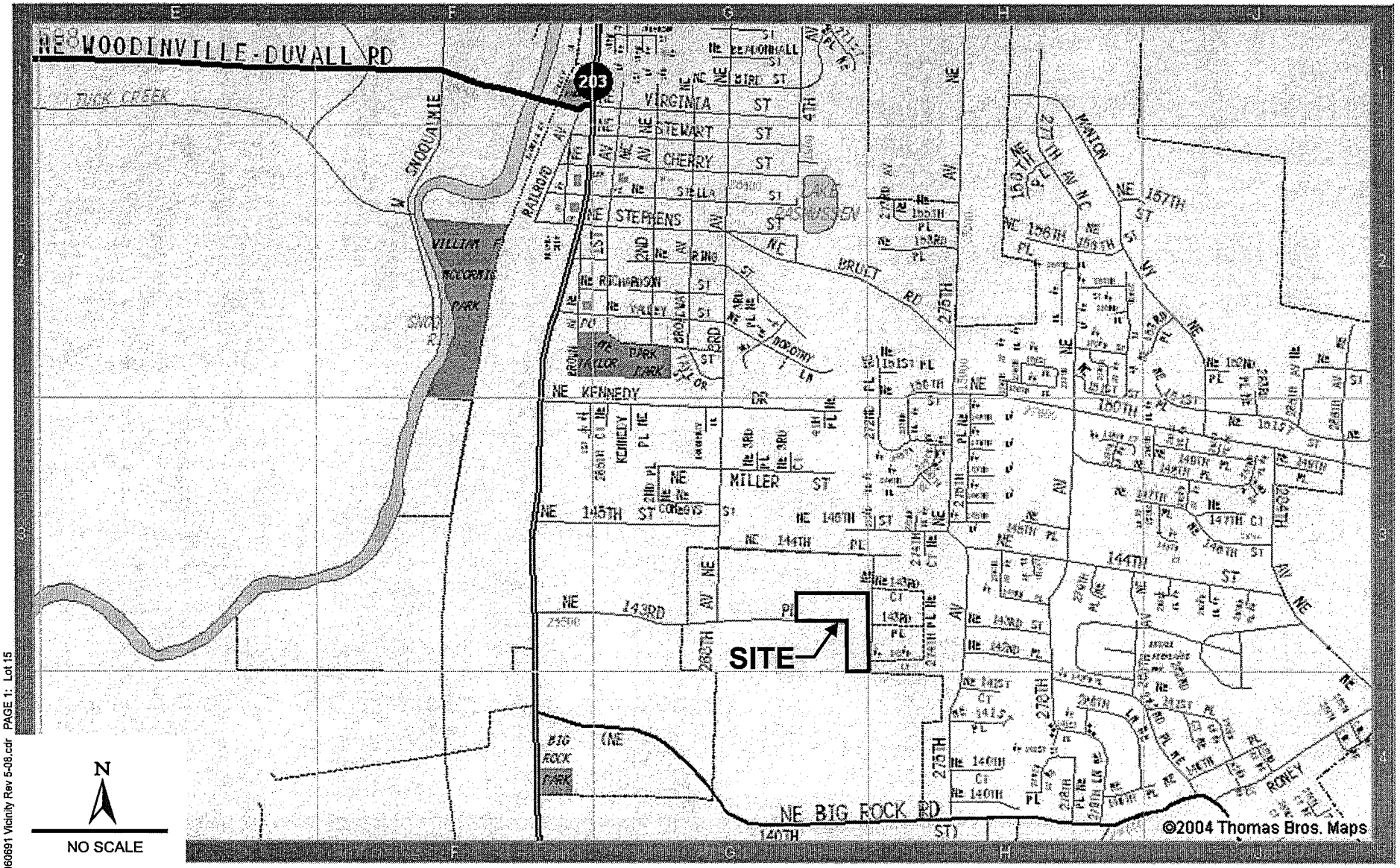
Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

MATTHEW A. MILLER
STATE OF WASHINGTON
36234
REGISTERED
PROFESSIONAL ENGINEER
5-23-08
EXPIRES 2/6/10

Jesse P. Overton, P.G.
Project Geologist

Matthew A. Miller, P.E.
Associate Engineer

Attachments: Figure 1: Vicinity Map
Figure 2: Site and Exploration Plan
Appendix: Exploration Pit Logs



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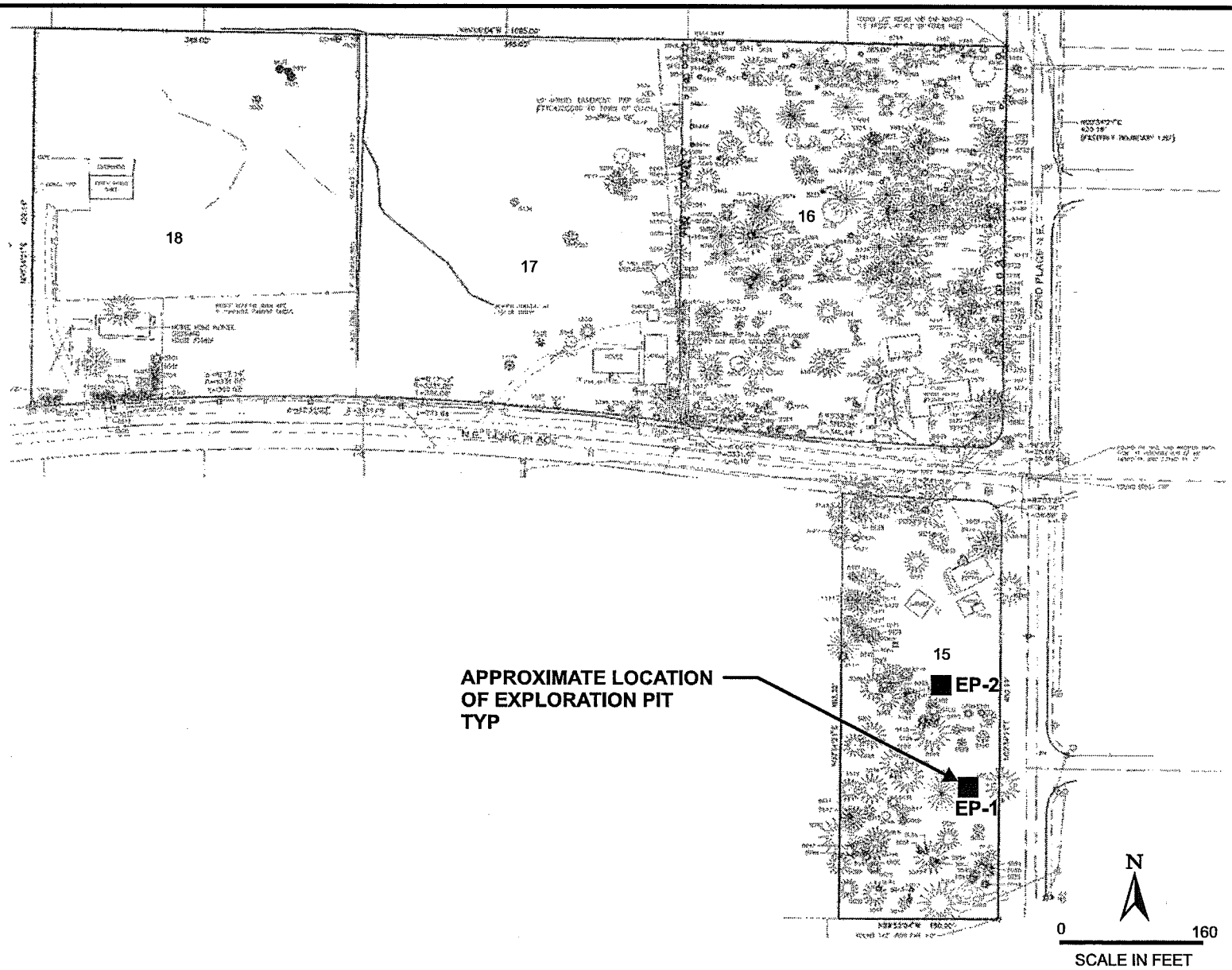
VICINITY MAP
RIO VISTA - LOT 15
DUVALL, WASHINGTON

FIGURE 1

DATE 5/08

PROJ. NO. KE060691A

060691 Rio Vista-Lots 15-18 \ 060691 Vicinity Rev 5-08.cdr PAGE 1: Lot 15



Reference: ESM

Associated Earth Sciences, Inc.

SITE AND EXPLORATION PLAN
RIO VISTA LOT 15
DUVALL, WASHINGTON

FIGURE 2

DATE 5/08

PROJ. NO. KE060691A

060691 Rio Vista Lots 15-18 \ 060691 Site and Explr-A-Rev 5-08.cdr

APPENDIX

Coarse-Grained Soils - More than 50%[1] Retained on No. 200 Sieve	Gravels - More than 50%[1] of Coarse Fraction Retained on No. 4 Sieve	≤5% Fines[5]	GW	Well-graded gravel and gravel with sand, little to no fines
			GP	Poorly-graded gravel and gravel with sand, little to no fines
		≥15% Fines[5]	GM	Silty gravel and silty gravel with sand
			GC	Clayey gravel and clayey gravel with sand
	Sands - 50%[1] or More of Coarse Fraction Passes No. 4 Sieve	≤5% Fines[5]	SW	Well-graded sand and sand with gravel, little to no fines
			SP	Poorly-graded sand and sand with gravel, little to no fines
		≥15% Fines[5]	SM	Silty sand and silty sand with gravel
			SC	Clayey sand and clayey sand with gravel
Fine-Grained Soils - 50%[1] or More Passes No. 200 Sieve	Silts and Clays Liquid Limit Less than 50		ML	Silt, sandy silt, gravelly silt, silt with sand or gravel
			CL	Clay of low to medium plasticity; silty, sandy, or gravelly clay, lean clay
			OL	Organic clay or silt of low plasticity
	Silts and Clays Liquid Limit 50 or More		MH	Elastic silt, clayey silt, silt with micaceous or diatomaceous fine sand or silt
			CH	Clay of high plasticity, sandy or gravelly clay, fat clay with sand or gravel
			OH	Organic clay or silt of medium to high plasticity
Highly Organic Soils			PT	Peat, muck and other highly organic soils

Terms Describing Relative Density and Consistency

	Density	SPT[2] blows/foot
Coarse-Grained Soils	Very Loose	0 to 4
	Loose	4 to 10
	Medium Dense	10 to 30
	Dense	30 to 50
	Very Dense	>50

	Consistency	SPT[2] blows/foot
Fine-Grained Soils	Very Soft	0 to 2
	Soft	2 to 4
	Medium Stiff	4 to 8
	Stiff	8 to 15
	Very Stiff	15 to 30
	Hard	>30

Test Symbols

G = Grain Size
M = Moisture Content
A = Atterberg Limits
C = Chemical
DD = Dry Density
K = Permeability

Component Definitions

Descriptive Term	Size Range and Sieve Number
Boulders	Larger than 12"
Cobbles	3" to 12"
Gravel	3" to No. 4 (4.75 mm)
Coarse Gravel	3" to 3/4"
Fine Gravel	3/4" to No. 4 (4.75 mm)
Sand	No. 4 (4.75 mm) to No. 200 (0.075 mm)
Coarse Sand	No. 4 (4.75 mm) to No. 10 (2.00 mm)
Medium Sand	No. 10 (2.00 mm) to No. 40 (0.425 mm)
Fine Sand	No. 40 (0.425 mm) to No. 200 (0.075 mm)
Silt and Clay	Smaller than No. 200 (0.075 mm)

[3] Estimated Percentage

Component	Percentage by Weight
Trace	<5
Few	5 to 10
Little	15 to 25
With	- Non-primary coarse constituents: ≥ 15% - Fines content between 5% and 15%

Moisture Content

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

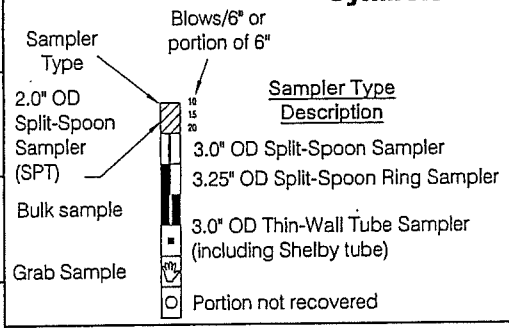
Slightly Moist - Perceptible moisture

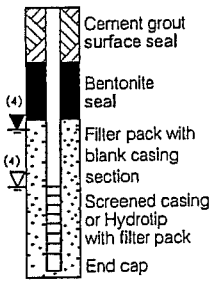
Moist - Damp but no visible water

Very Moist - Water visible but not free draining

Wet - Visible free water, usually from below water table

Symbols





[1] Percentage by dry weight
[2] (SPT) Standard Penetration Test (ASTM D-1586)
[3] In General Accordance with Standard Practice for Description and Identification of Soils (ASTM D-2488)
[4] Depth of ground water
ATD = At time of drilling
Static water level (date)
[5] Combined USCS symbols used for fines between 5% and 15%

Classifications of soils in this report are based on visual field and/or laboratory observations, which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field or laboratory testing unless presented herein. Visual-manual and/or laboratory classification methods of ASTM D-2487 and D-2488 were used as an identification guide for the Unified Soil Classification System.

Associated Earth Sciences, Inc.

EXPLORATION LOG KEY

FIGURE A1

blockslog_key.dwg LAYOUT: Layout2

LOG OF EXPLORATION PIT NO. EP-1

Depth (ft)	This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered. **DESCRIPTION**
	Topsoil
1	**Weathered Till** Medium dense, moist, brown, silty fine SAND, trace gravel.
2	
3	**Vashon Till** Dense, moist, gray, silty SAND, little gravel.
4	
5	
6	More dense with depth.
7	
8	
9	Bottom of exploration pit at depth 8 feet Very difficult digging.
10	
11	
12	
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19	
20	

Rio Vista, Lot 15
Duvall, WA

Associated Earth Sciences, Inc.

Logged by: JPO

Approved by:

Project No. KE060691A

May 2008

KCTP3 060691A.GPJ May 20, 2008

LOG OF EXPLORATION PIT NO. EP-2

Depth (ft)	This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered. **DESCRIPTION**
	Topsoil
1	**Vashon Till**
	Dense, moist, gray, silty SAND, little gravel.
2	Upper 1 1/2 feet less dense, weathered.
3	
4	
5	
6	More dense with depth.
7	
8	
9	Bottom of exploration pit at depth 8 feet Very difficult digging.
10	
11	
12	
13	
14	
15	
16	
17	
18	
19	
20	

Rio Vista, Lot 15
Duvall, WA

Associated Earth Sciences, Inc.

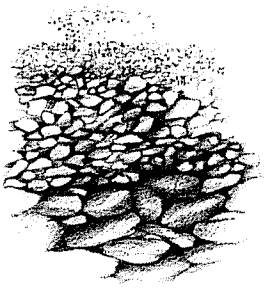
Logged by: JPO

Approved by:

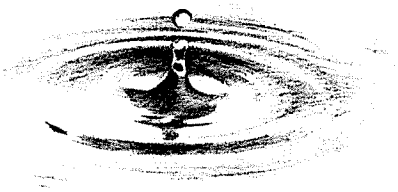
Project No. KE060691A

May 2008

KCTP3 060691A.GPJ May 20, 2008



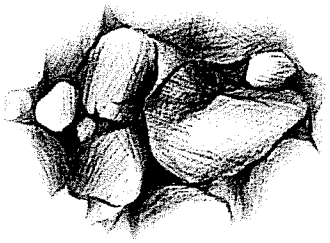
Geotechnical Engineering



Water Resources

Environmental Assessments and Remediation

Sustainable Development Services



Geologic Assessments

Associated Earth Sciences, Inc.

Celebrating 25 Years of Service

Subsurface Exploration, Geologic Hazard, and Preliminary Geotechnical Engineering Report

RIO VISTA – LOT 16

Duvall, Washington

Prepared for

Rio Vista, LLC

Project No. KE060691B
August 7, 2008

Associated Earth Sciences, Inc.

Celebrating Over 25 Years of Service

August 7, 2008
Project No. KE060691B

Rio Vista, LLC
P.O. Box 1282
Bellevue, Washington 98009-1282

Attention: Mr. Mike Reid

Subject: Subsurface Exploration, Geologic Hazard, and Preliminary Geotechnical Engineering Report
Rio Vista - Lot 16
27066 NE 143rd Place
Duvall, Washington

Dear Mr. Reid:

We are pleased to present the enclosed copies of the referenced report. This report summarizes the results of our subsurface exploration, geologic hazard, and preliminary geotechnical engineering studies and offers recommendations for the preliminary design and development of the proposed project. Our recommendations are preliminary in that project plans are still under development at the time of this report.

We have enjoyed working with you on this study and are confident that the recommendations presented in this report will aid in the successful completion of your project. If you should have any questions or if we can be of additional help to you, please do not hesitate to call.

Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

FOR

Jesse P. Overton, P.G.
Project Geologist'

JPO/tb
KE060691B1
Projects\20060691\KE\WP

Kirkland ▪ Everett ▪ Tacoma
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SUBSURFACE EXPLORATION, GEOLOGIC HAZARD, AND PRELIMINARY GEOTECHNICAL ENGINEERING REPORT

RIO VISTA
LOT 16

Duvall, Washington

Prepared for:
Rio Vista, LLC
P.O. Box 1282
Bellevue, Washington 98009-1282

Prepared by:
Associated Earth Sciences, Inc.
911 5th Avenue, Suite 100
Kirkland, Washington 98033
425-827-7701
Fax: 425-827-5424

August 7, 2008
Project No. KE060691B

I. PROJECT AND SITE CONDITIONS

1.0 INTRODUCTION

This report presents the results of our subsurface exploration, geologic hazard, and preliminary geotechnical engineering study for the proposed short-plat development. Our recommendations are preliminary in that project plans are still under development at the time of our exploration and preparation of this report. The location of the project site is shown on the "Vicinity Map," Figure 1. The existing site conditions and approximate locations of the explorations accomplished for this study are presented on the "Site and Exploration Plan," Figure 2. If there are any substantial changes in the nature, design, or location of the proposed development, the conclusions and recommendations contained in this report should be reviewed and modified, or verified, as necessary.

1.1 Purpose and Scope

The purpose of this study was to provide subsurface data to be used in the preliminary design of the project. Our study included a review of selected geologic literature, excavating exploration pits, and performing geologic studies to assess the type, thickness, distribution, and physical properties of the subsurface sediments and shallow ground water conditions. Preliminary geotechnical engineering studies were completed to formulate our recommendations for site preparation, site grading, home construction, drainage, and paving. This report summarizes our current fieldwork and offers development recommendations based on our present understanding of the project. We recommend that we be allowed to review project plans prior to construction to verify that our geotechnical recommendations have been correctly interpreted and incorporated into the design. Additional exploration or design modifications/review may be required to finalize project documentation.

1.2 Authorization

Verbal authorization to proceed with this study was granted by Mr. Mike Reid of Rio Vista, LLC during our office meeting on August 16, 2007. Our study was accomplished in general accordance with our written proposal and scope of work for geotechnical services dated July 23, 2007. This report has been prepared for the exclusive use of Rio Vista, LLC and their agents for specific application to this project.

Within the limitations of scope, schedule, and budget, our services have been performed in accordance with generally accepted geotechnical engineering and engineering geology practices in effect in this area at the time our report was prepared. No other warranty, express or implied, is made.

2.0 PROJECT AND SITE DESCRIPTION

This report was completed with an understanding of the project based on the boundary and topographic survey prepared by ESM Consulting Engineers, LLC (ESM) dated August 10, 2006, and our conversations with Mr. Mike Reid. It is our understanding that the existing residence and outbuildings will be demolished and replaced with a short-plat development consisting of an undetermined number of dwelling units.

The project site is located over 3.7 acres within Township 26N, Range 6E, Section 13 at 27066 NE 143rd Place in Duvall, King County, Washington. The site is bordered to the west and north by private, residential lots, to the south by NE 143rd Place, and to the east by 272nd Place NE. There exists a 10-foot wide, City of Duvall sewer easement along the west property line. Site topography exhibits a gentle slope on the order of 10 percent from east to west across the property.

At present, the site supports one single-family residence with a detached garage/shop building within the southeast corner of the property with driveway access from NE 143rd Place. Vegetation across the property consists of a dense canopy of Douglas Fir, Western Red Cedar, and Hemlock trees with a limited undergrowth of blackberries, salmonberries, and other native shrubs.

3.0 SUBSURFACE EXPLORATION

Our field study included the excavation of three exploration pits to gain subsurface information about the site. The various types of sediments, as well as the depths where characteristics of the sediments changed, are indicated on the exploration logs presented in the Appendix. The depths indicated on the logs where conditions changed may represent gradational variations between sediment types in the field.

The conclusions and recommendations presented in this report are based on the two exploration pits completed for this study. The number, locations, and depths of the explorations were completed within site and budgetary constraints. Because of the nature of exploratory work below ground, extrapolation of subsurface conditions between field explorations is necessary. It should be noted that differing subsurface conditions may sometimes be present due to the random nature of deposition and the alteration of topography by past grading and/or filling. The nature and extent of any variations between the field explorations may not become fully evident until construction. If variations are observed at that time, it may be necessary to re-evaluate specific recommendations in this report and make appropriate changes.

3.1 Exploration Test Pits

The exploration pits were excavated using a rubber-tired backhoe under sub-contract to our firm. The pits permitted direct, visual observation of subsurface conditions. Materials encountered in the exploration pits were studied and classified in the field by an engineering geologist from our firm. All exploration pits were backfilled immediately after examination and logging. Selected samples were then transported to our laboratory for further visual classification and testing, as necessary.

4.0 SUBSURFACE CONDITIONS

Subsurface conditions at the project site were inferred from the field explorations accomplished for this study, visual reconnaissance of the site, and review of selected geologic literature. As shown on the field logs, the explorations encountered a surficial layer of topsoil over dense till soils.

4.1 Stratigraphy

Topsoil

We encountered a thin, surficial layer of topsoil within each of our exploration pits EP-3 through EP-5. The topsoil generally consisted of loose, moist, dark brown, silty sand, rich with organic material. We observed the topsoil to a maximum depth of 8 inches. This unit is not considered suitable for the support of foundations and should be removed from the foundation areas.

Lodgement Till

Natural sediments encountered below the topsoil consisted of medium dense to very dense, medium to fine grained sand with variable amounts of silt, gravel, and cobbles with occasional boulders. These sediments were interpreted to be representative of Vashon lodgement till. Vashon lodgement till consists of an unsorted mixture of silt, sand, gravel, and cobbles that was deposited by basal, debris-laden glacial ice during the Vashon Stade of the Fraser Glaciation approximately 12,500 to 15,000 years ago. Lodgement till typically possesses high strength and low compressibility attributes that are favorable for support of foundations, floor slabs, and paving with proper preparation. The high relative density characteristic of Vashon lodgement till is due to its consolidation by the massive weight of the glacial ice from which it was deposited. In the areas such as the subject site, where Vashon lodgement till sediments are exposed at or near the ground surface, the density of the upper several feet of the Vashon lodgement till typically becomes reduced to a loose to medium dense state by weathering. Where the weathering process rendered these soils loose, the weathered till is not suitable for

direct foundation support, but may be suitable for reuse as structural fill provided it can be properly moisture conditioned and compacted to project specifications.

Vashon Advance Outwash Deposits

Underlying the lodgement till sediments at a depth of 5 feet within exploration pit EP-4 were sand and gravel sediments that were deposited during the advance of the Vashon ice sheet. The outwash sediments were found to be dense with depth and varied in grain size distribution. The outwash sediments typically consisted of a fine to coarse sand with some gravel. The outwash sediments extended beyond the termination depths of our exploration pits.

Our interpretations of subsurface conditions on-site are consistent with a published geologic map of the area (*Geologic Map of King County, Washington,* by Booth, Haugerud, and Sackett, 2002).

4.2 Hydrology

Ground water seepage was observed within exploration pit EP-4 at the time of our field study. Ground water seepage was observed within the Advance Outwash deposits at a depth of 11 feet. We interpreted this level of ground water as representative of the seasonal ground water table at this location. Specific depths, locations, and rates of ground water seepage are indicated on the exploration logs included in the Appendix. It should be noted that the depth and occurrence of ground water seepage at the site may vary in response to such factors as changes in season, precipitation, and site use.

II. GEOLOGIC HAZARDS AND MITIGATIONS

The following discussion of potential geologic hazards is based on the geologic, slope, and shallow ground water conditions as observed and discussed herein.

5.0 SEISMIC HAZARDS AND RECOMMENDED MITIGATION

Earthquakes occur in the Puget Lowland with great regularity. The vast majority of these events are small and are usually not felt. However, large earthquakes do occur as evidenced by the 1949, 7.2-magnitude event; the 1965, 6.5-magnitude event; and the 2001, 6.8-magnitude event. The 1949 earthquake appears to have been the largest in this area during recorded history and was centered in the Olympia area. Evaluation of earthquake return rates in the Puget Sound area indicates that an earthquake of the magnitude between 5.5 and 6.0 will likely occur every 25 to 40 years in the Puget Sound area.

Generally, there are four types of potential geologic hazards associated with large seismic events: 1) surficial ground rupture, 2) seismically induced landslides, 3) liquefaction, and 4) ground motion. The potential for each of these hazards to adversely impact the proposed project is discussed below.

5.1 Surficial Ground Rupture

The nearest known fault trace to the project site is the Seattle Fault located approximately 12 miles to the south. Recent studies by the U.S. Geological Survey (USGS) (e.g., Johnson et al., 1994, *Origin and Evolution of the Seattle Fault and Seattle Basin, Washington,* Geology, v. 22, pp. 71-74; and Johnson et al., 1999, *Active Tectonics of the Seattle Fault and Central Puget Sound Washington – Implications for Earthquake Hazards*, Geological Society of America Bulletin, July 1999, v. 111, n. 7, pp. 1042-1053) have provided evidence of surficial ground rupture along a northern splay of the Seattle Fault. The recognition of this fault splay is relatively new, and data pertaining to it are limited, with the studies still ongoing. According to the USGS studies, the latest movement of this fault was about 1,100 years ago when about 20 feet of surficial displacement took place. This displacement can presently be seen in the form of raised, wave-cut beach terraces along Alki Point in West Seattle and Restoration Point at the south end of Bainbridge Island. The recurrence interval of movement along this fault system is still unknown, although it is hypothesized to be in excess of several thousand years. Due to the suspected long recurrence interval, the potential for surficial ground rupture is considered to be low during the expected life of the proposed structures.

5.2 Seismically Induced Landslides

Based upon the relatively flat topography across the site, it is our opinion that the potential risk of damage to the proposed dwellings by seismically induced landsliding is low provided the recommendations contained herein are properly followed.

5.3 Liquefaction

Liquefaction is a condition that occurs when loose, saturated sand is subjected to a high intensity, cyclic loading (such as an earthquake) and it loses its shear strength. The encountered stratigraphy has a low potential for liquefaction due to its relatively dense, consolidated state and lack of adverse ground water conditions.

5.4 Ground Motion

Based on the encountered stratigraphy and our visual reconnaissance of the site, it is our opinion that earthquake damage to the proposed dwellings, when founded on a suitable bearing stratum in accordance with the recommendations presented in this report, would likely be caused by the intensity and acceleration associated with the event.

Guidelines presented in the 2006 *International Building Code* (IBC) standards using a Site Class "C" designation, as defined in Table 1613.5.2, were used. The 2006 IBC seismic design parameters for short period (S_S) and 1-second period (S_1) spectral acceleration values were determined by the latitude and longitude of the project site using the USGS National Seismic Hazard Mapping Project website (http://earthquake.usgs.gov/hazmaps/). Based on the 2002 data, the USGS website interpolated ground motions at the project site for the period of 0.2 seconds at 1.018 and for a 1-second period at 0.33, with a 2 percent chance of exceedence in 50 years.

6.0 EROSION HAZARDS AND MITIGATION

As of October 1, 2008, the Washington State Department of Ecology (Ecology) Construction Storm Water General Permit (also known as the National Pollutant Discharge Elimination System [NPDES] permit) requires weekly Temporary Erosion and Sedimentation Control (TESC) inspections AND weekly turbidity monitoring of storm water leaving the site for all sites 1 or more acres in size that discharge storm water to surface waters of the state. The TESC inspections and turbidity monitoring must be completed by a Certified Erosion and Sediment Control Lead (CESCL) for the duration of the construction. TESC reports and weekly turbidity levels do not need to be sent to Ecology, but should be logged into the project Storm Water Pollution Prevention Plan (SWPPP). If the project does not require a SWPPP, the TESC reports should be kept in a file on-site, or by the permit holder if there is no facility

on-site. Ecology requires a monthly summary report of the turbidity monitoring results (if performed) signed by the NPDES permit holder. If the monitored turbidity equals or exceeds 25 nephelometric turbidity units (NTUs) (Ecology benchmark standard), the project best management practices (BMPs) should be modified to decrease the turbidity of storm water leaving the site. Changes and upgrades to the BMPs should be continued until the weekly turbidity reading is 25 NTU or lower. If the monitored turbidity exceeds 250 NTU, the results must be reported to Ecology within 24 hours and corrective action taken. Daily turbidity monitoring is continued until the corrective action lowers the turbidity to below 25 NTU.

In order to meet the current Ecology requirements, a properly developed, constructed, and maintained erosion control plan consistent with the City of Duvall standards and best management erosion control practices will be required for this project. Associated Earth Sciences, Inc. (AESI) is available to assist the project civil engineer in developing site-specific erosion control plans. Based on past experience, it will be necessary to make adjustments and provide additional measures to the TESC plan in order to optimize its effectiveness. Ultimately, the success of the TESC plan depends on a proactive approach to project planning and contractor implementation and maintenance.

The erosion hazard of the site soils is high. The most effective erosion control measure is the maintenance of adequate ground cover. Maintaining cover measures atop disturbed ground provides the greatest reduction to the potential generation of turbid runoff and sediment transport. During the local wet season (October 1st through March 31st), exposed soil should not remain uncovered for more than 2 days unless it is actively being worked. Ground cover measures can include erosion control matting, plastic sheeting, straw mulch, crushed rock or recycled concrete, or mature hydroseed.

Flow-control measures are also essential for collecting and controlling the site runoff. Flow paths across slopes should be kept to less than 50 feet in order to reduce the erosion and sediment transport potential of concentrated flow. Ditch/swale spacing will need to be shortened with increasing slope gradient. Ditches and swales that exceed a gradient of about 7 to 10 percent, depending on their flow length, should have properly constructed check dams installed to reduce the flow velocity of the runoff and reduce the erosion potential within the ditch. Flow paths that are required to be constructed on gradients between 10 to 15 percent should be placed in a riprap-lined swale with the riprap properly sized for the flow conditions. Flow paths constructed on slope gradients steeper than 15 percent should be placed in a pipe slope drain. AESI is available to assist the project civil engineer in developing a suitable erosion control plan with proper flow control.

Some fine-grained surface soils are the result of natural weathering processes that have broken down parent materials into their mineral components. These mineral components can have an inherent electrical charge. Electrically charged mineral fines will attract oppositely charged particles and can combine (flocculate) to form larger particles that will settle out of suspension.

The sediments produced during the recent glaciation of Puget Sound are, however, most commonly the suspended soils that are carried by site storm water. The fine-grained fraction of the glacially derived soil is referred to as "rock flour," which is primarily a silt-sized particle with no electrical charge. These particles, once suspended in water, may have settling times in periods of months, not hours.

Therefore, the flow length within a temporary sediment control trap or pond has virtually no effect on the water quality of the discharge since it is not going to settle out of suspension in the time it takes to flow from one end of the pond to the other. Reduction of turbidity from a construction site is almost entirely a function of cover measures and flow control. Temporary sediment traps and ponds are necessary to control the release rate of the runoff and to provide a catchment for sand-sized and larger soil particles, but are very ineffective at reducing the turbidity of the runoff

Silt fencing should be utilized as buffer protection and not as a flow-control measure. Silt fencing is meant to be placed parallel with topographic contours to prevent sediment-laden runoff from leaving a work area or entering a sensitive area. Silt fences should not be placed to cross contour lines without having separate flow control in front of the silt fence. A swale/berm combination should be constructed to provide flow control rather than let the runoff build up behind the silt fence and utilize the silt fence as the flow-control measure. Runoff flowing in front of a silt fence will cause additional erosion and usually will cause a failure of the silt fence. Improperly installed silt fencing has the potential to cause a much larger erosion hazard than if the silt fence was not installed at all. The use of silt fencing should be limited to protect sensitive areas, and swales should be used to provide flow control.

6.1 Erosion Hazard Mitigation

To mitigate and reduce the erosion hazard and potential for off-site sediment transport, we would recommend the following:

1. Surface water should not be allowed to flow across the site over unprotected surfaces.

2. Silt fences should be placed and maintained around the perimeter of the proposed construction area throughout the entire construction phase of the project until permanent landscaping and permanent storm water collection facilities have been installed.

3. Soils that are to be reused around the site should be stored in such a manner as to reduce erosion from the stockpile. Protective measures may include, but are not necessarily limited to, covering with plastic sheeting, the use of low stockpiles in flat areas behind the residence, or the use of straw bales and/or additional silt fences around pile perimeters. Soils should not be stockpiled on the steeply sloping portions of the lot.

4. Areas stripped of natural vegetation during construction should be replanted and mulched as soon as possible or otherwise protected.

5. All storm water from impermeable surfaces, including driveways and roofs, should be permanently tightlined to a suitable storm water collection system.

6. A rocked construction entrance should be provided for truck traffic onto and off the site.

III. PRELIMINARY DESIGN RECOMMENDATIONS

7.0 INTRODUCTION

Our exploration indicates that, from a geotechnical standpoint, the proposed project is feasible provided the recommendations contained herein are properly followed. The bearing stratum appears relatively shallow across the property; therefore, we recommend the use of standard spread footing foundations to support the new structures.

8.0 SITE PREPARATION

Any existing buildings, paving, buried utilities, and any other structures should be removed from areas where structures or paving are planned. Any existing underground heating oil storage tanks (USTs) should be emptied, inerted, and removed in accordance with Ecology requirements and the resulting excavation backfilled with structural fill. AESI can assist the owner with proper UST abandonment, sampling, contaminated soil remediation (in the unlikely event that contamination is encountered), and State-required documentation.

Adequate TESCs should be constructed in accordance with City of Duvall requirements and the project civil engineering TESC design. We recommend that the project contractor work together with the design team and the City of Duvall to design, install, and maintain the erosion control measures. It is easier and much less costly to keep fine-grained soils in place than it is to remove suspended sediment from site storm runoff.

Site preparation of planned structural fill, building, and paving areas should include removal of all trees, brush, landscaping, debris, and any other deleterious material. Additionally, the upper, organic topsoil should be removed and the remaining roots grubbed. One should refer to topsoil thicknesses depicted on the exploration logs in the Appendix for specific observed values. Topsoil materials will "swell" some 20 to 30 percent upon excavation. Project stripping volume estimates should include this swell factor. Areas where loose surficial soils exist below finished grade due to demolition or grubbing operations should be considered as fill to the depth of disturbance and treated as subsequently recommended for structural fill placement. Topsoil should be processed and reused as topsoil, if allowed by the project plans and specifications, or should be removed from the site or used as fill in non-structural areas. If some minor settlement can be tolerated, the topsoil could be placed as fill in non-structural areas as long as it is free of stumps and roots larger than 3 inches in diameter and is compacted to a firm and unyielding condition in lifts, as described for structural fill. We should be involved in the planning of the location and thickness of all fill and should observe the placement and compaction operations.

8.1 Site Drainage and Surface Water Control

Adequate temporary and permanent control of surface water runoff will be required in order to allow site access and grading for construction of the new buildings, access driveways, installation of underground utilities, and other proposed improvements. Excavation, filling, subgrade, and grade preparation should be performed in a manner and sequence that will provide controlled drainage at all times and proper control of erosion. Surface water should be collected and pumped or drained to provide a suitable working platform. Successful drainage of wet zones due to ground water flow and accumulations of surface water runoff could be accomplished by ditching and/or the installation of cut-off trenches or "French" drains, where necessary.

The site should be graded to prevent water from ponding in construction areas and/or flowing into excavations. Exposed grades should be crowned, sloped, and smooth-drum rolled at the end of each day to facilitate drainage. Accumulated water must be removed from subgrades and work areas immediately prior to performing further work in the area. Equipment access may be limited, and the amount of soil rendered unfit for use as structural fill may be greatly increased if drainage efforts are not accomplished in a timely sequence. If an effective drainage system is not utilized, project delays and increased costs could be incurred due to the greater quantities of wet and unsuitable fill, or poor access and unstable conditions.

Final exterior grades should promote free and positive drainage away from the building at all times. Water must not be allowed to pond or to collect adjacent to foundations or within the immediate building area. We recommend that a gradient of at least 3 percent for a minimum distance of 10 feet from the building perimeters be provided, except in paved locations. In paved locations, a minimum gradient of 1 percent should be provided unless provisions are included for collection and disposal of surface water adjacent to the structures.

8.2 Wet Weather Conditions

If construction proceeds during an extended wet weather construction period and the moisture-sensitive, silty site soils become wet, they will become unstable. Therefore, the bids for site grading operations should be based upon the time of year that construction will proceed. Construction during wet weather is expected to require protection of subgrades in staging and construction areas, as recommended in the "Subgrade Protection" section of this report. It is expected that in wet conditions, additional soils may need to be removed and/or other stabilization methods used, such as a coarse, crushed rock working mat to develop a stable condition if silty subgrade soils are disturbed in the presence of excess moisture. The severity of construction problems will be dependent, in part, on the precautions that are taken by the contractor to protect the moisture- and disturbance-sensitive site soils. If overexcavation is necessary, it should be confirmed through continuous observation and testing by a representative of our firm. The site contractor should provide properly surfaced access roads

to fill areas. Maintenance and reconstruction of access roads will be necessary. Daily "sealing" of fill surfaces and drainage improvements is required. It is the contractor's responsibility to keep silt-laden runoff to a minimum by protecting exposed subgrades and maintaining erosion control measures.

8.3 Subgrade Protection

The site soils that are expected below the building pad and paving subgrades contain a significant silt fraction and are considered to be moisture- and disturbance-sensitive. These soils will become unstable if disturbed by construction equipment while at elevated moisture contents, requiring additional soil removal at an increased cost. Therefore, in addition to the recommendations presented in the "Site Drainage and Surface Water Control" section of this report, site preparation and initial construction activities should be planned to minimize disturbance to the existing ground surface, particularly during extended wet weather periods and the wet season (typically October through May). Construction traffic should be restricted to specific rock-surfaced drive areas to limit the area where disturbance of the subgrade will occur. If site stripping and grading activities are performed during extended dry weather periods, we anticipate that site stabilization requirements will be much less.

If construction will proceed in the winter, we recommend the use of a temporary working surface of sand and gravel, crushed rock, or quarry spalls to protect the silty soils, particularly in areas supporting concentrated equipment traffic. In winter construction staging areas, a minimum thickness of 12 inches of quarry spalls or 18 inches of pit run sand and gravel is recommended. If subgrade conditions are soft and silty, a geotextile separation fabric, such as Mirafi 500x or approved equivalent, should be used between the subgrade and the new fill. For building pads where floor slabs and foundation construction will be completed in the winter, a similar working surface, composed of at least 12 inches of pit run sand and gravel or crushed rock, should be used. Construction of working surfaces from advancing fill pads could be used to avoid directly exposing the subgrade soils to vehicular traffic. Similar minimum (or greater) sections will be required for temporary access road construction.

Foundation subgrades may require protection from foot and equipment traffic, and ponding of runoff during wet weather conditions. Typically, compacted crushed rock or a lean-mix concrete mat placed over a properly prepared subgrade provides adequate subgrade protection. Foundation concrete should be placed and excavations backfilled as soon as possible to protect the bearing grade.

8.4 Proof-Rolling and Subgrade Compaction

Following the recommended demolition, site stripping procedures, and required excavation to grade, the stripped subgrade within the building and pavement areas should be proof-rolled with heavy, rubber-tired construction equipment, such as a fully loaded, tandem-axle dump

truck. Proof-rolling should be performed prior to structural fill placement or foundation excavation. The proof-roll should be observed by the geotechnical engineer so that any soft or yielding subgrade soils can be identified. Any soft/loose, yielding soils should be removed to a stable subgrade. The subgrade should then be scarified, adjusted in moisture content, and recompacted to the required density. Proof-rolling should only be attempted if soil moisture contents are at or near optimum moisture content. Proof-rolling of wet subgrades could result in further degradation. Low areas and excavations may then be raised to the planned finished grade with compacted structural fill. Subgrade preparation and selection, placement, and compaction of structural fill should be performed under engineering-controlled conditions in accordance with the project specifications.

8.5 Overexcavation/Stabilization

Construction during extended wet weather periods could create the need to overexcavate exposed soils if they become disturbed and cannot be recompacted due to elevated moisture content and/or weather conditions. During dry weather periods, soft/wet soils, which may need to be overexcavated, may be encountered in some portions of the site. If necessary, this should be confirmed through continuous observation and testing by AESI. Soils that have become unstable may require remedial measures in the form of one or more of the following:

1. Drying and recompaction. Selective drying may be accomplished by scarifying or windrowing surficial material during extended periods of dry and warm weather.

2. Removal of affected soils to expose a suitable bearing subgrade and replacement with compacted structural fill.

3. Mechanical stabilization with a coarse, crushed aggregate compacted into the subgrade, possibly in conjunction with a geotextile.

4. Admixture stabilization with cement powder. Admixture design and installation procedures need to be reviewed and approved by the design team and City prior to site use.

8.6 Temporary and Permanent Cut Slopes

In our opinion, stable construction slopes should be the responsibility of the contractor and should be determined during construction. For estimating purposes, however, we anticipate that temporary, unsupported cut slopes in the upper, weathered till section should not exceed a slope of 1.5H:1V (Horizontal:Vertical). Unsupported cut slopes within the unsaturated till at depth can be made at a maximum slope of 1H:1V or flatter. As is typical with earthwork operations, some sloughing and raveling may occur, and cut slopes may have to be adjusted in the field. If ground water seepage is encountered in cut slopes or if surface water is not routed

away from temporary cut slope faces, flatter slopes will be required. In addition, WISHA/OSHA regulations should be followed at all times. Permanent cut and structural fill slopes that are not intended to be exposed to surface water should be designed at inclinations of 2H:1V or flatter. Slopes that are intended to be exposed to surface water, such as bioswale side slopes, if planned, should be designed at inclinations of 3H:1V or flatter. All permanent cut or fill slopes should be compacted to at least 95 percent of the modified Proctor maximum dry density, as determined by *American Society for Testing and Materials* (ASTM):D 1557, and the slopes should be protected from erosion by sheet plastic until vegetation cover can be established during favorable weather.

8.7 Frozen Subgrades

If earthwork takes place during freezing conditions, all exposed subgrades should be allowed to thaw and then be recompacted prior to placing subsequent lifts of structural fill or foundation components. Alternatively, the frozen material could be stripped from the subgrade to reveal unfrozen soil prior to placing subsequent lifts of fill or foundation components. The frozen soil should not be reused as structural fill until allowed to thaw and adjusted to the proper moisture content, which may not be possible during winter months.

9.0 STRUCTURAL FILL

Structural fill may be necessary to establish desired grades. All references to structural fill in this report refer to subgrade preparation, fill type, placement, and compaction of materials as discussed in this section. If a percentage of compaction is specified under another section of this report, the value given in that section should be used.

After stripping, planned excavation, and any required overexcavation have been performed to the satisfaction of the geotechnical engineer/engineering geologist, the upper 12 inches of exposed ground should be recompacted to 90 percent of ASTM:D 1557. If the subgrade contains too much moisture, adequate recompaction may be difficult or impossible to obtain and should probably not be attempted. In lieu of recompaction, the area to receive fill should be blanketed with washed rock or quarry spalls to act as a capillary break between the new fill and the wet subgrade. Where the exposed ground remains soft and further overexcavation is impractical, placement of an engineering stabilization fabric may be necessary to prevent contamination of the free-draining layer by silt migration from below.

After recompaction of the exposed ground is tested and approved, or a free-draining rock course is laid, structural fill may be placed to attain desired grades. Structural fill is defined as non-organic soil, acceptable to the geotechnical engineer, placed in maximum 8-inch loose lifts with each lift being compacted to 95 percent of ASTM:D 1557. In the case of roadway and utility trench filling, the backfill should be placed and compacted in accordance with City of

Duvall codes and standards. The top of the compacted fill should extend horizontally outward a minimum distance of 3 feet beyond the location of the perimeter footings or roadway edges before sloping down at a maximum angle of 2H:1V.

The contractor should note that any proposed fill soils must be evaluated by AESI prior to their use in fills. This would require that we have a sample of the material 72 hours in advance of filling activities to perform a Proctor test and determine its field compaction standard. Soils in which the amount of fine-grained material (smaller than the No. 200 sieve) is greater than approximately 5 percent (measured on the minus No. 4 sieve size) should be considered moisture-sensitive. Use of moisture-sensitive soil in structural fills should be limited to favorable dry weather conditions. The on-site, native till soils contain substantial amounts of silt and fine sand and are considered highly moisture-sensitive when excavated and used as fill materials. We anticipate that all excavated site soils will require aeration and drying prior to compaction in structural fill applications. Construction equipment traversing the site when the soils are wet can cause considerable disturbance. If fill is placed during wet weather or if proper compaction cannot be obtained, a select, import material consisting of a clean, free-draining gravel and/or sand should be used. Free-draining fill consists of non-organic soil with the amount of fine-grained material limited to 5 percent by weight when measured on the minus No. 4 sieve fraction and with at least 25 percent retained on the No. 4 sieve.

If fill is placed during wet weather or if proper compaction cannot be obtained, a select import material consisting of a clean, free-draining gravel and/or sand should be used. Free-draining fill consists of non-organic soil with the amount of fine-grained material limited to 5 percent by weight when measured on the minus No. 4 sieve fraction and at least 25 percent retained on the No. 4 sieve.

10.0 HOUSE FOUNDATIONS

Spread footings may be used for building support when founded upon the native, at least medium dense till soils prepared as recommended in this report. Based on our observations, suitable foundation bearing soils are expected approximately 2 feet below the existing ground surface. If existing fill is discovered around existing structures, it should be removed and replaced with structural fill, which is also suitable for foundation support.

The footings for the proposed dwelling units may be designed for an allowable foundation soil bearing pressure of 2,500 pounds per square foot (psf) including both dead and live loads. An increase of one-third may be used for short-term wind or seismic loading. Perimeter footings should be buried at least 18 inches into the surrounding soil for frost protection. However, all foundation elements must penetrate to the prescribed bearing stratum and no foundation elements should be constructed in or above loose, organic, or existing fill soils.

Anticipated settlement of footings founded as recommended should be on the order of ¾ inch or less, with differential settlement of ½ inch or less. However, disturbed material not removed from footing trenches prior to footing placement could result in increased settlements. All footing areas should be inspected by AESI prior to placing concrete to verify that the foundation subgrades are undisturbed and construction conforms to the recommendations contained in this report. Such inspections may be required by City of Duvall. Perimeter footing drains should be provided as discussed under the section on "Drainage Considerations."

It should be noted that the area bounded by lines extending downward at 1H:1V from any footing must not intersect another footing or intersect a filled area which has not been compacted to at least 95 percent of ASTM:D 1557. In addition, a 1.5H:1V line extending down and away from any footing must not daylight because sloughing or raveling may eventually undermine the footing. Thus, footings should not be placed near the edge of steps or cuts in the bearing soils.

11.0 FLOOR SUPPORT

We anticipate the use of concrete, slab-on-grade floors within the proposed dwelling units. The concrete, slab-on-grade floor should be cast atop a minimum of 4 inches of pea gravel to act as a capillary break. The floors should also be protected from dampness by covering the capillary break layer with an impervious moisture barrier at least 10 mils in thickness. Floor slabs may be supported by native soils.

12.0 DRAINAGE CONSIDERATIONS

All retaining and perimeter footing walls should be provided with a drain at the footing elevation. Drains should consist of rigid, perforated, polyvinyl chloride (PVC) pipe surrounded by washed pea gravel or drain rock. The level of the perforations in the pipe should be set approximately 2 inches below the bottom of the footing and should be constructed with sufficient gradient to allow gravity discharge away from the buildings. In addition, all retaining walls should be lined with a minimum, 12-inch-thick, washed gravel or washed crushed rock blanket provided over the full height of the wall that ties into the footing drain. Roof and surface runoff should not be discharged into the footing drain system, but should be handled by a separate, rigid, tightline drain. In planning, exterior grades adjacent to walls should be sloped downward away from the structures to achieve surface drainage.

13.0 PROJECT DESIGN AND CONSTRUCTION MONITORING

At the time of this report, site grading, structural plans, and construction methods have not been finalized, and the recommendations presented herein are preliminary. We are available to provide additional geotechnical consultation as the project design develops and possibly changes from that upon which this report is based. We recommend that AESI perform a geotechnical review of the plans prior to final design completion. In this way, our earthwork and foundation recommendations may be properly interpreted and implemented in the design.

We are also available to provide geotechnical engineering and monitoring services during construction. The integrity of the foundations for buildings and of new pavement depends on proper site preparation and construction procedures. In addition, engineering decisions may have to be made in the field in the event that variations in subsurface conditions become apparent. Construction monitoring services are not part of the current scope of work. If these services are desired, please let us know and we will prepare a cost proposal.

We have enjoyed working with you on this study and are confident that these recommendations will aid in the successful completion of your project. If you should have any questions, or require further assistance, please do not hesitate to call.

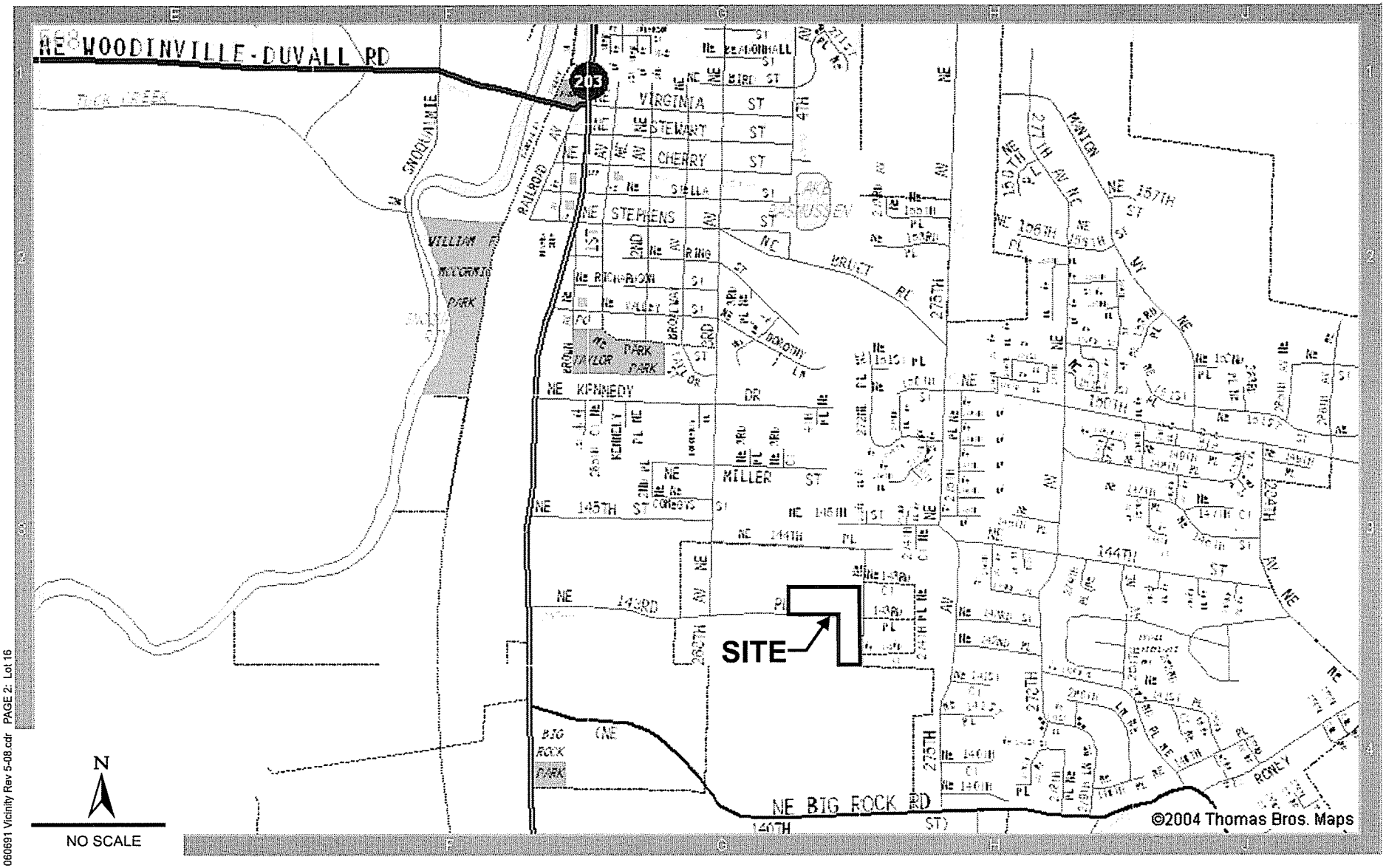
Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

MATTHEW A. MILLER
STATE OF WASHINGTON
30284
REGISTERED
PROFESSIONAL ENGINEER
8-7-08

Jesse P. Overton, P.G.
Project Geologist

Matthew A. Miller, P.E.
Associate Engineer

Attachments: Figure 1: Vicinity Map
Figure 2: Site and Exploration Plan
Appendix: Exploration Logs



Associated Earth Sciences, Inc.

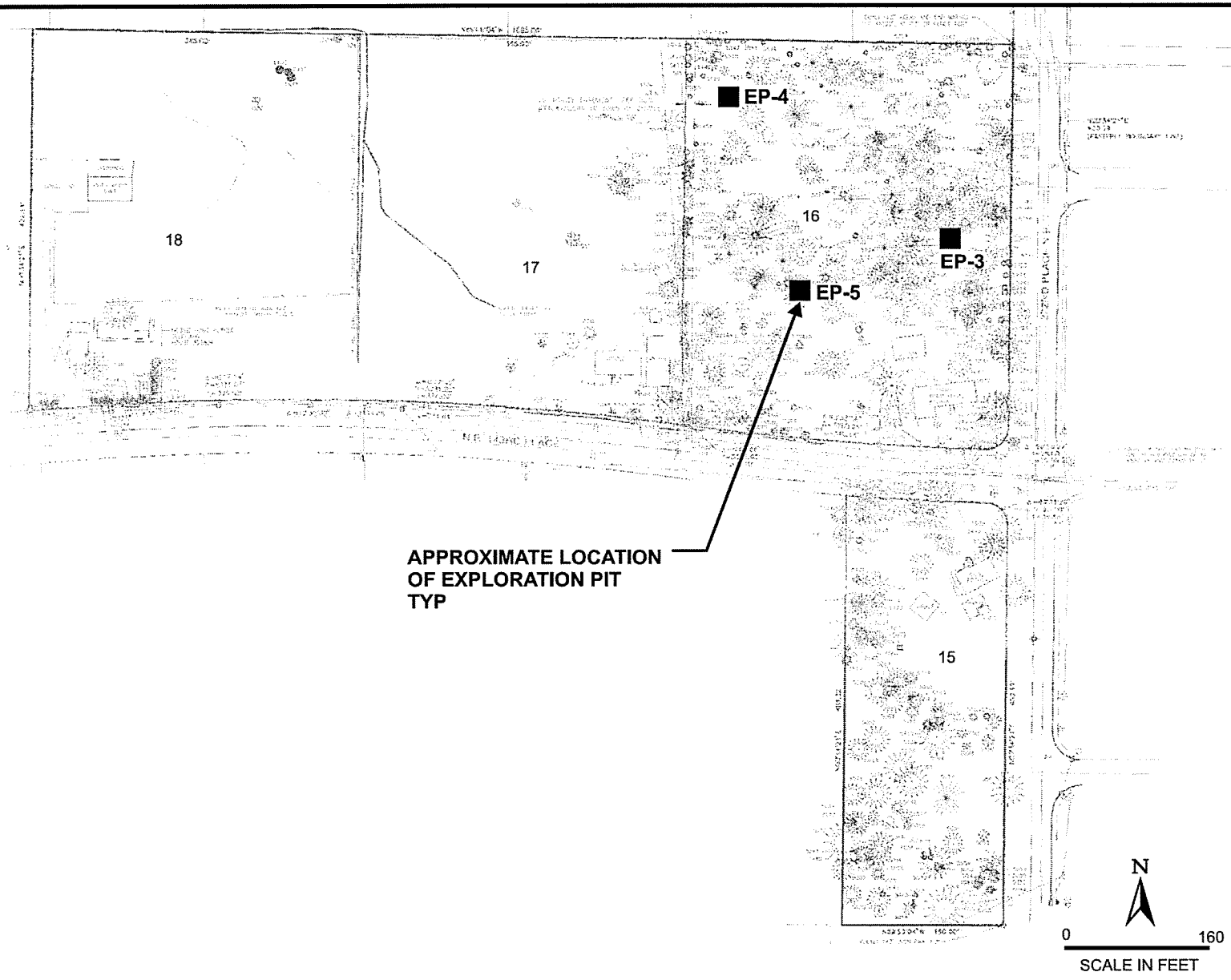
VICINITY MAP
RIO VISTA - LOT 16
DUVALL, WASHINGTON

FIGURE 1

DATE 5/08

PROJ. NO. KE060691B

060691 Rio Vista-Lots 15-18 \ 060691 Vicinity Rev 5-08.cdr PAGE 2: Lot 16



Reference: ESM

Associated Earth Sciences, Inc.

SITE AND EXPLORATION PLAN
RIO VISTA - LOT 16
DUVALL, WASHINGTON

FIGURE 2

DATE 5/08

PROJ. NO. KE060691B

060691 Rio Vista Lots 15-18 \ 060691 Site and Explr-B-Rev 5-08.cdr

APPENDIX

Coarse-Grained Soils - More than 50%[1] Retained on No. 200 Sieve	Gravels - More than 50%[1] of Coarse Fraction Retained on No. 4 Sieve	≤5% Fines[5]	GW	Well-graded gravel and gravel with sand, little to no fines
			GP	Poorly-graded gravel and gravel with sand, little to no fines
		≥15% Fines[5]	GM	Silty gravel and silty gravel with sand
			GC	Clayey gravel and clayey gravel with sand
	Sands - 50%[1] or More of Coarse Fraction Passes No. 4 Sieve	≤5% Fines[5]	SW	Well-graded sand and sand with gravel, little to no fines
			SP	Poorly-graded sand and sand with gravel, little to no fines
		≥15% Fines[5]	SM	Silty sand and silty sand with gravel
			SC	Clayey sand and clayey sand with gravel
Fine-Grained Soils - 50%[1] or More Passes No. 200 Sieve	Silts and Clays Liquid Limit Less than 50		ML	Silt, sandy silt, gravelly silt, silt with sand or gravel
			CL	Clay of low to medium plasticity; silty, sandy, or gravelly clay, lean clay
			OL	Organic clay or silt of low plasticity
	Silts and Clays Liquid Limit 50 or More		MH	Elastic silt, clayey silt, silt with micaceous or diatomaceous fine sand or silt
			CH	Clay of high plasticity, sandy or gravelly clay, fat clay with sand or gravel
			OH	Organic clay or silt of medium to high plasticity
Highly Organic Soils			PT	Peat, muck and other highly organic soils

Terms Describing Relative Density and Consistency

Coarse-Grained Soils

Density	SPT[2] blows/foot
Very Loose	0 to 4
Loose	4 to 10
Medium Dense	10 to 30
Dense	30 to 50
Very Dense	>50

Fine-Grained Soils

Consistency	SPT[2] blows/foot
Very Soft	0 to 2
Soft	2 to 4
Medium Stiff	4 to 8
Stiff	8 to 15
Very Stiff	15 to 30
Hard	>30

Test Symbols

G = Grain Size
M = Moisture Content
A = Atterberg Limits
C = Chemical
DD = Dry Density
K = Permeability

Component Definitions

Descriptive Term	Size Range and Sieve Number
Boulders	Larger than 12"
Cobbles	3" to 12"
Gravel	3" to No. 4 (4.75 mm)
Coarse Gravel	3" to 3/4"
Fine Gravel	3/4" to No. 4 (4.75 mm)
Sand	No. 4 (4.75 mm) to No. 200 (0.075 mm)
Coarse Sand	No. 4 (4.75 mm) to No. 10 (2.00 mm)
Medium Sand	No. 10 (2.00 mm) to No. 40 (0.425 mm)
Fine Sand	No. 40 (0.425 mm) to No. 200 (0.075 mm)
Silt and Clay	Smaller than No. 200 (0.075 mm)

[3] Estimated Percentage

Component	Percentage by Weight
Trace	<5
Few	5 to 10
Little	15 to 25
With	- Non-primary coarse constituents: ≥ 15% - Fines content between 5% and 15%

Moisture Content

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

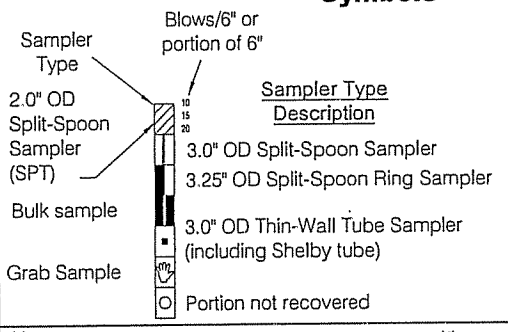
Slightly Moist - Perceptible moisture

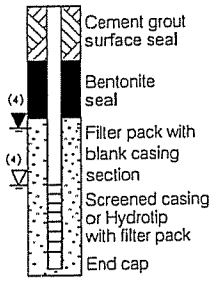
Moist - Damp but no visible water

Very Moist - Water visible but not free draining

Wet - Visible free water, usually from below water table

Symbols





[1] Percentage by dry weight
[2] (SPT) Standard Penetration Test (ASTM D-1586)
[3] In General Accordance with Standard Practice for Description and Identification of Soils (ASTM D-2488)
[4] Depth of ground water
ATD = At time of drilling
Static water level (date)
[5] Combined USCS symbols used for fines between 5% and 15%

Classifications of soils in this report are based on visual field and/or laboratory observations, which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field or laboratory testing unless presented herein. Visual-manual and/or laboratory classification methods of ASTM D-2487 and D-2488 were used as an identification guide for the Unified Soil Classification System.

Associated Earth Sciences, Inc.

EXPLORATION LOG KEY

FIGURE A1

blocks\log_key.dwg LAYOUT: Layout2

LOG OF EXPLORATION PIT NO. EP-3

Depth (ft)	This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered. **DESCRIPTION**
	Topsoil
1	**Weathered Till** Medium dense, dry, brown, silty fine SAND, trace gravel.
2	
3	**Vashon Till** Dense, moist, gray, silty SAND, little gravel.
4	
5	
6	More dense with depth.
7	
8	
9	Bottom of exploration pit at depth 8 feet Very difficult digging.
10	
11	
12	
13	
14	
15	
16	
17	
18	
19	
20	

Rio Vista, Lot 16
Duvall, WA

Associated Earth Sciences, Inc.

Logged by: JPO

Approved by:

Project No. KE060691B

May 2008

LOG OF EXPLORATION PIT NO. EP-4

Depth (ft)	This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered. **DESCRIPTION**
	Topsoil
1	**Vashon Till** Dense, moist, gray, silty SAND, little gravel.
2	Upper 1 1/2 feet less dense, weathered.
3	
4	More dense with depth.
5	**Vashon Advance Outwash** Dense, moist, orange mottled gray, fine to medium SAND, little gravel, little silt, trace cobbles.
6	
7	
8	No mottling below 8 feet.
9	Fine SAND, little silt, trace gravel at 9 to 9 1/2 feet.
10	
11	
12	
13	Bottom of exploration pit at depth 12 feet Light ground water seepage at ~11 feet.
14	
15	
16	
17	
18	
19	
20	

Rio Vista, Lot 16
Duvall, WA

Associated Earth Sciences, Inc.

Logged by: JPO

Approved by:

Project No. KE060691B

May 2008

KCTP3 060691B.GPJ May 20, 2008

LOG OF EXPLORATION PIT NO. EP-5

Depth (ft)	This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered. **DESCRIPTION**
	Topsoil
1	**Weathered Till** Medium dense, dry, silty fine SAND, trace gravel, roots.
2	
3	**Vashon Till** Dense, moist, gray, silty SAND, little gravel.
4	
5	
6	More dense with depth.
7	
8	
9	Bottom of exploration pit at depth 8 feet Very difficult digging
10	
11	
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Rio Vista, Lot 16
Duvall, WA

Associated Earth Sciences, Inc.

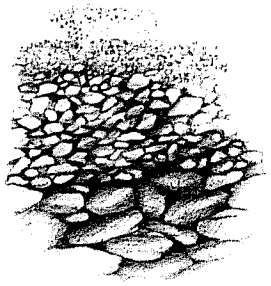
Logged by: JPO

Approved by:

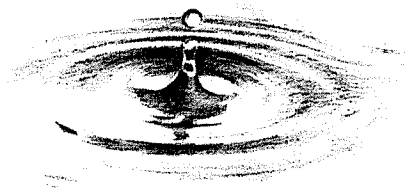
Project No. KE060691B

May 2008

KCTP3 060691B.GPJ May 20, 2008



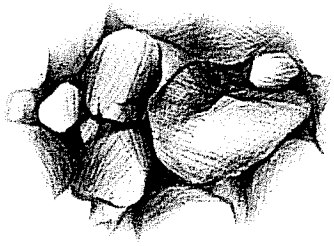
Geotechnical Engineering



Water Resources

Environmental Assessments and Remediation

Sustainable Development Services



Geologic Assessments

Associated Earth Sciences, Inc.

Celebrating 25 Years of Service

Subsurface Exploration, Geologic Hazard, and Preliminary Geotechnical Engineering Report

RIO VISTA – LOT 17

Duvall, Washington

Prepared for

Rio Vista, LLC

Project No. KE060691C
August 7, 2008

Associated Earth Sciences, Inc.

Celebrating Over 25 Years of Service

August 7, 2008
Project No. KE060691C

Rio Vista, LLC
P.O. Box 1282
Bellevue, Washington 98009-1282

Attention: Mr. Mike Reid

Subject: Subsurface Exploration, Geologic Hazard, and Preliminary Geotechnical Engineering Report
Rio Vista, Lot 17
27028 NE 143rd Place
Duvall, Washington

Dear Mr. Reid:

We are pleased to present the enclosed copies of the referenced report. This report summarizes the results of our subsurface exploration, geologic hazard, and preliminary geotechnical engineering studies and offers recommendations for the preliminary design and development of the proposed project. Our recommendations are preliminary in that project plans are still under development at the time of this report.

We have enjoyed working with you on this study and are confident that the recommendations presented in this report will aid in the successful completion of your project. If you should have any questions or if we can be of additional help to you, please do not hesitate to call.

Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

For

Jesse P. Overton, P.G.
Project Geologist

JPO/tb
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Projects\20060691\KE\WP

Kirkland ▪ Everett ▪ Tacoma
425-827-7701 425-259-0522 253-722-2992
www.aesgeo.com

SUBSURFACE EXPLORATION, GEOLOGIC HAZARD, AND PRELIMINARY GEOTECHNICAL ENGINEERING REPORT

RIO VISTA
LOT 17

Duvall, Washington

Prepared for:
Rio Vista, LLC
P.O. Box 1282
Bellevue, Washington 98009-1282

Prepared by:
Associated Earth Sciences, Inc.
911 5th Avenue, Suite 100
Kirkland, Washington 98033
425-827-7701
Fax: 425-827-5424

August 7, 2008
Project No. KE060691C

I. PROJECT AND SITE CONDITIONS

1.0 INTRODUCTION

This report presents the results of our subsurface exploration, geologic hazard, and preliminary geotechnical engineering study for the proposed short-plat development. Our recommendations are preliminary in that project plans are still under development at the time of our exploration and preparation of this report. The location of the project site is shown on the "Vicinity Map," Figure 1. The existing site conditions and approximate locations of the explorations accomplished for this study are presented on the "Site and Exploration Plan," Figure 2. If there are any substantial changes in the nature, design, or location of the proposed development, the conclusions and recommendations contained in this report should be reviewed and modified, or verified, as necessary.

1.1 Purpose and Scope

The purpose of this study was to provide subsurface data to be used in the preliminary design of the project. Our study included a review of selected geologic literature, excavating exploration pits, and performing geologic studies to assess the type, thickness, distribution, and physical properties of the subsurface sediments and shallow ground water conditions. Preliminary geotechnical engineering studies were completed to formulate our recommendations for site preparation, site grading, home construction, drainage, and paving. This report summarizes our current fieldwork and offers development recommendations based on our present understanding of the project. We recommend that we be allowed to review project plans prior to construction to verify that our geotechnical recommendations have been correctly interpreted and incorporated into the design. Additional exploration or design modifications/review may be required to finalize project documentation.

1.2 Authorization

Verbal authorization to proceed with this study was granted by Mr. Mike Reid of Rio Vista, LLC during our office meeting on August 16, 2007. Our study was accomplished in general accordance with our written proposal and scope of work for geotechnical services dated July 23, 2007. This report has been prepared for the exclusive use of Rio Vista, LLC and their agents, for specific application to this project.

Within the limitations of scope, schedule, and budget, our services have been performed in accordance with generally accepted geotechnical engineering and engineering geology practices in effect in this area at the time our report was prepared. No other warranty, express or implied, is made.

2.0 PROJECT AND SITE DESCRIPTION

This report was completed with an understanding of the project based on the boundary and topographic survey prepared by ESM Consulting Engineers, LLC (ESM) dated August 10, 2006, and our conversations with Mr. Mike Reid. It is our understanding that the existing residence and outbuildings will be demolished and replaced with a short-plat development consisting of an undetermined number of dwelling units.

The project site is located over 3.54 acres within Township 26N, Range 6E, Section 13 at 27028 NE 143rd Place in Duvall, King County, Washington. The site is bordered to the east, west, and north by private, residential lots, and to the south by NE 143rd Place. Site topography is relatively flat across the property with a seasonal drainage channel that trends to the northwest across the western half of the site.

At present, the site supports one single-family residence with a detached garage and a pair of small outbuildings within the southeast corner of the property, with driveway access from NE 143rd Place. Vegetation across the property consists of native field grass, a few stands of blackberry vines, and a couple of solitary deciduous trees in the northeast section of the property.

3.0 SUBSURFACE EXPLORATION

Our field study included the excavation of two exploration pits to gain subsurface information about the site. The various types of sediments, as well as the depths where characteristics of the sediments changed, are indicated on the exploration logs presented in the Appendix. The depths indicated on the logs where conditions changed may represent gradational variations between sediment types in the field.

The conclusions and recommendations presented in this report are based on the two exploration pits completed for this study. The number, locations, and depths of the explorations were completed within site and budgetary constraints. Because of the nature of exploratory work below ground, extrapolation of subsurface conditions between field explorations is necessary. It should be noted that differing subsurface conditions may sometimes be present due to the random nature of deposition and the alteration of topography by past grading and/or filling. The nature and extent of any variations between the field explorations may not become fully evident until construction. If variations are observed at that time, it may be necessary to re-evaluate specific recommendations in this report and make appropriate changes.

3.1 Exploration Test Pits

The exploration pits were excavated using a rubber-tired backhoe under sub-contract to our firm. The pits permitted direct, visual observation of subsurface conditions. Materials encountered in the exploration pits were studied and classified in the field by an engineering geologist from our firm. All exploration pits were backfilled immediately after examination and logging. Selected samples were then transported to our laboratory for further visual classification and testing, as necessary.

4.0 SUBSURFACE CONDITIONS

Subsurface conditions at the project site were inferred from the field explorations accomplished for this study, visual reconnaissance of the site, and review of selected geologic literature. As shown on the field logs, the explorations encountered a surficial layer of topsoil over advance outwash deposits.

4.1 Stratigraphy

Topsoil

We encountered a thin, surficial layer of topsoil within each of our exploration pits EP-8 and EP-9. The topsoil generally consisted of loose, moist, dark brown, silty sand rich with organic material. We observed the topsoil to a maximum depth of 18 inches within EP-8 in the southwest property corner. This unit is not considered suitable for the support of foundations and should be removed from the foundation areas.

Lodgement Till

Natural sediments encountered below the topsoil within EP-9 consisted of medium dense to very dense, medium to fine grained sand with variable amounts of silt, gravel, and cobbles with occasional boulders. These sediments were interpreted to be representative of Vashon lodgement till. Vashon lodgement till consists of an unsorted mixture of silt, sand, gravel, and cobbles that was deposited by basal, debris-laden glacial ice during the Vashon Stade of the Fraser Glaciation approximately 12,500 to 15,000 years ago. Lodgement till typically possesses high strength and low compressibility attributes that are favorable for support of foundations, floor slabs, and paving with proper preparation. The high relative density characteristic of Vashon lodgement till is due to its consolidation by the massive weight of the glacial ice from which it was deposited. In the areas such as the subject site, where Vashon lodgement till sediments are exposed at or near the ground surface, the density of the upper several feet of the Vashon lodgement till typically becomes reduced to a loose to medium dense state by weathering. Where the weathering process rendered these soils loose, the weathered

till is not suitable for direct foundation support, but may be suitable for reuse as structural fill provided it can be properly moisture conditioned and compacted to project specifications.

Vashon Advance Outwash Deposits

Natural sediments encountered below the topsoil within EP-8 and underlying the lodgement till sediments at a depth of 4 feet within exploration pit EP-9 were sand and gravel sediments that were deposited during the advance of the Vashon ice sheet. The outwash sediments were found to be dense with depth and varied in grain size distribution. The outwash sediments typically consisted of a fine to coarse sand with some gravel. The outwash sediments extended beyond the termination depths of our exploration pits. Advance outwash deposits typically possess high strength and low compressibility attributes that are favorable for support of foundations, floor slabs, and paving with proper preparation.

Our interpretations of subsurface conditions on-site are consistent with a published geologic map of the area (*Geologic Map of King County, Washington,* by Booth, Haugerud, and Sackett, 2002).

4.2 Hydrology

Ground water seepage was observed within exploration pit EP-8 at the time of our field study. Ground water seepage was observed within the Advance Outwash deposits at a depth of 10 feet. We interpreted this level of ground water as representative of the seasonal ground water table at this location. Specific depths, locations, and rates of ground water seepage are indicated on the exploration logs included in the Appendix. It should be noted that the depth and occurrence of ground water seepage at the site may vary in response to such factors as changes in season, precipitation, and site use.

II. GEOLOGIC HAZARDS AND MITIGATIONS

The following discussion of potential geologic hazards is based on the geologic, slope, and shallow ground water conditions as observed and discussed herein.

5.0 SEISMIC HAZARDS AND RECOMMENDED MITIGATION

Earthquakes occur in the Puget Lowland with great regularity. The vast majority of these events are small and are usually not felt. However, large earthquakes do occur as evidenced by the 1949, 7.2-magnitude event; the 1965, 6.5-magnitude event; and the 2001, 6.8-magnitude event. The 1949 earthquake appears to have been the largest in this area during recorded history and was centered in the Olympia area. Evaluation of earthquake return rates in the Puget Sound area indicates that an earthquake of the magnitude between 5.5 and 6.0 will likely occur every 25 to 40 years in the Puget Sound area.

Generally, there are four types of potential geologic hazards associated with large seismic events: 1) surficial ground rupture, 2) seismically induced landslides, 3) liquefaction, and 4) ground motion. The potential for each of these hazards to adversely impact the proposed project is discussed below.

5.1 Surficial Ground Rupture

The nearest known fault trace to the project site is the Seattle Fault located approximately 12 miles to the south. Recent studies by the U.S. Geological Survey (USGS) (e.g., Johnson et al., 1994, *Origin and Evolution of the Seattle Fault and Seattle Basin, Washington,* Geology, v. 22, pp. 71-74; and Johnson et al., 1999, *Active Tectonics of the Seattle Fault and Central Puget Sound Washington - Implications for Earthquake Hazards*, Geological Society of America Bulletin, July 1999, v. 111, n. 7, pp. 1042-1053) have provided evidence of surficial ground rupture along a northern splay of the Seattle Fault. The recognition of this fault splay is relatively new, and data pertaining to it are limited, with the studies still ongoing. According to the USGS studies, the latest movement of this fault was about 1,100 years ago when about 20 feet of surficial displacement took place. This displacement can presently be seen in the form of raised, wave-cut beach terraces along Alki Point in West Seattle and Restoration Point at the south end of Bainbridge Island. The recurrence interval of movement along this fault system is still unknown, although it is hypothesized to be in excess of several thousand years. Due to the suspected long recurrence interval, the potential for surficial ground rupture is considered to be low during the expected life of the proposed structures.

5.2 Seismically Induced Landslides

Based upon the relatively flat topography across the site, it is our opinion that the potential risk of damage to the proposed dwellings by seismically induced landsliding is low provided the recommendations contained herein are properly followed.

5.3 Liquefaction

Liquefaction is a condition that occurs when loose, saturated sand is subjected to a high intensity, cyclic loading (such as an earthquake) and it loses its shear strength. The encountered stratigraphy has a low potential for liquefaction due to its relatively dense, consolidated state and lack of adverse ground water conditions.

5.4 Ground Motion

Based on the encountered stratigraphy and our visual reconnaissance of the site, it is our opinion that earthquake damage to the proposed dwellings, when founded on a suitable bearing stratum in accordance with the recommendations presented in this report, would likely be caused by the intensity and acceleration associated with the event.

Guidelines presented in the 2006 *International Building Code* (IBC) standards using a Site Class "C" designation, as defined in Table 1613.5.2, were used. The 2006 IBC seismic design parameters for short period (S_s) and 1-second period (S_1) spectral acceleration values were determined by the latitude and longitude of the project site using the USGS National Seismic Hazard Mapping Project website (http://earthquake.usgs.gov/hazmaps/). Based on the 2002 data, the USGS website interpolated ground motions at the project site for the period of 0.2 seconds at 1.018 and for a 1-second period at 0.33, with a 2 percent chance of exceedence in 50 years.

6.0 EROSION HAZARDS AND MITIGATION

As of October 1, 2008, the Washington State Department of Ecology (Ecology) Construction Storm Water General Permit (also known as the National Pollutant Discharge Elimination System [NPDES] permit) requires weekly Temporary Erosion and Sedimentation Control (TESC) inspections AND weekly turbidity monitoring of storm water leaving the site for all sites 1 or more acres in size that discharge storm water to surface waters of the state. The TESC inspections and turbidity monitoring must be completed by a Certified Erosion and Sediment Control Lead (CESCL) for the duration of the construction. TESC reports and weekly turbidity levels do not need to be sent to Ecology, but should be logged into the project Storm Water Pollution Prevention Plan (SWPPP). If the project does not require a SWPPP, the TESC reports should be kept in a file on-site, or by the permit holder if there is no facility

on-site. Ecology requires a monthly summary report of the turbidity monitoring results (if performed) signed by the NPDES permit holder. If the monitored turbidity equals or exceeds 25 nephelometric turbidity units (NTUs) (Ecology benchmark standard), the project best management practices (BMPs) should be modified to decrease the turbidity of storm water leaving the site. Changes and upgrades to the BMPs should be continued until the weekly turbidity reading is 25 NTU or lower. If the monitored turbidity exceeds 250 NTU, the results must be reported to Ecology within 24 hours and corrective action taken. Daily turbidity monitoring is continued until the corrective action lowers the turbidity to below 25 NTU.

In order to meet the current Ecology requirements, a properly developed, constructed, and maintained erosion control plan consistent with the City of Duvall standards and best management erosion control practices will be required for this project. Associated Earth Sciences, Inc. (AESI) is available to assist the project civil engineer in developing site-specific erosion control plans. Based on past experience, it will be necessary to make adjustments and provide additional measures to the TESC plan in order to optimize its effectiveness. Ultimately, the success of the TESC plan depends on a proactive approach to project planning and contractor implementation and maintenance.

The erosion hazard of the site soils is high. The most effective erosion control measure is the maintenance of adequate ground cover. Maintaining cover measures atop disturbed ground provides the greatest reduction to the potential generation of turbid runoff and sediment transport. During the local wet season (October 1st through March 31st), exposed soil should not remain uncovered for more than 2 days unless it is actively being worked. Ground cover measures can include erosion control matting, plastic sheeting, straw mulch, crushed rock or recycled concrete, or mature hydroseed.

Flow-control measures are also essential for collecting and controlling the site runoff. Flow paths across slopes should be kept to less than 50 feet in order to reduce the erosion and sediment transport potential of concentrated flow. Ditch/swale spacing will need to be shortened with increasing slope gradient. Ditches and swales that exceed a gradient of about 7 to 10 percent, depending on their flow length, should have properly constructed check dams installed to reduce the flow velocity of the runoff and reduce the erosion potential within the ditch. Flow paths that are required to be constructed on gradients between 10 to 15 percent should be placed in a riprap-lined swale with the riprap properly sized for the flow conditions. Flow paths constructed on slope gradients steeper than 15 percent should be placed in a pipe slope drain. AESI is available to assist the project civil engineer in developing a suitable erosion control plan with proper flow control.

Some fine-grained surface soils are the result of natural weathering processes that have broken down parent materials into their mineral components. These mineral components can have an inherent electrical charge. Electrically charged mineral fines will attract oppositely charged particles and can combine (flocculate) to form larger particles that will settle out of suspension.

The sediments produced during the recent glaciation of Puget Sound are, however, most commonly the suspended soils that are carried by site storm water. The fine-grained fraction of the glacially derived soil is referred to as "rock flour," which is primarily a silt-sized particle with no electrical charge. These particles, once suspended in water, may have settling times in periods of months, not hours.

Therefore, the flow length within a temporary sediment control trap or pond has virtually no effect on the water quality of the discharge since it is not going to settle out of suspension in the time it takes to flow from one end of the pond to the other. Reduction of turbidity from a construction site is almost entirely a function of cover measures and flow control. Temporary sediment traps and ponds are necessary to control the release rate of the runoff and to provide a catchment for sand-sized and larger soil particles, but are very ineffective at reducing the turbidity of the runoff

Silt fencing should be utilized as buffer protection and not as a flow-control measure. Silt fencing is meant to be placed parallel with topographic contours to prevent sediment-laden runoff from leaving a work area or entering a sensitive area. Silt fences should not be placed to cross contour lines without having separate flow control in front of the silt fence. A swale/berm combination should be constructed to provide flow control rather than let the runoff build up behind the silt fence and utilize the silt fence as the flow-control measure. Runoff flowing in front of a silt fence will cause additional erosion and usually will cause a failure of the silt fence. Improperly installed silt fencing has the potential to cause a much larger erosion hazard than if the silt fence was not installed at all. The use of silt fencing should be limited to protect sensitive areas, and swales should be used to provide flow control.

6.1 Erosion Hazard Mitigation

To mitigate and reduce the erosion hazard and potential for off-site sediment transport, we would recommend the following:

1. Surface water should not be allowed to flow across the site over unprotected surfaces.

2. Silt fences should be placed and maintained around the perimeter of the proposed construction area throughout the entire construction phase of the project until permanent landscaping and permanent storm water collection facilities have been installed.

3. Soils that are to be reused around the site should be stored in such a manner as to reduce erosion from the stockpile. Protective measures may include, but are not necessarily limited to, covering with plastic sheeting, the use of low stockpiles in flat areas behind the residence, or the use of straw bales and/or additional silt fences around pile perimeters. Soils should not be stockpiled on the steeply sloping portions of the lot.

4. Areas stripped of natural vegetation during construction should be replanted and mulched as soon as possible or otherwise protected.

5. All storm water from impermeable surfaces, including driveways and roofs, should be permanently tightlined to a suitable storm water collection system.

6. A rocked construction entrance should be provided for truck traffic onto and off the site.

III. PRELIMINARY DESIGN RECOMMENDATIONS

7.0 INTRODUCTION

Our exploration indicates that, from a geotechnical standpoint, the proposed project is feasible provided the recommendations contained herein are properly followed. The bearing stratum appears relatively shallow across the property; therefore, we recommend the use of standard spread footing foundations to support the new structures.

8.0 SITE PREPARATION

Any existing buildings, paving, buried utilities, and any other structures should be removed from areas where structures or paving are planned. Any existing underground heating oil storage tanks (USTs) should be emptied, inerted, and removed in accordance with Ecology requirements and the resulting excavation backfilled with structural fill. AESI can assist the owner with proper UST abandonment, sampling, contaminated soil remediation (in the unlikely event that contamination is encountered), and State-required documentation.

Adequate TESCs should be constructed in accordance with City of Duvall requirements and the project civil engineering TESC design. We recommend that the project contractor work together with the design team and the City of Duvall to design, install, and maintain the erosion control measures. It is easier and much less costly to keep fine-grained soils in place than it is to remove suspended sediment from site storm runoff.

Site preparation of planned structural fill, building, and paving areas should include removal of all trees, brush, landscaping, debris, and any other deleterious material. Additionally, the upper, organic topsoil should be removed and the remaining roots grubbed. One should refer to topsoil thicknesses depicted on the exploration logs in the Appendix for specific observed values. Topsoil materials will "swell" some 20 to 30 percent upon excavation. Project stripping volume estimates should include this swell factor. Areas where loose surficial soils exist below finished grade due to demolition or grubbing operations should be considered as fill to the depth of disturbance and treated as subsequently recommended for structural fill placement. Topsoil should be processed and reused as topsoil, if allowed by the project plans and specifications, or should be removed from the site or used as fill in non-structural areas. If some minor settlement can be tolerated, the topsoil could be placed as fill in non-structural areas as long as it is free of stumps and roots larger than 3 inches in diameter and is compacted to a firm and unyielding condition in lifts, as described for structural fill. We should be involved in the planning of the location and thickness of all fill and should observe the placement and compaction operations.

8.1 Site Drainage and Surface Water Control

Adequate temporary and permanent control of surface water runoff will be required in order to allow site access and grading for construction of the new buildings, access driveways, installation of underground utilities, and other proposed improvements. Excavation, filling, subgrade, and grade preparation should be performed in a manner and sequence that will provide controlled drainage at all times and proper control of erosion. Surface water should be collected and pumped or drained to provide a suitable working platform. Successful drainage of wet zones due to ground water flow and accumulations of surface water runoff could be accomplished by ditching and/or the installation of cut-off trenches or "French" drains, where necessary.

The site should be graded to prevent water from ponding in construction areas and/or flowing into excavations. Exposed grades should be crowned, sloped, and smooth-drum rolled at the end of each day to facilitate drainage. Accumulated water must be removed from subgrades and work areas immediately prior to performing further work in the area. Equipment access may be limited, and the amount of soil rendered unfit for use as structural fill may be greatly increased if drainage efforts are not accomplished in a timely sequence. If an effective drainage system is not utilized, project delays and increased costs could be incurred due to the greater quantities of wet and unsuitable fill, or poor access and unstable conditions.

Final exterior grades should promote free and positive drainage away from the building at all times. Water must not be allowed to pond or to collect adjacent to foundations or within the immediate building area. We recommend that a gradient of at least 3 percent for a minimum distance of 10 feet from the building perimeters be provided, except in paved locations. In paved locations, a minimum gradient of 1 percent should be provided unless provisions are included for collection and disposal of surface water adjacent to the structure.

8.2 Wet Weather Conditions

If construction proceeds during an extended wet weather construction period and the moisture-sensitive, silty site soils become wet, they will become unstable. Therefore, the bids for site grading operations should be based upon the time of year that construction will proceed. Construction during wet weather is expected to require protection of subgrades in staging and construction areas, as recommended in the "Subgrade Protection" section of this report. It is expected that in wet conditions, additional soils may need to be removed and/or other stabilization methods used, such as a coarse, crushed rock working mat to develop a stable condition if silty subgrade soils are disturbed in the presence of excess moisture. The severity of construction problems will be dependent, in part, on the precautions that are taken by the contractor to protect the moisture- and disturbance-sensitive site soils. If overexcavation is necessary, it should be confirmed through continuous observation and testing by a representative of our firm. The site contractor should provide properly surfaced access roads

to fill areas. Maintenance and reconstruction of access roads will be necessary. Daily "sealing" of fill surfaces and drainage improvements is required. It is the contractor's responsibility to keep silt-laden runoff to a minimum by protecting exposed subgrades and maintaining erosion control measures.

8.3 Subgrade Protection

The site soils that are expected below the building pad and paving subgrades contain a significant silt fraction and are considered to be moisture- and disturbance-sensitive. These soils will become unstable if disturbed by construction equipment while at elevated moisture contents, requiring additional soil removal at an increased cost. Therefore, in addition to the recommendations presented in the "Site Drainage and Surface Water Control" section of this report, site preparation and initial construction activities should be planned to minimize disturbance to the existing ground surface, particularly during extended wet weather periods and the wet season (typically October through May). Construction traffic should be restricted to specific rock-surfaced drive areas to limit the area where disturbance of the subgrade will occur. If site stripping and grading activities are performed during extended dry weather periods, we anticipate that site stabilization requirements will be much less.

If construction will proceed in the winter, we recommend the use of a temporary working surface of sand and gravel, crushed rock, or quarry spalls to protect the silty soils, particularly in areas supporting concentrated equipment traffic. In winter construction staging areas, a minimum thickness of 12 inches of quarry spalls or 18 inches of pit run sand and gravel is recommended. If subgrade conditions are soft and silty, a geotextile separation fabric, such as Mirafi 500x or approved equivalent, should be used between the subgrade and the new fill. For building pads where floor slabs and foundation construction will be completed in the winter, a similar working surface, composed of at least 12 inches of pit run sand and gravel or crushed rock, should be used. Construction of working surfaces from advancing fill pads could be used to avoid directly exposing the subgrade soils to vehicular traffic. Similar minimum (or greater) sections will be required for temporary access road construction.

Foundation subgrades may require protection from foot and equipment traffic, and ponding of runoff during wet weather conditions. Typically, compacted crushed rock or a lean-mix concrete mat placed over a properly prepared subgrade provides adequate subgrade protection. Foundation concrete should be placed and excavations backfilled as soon as possible to protect the bearing grade.

8.4 Proof-Rolling and Subgrade Compaction

Following the recommended demolition, site stripping procedures, and required excavation to grade, the stripped subgrade within the building and pavement areas should be proof-rolled with heavy, rubber-tired construction equipment, such as a fully loaded, tandem-axle dump

truck. Proof-rolling should be performed prior to structural fill placement or foundation excavation. The proof-roll should be observed by the geotechnical engineer so that any soft or yielding subgrade soils can be identified. Any soft/loose, yielding soils should be removed to a stable subgrade. The subgrade should then be scarified, adjusted in moisture content, and recompacted to the required density. Proof-rolling should only be attempted if soil moisture contents are at or near optimum moisture content. Proof-rolling of wet subgrades could result in further degradation. Low areas and excavations may then be raised to the planned finished grade with compacted structural fill. Subgrade preparation and selection, placement, and compaction of structural fill should be performed under engineering-controlled conditions in accordance with the project specifications.

8.5 Overexcavation/Stabilization

Construction during extended wet weather periods could create the need to overexcavate exposed soils if they become disturbed and cannot be recompacted due to elevated moisture content and/or weather conditions. During dry weather periods, soft/wet soils, which may need to be overexcavated, may be encountered in some portions of the site. If necessary, this should be confirmed through continuous observation and testing by AESI. Soils that have become unstable may require remedial measures in the form of one or more of the following:

1. Drying and recompaction. Selective drying may be accomplished by scarifying or windrowing surficial material during extended periods of dry and warm weather.

2. Removal of affected soils to expose a suitable bearing subgrade and replacement with compacted structural fill.

3. Mechanical stabilization with a coarse, crushed aggregate compacted into the subgrade, possibly in conjunction with a geotextile.

4. Admixture stabilization with cement powder. Admixture design and installation procedures need to be reviewed and approved by the design team and City prior to site use.

8.6 Temporary and Permanent Cut Slopes

In our opinion, stable construction slopes should be the responsibility of the contractor and should be determined during construction. For estimating purposes, however, we anticipate that temporary, unsupported cut slopes in the upper, weathered till section and within the advance outwash deposits should not exceed a slope of 1.5H:1V (Horizontal:Vertical). Unsupported cut slopes within the unsaturated till at depth can be made at a maximum slope of 1H:1V or flatter. As is typical with earthwork operations, some sloughing and raveling may occur, and cut slopes may have to be adjusted in the field. If ground water seepage is

encountered in cut slopes or if surface water is not routed away from temporary cut slope faces, flatter slopes will be required. In addition, WISHA/OSHA regulations should be followed at all times. Permanent cut and structural fill slopes that are not intended to be exposed to surface water should be designed at inclinations of 2H:1V or flatter. Slopes that are intended to be exposed to surface water, such as bioswale side slopes, if planned, should be designed at inclinations of 3H:1V or flatter. All permanent cut or fill slopes should be compacted to at least 95 percent of the modified Proctor maximum dry density, as determined by *American Society for Testing and Materials* (ASTM):D 1557, and the slopes should be protected from erosion by sheet plastic until vegetation cover can be established during favorable weather.

8.7 Frozen Subgrades

If earthwork takes place during freezing conditions, all exposed subgrades should be allowed to thaw and then be recompacted prior to placing subsequent lifts of structural fill or foundation components. Alternatively, the frozen material could be stripped from the subgrade to reveal unfrozen soil prior to placing subsequent lifts of fill or foundation components. The frozen soil should not be reused as structural fill until allowed to thaw and adjusted to the proper moisture content, which may not be possible during winter months.

9.0 STRUCTURAL FILL

Structural fill may be necessary to establish desired grades. All references to structural fill in this report refer to subgrade preparation, fill type, placement, and compaction of materials as discussed in this section. If a percentage of compaction is specified under another section of this report, the value given in that section should be used.

After stripping, planned excavation, and any required overexcavation have been performed to the satisfaction of the geotechnical engineer/engineering geologist, the upper 12 inches of exposed ground should be recompacted to 90 percent of ASTM:D 1557. If the subgrade contains too much moisture, adequate recompaction may be difficult or impossible to obtain and should probably not be attempted. In lieu of recompaction, the area to receive fill should be blanketed with washed rock or quarry spalls to act as a capillary break between the new fill and the wet subgrade. Where the exposed ground remains soft and further overexcavation is impractical, placement of an engineering stabilization fabric may be necessary to prevent contamination of the free-draining layer by silt migration from below.

After recompaction of the exposed ground is tested and approved, or a free-draining rock course is laid, structural fill may be placed to attain desired grades. Structural fill is defined as non-organic soil, acceptable to the geotechnical engineer, placed in maximum 8-inch loose lifts with each lift being compacted to 95 percent of ASTM:D 1557. In the case of roadway and

utility trench filling, the backfill should be placed and compacted in accordance with City of Duvall codes and standards. The top of the compacted fill should extend horizontally outward a minimum distance of 3 feet beyond the location of the perimeter footings or roadway edges before sloping down at a maximum angle of 2H:1V.

The contractor should note that any proposed fill soils must be evaluated by AESI prior to their use in fills. This would require that we have a sample of the material 72 hours in advance of filling activities to perform a Proctor test and determine its field compaction standard. Soils in which the amount of fine-grained material (smaller than the No. 200 sieve) is greater than approximately 5 percent (measured on the minus No. 4 sieve size) should be considered moisture-sensitive. Use of moisture-sensitive soil in structural fills should be limited to favorable dry weather conditions. The on-site, native till soils contain substantial amounts of silt and fine sand and are considered highly moisture-sensitive when excavated and used as fill materials. We anticipate that all excavated site soils will require aeration and drying prior to compaction in structural fill applications. Construction equipment traversing the site when the soils are wet can cause considerable disturbance. If fill is placed during wet weather or if proper compaction cannot be obtained, a select, import material consisting of a clean, free-draining gravel and/or sand should be used. Free-draining fill consists of non-organic soil with the amount of fine-grained material limited to 5 percent by weight when measured on the minus No. 4 sieve fraction and with at least 25 percent retained on the No. 4 sieve.

If fill is placed during wet weather or if proper compaction cannot be obtained, a select import material consisting of a clean, free-draining gravel and/or sand should be used. Free-draining fill consists of non-organic soil with the amount of fine-grained material limited to 5 percent by weight when measured on the minus No. 4 sieve fraction and at least 25 percent retained on the No. 4 sieve.

10.0 HOUSE FOUNDATIONS

Spread footings may be used for building support when founded upon the native, at least medium dense till soils and advance outwash deposits prepared as recommended in this report. Based on our observations, suitable foundation bearing soils are expected approximately 2 feet below the existing ground surface. If existing fill is discovered around existing structures, it should be removed and replaced with structural fill, which is also suitable for foundation support.

The footings for the proposed dwelling units may be designed for an allowable foundation soil bearing pressure of 2,500 pounds per square foot (psf) including both dead and live loads. An increase of one-third may be used for short-term wind or seismic loading. Perimeter footings should be buried at least 18 inches into the surrounding soil for frost protection. However, all

foundation elements must penetrate to the prescribed bearing stratum and no foundation elements should be constructed in or above loose, organic, or existing fill soils.

Anticipated settlement of footings founded as recommended should be on the order of ¾ inch or less, with differential settlement of ½ inch or less. However, disturbed material not removed from footing trenches prior to footing placement could result in increased settlements. All footing areas should be inspected by AESI prior to placing concrete to verify that the foundation subgrades are undisturbed and construction conforms to the recommendations contained in this report. Such inspections may be required by City of Duvall. Perimeter footing drains should be provided as discussed under the section on "Drainage Considerations."

It should be noted that the area bounded by lines extending downward at 1H:1V from any footing must not intersect another footing or intersect a filled area which has not been compacted to at least 95 percent of ASTM:D 1557. In addition, a 1.5H:1V line extending down and away from any footing must not daylight because sloughing or raveling may eventually undermine the footing. Thus, footings should not be placed near the edge of steps or cuts in the bearing soils.

11.0 FLOOR SUPPORT

We anticipate the use of concrete, slab-on-grade floors within the proposed dwelling units. The concrete, slab-on-grade floor should be cast atop a minimum of 4 inches of pea gravel to act as a capillary break. The floors should also be protected from dampness by covering the capillary break layer with an impervious moisture barrier at least 10 mils in thickness. Floor slabs may be supported by native soils.

12.0 DRAINAGE CONSIDERATIONS

All retaining and perimeter footing walls should be provided with a drain at the footing elevation. Drains should consist of rigid, perforated, polyvinyl chloride (PVC) pipe surrounded by washed pea gravel or drain rock. The level of the perforations in the pipe should be set approximately 2 inches below the bottom of the footing and should be constructed with sufficient gradient to allow gravity discharge away from the buildings. In addition, all retaining walls should be lined with a minimum, 12-inch-thick, washed gravel or washed crushed rock blanket provided over the full height of the wall that ties into the footing drain. Roof and surface runoff should not be discharged into the footing drain system, but should be handled by a separate, rigid, tightline drain. In planning, exterior grades adjacent to walls should be sloped downward away from the structures to achieve surface drainage.

13.0 PROJECT DESIGN AND CONSTRUCTION MONITORING

At the time of this report, site grading, structural plans, and construction methods have not been finalized, and the recommendations presented herein are preliminary. We are available to provide additional geotechnical consultation as the project design develops and possibly changes from that upon which this report is based. We recommend that AESI perform a geotechnical review of the plans prior to final design completion. In this way, our earthwork and foundation recommendations may be properly interpreted and implemented in the design.

We are also available to provide geotechnical engineering and monitoring services during construction. The integrity of the foundations for buildings and of new pavement depends on proper site preparation and construction procedures. In addition, engineering decisions may have to be made in the field in the event that variations in subsurface conditions become apparent. Construction monitoring services are not part of the current scope of work. If these services are desired, please let us know and we will prepare a cost proposal.

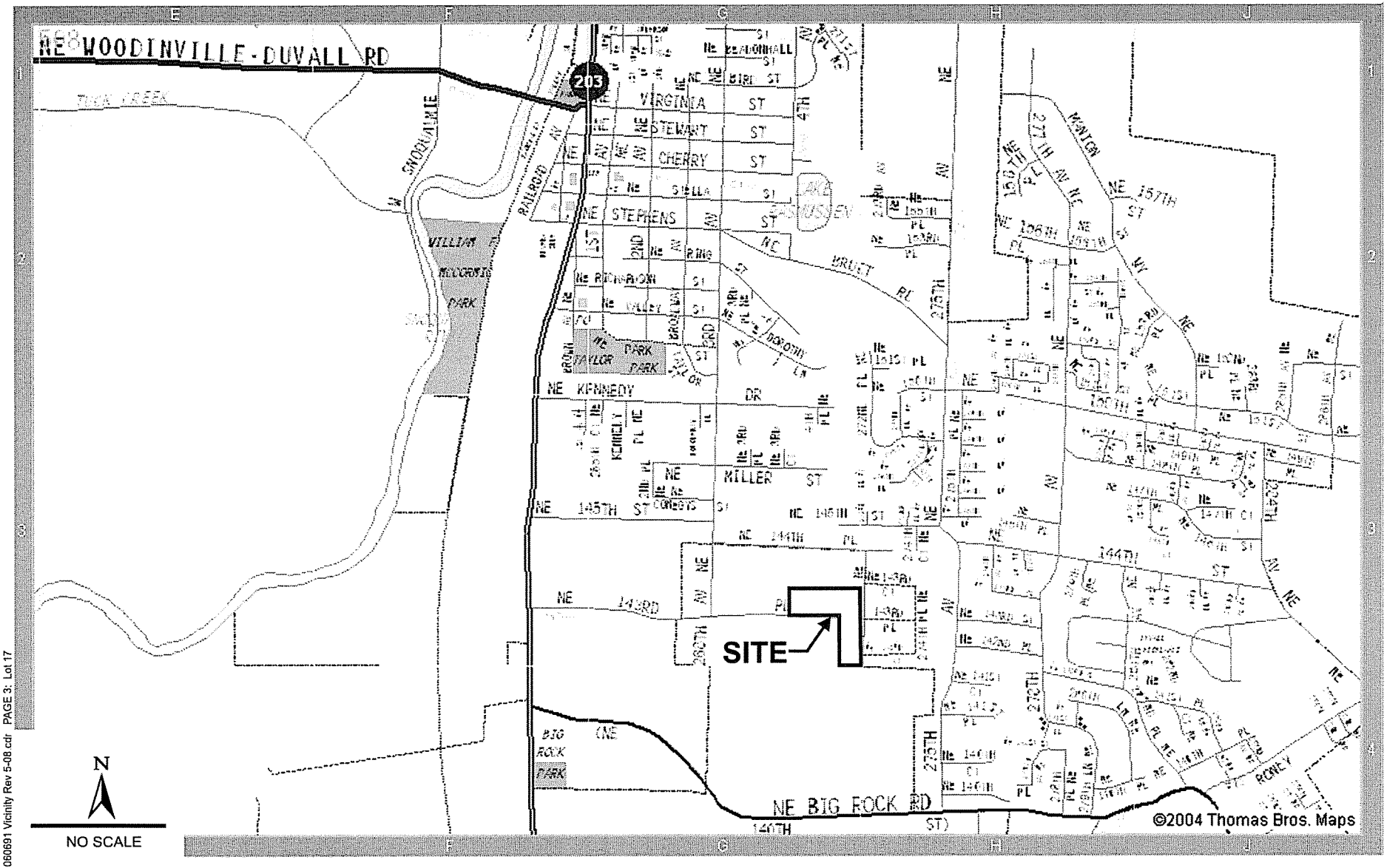
We have enjoyed working with you on this study and are confident that these recommendations will aid in the successful completion of your project. If you should have any questions, or require further assistance, please do not hesitate to call.

Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

Jesse P. Overton, P.G.
Project Geologist

Matthew A. Miller, P.E.
Associate Engineer

Attachments: Figure 1: Vicinity Map
Figure 2: Site and Exploration Plan
Appendix: Exploration Logs



Associated Earth Sciences, Inc.

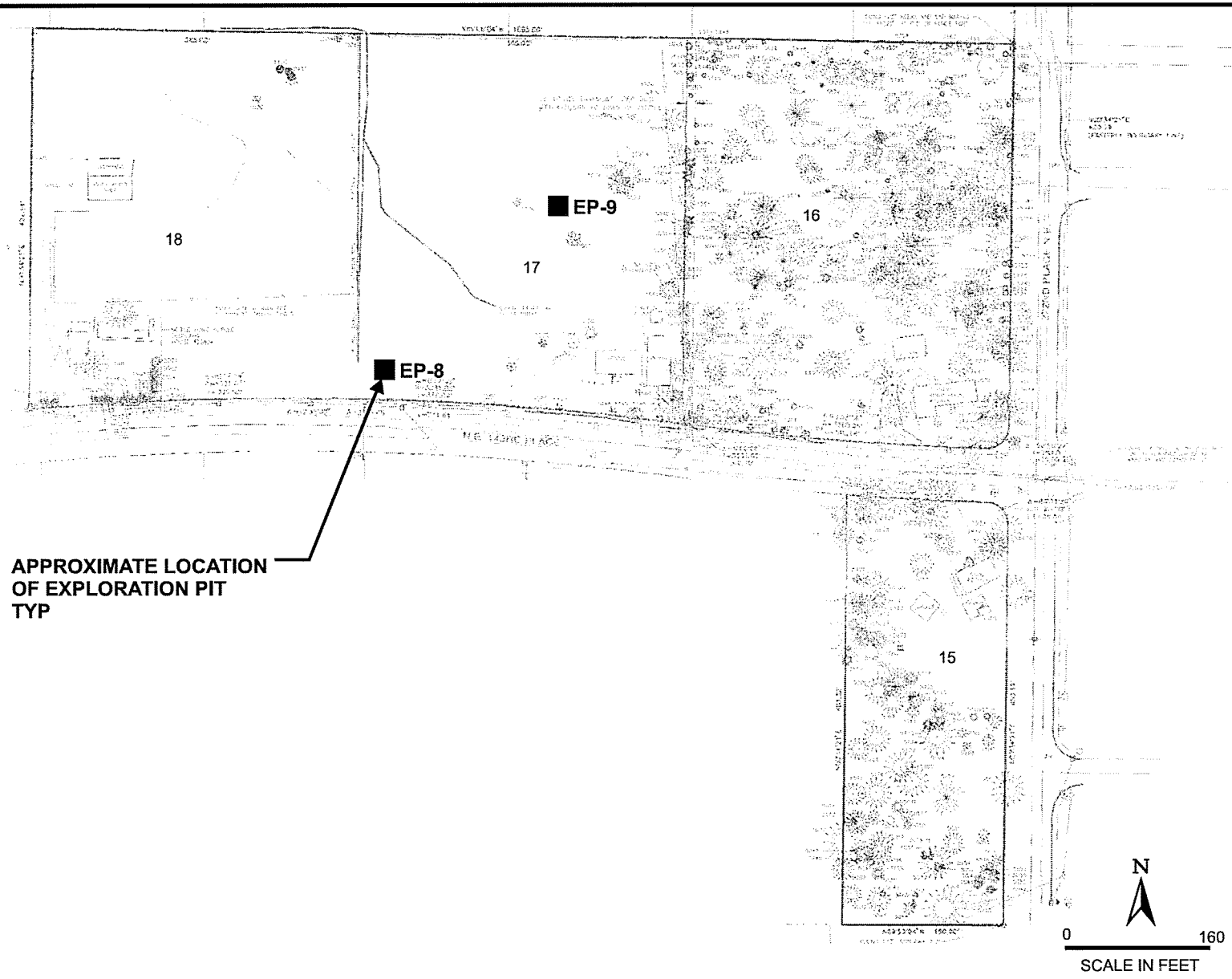
VICINITY MAP
RIO VISTA - LOT 17
DUVALL, WASHINGTON

FIGURE 1

DATE 5/08

PROJ. NO. KE060691C

060691 Rio Vista-Lots 15-18 \ 060691 Vicinity Rev 5-08.cdr PAGE 3: Lot 17



Reference: ESM

Associated Earth Sciences, Inc.

SITE AND EXPLORATION PLAN
RIO VISTA - LOT 17
DUVALL, WASHINGTON

FIGURE 2

DATE 5/08

PROJ. NO. KE060691C

060691 Rio Vista Lots 15-18 \ 060691 Site and Expl-C-Rev 5-08.cdr

APPENDIX

Coarse-Grained Soils - More than 50% [1] Retained on No. 200 Sieve	Gravels - More than 50% [1] of Coarse Fraction Retained on No. 4 Sieve	≤5% Fines [5]	GW	Well-graded gravel and gravel with sand, little to no fines
			GP	Poorly-graded gravel and gravel with sand, little to no fines
		≥15% Fines [5]	GM	Silty gravel and silty gravel with sand
			GC	Clayey gravel and clayey gravel with sand
	Sands - 50% [1] or More of Coarse Fraction Passes No. 4 Sieve	≤5% Fines [5]	SW	Well-graded sand and sand with gravel, little to no fines
			SP	Poorly-graded sand and sand with gravel, little to no fines
		≥15% Fines [5]	SM	Silty sand and silty sand with gravel
			SC	Clayey sand and clayey sand with gravel
Fine-Grained Soils - 50% [1] or More Passes No. 200 Sieve	Silts and Clays Liquid Limit Less than 50		ML	Silt, sandy silt, gravelly silt, silt with sand or gravel
			CL	Clay of low to medium plasticity; silty, sandy, or gravelly clay, lean clay
			OL	Organic clay or silt of low plasticity
	Silts and Clays Liquid Limit 50 or More		MH	Elastic silt, clayey silt, silt with micaceous or diatomaceous fine sand or silt
			CH	Clay of high plasticity, sandy or gravelly clay, fat clay with sand or gravel
			OH	Organic clay or silt of medium to high plasticity
Highly Organic Soils			PT	Peat, muck and other highly organic soils

Terms Describing Relative Density and Consistency

	Density	SPT[2] blows/foot
Coarse-Grained Soils	Very Loose	0 to 4
	Loose	4 to 10
	Medium Dense	10 to 30
	Dense	30 to 50
	Very Dense	>50
	Consistency	**SPT[2] blows/foot**
Fine-Grained Soils	Very Soft	0 to 2
	Soft	2 to 4
	Medium Stiff	4 to 8
	Stiff	8 to 15
	Very Stiff	15 to 30
	Hard	>30

Test Symbols

G = Grain Size
M = Moisture Content
A = Atterberg Limits
C = Chemical
DD = Dry Density
K = Permeability

Component Definitions

Descriptive Term	Size Range and Sieve Number
Boulders	Larger than 12"
Cobbles	3" to 12"
Gravel	3" to No. 4 (4.75 mm)
Coarse Gravel	3" to 3/4"
Fine Gravel	3/4" to No. 4 (4.75 mm)
Sand	No. 4 (4.75 mm) to No. 200 (0.075 mm)
Coarse Sand	No. 4 (4.75 mm) to No. 10 (2.00 mm)
Medium Sand	No. 10 (2.00 mm) to No. 40 (0.425 mm)
Fine Sand	No. 40 (0.425 mm) to No. 200 (0.075 mm)
Silt and Clay	Smaller than No. 200 (0.075 mm)

[3] Estimated Percentage

Component	Percentage by Weight
Trace	<5
Few	5 to 10
Little	15 to 25
With	- Non-primary coarse constituents: ≥ 15% - Fines content between 5% and 15%

Moisture Content

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

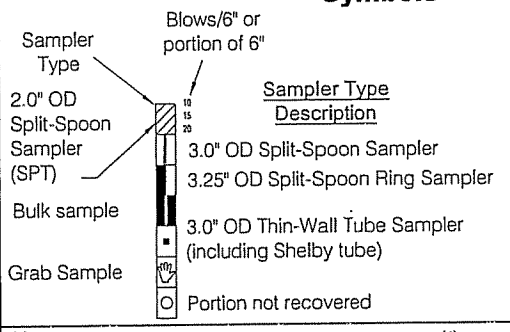
Slightly Moist - Perceptible moisture

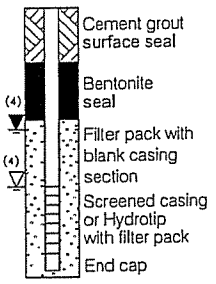
Moist - Damp but no visible water

Very Moist - Water visible but not free draining

Wet - Visible free water, usually from below water table

Symbols





(1) Percentage by dry weight
(2) (SPT) Standard Penetration Test (ASTM D-1586)
(3) In General Accordance with Standard Practice for Description and Identification of Soils (ASTM D-2488)
(4) Depth of ground water
▼ ATD = At time of drilling
▽ Static water level (date)
(5) Combined USCS symbols used for fines between 5% and 15%

Classifications of soils in this report are based on visual field and/or laboratory observations, which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field or laboratory testing unless presented herein. Visual-manual and/or laboratory classification methods of ASTM D-2487 and D-2488 were used as an identification guide for the Unified Soil Classification System.

Associated Earth Sciences, Inc.

EXPLORATION LOG KEY

FIGURE A1

blockslog_key.dwg LAYOUT: Layout2

LOG OF EXPLORATION PIT NO. EP-8

Depth (ft)	This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered. **DESCRIPTION**
	Topsoil
1	Organic-rich, dark brown, silty fine SAND.
	Vashon Advance Outwash
2	Dense, moist, gray, medium SAND, little gravel, little silt.
3	
4	
5	Silty fine to medium SAND, little gravel below 5 feet.
6	Increased moisture.
7	
8	Less silt below 8 feet.
9	
10	
11	
12	
13	Bottom of exploration pit at depth 12 feet Light seepage at ~10 feet.
14	
15	
16	
17	
18	
19	
20	

Rio Vista, Lot 17
Duvall, WA

Associated Earth Sciences, Inc.

Logged by: JPO

Approved by:

Project No. KE060691C

May 2008

KCTP3 060691C.GPJ May 20, 2008

LOG OF EXPLORATION PIT NO. EP-9

Depth (ft)	This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered. **DESCRIPTION**
	Topsoil
1	**Weathered Till** Medium dense, damp, brown, silty fine SAND, trace gravel.
2	**Vashon Till** Dense, moist, gray, silty SAND, little gravel.
3	
4	**Vashon Advance Outwash** Dense, moist, gray, fine to medium SAND, little gravel, little silt, trace cobbles.
5	
6	
7	Less gravel below 7 feet.
8	
9	Increased moisture below 9 feet.
10	
11	
12	
13	Bottom of exploration pit at depth 12 feet
14	
15	
16	
17	
18	
19	
20	

Rio Vista, Lot 17
Duvall, WA

Associated Earth Sciences, Inc.

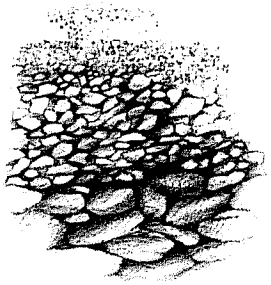
Logged by: JPO

Approved by:

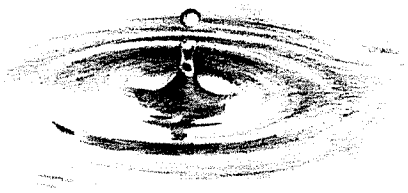
Project No. KE060691C

May 2008

KCTP3 060691C.GPJ May 20, 2008



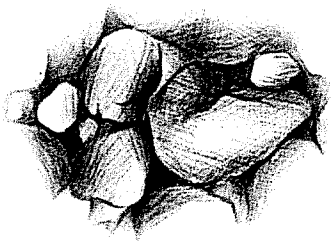
Geotechnical Engineering



Water Resources

Environmental Assessments and Remediation

Sustainable Development Services



Geologic Assessments

Associated Earth Sciences, Inc.

Celebrating 25 Years of Service

Subsurface Exploration, Geologic Hazard, and Preliminary Geotechnical Engineering Report

RIO VISTA – LOT 18

Duvall, Washington

Prepared for

Rio Vista, LLC

Project No. KE060691D
August 7, 2008

Associated Earth Sciences, Inc.

Celebrating Over 25 Years of Service

August 7, 2008
Project No. KE060691D

Rio Vista, LLC
P.O. Box 1282
Bellevue, Washington 98009-1282

Attention: Mr. Mike Reid

Subject: Subsurface Exploration, Geologic Hazard, and Preliminary Geotechnical Engineering Report
Rio Vista, Lot 18
26854 NE 143rd Place
Duvall, Washington

Dear Mr. Reid:

We are pleased to present the enclosed copies of the referenced report. This report summarizes the results of our subsurface exploration, geologic hazard, and preliminary geotechnical engineering studies and offers recommendations for the preliminary design and development of the proposed project. Our recommendations are preliminary in that project plans are still under development at the time of this report.

We have enjoyed working with you on this study and are confident that the recommendations presented in this report will aid in the successful completion of your project. If you should have any questions or if we can be of additional help to you, please do not hesitate to call.

Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

FOR

Jesse P. Overton, P.G.
Project Geologist

JPO/tb
KE060691D1
Projects\20060691\KE\WP

SUBSURFACE EXPLORATION, GEOLOGIC HAZARD, AND PRELIMINARY GEOTECHNICAL ENGINEERING REPORT

RIO VISTA
LOT 18

Duvall, Washington

Prepared for:
Rio Vista, LLC
P.O. Box 1282
Bellevue, Washington 98009-1282

Prepared by:
Associated Earth Sciences, Inc.
911 5th Avenue, Suite 100
Kirkland, Washington 98033
425-827-7701
Fax: 425-827-5424

August 7, 2008
Project No. KE060691D

I. PROJECT AND SITE CONDITIONS

1.0 INTRODUCTION

This report presents the results of our subsurface exploration, geologic hazard, and preliminary geotechnical engineering study for the proposed short-plat development. Our recommendations are preliminary in that project plans are still under development at the time of our exploration and preparation of this report. The location of the project site is shown on the "Vicinity Map," Figure 1. The existing site conditions and approximate locations of the explorations accomplished for this study are presented on the "Site and Exploration Plan," Figure 2. If there are any substantial changes in the nature, design, or location of the proposed development, the conclusions and recommendations contained in this report should be reviewed and modified, or verified, as necessary.

1.1 Purpose and Scope

The purpose of this study was to provide subsurface data to be used in the preliminary design of the project. Our study included a review of selected geologic literature, excavating exploration pits, and performing geologic studies to assess the type, thickness, distribution, and physical properties of the subsurface sediments and shallow ground water conditions. Preliminary geotechnical engineering studies were completed to formulate our recommendations for site preparation, site grading, home construction, drainage, and paving. This report summarizes our current fieldwork and offers development recommendations based on our present understanding of the project. We recommend that we be allowed to review project plans prior to construction to verify that our geotechnical recommendations have been correctly interpreted and incorporated into the design. Additional exploration or design modifications/review may be required to finalize project documentation.

1.2 Authorization

Verbal authorization to proceed with this study was granted by Mr. Mike Reid of Rio Vista, LLC during our office meeting on August 16, 2007. Our study was accomplished in general accordance with our written proposal and scope of work for geotechnical services dated July 23, 2007. This report has been prepared for the exclusive use of Rio Vista, LLC and their agents, for specific application to this project.

Within the limitations of scope, schedule, and budget, our services have been performed in accordance with generally accepted geotechnical engineering and engineering geology practices in effect in this area at the time our report was prepared. No other warranty, express or implied, is made.

2.0 PROJECT AND SITE DESCRIPTION

This report was completed with an understanding of the project based on the boundary and topographic survey prepared by ESM Consulting Engineers, LLC (ESM) dated August 10, 2006 and our conversations with Mr. Mike Reid. It is our understanding that the existing residence and outbuildings will be demolished and replaced with a short-plat development consisting of an undetermined number of dwelling units.

The project site is located over 3.52 acres within Township 26N, Range 6E, Section 13 at 27066 NE 143rd Place in Duvall, King County, Washington. The site is bordered to the east, west and north by private, residential lots, and to the south by NE 143rd Place. Site topography is relatively flat with a very slight downward grade towards the northeast.

At present, the site supports one single-family residence with a detached garage and shop outbuilding across the west side of the property with driveway access from NE 143rd Place. Vegetation across the property consists of native field grass, a few stands of blackberry vines, and a small orchard of fruit trees in the southwest corner of the property.

3.0 SUBSURFACE EXPLORATION

Our field study included the excavation of two exploration pits to gain subsurface information about the site. The various types of sediments, as well as the depths where characteristics of the sediments changed, are indicated on the exploration logs presented in the Appendix. The depths indicated on the logs where conditions changed may represent gradational variations between sediment types in the field.

The conclusions and recommendations presented in this report are based on the two exploration pits completed for this study. The number, locations, and depths of the explorations were completed within site and budgetary constraints. Because of the nature of exploratory work below ground, extrapolation of subsurface conditions between field explorations is necessary. It should be noted that differing subsurface conditions may sometimes be present due to the random nature of deposition and the alteration of topography by past grading and/or filling. The nature and extent of any variations between the field explorations may not become fully evident until construction. If variations are observed at that time, it may be necessary to re-evaluate specific recommendations in this report and make appropriate changes.

3.1 Exploration Test Pits

The exploration pits were excavated using a rubber-tired backhoe under sub-contract to our firm. The pits permitted direct, visual observation of subsurface conditions. Materials encountered in the exploration pits were studied and classified in the field by an engineering

geologist from our firm. All exploration pits were backfilled immediately after examination and logging. Selected samples were then transported to our laboratory for further visual classification and testing, as necessary.

4.0 SUBSURFACE CONDITIONS

Subsurface conditions at the project site were inferred from the field explorations accomplished for this study, visual reconnaissance of the site, and review of selected geologic literature. As shown on the field logs, the explorations encountered a surficial layer of topsoil over dense till soils.

4.1 Stratigraphy

Topsoil

We encountered a thin, surficial layer of topsoil within each of our exploration pits EP-6 and EP-7. The topsoil generally consisted of loose, moist, dark brown, silty sand rich with organic material. We observed the topsoil to a maximum depth of 8 inches. This unit is not considered suitable for the support of foundations and should be removed from the foundation areas.

Lodgement Till

Natural sediments encountered below the topsoil consisted of medium dense to very dense, medium to fine grained sand with variable amounts of silt, gravel, and cobbles with occasional boulders. These sediments were interpreted to be representative of Vashon lodgement till. Vashon lodgement till consists of an unsorted mixture of silt, sand, gravel, and cobbles that was deposited by basal, debris-laden glacial ice during the Vashon Stade of the Fraser Glaciation approximately 12,500 to 15,000 years ago. Lodgement till typically possesses high strength and low compressibility attributes that are favorable for support of foundations, floor slabs, and paving with proper preparation. The high relative density characteristic of Vashon lodgement till is due to its consolidation by the massive weight of the glacial ice from which it was deposited. In the areas such as the subject site, where Vashon lodgement till sediments are exposed at or near the ground surface, the density of the upper several feet of the Vashon lodgement till typically becomes reduced to a loose to medium dense state by weathering. Where the weathering process rendered these soils loose, the weathered till is not suitable for direct foundation support, but may be suitable for reuse as structural fill provided it can be properly moisture conditioned and compacted to project specifications.

Our interpretations of subsurface conditions on-site are consistent with a published geologic map of the area (*Geologic Map of King County, Washington,* by Booth, Haugerud, and Sackett, 2002).

4.2 Hydrology

No ground water was encountered in any of our exploration pits at the time of our site exploration. It should be noted that fluctuations in the level of ground water may occur due to the time of the year and variations in rainfall, and may occur randomly from fill/disturbed soil layers.

II. GEOLOGIC HAZARDS AND MITIGATIONS

The following discussion of potential geologic hazards is based on the geologic, slope, and shallow ground water conditions as observed and discussed herein.

5.0 SEISMIC HAZARDS AND RECOMMENDED MITIGATION

Earthquakes occur in the Puget Lowland with great regularity. The vast majority of these events are small and are usually not felt. However, large earthquakes do occur as evidenced by the 1949, 7.2-magnitude event; the 1965, 6.5-magnitude event; and the 2001, 6.8-magnitude event. The 1949 earthquake appears to have been the largest in this area during recorded history and was centered in the Olympia area. Evaluation of earthquake return rates in the Puget Sound area indicates that an earthquake of the magnitude between 5.5 and 6.0 will likely occur every 25 to 40 years in the Puget Sound area.

Generally, there are four types of potential geologic hazards associated with large seismic events: 1) surficial ground rupture, 2) seismically induced landslides, 3) liquefaction, and 4) ground motion. The potential for each of these hazards to adversely impact the proposed project is discussed below.

5.1 Surficial Ground Rupture

The nearest known fault trace to the project site is the Seattle Fault located approximately 12 miles to the south. Recent studies by the U.S. Geological Survey (USGS) (e.g., Johnson et al., 1994, *Origin and Evolution of the Seattle Fault and Seattle Basin, Washington,* Geology, v. 22, pp. 71-74; and Johnson et al., 1999, *Active Tectonics of the Seattle Fault and Central Puget Sound Washington - Implications for Earthquake Hazards*, Geological Society of America Bulletin, July 1999, v. 111, n. 7, pp. 1042-1053) have provided evidence of surficial ground rupture along a northern splay of the Seattle Fault. The recognition of this fault splay is relatively new, and data pertaining to it are limited, with the studies still ongoing. According to the USGS studies, the latest movement of this fault was about 1,100 years ago when about 20 feet of surficial displacement took place. This displacement can presently be seen in the form of raised, wave-cut beach terraces along Alki Point in West Seattle and Restoration Point at the south end of Bainbridge Island. The recurrence interval of movement along this fault system is still unknown, although it is hypothesized to be in excess of several thousand years. Due to the suspected long recurrence interval, the potential for surficial ground rupture is considered to be low during the expected life of the proposed structures.

5.2 Seismically Induced Landslides

Based upon the relatively flat topography across the site, it is our opinion that the potential risk of damage to the proposed dwellings by seismically induced landsliding is low provided the recommendations contained herein are properly followed.

5.3 Liquefaction

Liquefaction is a condition that occurs when loose, saturated sand is subjected to a high intensity, cyclic loading (such as an earthquake) and it loses its shear strength. The encountered stratigraphy has a low potential for liquefaction due to its relatively dense, consolidated state and lack of adverse ground water conditions.

5.4 Ground Motion

Based on the encountered stratigraphy and our visual reconnaissance of the site, it is our opinion that earthquake damage to the proposed dwellings, when founded on a suitable bearing stratum in accordance with the recommendations presented in this report, would likely be caused by the intensity and acceleration associated with the event.

Guidelines presented in the 2006 *International Building Code* (IBC) standards using a Site Class "C" designation, as defined in Table 1613.5.2, were used. The 2006 IBC seismic design parameters for short period (S_S) and 1-second period (S_1) spectral acceleration values were determined by the latitude and longitude of the project site using the USGS National Seismic Hazard Mapping Project website (http://earthquake.usgs.gov/hazmaps/). Based on the 2002 data, the USGS website interpolated ground motions at the project site for the period of 0.2 seconds at 1.018 and for a 1-second period at 0.33, with a 2 percent chance of exceedence in 50 years.

6.0 EROSION HAZARDS AND MITIGATION

As of October 1, 2008, the Washington State Department of Ecology (Ecology) Construction Storm Water General Permit (also known as the National Pollutant Discharge Elimination System [NPDES] permit) requires weekly Temporary Erosion and Sedimentation Control (TESC) inspections AND weekly turbidity monitoring of storm water leaving the site for all sites 1 or more acres in size that discharge storm water to surface waters of the state. The TESC inspections and turbidity monitoring must be completed by a Certified Erosion and Sediment Control Lead (CESCL) for the duration of the construction. TESC reports and weekly turbidity levels do not need to be sent to Ecology, but should be logged into the project Storm Water Pollution Prevention Plan (SWPPP). If the project does not require a SWPPP, the TESC reports should be kept in a file on-site, or by the permit holder if there is no facility

on-site. Ecology requires a monthly summary report of the turbidity monitoring results (if performed) signed by the NPDES permit holder. If the monitored turbidity equals or exceeds 25 nephelometric turbidity units (NTUs) (Ecology benchmark standard), the project best management practices (BMPs) should be modified to decrease the turbidity of storm water leaving the site. Changes and upgrades to the BMPs should be continued until the weekly turbidity reading is 25 NTU or lower. If the monitored turbidity exceeds 250 NTU, the results must be reported to Ecology within 24 hours and corrective action taken. Daily turbidity monitoring is continued until the corrective action lowers the turbidity to below 25 NTU.

In order to meet the current Ecology requirements, a properly developed, constructed, and maintained erosion control plan consistent with the City of Duvall standards and best management erosion control practices will be required for this project. Associated Earth Sciences, Inc. (AESI) is available to assist the project civil engineer in developing site-specific erosion control plans. Based on past experience, it will be necessary to make adjustments and provide additional measures to the TESC plan in order to optimize its effectiveness. Ultimately, the success of the TESC plan depends on a proactive approach to project planning and contractor implementation and maintenance.

The erosion hazard of the site soils is high. The most effective erosion control measure is the maintenance of adequate ground cover. Maintaining cover measures atop disturbed ground provides the greatest reduction to the potential generation of turbid runoff and sediment transport. During the local wet season (October 1st through March 31st), exposed soil should not remain uncovered for more than 2 days unless it is actively being worked. Ground cover measures can include erosion control matting, plastic sheeting, straw mulch, crushed rock or recycled concrete, or mature hydroseed.

Flow-control measures are also essential for collecting and controlling the site runoff. Flow paths across slopes should be kept to less than 50 feet in order to reduce the erosion and sediment transport potential of concentrated flow. Ditch/swale spacing will need to be shortened with increasing slope gradient. Ditches and swales that exceed a gradient of about 7 to 10 percent, depending on their flow length, should have properly constructed check dams installed to reduce the flow velocity of the runoff and reduce the erosion potential within the ditch. Flow paths that are required to be constructed on gradients between 10 to 15 percent should be placed in a riprap-lined swale with the riprap properly sized for the flow conditions. Flow paths constructed on slope gradients steeper than 15 percent should be placed in a pipe slope drain. AESI is available to assist the project civil engineer in developing a suitable erosion control plan with proper flow control.

Some fine-grained surface soils are the result of natural weathering processes that have broken down parent materials into their mineral components. These mineral components can have an inherent electrical charge. Electrically charged mineral fines will attract oppositely charged particles and can combine (flocculate) to form larger particles that will settle out of suspension.

The sediments produced during the recent glaciation of Puget Sound are, however, most commonly the suspended soils that are carried by site storm water. The fine-grained fraction of the glacially derived soil is referred to as "rock flour," which is primarily a silt-sized particle with no electrical charge. These particles, once suspended in water, may have settling times in periods of months, not hours.

Therefore, the flow length within a temporary sediment control trap or pond has virtually no effect on the water quality of the discharge since it is not going to settle out of suspension in the time it takes to flow from one end of the pond to the other. Reduction of turbidity from a construction site is almost entirely a function of cover measures and flow control. Temporary sediment traps and ponds are necessary to control the release rate of the runoff and to provide a catchment for sand-sized and larger soil particles, but are very ineffective at reducing the turbidity of the runoff

Silt fencing should be utilized as buffer protection and not as a flow-control measure. Silt fencing is meant to be placed parallel with topographic contours to prevent sediment-laden runoff from leaving a work area or entering a sensitive area. Silt fences should not be placed to cross contour lines without having separate flow control in front of the silt fence. A swale/berm combination should be constructed to provide flow control rather than let the runoff build up behind the silt fence and utilize the silt fence as the flow-control measure. Runoff flowing in front of a silt fence will cause additional erosion and usually will cause a failure of the silt fence. Improperly installed silt fencing has the potential to cause a much larger erosion hazard than if the silt fence was not installed at all. The use of silt fencing should be limited to protect sensitive areas, and swales should be used to provide flow control.

6.1 Erosion Hazard Mitigation

To mitigate and reduce the erosion hazard and potential for off-site sediment transport, we would recommend the following:

1. Surface water should not be allowed to flow across the site over unprotected surfaces.

2. Silt fences should be placed and maintained around the perimeter of the proposed construction area throughout the entire construction phase of the project until permanent landscaping and permanent storm water collection facilities have been installed.

3. Soils that are to be reused around the site should be stored in such a manner as to reduce erosion from the stockpile. Protective measures may include, but are not necessarily limited to, covering with plastic sheeting, the use of low stockpiles in flat areas behind the residence, or the use of straw bales and/or additional silt fences around pile perimeters. Soils should not be stockpiled on the steeply sloping portions of the lot.

4. Areas stripped of natural vegetation during construction should be replanted and mulched as soon as possible or otherwise protected.

5. All storm water from impermeable surfaces, including driveways and roofs, should be permanently tightlined to a suitable storm water collection system.

6. A rocked construction entrance should be provided for truck traffic onto and off the site.

III. PRELIMINARY DESIGN RECOMMENDATIONS

7.0 INTRODUCTION

Our exploration indicates that, from a geotechnical standpoint, the proposed project is feasible provided the recommendations contained herein are properly followed. The bearing stratum appears relatively shallow across the property; therefore, we recommend the use of standard spread footing foundations to support the new structures.

8.0 SITE PREPARATION

Any existing buildings, paving, buried utilities, and any other structures should be removed from areas where structures or paving are planned. Any existing underground heating oil storage tanks (USTs) should be emptied, inerted, and removed in accordance with Ecology requirements and the resulting excavation backfilled with structural fill. AESI can assist the owner with proper UST abandonment, sampling, contaminated soil remediation (in the unlikely event that contamination is encountered), and State-required documentation.

Adequate TESCs should be constructed in accordance with City of Duvall requirements and the project civil engineering TESC design. We recommend that the project contractor work together with the design team and the City of Duvall to design, install, and maintain the erosion control measures. It is easier and much less costly to keep fine-grained soils in place than it is to remove suspended sediment from site storm runoff.

Site preparation of planned structural fill, building, and paving areas should include removal of all trees, brush, landscaping, debris, and any other deleterious material. Additionally, the upper, organic topsoil should be removed and the remaining roots grubbed. One should refer to topsoil thicknesses depicted on the exploration logs in the Appendix for specific observed values. Topsoil materials will "swell" some 20 to 30 percent upon excavation. Project stripping volume estimates should include this swell factor. Areas where loose surficial soils exist below finished grade due to demolition or grubbing operations should be considered as fill to the depth of disturbance and treated as subsequently recommended for structural fill placement. Topsoil should be processed and reused as topsoil, if allowed by the project plans and specifications, or should be removed from the site or used as fill in non-structural areas. If some minor settlement can be tolerated, the topsoil could be placed as fill in non–structural areas as long as it is free of stumps and roots larger than 3 inches in diameter and is compacted to a firm and unyielding condition in lifts, as described for structural fill. We should be involved in the planning of the location and thickness of all fill and should observe the placement and compaction operations.

8.1 Site Drainage and Surface Water Control

Adequate temporary and permanent control of surface water runoff will be required in order to allow site access and grading for construction of the new buildings, access driveways, installation of underground utilities, and other proposed improvements. Excavation, filling, subgrade, and grade preparation should be performed in a manner and sequence that will provide controlled drainage at all times and proper control of erosion. Surface water should be collected and pumped or drained to provide a suitable working platform. Successful drainage of wet zones due to ground water flow and accumulations of surface water runoff could be accomplished by ditching and/or the installation of cut-off trenches or "French" drains, where necessary.

The site should be graded to prevent water from ponding in construction areas and/or flowing into excavations. Exposed grades should be crowned, sloped, and smooth-drum rolled at the end of each day to facilitate drainage. Accumulated water must be removed from subgrades and work areas immediately prior to performing further work in the area. Equipment access may be limited, and the amount of soil rendered unfit for use as structural fill may be greatly increased if drainage efforts are not accomplished in a timely sequence. If an effective drainage system is not utilized, project delays and increased costs could be incurred due to the greater quantities of wet and unsuitable fill, or poor access and unstable conditions.

Final exterior grades should promote free and positive drainage away from the building at all times. Water must not be allowed to pond or to collect adjacent to foundations or within the immediate building area. We recommend that a gradient of at least 3 percent for a minimum distance of 10 feet from the building perimeters be provided, except in paved locations. In paved locations, a minimum gradient of 1 percent should be provided unless provisions are included for collection and disposal of surface water adjacent to the structures.

8.2 Wet Weather Conditions

If construction proceeds during an extended wet weather construction period and the moisture-sensitive, silty site soils become wet, they will become unstable. Therefore, the bids for site grading operations should be based upon the time of year that construction will proceed. Construction during wet weather is expected to require protection of subgrades in staging and construction areas, as recommended in the "Subgrade Protection" section of this report. It is expected that in wet conditions, additional soils may need to be removed and/or other stabilization methods used, such as a coarse, crushed rock working mat to develop a stable condition if silty subgrade soils are disturbed in the presence of excess moisture. The severity of construction problems will be dependent, in part, on the precautions that are taken by the contractor to protect the moisture- and disturbance-sensitive site soils. If overexcavation is necessary, it should be confirmed through continuous observation and testing by a representative of our firm. The site contractor should provide properly surfaced access roads

to fill areas. Maintenance and reconstruction of access roads will be necessary. Daily "sealing" of fill surfaces and drainage improvements is required. It is the contractor's responsibility to keep silt-laden runoff to a minimum by protecting exposed subgrades and maintaining erosion control measures.

8.3 Subgrade Protection

The site soils that are expected below the building pad and paving subgrades contain a significant silt fraction and are considered to be moisture- and disturbance-sensitive. These soils will become unstable if disturbed by construction equipment while at elevated moisture contents, requiring additional soil removal at an increased cost. Therefore, in addition to the recommendations presented in the "Site Drainage and Surface Water Control" section of this report, site preparation and initial construction activities should be planned to minimize disturbance to the existing ground surface, particularly during extended wet weather periods and the wet season (typically October through May). Construction traffic should be restricted to specific rock-surfaced drive areas to limit the area where disturbance of the subgrade will occur. If site stripping and grading activities are performed during extended dry weather periods, we anticipate that site stabilization requirements will be much less.

If construction will proceed in the winter, we recommend the use of a temporary working surface of sand and gravel, crushed rock, or quarry spalls to protect the silty soils, particularly in areas supporting concentrated equipment traffic. In winter construction staging areas, a minimum thickness of 12 inches of quarry spalls or 18 inches of pit run sand and gravel is recommended. If subgrade conditions are soft and silty, a geotextile separation fabric, such as Mirafi 500x or approved equivalent, should be used between the subgrade and the new fill. For building pads where floor slabs and foundation construction will be completed in the winter, a similar working surface, composed of at least 12 inches of pit run sand and gravel or crushed rock, should be used. Construction of working surfaces from advancing fill pads could be used to avoid directly exposing the subgrade soils to vehicular traffic. Similar minimum (or greater) sections will be required for temporary access road construction.

Foundation subgrades may require protection from foot and equipment traffic, and ponding of runoff during wet weather conditions. Typically, compacted crushed rock or a lean-mix concrete mat placed over a properly prepared subgrade provides adequate subgrade protection. Foundation concrete should be placed and excavations backfilled as soon as possible to protect the bearing grade.

8.4 Proof-Rolling and Subgrade Compaction

Following the recommended demolition, site stripping procedures, and required excavation to grade, the stripped subgrade within the building and pavement areas should be proof-rolled with heavy, rubber-tired construction equipment, such as a fully loaded, tandem-axle dump

truck. Proof-rolling should be performed prior to structural fill placement or foundation excavation. The proof-roll should be observed by the geotechnical engineer so that any soft or yielding subgrade soils can be identified. Any soft/loose, yielding soils should be removed to a stable subgrade. The subgrade should then be scarified, adjusted in moisture content, and recompacted to the required density. Proof-rolling should only be attempted if soil moisture contents are at or near optimum moisture content. Proof-rolling of wet subgrades could result in further degradation. Low areas and excavations may then be raised to the planned finished grade with compacted structural fill. Subgrade preparation and selection, placement, and compaction of structural fill should be performed under engineering-controlled conditions in accordance with the project specifications.

8.5 Overexcavation/Stabilization

Construction during extended wet weather periods could create the need to overexcavate exposed soils if they become disturbed and cannot be recompacted due to elevated moisture content and/or weather conditions. During dry weather periods, soft/wet soils, which may need to be overexcavated, may be encountered in some portions of the site. If necessary, this should be confirmed through continuous observation and testing by AESI. Soils that have become unstable may require remedial measures in the form of one or more of the following:

1. Drying and recompaction. Selective drying may be accomplished by scarifying or windrowing surficial material during extended periods of dry and warm weather.

2. Removal of affected soils to expose a suitable bearing subgrade and replacement with compacted structural fill.

3. Mechanical stabilization with a coarse, crushed aggregate compacted into the subgrade, possibly in conjunction with a geotextile.

4. Admixture stabilization with cement powder. Admixture design and installation procedures need to be reviewed and approved by the design team and City prior to site use.

8.6 Temporary and Permanent Cut Slopes

In our opinion, stable construction slopes should be the responsibility of the contractor and should be determined during construction. For estimating purposes, however, we anticipate that temporary, unsupported cut slopes in the upper, weathered till section should not exceed a slope of 1.5H:1V (Horizontal:Vertical). Unsupported cut slopes within the unsaturated till at depth can be made at a maximum slope of 1H:1V or flatter. As is typical with earthwork operations, some sloughing and raveling may occur, and cut slopes may have to be adjusted in the field. If ground water seepage is encountered in cut slopes or if surface water is not routed

away from temporary cut slope faces, flatter slopes will be required. In addition, WISHA/OSHA regulations should be followed at all times. Permanent cut and structural fill slopes that are not intended to be exposed to surface water should be designed at inclinations of 2H:1V or flatter. Slopes that are intended to be exposed to surface water, such as bioswale side slopes, if planned, should be designed at inclinations of 3H:1V or flatter. All permanent cut or fill slopes should be compacted to at least 95 percent of the modified Proctor maximum dry density, as determined by *American Society for Testing and Materials* (ASTM):D 1557, and the slopes should be protected from erosion by sheet plastic until vegetation cover can be established during favorable weather.

8.7 Frozen Subgrades

If earthwork takes place during freezing conditions, all exposed subgrades should be allowed to thaw and then be recompacted prior to placing subsequent lifts of structural fill or foundation components. Alternatively, the frozen material could be stripped from the subgrade to reveal unfrozen soil prior to placing subsequent lifts of fill or foundation components. The frozen soil should not be reused as structural fill until allowed to thaw and adjusted to the proper moisture content, which may not be possible during winter months.

9.0 STRUCTURAL FILL

Structural fill may be necessary to establish desired grades. All references to structural fill in this report refer to subgrade preparation, fill type, placement, and compaction of materials as discussed in this section. If a percentage of compaction is specified under another section of this report, the value given in that section should be used.

After stripping, planned excavation, and any required overexcavation have been performed to the satisfaction of the geotechnical engineer/engineering geologist, the upper 12 inches of exposed ground should be recompacted to 90 percent of ASTM:D 1557. If the subgrade contains too much moisture, adequate recompaction may be difficult or impossible to obtain and should probably not be attempted. In lieu of recompaction, the area to receive fill should be blanketed with washed rock or quarry spalls to act as a capillary break between the new fill and the wet subgrade. Where the exposed ground remains soft and further overexcavation is impractical, placement of an engineering stabilization fabric may be necessary to prevent contamination of the free-draining layer by silt migration from below.

After recompaction of the exposed ground is tested and approved, or a free-draining rock course is laid, structural fill may be placed to attain desired grades. Structural fill is defined as non-organic soil, acceptable to the geotechnical engineer, placed in maximum 8-inch loose lifts with each lift being compacted to 95 percent of ASTM:D 1557. In the case of roadway and utility trench filling, the backfill should be placed and compacted in accordance with City of

Duvall codes and standards. The top of the compacted fill should extend horizontally outward a minimum distance of 3 feet beyond the location of the perimeter footings or roadway edges before sloping down at a maximum angle of 2H:1V.

The contractor should note that any proposed fill soils must be evaluated by AESI prior to their use in fills. This would require that we have a sample of the material 72 hours in advance of filling activities to perform a Proctor test and determine its field compaction standard. Soils in which the amount of fine-grained material (smaller than the No. 200 sieve) is greater than approximately 5 percent (measured on the minus No. 4 sieve size) should be considered moisture-sensitive. Use of moisture-sensitive soil in structural fills should be limited to favorable dry weather conditions. The on-site, native till soils contain substantial amounts of silt and fine sand and are considered highly moisture-sensitive when excavated and used as fill materials. We anticipate that all excavated site soils will require aeration and drying prior to compaction in structural fill applications. Construction equipment traversing the site when the soils are wet can cause considerable disturbance. If fill is placed during wet weather or if proper compaction cannot be obtained, a select, import material consisting of a clean, free-draining gravel and/or sand should be used. Free-draining fill consists of non-organic soil with the amount of fine-grained material limited to 5 percent by weight when measured on the minus No. 4 sieve fraction and with at least 25 percent retained on the No. 4 sieve.

If fill is placed during wet weather or if proper compaction cannot be obtained, a select import material consisting of a clean, free-draining gravel and/or sand should be used. Free-draining fill consists of non-organic soil with the amount of fine-grained material limited to 5 percent by weight when measured on the minus No. 4 sieve fraction and at least 25 percent retained on the No. 4 sieve.

10.0 HOUSE FOUNDATIONS

Spread footings may be used for building support when founded upon the native, at least medium dense till soils prepared as recommended in this report. Based on our observations, suitable foundation bearing soils are expected approximately 2 feet below the existing ground surface. If existing fill is discovered around existing structures, it should be removed and replaced with structural fill, which is also suitable for foundation support.

The footings for the proposed dwelling units may be designed for an allowable foundation soil bearing pressure of 2,500 pounds per square foot (psf) including both dead and live loads. An increase of one-third may be used for short-term wind or seismic loading. Perimeter footings should be buried at least 18 inches into the surrounding soil for frost protection. However, all foundation elements must penetrate to the prescribed bearing stratum and no foundation elements should be constructed in or above loose, organic, or existing fill soils.

Anticipated settlement of footings founded as recommended should be on the order of ¾ inch or less, with differential settlement of ½ inch or less. However, disturbed material not removed from footing trenches prior to footing placement could result in increased settlements. All footing areas should be inspected by AESI prior to placing concrete to verify that the foundation subgrades are undisturbed and construction conforms to the recommendations contained in this report. Such inspections may be required by City of Duvall. Perimeter footing drains should be provided as discussed under the section on "Drainage Considerations."

It should be noted that the area bounded by lines extending downward at 1H:1V from any footing must not intersect another footing or intersect a filled area which has not been compacted to at least 95 percent of ASTM:D 1557. In addition, a 1.5H:1V line extending down and away from any footing must not daylight because sloughing or raveling may eventually undermine the footing. Thus, footings should not be placed near the edge of steps or cuts in the bearing soils.

11.0 FLOOR SUPPORT

We anticipate the use of concrete, slab-on-grade floors within the proposed dwelling units. The concrete, slab-on-grade floor should be cast atop a minimum of 4 inches of pea gravel to act as a capillary break. The floors should also be protected from dampness by covering the capillary break layer with an impervious moisture barrier at least 10 mils in thickness. Floor slabs may be supported by native soils.

12.0 DRAINAGE CONSIDERATIONS

All retaining and perimeter footing walls should be provided with a drain at the footing elevation. Drains should consist of rigid, perforated, polyvinyl chloride (PVC) pipe surrounded by washed pea gravel or drain rock. The level of the perforations in the pipe should be set approximately 2 inches below the bottom of the footing and should be constructed with sufficient gradient to allow gravity discharge away from the buildings. In addition, all retaining walls should be lined with a minimum, 12-inch-thick, washed gravel or washed crushed rock blanket provided over the full height of the wall that ties into the footing drain. Roof and surface runoff should not be discharged into the footing drain system, but should be handled by a separate, rigid, tightline drain. In planning, exterior grades adjacent to walls should be sloped downward away from the structures to achieve surface drainage.

13.0 PROJECT DESIGN AND CONSTRUCTION MONITORING

At the time of this report, site grading, structural plans, and construction methods have not been finalized, and the recommendations presented herein are preliminary. We are available to provide additional geotechnical consultation as the project design develops and possibly changes from that upon which this report is based. We recommend that AESI perform a geotechnical review of the plans prior to final design completion. In this way, our earthwork and foundation recommendations may be properly interpreted and implemented in the design.

We are also available to provide geotechnical engineering and monitoring services during construction. The integrity of the foundations for buildings and of new pavement depends on proper site preparation and construction procedures. In addition, engineering decisions may have to be made in the field in the event that variations in subsurface conditions become apparent. Construction monitoring services are not part of the current scope of work. If these services are desired, please let us know and we will prepare a cost proposal.

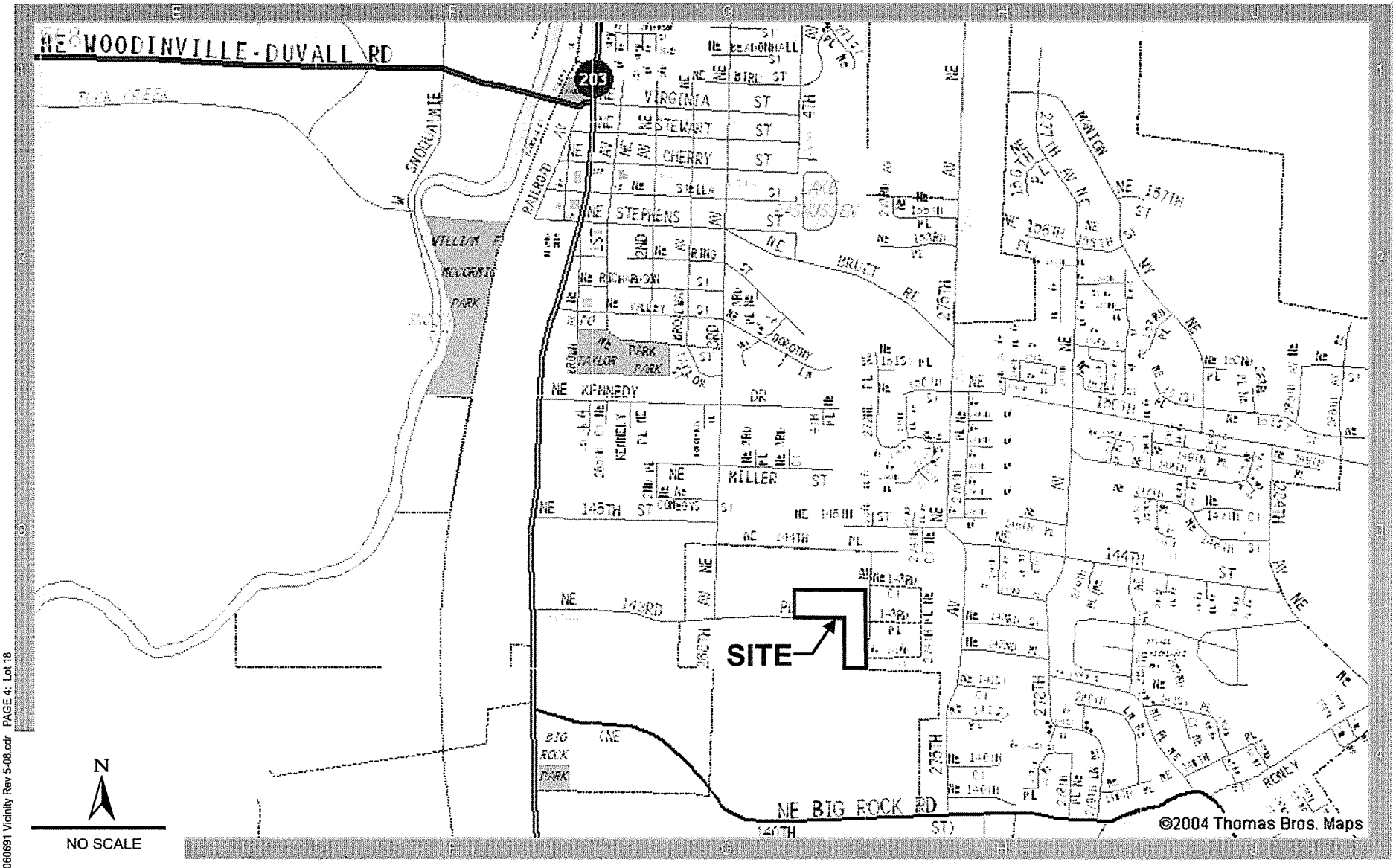
We have enjoyed working with you on this study and are confident that these recommendations will aid in the successful completion of your project. If you should have any questions, or require further assistance, please do not hesitate to call.

Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

Jesse P. Overton, P.G.
Project Geologist

Matthew A. Miller, P.E.
Associate Engineer

Attachments: Figure 1: Vicinity Map
Figure 2: Site and Exploration Plan
Appendix: Exploration Logs



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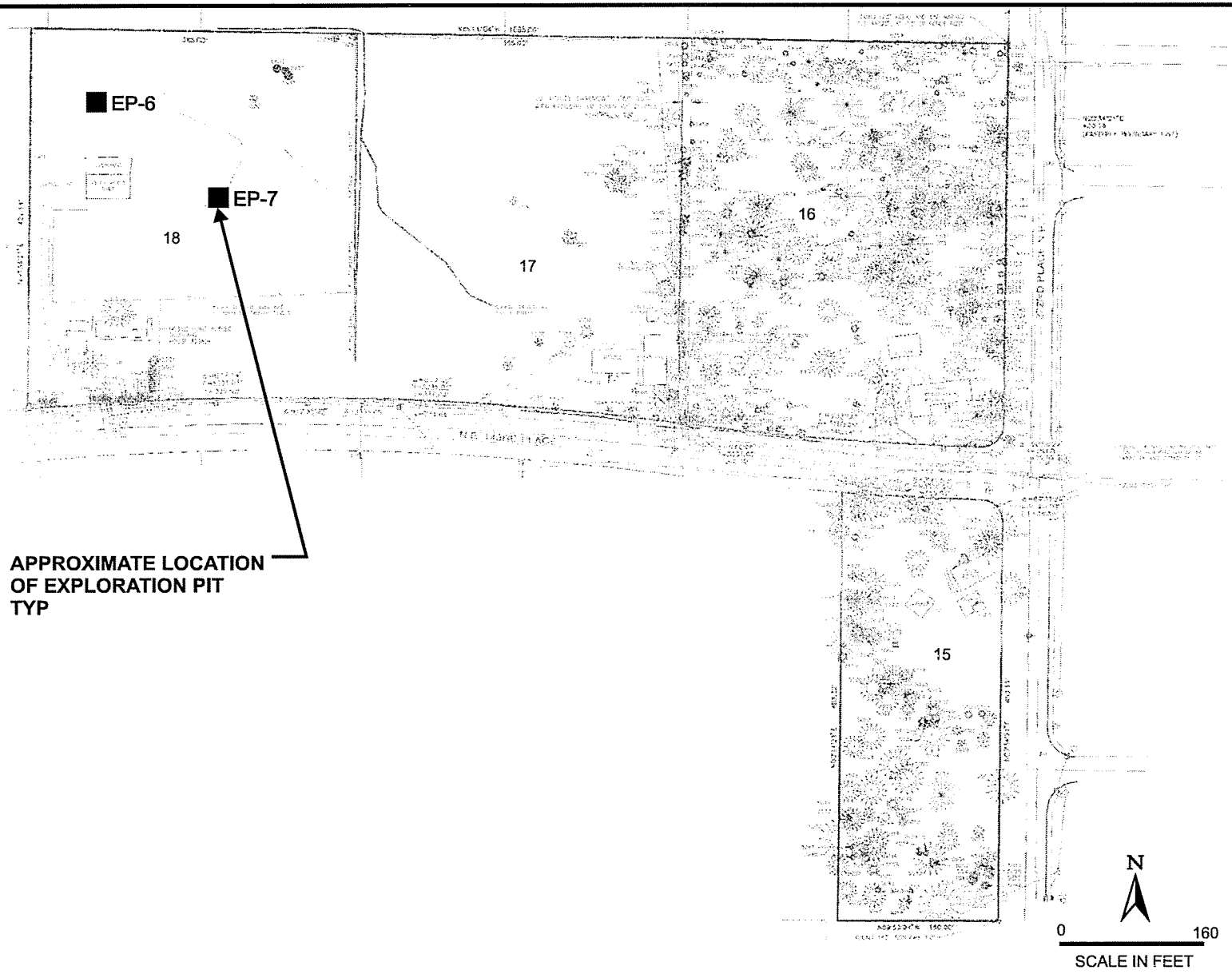
VICINITY MAP
RIO VISTA - LOT 18
DUVALL, WASHINGTON

FIGURE 1

DATE 5/08

PROJ. NO. KE060691D

060691 Rio Vista-Lots 15-18 \ 060691 Vicinity Rev 5-08.cdr PAGE 4: Lot 18



Reference: ESM

Associated Earth Sciences, Inc.

SITE AND EXPLORATION PLAN
RIO VISTA - LOT 18
DUVALL, WASHINGTON

FIGURE 2

DATE 5/08

PROJ. NO. KE060691D

060691 Rio Vista Lots 15-18 \ 060691 Site and Expl-D-Rev 5-08 cdr

APPENDIX

Coarse-Grained Soils - More than 50%[1] Retained on No. 200 Sieve	Gravels - More than 50%[1] of Coarse Fraction Retained on No. 4 Sieve	≤5% Fines[5]	GW	Well-graded gravel and gravel with sand, little to no fines
			GP	Poorly-graded gravel and gravel with sand, little to no fines
		≥15% Fines[5]	GM	Silty gravel and silty gravel with sand
			GC	Clayey gravel and clayey gravel with sand
	Sands - 50%[1] or More of Coarse Fraction Passes No. 4 Sieve	≤5% Fines[5]	SW	Well-graded sand and sand with gravel, little to no fines
			SP	Poorly-graded sand and sand with gravel, little to no fines
		≥15% Fines[5]	SM	Silty sand and silty sand with gravel
			SC	Clayey sand and clayey sand with gravel
Fine-Grained Soils - 50%[1] or More Passes No. 200 Sieve	Silts and Clays Liquid Limit Less than 50		ML	Silt, sandy silt, gravelly silt, silt with sand or gravel
			CL	Clay of low to medium plasticity; silty, sandy, or gravelly clay, lean clay
			OL	Organic clay or silt of low plasticity
	Silts and Clays Liquid Limit 50 or More		MH	Elastic silt, clayey silt, silt with micaceous or diatomaceous fine sand or silt
			CH	Clay of high plasticity, sandy or gravelly clay, fat clay with sand or gravel
			OH	Organic clay or silt of medium to high plasticity
Highly Organic Soils			PT	Peat, muck and other highly organic soils

Terms Describing Relative Density and Consistency

	Density	SPT[2] blows/foot
Coarse-Grained Soils	Very Loose	0 to 4
	Loose	4 to 10
	Medium Dense	10 to 30
	Dense	30 to 50
	Very Dense	>50

	Consistency	SPT[2] blows/foot
Fine-Grained Soils	Very Soft	0 to 2
	Soft	2 to 4
	Medium Stiff	4 to 8
	Stiff	8 to 15
	Very Stiff	15 to 30
	Hard	>30

Test Symbols

G = Grain Size
M = Moisture Content
A = Atterberg Limits
C = Chemical
DD = Dry Density
K = Permeability

Component Definitions

Descriptive Term	Size Range and Sieve Number
Boulders	Larger than 12"
Cobbles	3" to 12"
Gravel	3" to No. 4 (4.75 mm)
Coarse Gravel	3" to 3/4"
Fine Gravel	3/4" to No. 4 (4.75 mm)
Sand	No. 4 (4.75 mm) to No. 200 (0.075 mm)
Coarse Sand	No. 4 (4.75 mm) to No. 10 (2.00 mm)
Medium Sand	No. 10 (2.00 mm) to No. 40 (0.425 mm)
Fine Sand	No. 40 (0.425 mm) to No. 200 (0.075 mm)
Silt and Clay	Smaller than No. 200 (0.075 mm)

[3] Estimated Percentage

Component	Percentage by Weight
Trace	<5
Few	5 to 10
Little	15 to 25
With	- Non-primary coarse constituents: ≥ 15% - Fines content between 5% and 15%

Moisture Content

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

Slightly Moist - Perceptible moisture

Moist - Damp but no visible water

Very Moist - Water visible but not free draining

Wet - Visible free water, usually from below water table

Symbols

Sampler Type

2.0" OD Split-Spoon Sampler (SPT)

Bulk sample

Grab Sample

Blows/6" or portion of 6"

10 15 20

Sampler Type Description
3.0" OD Split-Spoon Sampler
3.25" OD Split-Spoon Ring Sampler
3.0" OD Thin-Wall Tube Sampler (including Shelby tube)
Portion not recovered

Cement grout surface seal

Bentonite seal

(4)

Filter pack with blank casing section

(4)

Screened casing or Hydrotip with filter pack

End cap

(1) Percentage by dry weight
(2) (SPT) Standard Penetration Test (ASTM D-1586)
(3) In General Accordance with Standard Practice for Description and Identification of Soils (ASTM D-2488)
(4) Depth of ground water
ATD = At time of drilling
Static water level (date)
(5) Combined USCS symbols used for fines between 5% and 15%

Classifications of soils in this report are based on visual field and/or laboratory observations, which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field or laboratory testing unless presented herein. Visual-manual and/or laboratory classification methods of ASTM D-2487 and D-2488 were used as an identification guide for the Unified Soil Classification System.

Associated Earth Sciences, Inc.

EXPLORATION LOG KEY

FIGURE A1

blocks\log_key.dwg LAYOUT: Layout2

LOG OF EXPLORATION PIT NO. EP-6

Depth (ft)	This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered. **DESCRIPTION**
	Topsoil
1	Medium dense, moist, brown, silty fine SAND, trace gravel.
2	**Vashon Till**
3	Dense, moist, gray, silty SAND, little gravel.
4	Dense, moist, gray, fine to medium SAND, little gravel, little silt, trace cobbles.
5	
6	
7	
8	Very dense, moist, gray, silty SAND, little gravel.
9	
10	
11	Bottom of exploration pit at depth 10 feet
12	
13	
14	
15	
16	
17	
18	
19	
20	

Rio Vista, Lot 18
Duvall, WA

Associated Earth Sciences, Inc.

Logged by: JPO

Approved by:

Project No. KE060691D

9/5/07

KCTP3 060691D.GPJ May 20, 2008

LOG OF EXPLORATION PIT NO. EP-7

Depth (ft)	This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered. **DESCRIPTION**
	Topsoil
1	**Vashon Till** Dense, moist, gray, silty fine to medium SAND, little gravel, trace cobbles.
2	
3	
4	
5	
6	More dense with depth.
7	
8	
9	Bottom of exploration pit at depth 8 feet Very difficult digging.
10	
11	
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Rio Vista, Lot 18
Duvall, WA

Associated Earth Sciences, Inc.

Logged by: JPO

Approved by:

Project No. KE060691D

9/5/07

KCTP3 060691D.GPJ May 20, 2008

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