Pre-Design Report

160th Avenue NE Extension

Prepared for
City of Redmond

November 2012

CH2M HILL®
1100 112th Avenue NE
Suite 400
Bellevue, WA 98004
November 14, 2012

430676

Mr. Steven Gibbs, P.E., Project Manager
City of Redmond
PO Box 97010 MS: 1NPW
Redmond, WA 98073-9710

Subject: 160th Avenue NE Extension Pre-Design Report

Dear Steve:

We are pleased to submit the attached Pre-Design Report for your 160th Avenue NE Extension Project.

We have prepared this report to investigate existing conditions, evaluate alternatives, and estimate costs for construction and funding purposes. Included in this report are the following:

- Executive Summary
- Introduction and Purpose
- Alignment Alternatives Considered
- Alternative Screening and Selection
- Design Criteria and Assumptions
- Right-of-Way
- Environmental
- Cost Estimates
- Redmond-Woodinville Road Intersection Analysis

We would like to thank you and your staff for your assistance and support. We look forward to the opportunity to provide our services to the City of Redmond again in the future.

Sincerely,

[Signature]

Roger J. Mason, P.E.
Project Manager
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Executive Summary

Introduction

The purpose of this report is to provide a preliminary design concept for the 160th Avenue NE Extension Project (Project) in the City of Redmond (City) that accurately represents the current and future corridor needs of the City as well as the surrounding neighborhoods. This corridor has been analyzed previously and a final supplemental Environmental Impact Statement (EIS) was issued in 1999. Since that time, the city’s future vision for the corridor has broadened to include a greater variety of users and modes. This report provides details of the revised design and includes information on the design, costs, and environmental requirements.

For the Project, three primary alternatives were developed in conjunction with design preferences from City staff and City Zoning Codes that included varying horizontal and vertical alignments, right-of-way footprints, retaining walls, wetland and stream crossing points, and varying cross sections. Ultimately, an alternative was selected for further evaluation that provided reduced project cost and entailed the least amount of additional right-of-way acquisition by making use of previously acquired city parcels, intended for the corridor. Additionally, the selected alternative provides a more direct horizontal and vertical alignment and provides a safe corridor for all users.

The preferred alternative in the 1999 EIS consisted of a roadway section with five lanes. While this previous alternative more than doubles the roadway capacity of the adjacent Redmond-Woodinville Road (Red-Wood Road), it does not fit within the city’s current vision or the context of the adjacent roadway network and residential neighborhoods. The narrower section consisting of two traffic lanes, two bike lanes, and two sidewalks, was selected after several iterations internally within CH2M HILL and with input from City staff. This alternative reflects a design that satisfies the City’s vision of a future corridor that would reduce costs and environmental impacts resulting from a 5-lane roadway but would still accommodate the needs of pedestrians, bicyclists, and trail users. The selected roadway section is adaptable to a future higher roadway capacity configuration including a second uphill lane with minimal changes should the city determine the necessity for this future capacity increase.

After the preferred alternative and roadway section was selected, further analysis, including fieldwork to determine boundaries and categories, of the wetlands and streams and methods of crossing those critical areas was studied to minimize cost and impacts. At the largest wetland crossing immediately south of the Puget Sound Energy (PSE) transmission lines, several crossing structures were evaluated, including a bridge, precast concrete culverts, and steel culverts. A precast concrete arch culvert was selected for its simplicity, cost effectiveness, context within the Project setting, and PSE trail undercrossing adaptability.

Additional work included consideration of slope stability and suitability of Project-related walls, stormwater runoff processing and mitigation, and the environmental requirements including the National Environmental Protection Act (NEPA), State Environmental Protection Act (SEPA), and the required permits.

Budget Level Costs

A summary of the estimated budget-level costs for right-of-way, Project development, and construction of the preferred alternative for 160th Avenue NE Extension is provided in this report. The estimated costs for construction were developed assuming 2012 unit prices including 30 percent contingency and were escalated to a mid-point of construction for year 2015, assuming a 3 percent per year escalation factor.
The estimated costs for right-of-way, Project development and construction management included contingencies ranging from 25 to 35 percent.

The $13.4 million total cost estimated for the Project as currently envisioned is approximately 59 percent lower than the $33 million presented in the City’s 2011 Transportation Improvement Program. The primary differences are due to reducing the cross-section from five lanes to two lanes, and crossing the existing major stream crossing with a precast concrete culvert instead of a larger concrete bridge structure. These changes have a dramatic effect on the cost of the Project when considering the steep terrain and wetland conditions that exist along the corridor.

**Project Schedule**

The overall Project schedule is dependent on the timing of available Project funding. Assuming that adequate Project funding can be obtained during the 2013-2016 funding cycles, the following general schedule is anticipated for the Project.

- Obtain full Project and/or design funding: 2013-2014
- Preliminary engineering and environmental documentation: 2014
- Obtain right-of-way and construction funding (if necessary): 2014-2015
- Right-of-way acquisition: 2015
- Final design and plans and specifications: 2015
- Construction: 2016

**Funding**

Due to the nature of this Project being more of a missing link connection between two residential streets, the primary funding source for this Project would most likely be the City. City sources could involve developer mitigation, stormwater and water quality, utility, and perhaps Parks and Recreation for improvements and connections related to the Powerline Trail. Other potential sources could include Transportation Improvement Board; other State funds related to water quality, stream enhancement, or wetlands; federal, or Puget Sound Regional Council funds. The traffic signal at State Route (SR) 202 could be eligible for Washington State Department of Transportation (WSDOT) funds. Project readiness is a key criterion for funding agencies. Advancing the Project to obtain environmental approvals and needed right-of-way are key milestones that improve the Project’s rating and scoring in a competitive grant process. Based on the schedule noted above, the City should plan to secure funds for this activity as early as 2013.
SECTION 1
Introduction and Purpose

1.1 Introduction
The 160th Avenue NE Extension Project (Project) aims to improve the safety of all modes of travel, accommodate the increased need for additional vehicle capacity within the corridor, and provides essential arterial linkages out of Redmond’s downtown core. The Project’s solutions must balance the needs of the future vehicular traffic along with the needs of pedestrians, bicycles, and equestrians; minimize disturbance to the environment and properties; and define a solution that is practical and economically feasible for the City.

The Project would extend 160th Avenue NE from its current barricaded terminus location just north of NE 98th Street to Red-Wood Road near NE 107th Court. This extension is approximately 1,300 feet long. A vicinity map of the Project area is provided in Figure 1.

Within the corridor, a number of unique aspects will need to be addressed:

- Steep terrain and the presence of wetlands;
- Roadway retaining walls;
- Crossing of three ravines with streams and associated wetlands;
- Crossing of the Seattle Public Utilities (SPU) 54-inch water line;
- Undercrossing of overhead PSE transmission lines and support towers; and
- The need for a grade-separated crossing of the Powerline Trail located within the PSE right-of-way.

These components and key Project issues are summarized in Figure 2.

1.2 Pre-Design Report Purpose

The purpose of this Pre-Design Report is to document the alternatives development and selection process, identify design criteria, and define Project costs to support the funding application process for the final design and construction of 160th Avenue NE Extension. This report summarizes the alternative development and selection process, including:

- Identification of alternatives considered
- Development of the alternatives
- Evaluation, screening, and selection of the preferred alternative
- Development of budget-level costs for the preferred alternative
- Providing a matrix of required environmental permits

1.3 Project Purpose

The 160th Avenue NE Extension Project is a necessary improvement because of limited multimodal access for all users, and site constraints to expanding Red-Wood Road within the existing corridor. This proposed corridor would more efficiently handle congestion, increase safety, and provide multimodal access from the Downtown Redmond core to and from the north. The Project objectives have evolved from the 1999 Supplemental Environmental Impact Statement (SEIS) into the following:

- Redistribute traffic flow and reduce overall congestion.
- Provide a new arterial into the central business district.
• Improve safety in the corridor.
• Provide bicycle, pedestrian and equestrian facilities.
• Minimize impacts to the environment.
• Provide a corridor that accomplishes the aforementioned points in a cost efficient manner.

This Pre-Design phase of the Project consists of the preliminary design and feasibility of extending 160th Avenue NE. The preliminary design of the Project improvements will involve establishing a cross-section and optimizing a Project footprint to determine the right-of-way needs, environmental impacts, and estimated costs of the Project.

1.4 Existing Conditions

The existing site includes a steep longitudinal cross slope with interspersed low-quality wetlands and seasonal creeks. Vegetation is a combination of riparian grasses on the west and pockets of evergreen trees in the east. The existing hillside is unstable near the south end of the Project but becomes increasingly stable once the impacted areas extend to the north. Currently, overhead power lines and underground utility corridor crosses the proposed alignment as well as a moderately maintained, steep, multi-use trail. There are two dwelling units in the southeast quadrant of the Project area. Photos of the existing conditions can be found in Appendix C.

1.4.1 Survey and Basemapping

CH2M HILL performed a topographic survey in support of the proposed improvements for 160th Avenue NE covering the segment of undeveloped area between the northerly and southerly termini of the existing roadway. During the survey, the survey team recovered existing monuments based on a previous survey provided to CH2M HILL by the City.

The map is tied to City horizontal control points "GPS903E1," "GPS903E4," "GPS903D4," and "GLO3DS and City BM # COR9177. The horizontal datum is NAD 83 (91) and the vertical datum is NAVD 88. The mapping was collected using cross-section method at 50-foot intervals, with grade locations at a maximum of 25-foot spacing. It depicts existing features including roadways, right-of-way lines, parcel boundary lines and ownerships, easements, major utilities, walls, structures, culverts, fences, towers, trees, grade breaks, streams, and wetland flags. A digital terrain model (DTM) was generated from surveyed elevations to generate 2-foot contours. An electronic copy of the DTM and basemap can be found in Appendix D.
Figure 1: Vicinity Map
SECTION 1 - INTRODUCTION AND PURPOSE

Figure 2: Issues Map

Design Area 1
- Wetlands
- Slope stability at hillside

Design Area 2
- Crossing of streams/wetlands at ravines
- PSE corridor with trail crossing, SPU water main, and transmission lines

CITY OF REDMOND – 160TH AVENUE NE EXTENSION PRE-DESIGN REPORT
NOVEMBER 2012
SECTION 2
Alignment Alternatives Considered

2.1 Introduction
As shown in Figure 3, three horizontal alignments were initially considered for the City’s 160th Avenue Extension Pre-Design Project to evaluate, select, and optimize an alignment that balances right-of-way impacts, environmental impacts, and earthwork and structural impacts. Field survey data of this steep and varied terrain was utilized in conjunction with the horizontal alignments to develop optimized vertical alignments. Key metrics for horizontal and vertical alignment alternative development included the following:

- Economize material and construction costs.
- Minimize impacts to environmentally sensitive areas (wetlands, tree strands).
- Minimize right-of-way takes beyond existing ownership.
- Allow multimodal facilities within the corridor as well as crossing the corridor.
- Ensure design does not preclude Americans with Disabilities Act of 1990 (ADA) design compliant trail crossing and associated approaches.
- Minimize wall, structure, and earthwork costs as much as possible.

The 1999 SEIS identified a preferred corridor and alignment. Based on information from City staff, an isolated parcel within the corridor was acquired during an adjacent real estate transaction sometime in the early 2000s. This parcel was located based on the preferred alignment in the SEIS. In addition to developing an alignment alternative that maintained the roadway within the limits of the parcel, two other alignment alternatives were developed that would require additional adjacent parcel acquisitions. All alignment alternatives avoided impacts to existing overhead power lines and existing underground water and gas lines while not precluding future expansions within the current utility corridor (utilidor) that the proposed roadway crosses. Each alternative was also carefully designed to allow potential at grade or grade separated crossing of the existing Powerline Trail, which crosses the site and any future improvements to the trail. Section 4 of this report details the selection process.

2.2 Alignment Alternative 1
Reduced Right-of-Way Impacts
Alignment 1 (shown in orange on) utilizes direct geometry to minimize curves while avoiding wetland as much as possible. This alternative will require some realignment of the existing roadway adjacent to the Riverpoint complex and requires a steep grade exceeding 10 percent in the southern portion of the Project to minimize excessively large walls and closely follow the terrain. In the northern portion, the alignment has flatter grades that require additional walls and earthwork. Additionally, the alternative crosses the largest wetland, wetland 3A, at the widest point. Where possible, the alignment stays close to existing right-of-way pockets. However, the alternative would require additional “sliver” takes in
addition to existing city property. Parcel impacts include 0225059135 (Veal), 3526059123 (Veal), 3526059062 (PSE), and 3526059088 (King County).

2.3 Alignment Alternative 2

No Regard for Existing Right-of-Way
Alternative 2 (shown in magenta on Figure 3) follows a projection of the existing alignment and then turns sharply into the hillside to gain elevation and avoid wetland 1A while minimizing the crossing length of wetland 2A and requiring large amounts of earthwork and cut walls. With a maximum grade of 8 percent, the alignment climbs up the hillside and daylights at the bottom of wetland 3A. The alignment then curves to the west to minimize the crossing length of wetland 3 (adjacent to the PSE trail crossing) followed by a right turn to re-align with the portion of 160th Avenue adjacent to Redmond 74. The northern portion of the alignment closely follows the existing terrain. The portion would require only minimal walls and earthwork. This alignment requires require great amounts of additional right-of-way acquisitions due to its curving horizontal design. The alignment will affect the following parcels, 0225059135 (Veal), 3526059123 (Veal), 3526059062 (PSE), 3526059088 (PSE), and 3526059088 (King County).

2.4 Alignment Alternative 3

Maintains Predefined Corridor with the Least Right-of-Way Impacts
The main intent of Alternative 3 (shown in red on Figure 3) is to stay within the existing City right-of-way parcels. The alignment only features the curve just north of the Riverpoint development and affects parcels 0225059135 (Veal), 3526059062 (PSE), 3526059088 (PSE), and 3526059088 (King County). Generally, this is a straighter horizontal alignment, the maximum grade is limited to 8 percent, with only 650 feet of length greater than 5 percent, and optimizes the separation from the existing residence (Veal). Significant walls and earthwork are required along the entire length of this alignment. The crossing of wetland 3A is near the widest point, which creates the greatest impact.
SECTION 3

Alternative Screening and Selection

3.1 Overview of Public and Agency Involvement Process

A City review team comprised of several City departments has reviewed and guided the design effort to date. There has also been contact with key utility stakeholders as well as adjacent property owners. Moving forward, a public open house should be held to inform the public and other stakeholders about the Project.

3.2 Description of Screening Process

In a Project (Consultant) team meeting on April 17, 2011, the team developed screening criteria to provide a preliminary ranking of the three alternatives to focus our efforts on the most beneficial alignment. In support of this ranking, the team has considered key influences along the alignment that will drive Project costs as well as minimize impacts to the neighborhood and the environment. Ultimately, all Project attributes include an inherent cost that serves as a sufficient form of weighting and comparing Project attributes. For this particular Project, the Consultant team determined that the three top Project attributes to be examined with respect to cost are right-of-way impacts, environmental impacts, and earthwork and structures. During the evaluation process, the alignment alternatives were continuously compared against each other with the aforementioned attributes used to measure relative performance. Additional attributes that were used in comparison include:

- Accommodation of PSE trail crossing
- Roadway grade and safety criteria
- Traffic operations

3.3 Recommendations and Summary of Preferred Alignment Alternative Selection

As a result of the evaluation and screening process, alignment 3 is the preferred alternative for the 160th Avenue NE Extension. Alignment 3 provides a direct alignment centered in existing right-of-way parcels which reduces the need to acquire additional property and minimizes impacts to existing adjacent landowners. When analyzed in terms of cost with the environmental impacts of all three alignments, the cost of mitigating environmental impacts was roughly the same for all alignments whereas the right-of-way impacts were quite different. Alignment alternative 3 required the least amount of additional right-of-way when compared with the other alignments. While alignment alternative 3 required more earthwork and structures than alignment 1 and a longer crossing of wetland 3A than alignment 2, the relative costs of these additional expenditures were not enough to justify the additional right-of-way requirements for the other alignments. The maximum roadway grade and relatively simple horizontal layout of alignment 3 were also deciding factors as they were combined more desirable than either alternative alignment. Traffic operations should be nearly identical for all alignments.

It is important to note that while the three alignments were thoroughly compared and contrasted, they are very similar and all provide safe mobility for all users while minimizing impacts to existing utilities, not precluding new utilities, and accommodating a trail crossing of the PSE trail.
SECTION 4
Design Criteria and Assumptions

4.1 Roadway Geometrics and Cross Sections

Based on the City “Zoning Code, Appendix 2: Construction Specification and Design Standards for Streets and Access” the following design criteria were implemented in preliminary design. This geometric design criterion is summarized in Table 1. This is followed by additional detail regarding the basis of the selection.

Table 1: Geometric Design Criteria

<table>
<thead>
<tr>
<th>Design Element</th>
<th>Standard</th>
<th>Proposed Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>Functional classification</td>
<td>Minor Arterial</td>
<td>Minor Arterial</td>
</tr>
<tr>
<td>Posted speed</td>
<td>30 mph</td>
<td>30 mph</td>
</tr>
<tr>
<td>Design speed</td>
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<td>35 mph</td>
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<tr>
<td>Number of traffic lanes</td>
<td>4</td>
<td>2</td>
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<tr>
<td>Lane width</td>
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<td>Bike lanes</td>
<td>5.5 feet</td>
<td>5.5 feet</td>
</tr>
<tr>
<td>Sidewalk (both sides)</td>
<td>6 feet</td>
<td>6 feet</td>
</tr>
<tr>
<td>Landscape strip</td>
<td>5 feet</td>
<td>4 feet to 5 feet</td>
</tr>
<tr>
<td>Maximum grade</td>
<td>10 percent</td>
<td>8 percent</td>
</tr>
<tr>
<td>Minimum horizontal radius</td>
<td>510 feet</td>
<td>1300 feet</td>
</tr>
<tr>
<td>Minimum horizontal tangent runout</td>
<td>200 feet</td>
<td>915 feet</td>
</tr>
<tr>
<td>Flat stopping sight distance</td>
<td>305 feet</td>
<td>305 feet</td>
</tr>
<tr>
<td>Maximum stopping sight distance (based on maximum grade)</td>
<td>350 feet</td>
<td>350 feet</td>
</tr>
<tr>
<td>Minimum sag curve length (with lighting)</td>
<td>165 feet</td>
<td>400 feet</td>
</tr>
</tbody>
</table>

4.2 Functional Classification

Based on the City’s Transportation Master Plan the functional classification for 160th Avenue NE is a Minor Arterial. Geometric design criteria for minor arterials are shown in Table 2.

4.3 Posted Speed and Design Speed

Our recommendation is to use a posted and design speed of 30 miles per hour (mph) for the Project. This coincides with the existing portion of 160th Avenue NE that is adjacent to Downtown and the Riverpoint housing development. Per the City zoning codes, stopping sight distance is computed from a speed of 40 mph for a 30 mph corridor.
4.4 Roadway Cross-Section

The selection of the preferred section was an iterative one examining several interim configurations that provided flexibility for a final section. Although the preferred section does not meet all the design attributes in the City zoning code for a minor arterial, it will provide a safe and context sensitive design for the adjacent neighborhoods as well as natural areas.

4.4.1 Traffic Lanes

The 33 feet of roadway width leaves two configurations for the travelled lanes while providing width for bike lanes, sidewalks, and planter strips. This two-lane section is better aligned to the adjacent roadway network rather than a five-lane section that was originally proposed with the 1999 SEIS. The proposed “opening day” configuration will have two 11-foot lanes, one for each travel direction. AASHTO recommends 11-foot lanes on arterial streets as it helps moderate speeds and, in this particular situation, reduces the amount of impervious area and right-of-way impacts. With 22 feet of total lane width, this will leave another 11 feet of room for bike lanes. Based on city input and citing issues related to cost and environmental impacts, a two-lane section was selected. In the future, as travel demand increases, the roadway width can be repurposed to allow for three lanes - one downhill southbound lane and two uphill northbound lanes (*Figure 6*).

In the three-lane configuration, there will be no roadway width specifically purposed for bike lanes. In the southbound direction, the traveled lane will be a shared vehicle and bike lane. At the proposed 8 percent grade, it is reasonable to assume that bicycles will be traveling at the same 30 mph as the vehicles. In the northbound direction, there will be one general-purpose lane and one shared truck climbing and bike lane.

The placement of a climbing lane was evaluated; however, the AASHTO warrants and research for climbing lane placement indicate that at an 8 percent uphill grade, a speed reduction of less than 5 mph will occur over the distance of 130 feet (the length of the 8 percent grade) for a large truck with a starting speed of 70 mph. Furthermore, AASHTO indicates that a climbing lane may not be economically feasible if there is less than a 10 mph speed reduction for a heavy truck. Since the crawl speed for most trucks is between 20 and 30 mph, a truck driving up this grade will have a very small, if any, reduction in speed. Finally, the taper to add and remove the climbing lanes (300 and 600 feet, respectively, according to AASHTO) represent nearly 70 percent of the roadway length.

Ultimately, if the city sees fit, the three-lane configuration could be implemented at the roadways time of opening to mitigate effects of slower vehicles driving up the grade.

4.4.2 Bike Lanes

As mentioned above in Section 4.4.1, in the proposed “opening day” configuration, there will be 11 feet of roadway width that will be utilized as bike lanes. This width allows for a 5.5-foot bike lane in each direction. In the future configuration, the bike lanes will be eliminated and each direction will have a shared bike and vehicle lane.

4.4.3 Sidewalk

In both the northbound and southbound directions of the 160th Avenue NE Extension, there will be a 6-foot sidewalk. This sidewalk will meet all current ADA and accessibility guidelines, with a maximum cross-slope of 2 percent. Section R302.5 of the PROWAG (*Public Rights-of-Way Access Guide*), along with WSDOT’s *Field Guide for Accessible Rights-of-Way* – 2010 Edition, provides that longitudinal grades exceeding 5 percent are permitted on facilities adjacent to a street or road with a grade equal to or
greater than the adjacent sidewalk. In addition, at the intersection of 160th Avenue NE and Red-Wood Road, all designated pedestrian crossing locations will be appointed with ADA curb ramps and crosswalks.

4.4.4 Landscape Strip
There will be a 4-foot landscape strip located between the roadway curb and the sidewalk on the west side of 160th Avenue NE and a 5-foot landscape strip on the east side of 160th Avenue NE. These landscape strips will not only introduce green space onto the cross-section, but will also serve stormwater needs for the property. These landscaped strips will house stormwater filters that will infiltrate the roadway runoff as described in Section 4.15.

4.5 Maximum Grades
As shown in the preferred profile in Figure 7, a maximum 130-foot long grade of 8 percent is recommended to minimize earthwork quantities and walls. This will better align the roadway within the context of the existing topography.

4.6 Horizontal Radius
A minimum horizontal radius of 1,300 feet (on centerline) provides a safe and predictable driving path that includes sufficient horizontal sight distance so that sight lines are contained within the roadway prism.

4.7 Horizontal Tangent Runout
A distance of 915 feet between curves provides sufficient runout for superelevation and is long enough to prevent a condition of broken back curves.

4.8 Stopping Sight Distance
Based on City Zoning Code, Appendix 2, arterial stopping sight distance was computed based on a design speed of 10 mph above the assumed posted speed (30 mph).

4.9 Sag Curve Length
A 400-foot sag curve is provided near the southern end of the Project to provide a smooth driving experience while allowing sufficient stopping sight distance and headlight sight distance for drivers.

4.10 Equestrian Facilities
Based on the PSE Trail Report developed by Reid Middleton in 2009, a grade-separated trail crossing with vertical clearance minimum of 10 feet is to be provided in the design. As shown in the roadway profile in Figure 7, the 84-foot precast culvert at wetland 3 will allow for an equestrian crossing. The existing PSE trail will need to be realigned to descend underneath the 160th Avenue NE Extension and pass through the culvert. The 84-foot undercrossing can accommodate a 12-foot high by 12-foot wide box culvert adjacent to wetland 3 for equestrian use which meets the minimum criteria set forth in the 2009 PSE Trail Report.
4.11 Traffic Operations

Traffic operations along 160th Avenue NE will be influenced primarily by the major intersections at the northern terminus at Red-Wood Road and southern end at NE 90th Street. The intersection of NE 90th Street is currently signalized, while the intersection with Red-Wood Road is stop-controlled. Detailed traffic operations analysis was conducted for the 160th Avenue NE/Red-Wood Road intersection and is presented later in this report in Section 9, with the recommendation for a signalized intersection.

As discussed in Section 5.4.1., there are two proposed configurations for traffic lanes. The proposed “opening day” configuration will have two 11-foot lanes, one for each travel direction, with two 5.5-foot bike lanes. The ultimate configuration calls for three 11-foot travel lanes, with two northbound (uphill) lanes and one southbound lane. In the three-lane configuration, there will be no roadway width specifically purposed for bike lanes. In the southbound direction, the traveled lane will be a shared vehicle and bike lane. At the proposed 8 percent grade, it is reasonable to assume that bicycles will be traveling at the same 30 mph as the vehicles. In the northbound direction, there will be one general-purpose lane and one shared truck climbing and bike lane. In addition, bicyclists traveling northbound up the hill can use the adjacent sidewalk instead of the truck-climbing lane. The 2030 PM peak hour demand volume in the northbound direction is 815 vehicles per hour (vph). There is no existing or forecasted truck volume available for 160th Avenue NE, so 2 percent of the vehicles were assumed trucks (approximately 16 trucks). Based on methodology outlined in the HCM 2000, the arterial level of service (LOS) for northbound 160th Avenue NE is LOS C with one travel lane. Therefore, an additional climbing lane is not necessarily needed to improve traffic operational performance. Ultimately, if the city sees fit, the three-lane configuration could be implemented at the roadway’s time of opening to mitigate effects of slower vehicles driving up the grade.
4.12 Culvert Structures

The proposed roadway crosses over several ravines or lowland areas that require culverts for water passage. At all but one of these locations, relatively small diameter culverts can be used. However, the ravine located immediately south of the PSE transmission lines spans approximately 150 feet from bank to bank and is about 20 feet deep at the proposed road crossing. It contains seasonal stream flow and a wetland designated as wetland 3. It is assumed that a longer span culvert or bridge is required to provide for stream flow and minimize wetland impacts. In addition, it is desired to provide for an equestrian trail crossing under the roadway along the north bank of the ravine and connect to the PSE trail on each end. These culverts can be seen in Figure 7.

Several crossing structures were evaluated at wetland 3, including a pre-stressed concrete girder bridge as well as steel and precast concrete culverts of span lengths ranging from 48 to 100 feet. This evaluation is summarized in Appendix A-2, and based on the evaluation performed it is recommended than an 84-foot precast concrete culvert be used to cross wetland 3.

4.13 Retaining Walls

Retaining walls for this Project include both fill and cut walls. Fill walls are mainly needed on the west side of the proposed roadway to contain the roadway embankment fill. It is assumed that structural earth walls are the most economical fill wall type when the height is greater than about 10 to 12 feet. For fill walls less than this height, rockery walls are proposed as a cost-effective alternative.

Cut walls are required at two locations on the east side of the proposed roadway where the road is benched into the hillside. Typical cut wall types include concrete cantilever walls, soldier pile walls, and soil nail walls. It is presumed that soil nail walls are the most cost-effective choice, assuming that the soil can temporarily stand vertically several feet high to place the rows of soil nails. The cut walls can be significantly reduced or possibly eliminated if the cut slope can extend past the Veal property limit.

The retaining walls are shown in Figure 4, and an evaluation of retaining walls needed for the new roadway is included in Appendix A-1.

4.14 Geotechnical

Appendix A-3 contains a summary of the geotechnical work. It contains a geotechnical findings report summarizing field observations and a summary of work at or near this site that has been completed by others. The geotechnical work on the Project was based on field observations and review of published and existing reports. Appendix A-3 provides a geotechnical summary of findings and provides preliminary evaluation of geotechnical conditions along the proposed alignment. The geotechnical findings report also provides recommendations for the anticipated work associated with design and construction of the new roadway and discusses areas where additional subsurface investigations will be necessary.

No additional geotechnical fieldwork was completed for this phase of the Project. However, CH2M HILL noted the prior geotechnical work conducted by others along with careful field observation of site
conditions as a basis for the decisions that form the basis of the recommendations contained in the phase of the work. As the Project moves forward in the planning and design process, additional detailed field investigations will be required to confirm foundation conditions along selected areas of the alignment as required to develop final recommendations for types of structures, foundation conditions and preparation, and other important information necessary for design development.

4.15 Stormwater

Stormwater management (Figure 9) will need to address erosion and sedimentation control, pavement drainage, conveyance, water quality treatment, flow control and use of low impact development practices where feasible. Due to the steep topography of the site, both along the roadway and in the vicinity, the potential to infiltrate runoff completely within the roadway section is anticipated to be low. The current stormwater approach, as described in Appendix A-4, integrates bioretention planters within the planter strips of the roadway to provide water quality treatment. These planter strips will be connected to infiltration trenches to maximize the potential infiltration within the site and maintain existing hydrology to streams that cross the proposed roadway alignment.

Due to the close vicinity to the Sammamish River, flow control may not be required, if the Project can demonstrate adequate capacity in the conveyance system that delivers runoff to the river. As a portion of that conveyance system includes discharge through wetland 1, early design phase studies will need to evaluate the existing conditions within the wetland and the potential for the Project to affect the hydroperiod. The existing wetland is degraded and dominated by Reed Canary Grass; therefore, potential wetland restoration is included in the current design approach. Should discharge, without flow control, to the wetland not be deemed appropriate in these studies, additional flow control will likely need to be provided using underground vaults located at the southern extent of the roadway extension prior to discharge to the wetland. The pre-design memorandum in Appendix A-4 describes stormwater criteria applicable to the site and a proposed tiered approach to stormwater depending upon further study.
4.16 Utilities

4.16.1 Existing Utilities

The following existing utilities are contained within the Project area:

- *Puget Sound Energy (PSE).* PSE owns a 250-foot wide corridor established for their high-voltage transmission line and a secondary transmission line that crosses the proposed roadway alignment. Their steel tower supported transmission line supports six high-voltage transmission cables that cross the alignment at approximately Sta 35+50. The secondary overhead power line crosses at approximately Sta 34+00. Overhead clearances are adequate for proposed traffic operations; however, when installing the 84-foot precast, culvert construction methods will need to consider space constraints and overhead clearance restrictions by PSE.

- *Seattle Public Utilities (SPU).* SPU operates a 54-inch diameter water transmission main (Tolt Pipeline No. 2) that lies within a 30-foot wide easement abutting the north margin of the PSE right-of-way. Cascade Water Alliance supplies water to the City from SPU’s Tolt Pipeline No. 2. Based on record drawings, the water main currently has approximately 4.5 feet of cover at the crossing of the proposed roadway alignment. The proposed roadway grade will add approximately 1 to 3 feet of fill over the existing pipeline. Special protection of the existing water main is not anticipated, but should be confirmed during the next phase of design work.

- *Capped Utilities.* Currently both private and City utilities have been capped at both the north and south ends of the Project in anticipation of the future roadway extension. These utilities are described in Section 4.16.2.

4.16.2 Proposed Utilities

Extending 160th Avenue NE provides the opportunity to connect the various existing utility systems that have been capped for future extension. It also provides utility infrastructure to support future development. Together, City and private utilities have identified the following proposed utilities for this Project (*Figure 10*):

- *City of Redmond Sewer.* There is an existing sewer main capped at the southern termini of 160th Avenue NE. The City desires to extend the sewer main an additional 400 feet north, requiring two new manholes. The existing sewer is approximately 12 feet deep. Between the existing manhole and the first proposed manhole, the depth of the sewer main will range from 12 to 8 feet. The remainder of the sewer main extension between the first and second manhole will generally follow the roadway profile grade and be 8 feet deep. This sewer extension will serve the properties to the east of 160th Avenue NE south of the wetland 3. There will be no sewer line provided between the end of the 400 feet extension and the northern Project terminus (*Figure 7*).

- *City of Redmond Water.* Existing 12-inch waterlines are capped at both ends of the Project and will be connected by the Project. The City’s standard cover requirement for water mains is 4 feet. There are four proposed culvert crossings along the roadway alignment. Three culverts are 36 inches in diameter and will be installed at depths beneath the proposed waterline and clear of any conflicts. The waterline crossing of the proposed 84-inch culvert at wetland 3 cannot achieve the 4-foot minimum cover requirement. To accommodate headroom for the equestrian trail, the culvert crest cannot be deeper than 4.5 feet below the roadway, resulting in only 3 feet of cover over the proposed 12-inch water main. The proposed waterline crossing over the existing 54-inch SPU Tolt Pipeline 2 allows the 4-foot minimum cover to be met (*Figure 7*).
• **Comcast.** A 4-inch conduit will be routed beneath the western sidewalk for Comcast. In addition, two new vaults will be placed in the sidewalk for maintenance access.

• **PSE.** For electrical distribution, two 4-inch conduits will be routed beneath the western sidewalk for PSE. PSE has indicated that they may want a future gas line installed, but it would depend on the type of future development that is expected along the corridor. There appears to be adequate space for a gas line.

• **Frontier.** Four 4-inch conduits are currently capped at the southern end of the Project and would be extended north throughout the length of the Project.

Table 2 is a compilation of contact information for the utilities in the Project area. This contact information is subject to change.
### Table 2: Contact Information for Area Utilities

<table>
<thead>
<tr>
<th>Utility</th>
<th>Owner</th>
<th>Contact Name</th>
<th>Contact Information</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water</td>
<td>City of Redmond</td>
<td>Jeff Thompson, PE</td>
<td>15670 NE 85&lt;sup&gt;th&lt;/sup&gt; Street</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Senior Engineer</td>
<td>Redmond, WA 98073</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Water and Wastewater</td>
<td>Phone: (425) 556-2884</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Engineering</td>
<td><a href="mailto:jthompson@redmond.gov">jthompson@redmond.gov</a></td>
</tr>
<tr>
<td>Water</td>
<td>Seattle Public Utilities</td>
<td>Jennyfer Jacobsen</td>
<td>700 5&lt;sup&gt;th&lt;/sup&gt; Avenue, #4144</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Seattle, WA 98104</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Phone: (206) 684-8766</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Fax: (206) 233-1532</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td><a href="mailto:JacobsJS@seattle.gov">JacobsJS@seattle.gov</a></td>
</tr>
<tr>
<td>Sewer</td>
<td>City of Redmond</td>
<td>Jeff Thompson, PE</td>
<td>15670 NE 85&lt;sup&gt;th&lt;/sup&gt; Street</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Senior Engineer</td>
<td>Redmond, WA 98073</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Water and Wastewater</td>
<td>Phone: (425) 556-2884</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Engineering</td>
<td><a href="mailto:jthompson@redmond.gov">jthompson@redmond.gov</a></td>
</tr>
<tr>
<td>Cable TV &amp; Fiber</td>
<td>Comcast</td>
<td>Jill M. Look</td>
<td>1525 75&lt;sup&gt;th&lt;/sup&gt; Street SW Suite 200</td>
</tr>
<tr>
<td>Optics</td>
<td></td>
<td></td>
<td>Everett, WA 98203</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Phone: (425) 396-6032</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Fax: (425) 263-5352</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td><a href="mailto:Jill.Look@cable.comcast.com">Jill.Look@cable.comcast.com</a></td>
</tr>
<tr>
<td>Power &amp; Gas</td>
<td>Puget Sound Energy</td>
<td>Kelly Purnell</td>
<td>PO Box 97034, EST 11W</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Municipal Construction Planner</td>
<td>Bellevue, WA 98009</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Phone: (425) 462-3488</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td><a href="mailto:Kelly.Purnell@PSE.com">Kelly.Purnell@PSE.com</a></td>
</tr>
<tr>
<td>Telephone</td>
<td>Frontier</td>
<td>Mike HaKahan</td>
<td>1800 41&lt;sup&gt;st&lt;/sup&gt; Street SW</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Network Engineer,</td>
<td>Everett, WA 98203</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Outside Plant Engineering</td>
<td>Phone: (425) 263-4038</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Fax: (425) 263-4048</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td><a href="mailto:y.m.hakahan@ftr.com">y.m.hakahan@ftr.com</a></td>
</tr>
</tbody>
</table>
SECTION 5
Right-of-Way

5.1 Introduction

The right-of-way needs for this Project are shown on Figure 11. The existing 160th Avenue NE right-of-way is 84-feet wide through the Riverpoint residential neighborhood to the south; and is 84-feet wide through the Redmond 74 residential neighborhood to the north. This width provides for a full five-lane roadway with bike lanes, sidewalks, and planters; however, the existing street section is three traffic lanes, with bike lanes and on-street parking.

Based on earlier planning work to complete 160th Avenue NE, the City acquired an 84-foot wide by approximately 270-foot long parcel (parcel 3256059127) to accommodate the extension of 160th Avenue NE. The proposed roadway alignment, together with the use of retaining walls on the uphill side is designed for the improvements to fit within this existing right-of-way.

Additional right-of-way is required from three parcels to complete the extension and match to the existing termini.

- Approximately 31,000 square feet is required from the Veal parcel 0225059135 to accommodate the proposed roadway, along with its cut and fill slopes. To avoid leaving the property owner with an unusable remnant, the remaining parcel area lying west of the roadway is included in the acquisition area. The right-of-way acquisition area includes the proposed open cut slope along the steep hillside to the east instead of retaining walls.

- Approximately 20,000 square feet is required from PSE parcel 3526059062 to accommodate the proposed roadway and stream crossing culvert. It is assumed that PSE will grant the City a permanent easement for the proposed roadway and related infrastructure. An additional easement or permit may be necessary from SPU to cross over their 54-inch water main.

- A triangular shaped 1,500-square foot parcel is required from King County Parks parcel 3526059088 to accommodate roadway improvements. At this time, it is unknown if this would be acquired at a fee or as permanent easement.

In addition to the right-of-way acquisitions for roadway, additional right-of-way acquisitions will be required for onsite wetland buffer impacts. Based upon the roadway design and preliminary wetland work, approximately 34,000 square feet of wetland buffers would be impacted. As shown in Table 3, approximately 10,000 square feet can be used from the remnant parcel that will be acquired for roadway construction (parcel 0225059135). The remaining area will need to be acquired onsite as part of the right-of-way acquisition process to address the remaining approximately 24,000 square feet required for mitigation. Onsite wetland buffers result in the need to acquire approximately 75,000 square feet of right-of-way.

The Project also results in wetland impacts and for the Project. It was assumed that these impacts would be addressed through the purchase of credits at an offsite mitigation bank that is currently proposed in the Project area. If credits cannot be purchased, approximately 13,770 square feet of additional right-of-way would need to be acquired onsite to address the wetland impacts.
5.2 Parcel Table

*Table 3* provides a summary of the parcels that surround the Project area. Eight parcels border the Project; of those eight, only three require right-of-way acquisitions. Of those four parcels, two of them (owned by PSE and King County Parks) could result in an easement in lieu of an acquisition. A map of the parcels and potential right-of-way acquisitions can be found in *Figure 11.*

**Table 3: Parcel Table**

<table>
<thead>
<tr>
<th>Right-of-Way Purpose</th>
<th>Property Owner</th>
<th>Parcel ID</th>
<th>Current Use</th>
<th>Total Area (square feet)</th>
<th>Acquisition Area (square feet)</th>
<th>Net Area (square feet)</th>
<th>Notes</th>
</tr>
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<tr>
<td>Roadway</td>
<td>Good</td>
<td>0225059101</td>
<td>Single family</td>
<td>84,506</td>
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<td>Veal</td>
<td>0225059135</td>
<td>Single family</td>
<td>158,122</td>
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<td>137,517</td>
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<td></td>
<td>Veal</td>
<td>3526059123</td>
<td>Vacant lot</td>
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<td></td>
<td>La Prete</td>
<td>0225059002</td>
<td>Apartment</td>
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<td></td>
<td>Puget Sound Energy</td>
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<td>Distribution right-of-way</td>
<td>266,151</td>
<td>19,750</td>
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<td>King County Parks</td>
<td>3526059088</td>
<td>Samm River trail site</td>
<td>968,338</td>
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<td></td>
<td>556962TRCT</td>
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<td>Access tract</td>
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<td>70902000000</td>
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<td>Condominiums</td>
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<td>Roadway totals</td>
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<td>2,683,597</td>
<td>41,845</td>
<td>2,641,752</td>
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<td>Wetland Mitigation*</td>
<td>Veal</td>
<td>0225059135</td>
<td>Single family</td>
<td>137,517</td>
<td>10,000</td>
<td>127,517</td>
<td>Remnant of parcel to be used for wetland mitigation</td>
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<tr>
<td>Wetland mitigation totals</td>
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<td></td>
<td></td>
<td>33,155</td>
<td></td>
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<tr>
<td>Project totals</td>
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<td>2,683,597</td>
<td>75,000</td>
<td>2,608,597</td>
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</tr>
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</table>

*Note: assumes 13,770 square feet of credits acquired from offsite wetland mitigation bank.*
**PROPERTY OWNERS**

- **GOOD**: 0252059201, Single Family, 84,506 SF - 84,506 SF
  
- **VEAL**: 0250592135, Single Family, 137,517 SF - 137,517 SF
  
- **PUGET SOUND ENERGY**: 3526059062, Distribution Row, 246,401 SF - 246,401 SF
  
- **KING COUNTY PARKS**: 3526059288, Warm River Trail Site, 1,490 SF - 1,490 SF
  
- **556925TRCT**: Access Tract, 1,490 SF - 1,490 SF
  
- **GOOD 0225059101**: Single Family, 84,506 SF - 84,506 SF
  
- **VEAL 0225059135**: Single Family, 137,517 SF - 137,517 SF
  
- **VEAL 3526059123**: Vacant Lot, 134,600 SF - 134,600 SF
  
- **GOOD 0225059002**: Apartment, 738,000 SF - 738,000 SF
  
- **PUGET SOUND 3526059062**: Distribution Row, 246,401 SF - 246,401 SF
  
- **KING COUNTY PARKS 3526059288**: Warm River Trail Site, 1,490 SF - 1,490 SF
  
- **556925TRCT 1029020000**: Access Tract, 1,490 SF - 1,490 SF

**Net Area (SF)**

- **GOOD**: 84,506 SF - 84,506 SF
  
- **VEAL**: 137,517 SF - 137,517 SF
  
- **PUGET SOUND ENERGY**: 246,401 SF - 246,401 SF
  
- **KING COUNTY PARKS**: 1,490 SF - 1,490 SF
  
- **556925TRCT**: 1,490 SF - 1,490 SF
  
- **GOOD 0225059101**: 84,506 SF - 84,506 SF
  
- **VEAL 0225059135**: 137,517 SF - 137,517 SF
  
- **VEAL 3526059123**: 134,600 SF - 134,600 SF
  
- **GOOD 0225059002**: 738,000 SF - 738,000 SF
  
- **PUGET SOUND 3526059062**: 246,401 SF - 246,401 SF
  
- **KING COUNTY PARKS 3526059288**: 1,490 SF - 1,490 SF
  
- **556925TRCT 1029020000**: 1,490 SF - 1,490 SF

**Notes**

- Remnant of parcel to be used for wetland mitigation.
- Remnant of parcel to be identified.

**Total Net Area (SF)**

- **ROADWAY**: 2,683,597 SF - 2,683,597 SF
  
- **WETLAND MITIGATION**: 33,155 SF - 33,155 SF

**Total Project Area (SF)**

- **ROADWAY TOTALS**: 2,716,752 SF - 2,716,752 SF
  
- **WETLAND MITIGATION TOTALS**: 33,155 SF - 33,155 SF
  
- **PROJECT TOTALS**: 2,749,907 SF - 2,749,907 SF

**Notes**

- Assumes 13,770 SF of credits acquired from off-site wetland mitigation bank.

**City of Redmond 160th Avenue NE Extension**

**Figure 11:** Right-of-Way Acquisitions

- **ROW ACQUISITIONS**: City of Redmond 160th Avenue NE Extension
  
- **WETLAND**: Transition to match to existing to be refined after corridor concept approval.
  
- **BUFFER CL 3**: Transition to match to existing to be refined after corridor concept approval.
  
- **BUFFER CL 4**: Transition to match to existing to be refined after corridor concept approval.
  
- **CULVERT**: Transition to match to existing to be refined after corridor concept approval.
  
- **FILL LINE**: Transition to match to existing to be refined after corridor concept approval.
  
- **RETAINING WALL**: Transition to match to existing to be refined after corridor concept approval.

**Notes**

1. Row impacts to not include impacts from any future realignment of the Equestrian Trail.
6.1 Summary

The use of federal funds will require NEPA approval, which will be conducted through WSDOT Local Programs who act on behalf of the Federal Highway Administration. NEPA documentation will consist of the WSDOT Environmental Classification Summary with any required supporting documentation attached (air quality, noise, cultural resources, and aesthetics). The NEPA documentation could be adopted to support the required SEPA approval for the Project.

The Project’s impact to wetlands requires a Clean Water Act Section 404 permit from the U.S. Army Corps of Engineers and an additional state permit. Because Project effects to wetlands are anticipated to be less than 0.5 acre, a Nationwide Permit can be acquired which minimizes the review timeline. Additional state and local permits will also be required prior to construction. Overall, up to 18 months will be required to complete the necessary environmental documentation and permitting process. Table 4 provides a summary of anticipated environmental permits. Further details related to the permit matrix describing the anticipated environmental requirements are shown in Table 1 of Appendix A-6.
Table 4: Anticipated Environmental Requirements – 160th Avenue NE Extension

<table>
<thead>
<tr>
<th>Permit</th>
<th>Trigger</th>
<th>Submittal Information</th>
<th>Typical Review Timeline</th>
</tr>
</thead>
<tbody>
<tr>
<td>Federal Compliance/Permits</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>National Environmental Policy Act (NEPA)</td>
<td>Federal action – including Federal Funds, Federal Lands, or Federal Permits</td>
<td>WSDOT Environmental Classification Summary Supporting Documentation</td>
<td>Typically 6 to 12 months for a NEPA DCE. Depends on required supporting documentation</td>
</tr>
<tr>
<td>Endangered Species Act Compliance (Section 7)</td>
<td>If the project is located or could affect a listed species or critical habitat</td>
<td>WSDOT ECS Biological Evaluation/Assessment - potential</td>
<td>Tied to NEPA</td>
</tr>
<tr>
<td>Section 105</td>
<td>Federal action that may affect a historic property or site. Requires tribal consultation</td>
<td>Cultural Resources Report</td>
<td>Tied to NEPA</td>
</tr>
<tr>
<td>Section 4(f/6(f)</td>
<td>Temporary closure/detour of the Puget Power Trail Potential property acquisitions with federal funds</td>
<td>Section 4(f/6(f) Analysis and Determination</td>
<td>Tied to NEPA</td>
</tr>
<tr>
<td>Clean Water Act (CWA) Section 404 - USACE</td>
<td>Dredge and/or placement of fill within streams or wetlands. Under 0.5 acre wetland impact triggers a Nationwide Permit.</td>
<td>Joint Aquatic Resource Permit Application (JARPA) Wetland/Stream report Cultural Resources</td>
<td>Typically 3 to 6 months</td>
</tr>
<tr>
<td>U.S. Fish and Wildlife Bald Eagle Management Permit</td>
<td>Bald eagles nest within 000 feet of construction activities</td>
<td>Permit</td>
<td>Typically 3 months</td>
</tr>
<tr>
<td>State Permits</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NPDES for Construction Activity - Washington Department of Ecology (Ecology)</td>
<td>Soil disturbing activities of one acre or more</td>
<td>Construction Stormwater General Permit Application Stormwater Pollution Prevention Plan (SWPPP)</td>
<td>Typically less than 2 months. Requires SEPA to be complete prior to approval</td>
</tr>
<tr>
<td>CWA Section 401 Water Quality Certification - Ecology</td>
<td>Triggered by the CWA Section 404 Permit</td>
<td>JARPA</td>
<td>Typically 1 to 4 months</td>
</tr>
<tr>
<td>Hydraulic Project Approval - Washington Department of Fish and Wildlife</td>
<td>Work located within, over, or under the Ordinary High Water Mark of waters of the state.</td>
<td>JARPA</td>
<td>Typically within 45 days. Requires SEPA to be complete prior to approval</td>
</tr>
<tr>
<td>Local Permits - City of Redmond</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>State Environmental Policy Act (SEPA)</td>
<td>Proposals with an ‘action’</td>
<td>SEPA Checklist Supporting documentation</td>
<td>Typically 4 months</td>
</tr>
<tr>
<td>Clear and Grade Permit</td>
<td>Required to clear trees, excavate, fill and grade</td>
<td>Application Drainage Report Geotech report</td>
<td>Typically 4 months</td>
</tr>
<tr>
<td>Right-of-Way Permit</td>
<td>Work within existing City right-of-way</td>
<td>Right-of-Way Permit Plans TCP</td>
<td>Typically 4 months</td>
</tr>
<tr>
<td>Critical Areas Review</td>
<td>To address impacts to critical areas within the project area</td>
<td>Critical Areas Report</td>
<td>Varies based upon project impacts May trigger the need for a Reasonable Use Permit</td>
</tr>
<tr>
<td>Tree Removal</td>
<td>Removal of more than 10 significant trees or stand of trees.</td>
<td>Tree Removal Permit Tree Removal Exception Request (criteria in Redmond Zoning Code 21.72.090)</td>
<td>Typically 2 to 3 weeks – trees within critical area Expires 60 days from date of issuance More than 10 trees triggers Cleaning and Grading Permit</td>
</tr>
</tbody>
</table>
SECTION 7

Cost Estimates

7.1 Budget Level Cost Estimate

The following estimate uses an evolved master planning cost template specifically developed for the City. The estimate shown on the following pages accounts for critical Project costs as well as identifies specific high cost items.
CITY OF REDMOND Transportation Facilities Plan

Project: **160th Ave NE Extension**
Project ID: **74**

<table>
<thead>
<tr>
<th>Section</th>
<th>Cost</th>
<th>Risk Assessment</th>
<th>Contingency</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>I. Right of Way</td>
<td>$1,224,000</td>
<td>LOW-MEDIUM</td>
<td>25%</td>
<td>$305,000.00</td>
</tr>
<tr>
<td>II. Construction</td>
<td>$5,731,715</td>
<td>MEDIUM</td>
<td>30%</td>
<td>$1,719,514.51</td>
</tr>
<tr>
<td>III. Project Development</td>
<td>$1,715,515</td>
<td>MEDIUM-HIGH</td>
<td>35%</td>
<td>$601,430.08</td>
</tr>
<tr>
<td>IV. Construction Management</td>
<td>$887,757</td>
<td>MEDIUM</td>
<td>30%</td>
<td>$266,227.18</td>
</tr>
</tbody>
</table>

**V. Estimate of Probable Cost (2012)**

**Subtotal** $12,459,000

**VI. Escalation**

<table>
<thead>
<tr>
<th>Year of cost index</th>
<th>Project Escalation</th>
</tr>
</thead>
<tbody>
<tr>
<td>2012</td>
<td>$1,155,286</td>
</tr>
</tbody>
</table>

**TOTAL ESTIMATE OF PROBABLE COST** $13,614,286

See sheet 4 for Assumptions

*Terminology used herein is in 2012 dollars for Planning, Survey, The expenses included are labor costs, material costs, site development, right-of-way costs, permit and survey fee costs. These estimate does not include relocation, 311 calls, or operations and maintenance costs. In addition, there are no costs for the mitigation or remediation associated with the potential discovery of hazardous materials. The order of magnitude cost opinion shown has been prepared to guidance in project evaluation at the time of the estimate. The final costs of the project will depend on actual labor and material costs, actual project conditions, productivity, contract market conditions, final project scope, final project schedule, and all variable factors. As such, the final project cost will vary from the estimate prepared above. Because of these factors, funding needs must be carefully reviewed prior to making specific financial commitment or establishing final budgets.*
**CITY OF REDMOND Transportation Facilities Plan**

**Project:** 160th Ave NE Extension  
**Project ID:** 74  
**Concept No.:**  
**Entered by:** ARB  
**Reviewed by:** JAM  
**Updated:** 8/13/2012

### SECTION 7 - COST ESTIMATES

#### 1. RIGHT OF WAY

<table>
<thead>
<tr>
<th>Neighborhood</th>
<th>Education Hill</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>I. RIGHT OF WAY</strong></td>
<td><strong>Unit</strong></td>
</tr>
<tr>
<td>1 Land Purchase</td>
<td>SF</td>
</tr>
<tr>
<td>2 Damage / Cure</td>
<td>%</td>
</tr>
<tr>
<td>3 Partial Building Take</td>
<td>SF</td>
</tr>
<tr>
<td>4 Relocation</td>
<td>%</td>
</tr>
<tr>
<td>5 Acquiring/Admin. Costs (per Parcel)</td>
<td>EA</td>
</tr>
<tr>
<td>6 Condemnation Contingency (Estimated)</td>
<td></td>
</tr>
<tr>
<td>7 Right of Way Sub-Total</td>
<td></td>
</tr>
</tbody>
</table>

#### 2. CONSTRUCTION

| | **Unit** | **Quantity** | **Unit Cost** | **Total** |
| 8 Demolition/Clearing | SF | 91,305.00 | $1.00 | **$91,305** |
| 9 Roadway Excavation | CY | 8,820.00 | $25.00 | **$220,500** |
| 10 Embankment Compaction | CY | 4,060.00 | $5.00 | **$20,300** |
| 11 Gravel Borrow | CY | 5,351.00 | $25.00 | **$133,775** |
| 12 Bridge Demolition & Disposal | SF | - | $60.00 | **$0** |
| 13 New Bridge and Bridge Widening | LS | 1.00 | $660,000.00 | **$660,000** |
| 14 New Pavement - HMA | TON | 2,480.50 | $100.00 | **$248,050** |
| 15 CSBC | TON | 1,402.33 | $25.00 | **$35,083** |
| 16 Pavement Overlay | LANE-MILE | - | $175,000.00 | **$0** |
| 17 Sidewalks | SY | 1,760.00 | $60.00 | **$105,600** |
| 18 Curb and Gutter | LF | 2,640.00 | $25.00 | **$66,000** |
| 19 Concrete Barrier / Moment Slab | LF | 790.00 | $300.00 | **$237,000** |
| 20 Walls - Cut (Soil Nail) | SF | 2,910.00 | $120.00 | **$349,200** |
| 21 Walls - Fill (SE) | SF | 4,610.00 | $50.00 | **$230,500** |
| 22 Walls - Fill (Tail SE @ Steep Hillside) | SF | 7,110.00 | $90.00 | **$639,900** |
| 23 Walls - Fill (Rockery) | SF | 500.00 | $40.00 | **$20,000** |
| 24 Drainage for Roadway | LANE-MILE | 0.75 | $180,000.00 | **$135,000** |
| 25 Stormwater Management & Utility Fees | LS | 1.00 | $165,000.00 | **$165,000** |
| 26 Culvert - 36" | LF | 210.00 | $150.00 | **$31,500** |
| 27 Landscaping | LF | 1,320.00 | $46.00 | **$60,720** |
| 28 Sanitary Sewer and Utility Modifications | LANE-MILE | 0.75 | $100,000.00 | **$75,000** |
| 29 New Water Main | MILE | 0.25 | $528,000.00 | **$132,000** |
| 30 Utility Uncovering | LF | - | $280.00 | **$0** |
| 31 Traffic Signal New | EA | - | $280,000.00 | **$0** |
| 32 Traffic Signal Modification | EA | 1.00 | $150,000.00 | **$150,000** |
| 33 Channelization / Signing | LANE-MILE | 1.50 | $25,000.00 | **$37,500** |
| 34 Illumination | MILE | 0.43 | $260,000.00 | **$110,992** |
| 35 TESC | LS | 3% | of lines 8 through 31 | **$118,715** |
| 36 Wetland Mitigation | LS | 1.00 | $112,000.00 | **$112,000** |
| 37 Flow Control | LS | 1.00 | $35,000.00 | **$35,000** |
| 38 Construction Traffic Control | % | 10% | of lines 8 through 37 | **$418,787** |
| 39 Miscellaneous / Allowance | % | 12% | of lines 8 through 38 | **$556,998** |
| 40 Mobilization | % | 10% | of lines 8 through 39 | **$519,865** |
| 41 WA State Sales Tax (Non-city utilities) | % | 10% | of line 25 & 26 | **$13,200** |
| 42 Construction Sub-Total | | | | **$5,731,715** |

**Assumptions listed on page 4**  
**Continued on Page 4**

The above construction is in 2012 dollars for Planning Level. The cost does not include excavation, fill, or other operations and maintenance costs. In addition, there are no costs for the mitigation or remediation associated with the potential for oil spills or hazardous materials. The order of magnitude cost opinion shown has been prepared for guidance in project evaluation at the time of the estimate. The final costs of the project will depend on actual labor, material costs, actual site conditions, productivity, competitive market conditions, final project scope, final project schedule, and other variable factors. As a result, the final project costs will vary from the estimate presented above. Because of these factors, funding needs must be carefully reviewed prior to making specific financial decisions or establishing final budgets.
### SECTION 7 - COST ESTIMATES

CITY OF REDMOND Transportation Facilities Plan

<table>
<thead>
<tr>
<th>Project</th>
<th>160th Ave NE Extension</th>
<th>Entered by: ARB</th>
<th>Reviewed by: JAM</th>
<th>Updated: 8/13/2012</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project ID</td>
<td>74</td>
<td>Concept No.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

#### III. Project Development

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
<th>Unit</th>
<th>Quantity</th>
<th>Unit Cost</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>43</td>
<td>Environmental Documentation</td>
<td>%</td>
<td>5%</td>
<td>of line 42</td>
<td>$286,586</td>
</tr>
<tr>
<td>44</td>
<td>Design Engineering</td>
<td>%</td>
<td>15%</td>
<td>of line 42</td>
<td>$859,757</td>
</tr>
<tr>
<td>45</td>
<td>Agency Administration</td>
<td>%</td>
<td>7%</td>
<td>of line 42</td>
<td>$401,220</td>
</tr>
<tr>
<td>46</td>
<td>Public Art</td>
<td>%</td>
<td>1%</td>
<td>of line 42</td>
<td>$57,317</td>
</tr>
<tr>
<td>47</td>
<td>Community Engagement</td>
<td>%</td>
<td>2%</td>
<td>of line 42</td>
<td>$114,634</td>
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<tr>
<td>48</td>
<td>Project Development Sub-Total</td>
<td></td>
<td></td>
<td></td>
<td>$1,719,515</td>
</tr>
</tbody>
</table>

#### IV. Construction Management

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
<th>Unit</th>
<th>Quantity</th>
<th>Unit Cost</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>Construction Management</td>
<td>%</td>
<td>15%</td>
<td>of line 42</td>
<td>$859,757</td>
</tr>
<tr>
<td>51</td>
<td>Wetland Monitoring Agreement</td>
<td></td>
<td></td>
<td></td>
<td>$0</td>
</tr>
<tr>
<td>52</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$0</td>
</tr>
<tr>
<td>53</td>
<td>Wetland Monitoring Agreement</td>
<td>%</td>
<td>25%</td>
<td>of line 36</td>
<td>$28,000</td>
</tr>
<tr>
<td>54</td>
<td>Construction Management Sub-Total</td>
<td></td>
<td></td>
<td></td>
<td>$887,757</td>
</tr>
</tbody>
</table>

**Assumptions:**

- **Proj Length:** 1320' long
- **Road section:** 5.5+11+11+5.5 = 33'
- **Paving area:** 33*1320 = 43560sf
- 1 cy HMA = 2.05 tons HMA
- **Assume 6" CSBC**
- **CSBC area = Paving area**
- 1 cy CSBC = 1 ton
- **Sidewalk Area**
  - two 6' wide for 1320' = 15840sf = 1760sy
- **Model Analysis - planar surface area for C&G = 91305sf**

---

The above cost opinion is in 2012 dollars for Planning Level. The cost does not include escalation, financial costs, or operations and maintenance costs. In addition, there are no costs for the mitigation or remediation associated with the potential discovery of hazardous materials. The order of magnitude cost opinion shown has been prepared for guidance in project evaluation at the time of the estimate. The final costs of the project will depend on actual labor and material costs, actual site conditions, productivity, competitive market conditions, final project scope, final project schedule, and other variable factors. As a result, the final project costs will vary from the estimate presented above. Because of these factors, funding needs must be carefully reviewed prior to making specific financial decisions or establishing final budgets.
7.2 Planning Level Project Risk Assessment

The planning level risk assessment takes several factors of Project development and construction and assigns a risk level based on inputs by the design team. While the inputs are qualitative in nature, many of the factors at a preliminary level of design can be assigned risk based on engineering judgment. The assessment then creates an aggregate risk assessment that can be used to aid in the development of appropriate contingency levels for cost analysis. Each risk consideration is ranked according to the level of anticipated risk for the Project and assigned either a high, medium, or low ranking. A ranking of “high” corresponds to a large amount of risk and a ranking of “low” correlates to a small amount of risk. These risk assessment rankings are based on both the likelihood of the risk occurring as well as the impact that the risk would have on the Project.

The planning level risk assessment for the 160<sup>th</sup> Avenue NE Extension Project can be found in Figure 12. Four key risk considerations were taken into account:

- **Environmental Permitting.** This Project has four environmentally sensitive wetlands that will be disturbed temporarily or permanently during the construction of 160<sup>th</sup> Avenue NE. However, a minimal number of anticipated agency approvals will be required to build the Project. It is for these reasons that the environmental permitting risk assessment is at level medium. Section 0 provides more information on the Project’s environmental considerations.

- **Design and Construction.** The risk assessment level for design and construction has been assessed at a medium level. Most of these risk considerations are based on structures, geotechnical data, and utilities. As mentioned in Appendix A-3, only a limited amount of soil exploration has been performed thus far, which ranks the unknown soil conditions risk at medium. However, it is not predicted that the roadway will encounter any contaminated soils. There is currently a low level of risk assigned to utilities within the Project, because no existing utilities need to be relocated as part of the design. Because the Project is still in the pre-design phase and concepts are still being worked through, a medium level of risk has been assigned to the Project definition.

- **Right-of-Way.** Based on the proposed alignment there are four parcels that require right-of-way acquisition (Section 1.1). Although acquisition is required, the risk assessment for right-of-way has been assigned a medium score because the acquisition is small compared to the size of the parcels, and do not require the acquisition of any residential or commercial buildings or any relocations.

- **Other Factors.** The risk assessment for other factors deals with the influence level that other neighboring projects may have on this one, how the stakeholders and public view the Project (if the Project is controversial), if the Project has any federal funding, is located within multi-jurisdictions, and any time constraints that the Project may have. Based on the research for this Project, the risk assessment level for other factors has been ranked as none, because the other factors currently do not seem to have any adverse effects on the Project.
Figure 12: Planning Level Risk Assessment

<table>
<thead>
<tr>
<th>Risk Considerations</th>
<th>Likelihood</th>
<th>Impact</th>
<th>Risk Assessment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Environmental Permitting</td>
<td>High</td>
<td>High</td>
<td>HIGH</td>
</tr>
<tr>
<td>Presence of wetlands</td>
<td>High</td>
<td>Med</td>
<td>HIGH</td>
</tr>
<tr>
<td>Impacts to ecological sensitive areas</td>
<td>Med</td>
<td>Med</td>
<td>MEDIUM</td>
</tr>
<tr>
<td>Multi-agency approvals needed</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Design and Construction</td>
<td>Med</td>
<td>Med</td>
<td>MEDIUM</td>
</tr>
<tr>
<td>Unknown soil conditions</td>
<td>Low</td>
<td>Low</td>
<td>LOW</td>
</tr>
<tr>
<td>Contaminated soils</td>
<td>Low</td>
<td>Med</td>
<td>LOW</td>
</tr>
<tr>
<td>Unknown utilities</td>
<td>Low</td>
<td>Med</td>
<td>LOW</td>
</tr>
<tr>
<td>Underground utility project elements</td>
<td>Low</td>
<td>Med</td>
<td>LOW</td>
</tr>
<tr>
<td>Significant structures</td>
<td>High</td>
<td>Med</td>
<td>HIGH</td>
</tr>
<tr>
<td>Work within water table</td>
<td>Med</td>
<td>Med</td>
<td>MEDIUM</td>
</tr>
<tr>
<td>Little project definition, many unknowns</td>
<td>Med</td>
<td>Med</td>
<td>MEDIUM</td>
</tr>
<tr>
<td>New technology</td>
<td>None</td>
<td>None</td>
<td>NONE</td>
</tr>
<tr>
<td>Right of Way</td>
<td>Med</td>
<td>Med</td>
<td>MEDIUM</td>
</tr>
<tr>
<td>Significant property impacts</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Other Factors</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Project scope affected by other projects</td>
<td>Low</td>
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<td>NONE</td>
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<tr>
<td>Controversial project</td>
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<td>Low</td>
<td>LOW</td>
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<tr>
<td>Multi-jurisdictional project</td>
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<td>NONE</td>
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<tr>
<td>Federal funding</td>
<td>Low</td>
<td>Low</td>
<td>LOW</td>
</tr>
<tr>
<td>Time constraint</td>
<td>Low</td>
<td>None</td>
<td>NONE</td>
</tr>
</tbody>
</table>

Risk Matrix

- Environmental Permitting
- Design and Construction
- Right of Way
- Significant property impacts
- Other factors
- Aggregate project risk
SECTION 8
Redmond-Woodinville Road Intersection Analysis

8.1 Introduction

Three intersection configurations were evaluated for the intersection connection between the northern end of 160th Avenue Extension and Red-Wood Road (SR 202) based on metrics of constructability, operational and multimodal safety, cost, mobility, local traffic access, and aesthetics. These alternatives included a no-build alternative, a roundabout alternative, and a signalized intersection alternative. Methods and assumptions for the traffic operations analysis and forecasting are presented below, followed by a detailed description of each alternative. Finally, a summary of traffic operations results is presented for each alternative, followed by a preferred configuration.

8.1.1 Methods and Assumptions

Intersection LOS, delays, and volume-to-capacity (v/c) ratios were calculated at the study intersection of Red-Wood Road/160th Avenue NE based on methods contained in the Highway Capacity Manual 2010. Synchro (version 8.0) was used for these calculations. Appendix A-7 contains detailed LOS worksheets for the different alternatives.

Traffic forecast volumes for the year 2030 were based on information in the Summary of Traffic Operations and Forecasting – Red-Wood Road Corridor Study, dated December 2007 by Transpo Group. The study included 2007 as the year for existing conditions and 2030 as the horizon year, based on the latest forecast year for the Bellevue-Kirkland-Redmond travel demand forecast model at the time the study was conducted. In that study, several alternatives were studied for the configuration of Red-Wood Road, including three- and five-lane alternatives. Based on discussions with the City, the three-lane alternative on Red-Wood Road was used as the basis for traffic forecasts for this traffic analysis.

Recent traffic count data provided by the City was reviewed to ensure that traffic volumes near the intersection of Red-Wood Road/160th Avenue NE have not changed significantly since 2007. PM Peak intersection counts at two locations (NE 90th Street/Red-Wood Road and NE 109th Street/Red-Wood Road) were reviewed for years 2007 and 2010. In both locations, total intersection entering volume was similar but slightly lower in 2010 than in 2007:

- NE 90th Street/Red-Wood Road: 2007 volume = 2,055 vph; 2010 volume = 1,884 vph
- NE 109th Street/Red-Wood Road: 2007 volume = 1,908 vph; 2010 volume = 1,842 vph

Recent volumes show that little change in traffic volumes has occurred near the Red-Wood Road/160th Avenue NE intersection. Therefore, forecast volumes for the year 2030 from the Red-Wood Road Corridor Study were assumed reasonable and conservative for estimating capacity at the Red-Wood Road/160th Avenue NE intersection. Year 2030 PM peak hour intersection turn movement volumes for the intersection of Red-Wood Road/160th Avenue NE are presented in Figure 13. Based on review of the existing AM and PM peak hour volumes at the two locations noted above, traffic patterns on Red-Wood Road are highly directional (e.g., relatively the same amount of traffic heading southbound in the AM period can be seen heading northbound in the PM period). Therefore, year 2030 AM peak hour volumes were estimated by reversing 2030 PM peak hour traffic flows.
8.2 Alternatives

8.2.1 Alternative 1: No-Build Alternatives

The “No-Build” Alternative (Figure 14) maintains the existing intersection configuration. The current design terminates 160th Avenue at a tee intersection with NE 106th Street. NE 106th Street then shortly connects at an un-signalized intersection with Red-Wood Road. The channelization from 160th Avenue NE is that of one lane in each direction. The channelization of NE 106th Street has one westbound lane, one eastbound to southbound turn lane, and one eastbound to northbound turn lane. Red-Wood Road provides one northbound through lane, one southbound through lane, one southbound to westbound turn lane, and a middle lane that provides northbound to southbound turning traffic a refuge as well as eastbound to northbound traffic (flying “T”).

- **Construction Cost:** minimal, configuration is currently built; some additional signage and striping may be needed.
- **Construction Delay and Mitigation:** minimal, configuration is currently built and operating.
- **Safety for Vehicles:** not particularly safe for traffic that is turning onto Red-Wood Road without a protected signal. The current approach of 106th Street is up a hill to Red-Wood Road, which likely reduces intersection sight distance coupled with Red-Wood Road being in a horizontal curve in this area.
- **Safety for Pedestrians and Bicyclists:** no protected crossing across Red-Wood Road. Existing sidewalks are sufficiently wide but are steep. Marked crosswalks would need to be carefully placed in the future. Bike lanes are provided on Red-Wood Road in this segment but no facilities are included on 106th Street.
- **Traffic Operations:** The current configuration is not optimal and operations will quickly deteriorate because of awkward geometry requiring through movements to make several turns.
- **Local Traffic Access:** Access will remain as it is currently. Delays from traffic congestion are likely to reduce access quality, however.
- **Aesthetics:** the current condition of the area is landscaped providing views and open space. There is nothing particularly noteworthy or culturally significant.
8.2.2 Alternative 2: Roundabout

The roundabout alternative (Figure 15) would drastically alter the existing intersection geometry and traffic configuration. Using a minimum inscribed diameter of 100 feet per WSDOT standards, combined with a 21-foot rotary lane and sidewalks for pedestrian circulation, the roundabout alternative would occupy a substantial area of the available city right-of-way. Due to the sloping nature of the site, several walls are required on the south and west edges of the site to maintain a flat roundabout for traffic operational safety. Consequently, the neighborhood access at 160th Avenue would be closed and removed. With an approximate operating speed of 15 mph for vehicles up to and including WB-40 trucks (maximum vehicle design size is estimated as a WB-50) this configuration keeps southbound traffic moving smoothly into Downtown Redmond. Based on traffic forecasts, traffic heading northbound in a continuous stream from 160th Avenue may hinder northbound traffic entering the roundabout from Red-Wood Road.

- Construction Costs: A roundabout at this location will require a substantial amount of grading, walls, fill, new pavement, and potentially adjacent property adjustments.
- Construction Delay and Mitigation: The construction of a roundabout at this location would cause major traffic detours and delays to Red-Wood Road during construction.
- Safety for Vehicles: Roundabouts drastically reduce head on collisions and are generally safer than conventional intersections while still maintaining mobility.
- Safety for Pedestrians and Cyclists: Roundabouts require longer transition paths for pedestrians and have two conflict points at each leg. Either cyclists typically operate as pedestrians on paths external to the circulating roadway or speeds within the roundabout are generally low enough for cyclists to operate as traffic.
- Traffic Operations: Further analysis would be required to say exactly whether a roundabout at this particular location would improve traffic flow compared to a conventional signalized intersection. There is also no way to customize traffic signal timing at various times of the day to accommodate directional commuter traffic.
- Local Traffic Access: Existing access to and from the adjacent neighborhood by 106th Street would be closed and removed entirely.
FIGURE 15: ALTERNATIVE 2, ROUNDABOUT
8.2.3 Alternative 3: Signalized Intersection

The signalized intersection alternative (*Figure 16*) would first adjust the geometry of the existing intersection between NE 106th Street and Red-Wood Road to make the 160th Avenue NE movement the primary movement. This alternative would maintain the existing Flying “T” geometric configuration of Red-Wood Road in this area but move it slightly to the north. Separate right turn and left turn lane channelization would be provided from 160th Avenue to Red-Wood Road. The outside northbound lane could be signalized to keep northbound traffic on Red-Wood Road moving, while other phases of the signal are engaged. However, additional widening of Red-Wood Road north of this intersection would probably be required for safety and to achieve maximum operational benefits. A similar present implementation of this design is at NE 51st Street and West Lake Sammamish Parkway. The future proximity of the intersection of 106th Street NE and 160th Avenue requires that access to the homes near 106th Street be restricted to a “right-in, right-out” configuration. Full access to 160th Avenue NE would be provided by the intersection with NE 103rd Way and 160th Avenue NE.

- **Construction Cost**: Moderate requiring significant demolition and earthwork grading, including some walls adjacent to the properties to the west to accommodate a significant elevation differential. This could be reduced by using a terraced combination of fill walls and fill slopes.

- **Construction Delay and Mitigation**: Constructing the intersection separate of the 160th Avenue NE extension would permit adjacent homeowners access routes without building an interim roadway. Impacts to the existing operations on Red-Wood Road would be relatively minimal.

- **Safety for Vehicles**: A signalized intersection could provide dedicated phases for all movement, which would reduce the likelihood of collisions.

- **Safety for Pedestrians and Cyclists**: A signalized intersection would provide safe access across Red-Wood Road and provide greater neighborhood connectivity. Presently, the nearest crosswalks are 0.32 mile to the north and 0.23 mile to the south.

- **Traffic Operations**: Given traffic predictions in the 2007 report, a signal should be seriously considered at this intersection to accommodate future traffic shifts and patterns due to the 160th Avenue Extension Project.

- **Local Traffic Access**: Access would be restricted to one partial intersection for southbound 160th Avenue traffic only and one full intersection for safety and operational reasons.

- **Aesthetics**: A similar landscaping style is expected. There is nothing particularly noteworthy or culturally significant about this alternative.
FIGURE 16: ALTERNATIVE 3, SIGNALIZED INTERSECTION
8.3 Traffic Analysis

Results of the traffic analysis are presented in Table 5. The intersection would experience the lowest amount of delay in Alternative 3 (Signal) for both AM and PM peak hours. Despite this, the intersection would still operate at or near capacity in the Year 2030 PM peak hour, with the northbound and eastbound approaches experiencing approximately 65 seconds of control delay.

*Table 5: 2030 Intersection Operations at Red-Wood Road/160th Avenue NE Intersection*

<table>
<thead>
<tr>
<th>Alternative</th>
<th>Northbound</th>
<th>Eastbound</th>
<th>Southbound</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LOS</td>
<td>Delay (sec/veh)$^2$</td>
<td>V/C</td>
<td>LOS</td>
</tr>
<tr>
<td>1-No Build$^1$</td>
<td>C</td>
<td>15.3</td>
<td>0.06</td>
<td>F</td>
</tr>
<tr>
<td>2-Roundabout</td>
<td>C</td>
<td>22.8</td>
<td>0.73</td>
<td>F</td>
</tr>
<tr>
<td>3-Signal</td>
<td>B</td>
<td>13.8</td>
<td>0.41</td>
<td>D</td>
</tr>
<tr>
<td>1-No Build$^1$</td>
<td>B</td>
<td>10.3</td>
<td>0.02</td>
<td>F</td>
</tr>
<tr>
<td>2-Roundabout</td>
<td>F</td>
<td>&gt;200</td>
<td>1.84</td>
<td>F</td>
</tr>
<tr>
<td>3-Signal</td>
<td>E</td>
<td>66.2</td>
<td>1.01</td>
<td>E</td>
</tr>
</tbody>
</table>

Notes:
1. No Build assumes the intersection is one-way stop-controlled.
2. Delay values higher than 200 seconds are unreliable, and are therefore reported as >200.

Based on the operational results from Table 5, Alternative 3 (Signal) was evaluated for queues to help determine storage length requirements for turn lanes. Queue lengths for Alternative 3 are presented below in Table 6. Ninety-fifth percentile queues are defined as the queue length that has a 5 percent probability of being exceeded during the analysis period. Fiftieth percentile queue lengths are more typical of the driver experience, but 95$^{th}$ percentile queue lengths are a conservative estimate used to help design storage lengths for turn lanes where right-of-way is available.

The eastbound and northbound approaches experience the longest queues in the PM peak hour, with the 50$^{th}$ and 95$^{th}$ percentile queue lengths both exceeding the adjacent turn lane storage length. However, the eastbound right-turn and northbound left-turn lane demand volumes and corresponding queue length are quite low (approximately one vehicle per cycle), and should not cause an operational problem. The southbound approach experiences the longest queues in the AM peak hour, with the through lane 50$^{th}$ percentile queue length of 500 feet exceeding the right-turn lane storage length of 250 feet. This means that the southbound through lane queue will periodically block entrance into the right-turn lane, which could increase the southbound approach queue length beyond what is reported in Table 6 because right-turn vehicles are stuck in the through lane north of the NE 107$^{th}$ Street intersection. The southbound approach queue would also block the northbound left-turn lane at the NE 107$^{th}$ Street/Red-Wood Road intersection. A potential mitigation to this issue is to convert the southbound right-turn lane at the NE 107$^{th}$ Street/Red-Wood Road intersection into a shared through-right lane that lines up with the right-turn lane at 160$^{th}$ Avenue NE, as well as widen approximately
100 feet Red-Wood Road just south of the NE 107th Street intersection. This would create approximately 725 feet of storage length for southbound right-turn vehicles destined to 160th Avenue NE, which is longer than the 95th percentile queue length of 700 feet for the southbound through lane.

Table 6: 2030 Queue Lengths at Red-Wood Road/160th Avenue NE Intersection for Alternative 3 (Signal)

<table>
<thead>
<tr>
<th>Direction</th>
<th>Lane</th>
<th>Storage Length(^1) (ft)</th>
<th>50th Percentile Queue(^2) (ft)</th>
<th>95th Percentile Queue(^3) (ft)</th>
<th>50th Percentile Queue(^2) (ft)</th>
<th>95th Percentile Queue(^3) (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eastbound</td>
<td>Left</td>
<td>2,600</td>
<td>375</td>
<td>500</td>
<td>675</td>
<td>950</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>200</td>
<td>0</td>
<td>25</td>
<td>25</td>
<td>25</td>
</tr>
<tr>
<td>Northbound</td>
<td>Left</td>
<td>150</td>
<td>20</td>
<td>25</td>
<td>25</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>Through</td>
<td>2,600</td>
<td>200</td>
<td>300</td>
<td>675</td>
<td>950</td>
</tr>
<tr>
<td>Southbound</td>
<td>Through</td>
<td>1,300</td>
<td>500</td>
<td>700</td>
<td>275</td>
<td>375</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>250</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Notes:
1. Through lane storage is estimated based on distance to the adjacent signalized intersection or 1/2 mile, whichever is closer.
2. The 50th percentile queue is defined as the queue length that has a 50 percent probability of being exceeded during the analysis period.
3. The 95th percentile queue is defined as the queue length that has a 5 percent probability of being exceeded during the analysis period.

8.4 Recommendation

The traffic analysis and as well as the examination of constructability indicates that the signalized alternative best serves the needs of vehicles, pedestrians, and bicyclists, safely and cost-effectively. Construction costs and impacts are minimized by the alternative being smaller than the roundabout. The signalization provides a safe connection between 160th Avenue NE and Red-Wood Road without significantly affecting existing traffic patterns on Red-Wood Road. The alternative also provides a safe connection for pedestrians on both sides of Red-Wood Road and connects the existing neighborhoods better than a roundabout or the existing conditions.
City of Redmond 160th Avenue NE Extension
Preliminary Evaluation of Retaining Walls

PREPARED FOR: Steve Gibbs/City of Redmond
PREPARED BY: Mark Johnson/CH2M HILL
REVIEWED BY: Roger Mason, Ken Green/CH2M HILL
DATE: June 15, 2012

Introduction
This project consists of extending 160th Avenue NE from the current terminus at NE 99th to NE 102nd Street and the intersection with Redmond-Woodinville Road. The project includes several structures, including retaining walls and a crossing structure over a ravine and wetland. A previous technical memorandum addressed the crossing structure located at the wetland south of the PSE transmission lines. This memorandum identifies and evaluates the retaining walls needed to construct the 160th Avenue extension roadway.

Retaining Walls
Retaining walls for this project include both fill and cut walls. Typical fill wall types include structural earth (SE) walls, cast-in-place concrete cantilever walls, and rockery walls. Structural earth walls, which consist of a select structural backfill zone reinforced with layers or mats of reinforcement, are typically faced with precast concrete panels attached to the wall reinforcement. They are an economic wall choice in locations where there is sufficient wall area to justify their use. Concrete cantilever walls are another common wall type, but are typically more expensive than structural earth walls. Rockery walls are a cost-effective alternative for relatively short walls less than 10 feet high.

Typical cut wall types include concrete cantilever walls, soldier pile walls, and soil nail walls. Concrete cantilever walls are often an economical choice where there is sufficient space to overexcavate the soil to place the wall footing. Where overexcavation is not practical or cost-effective, soil nail or soldier pile walls are needed. Soil nail walls are typically used in soil conditions in which the soil can temporarily stand vertically several feet high to place the rows of soil nails.

A preliminary roadway plan and profile has been developed for the project to satisfy a number of objectives, including limiting the maximum roadway grade up the hill; minimizing impacts to existing wetlands; limiting right-of-way impacts; balancing the amount of cut and fill within the project site; and minimizing the number and size of retaining walls. The site plan showing the location and extent of retaining walls is shown in Figure 1. The corresponding roadway profile is shown in Figure 1B.
Wall A
This tall fill wall is located on the west side of the roadway along the steep hillside at the south end of the alignment where a significant amount of fill is needed. A structural earth wall is proposed, as shown in Section A in Figure 2. At the base of the hill adjacent to Wetland 1A, it is expected that the surface soils are poor and will need to be removed prior to placing structural backfill for the wall to prevent excessive settlement and bearing failure. As the roadway climbs the hill, the wall will likely need to be benched into the hillside, as shown in Section B in Figure 2, to ensure global stability of the wall on the steep slope. Benching will require a significant amount of overexcavation of soil at the base of the wall.

Walls B and C
These two fill walls are located on the west side of the roadway to retain the roadway fill within the property limits. Given the length and height of the walls, structural earth walls are proposed. A typical section is shown in Section A in Figure 2.

Walls D and F
These two cut walls are located near the top of the steep hillside where the roadway is benched into the hill. Between the two walls, the roadway crosses over a shallow ravine and Wetland 2A, creating a short fill section retained by Wall E. The cut walls are needed to prevent the cut slope from extending past the right-of-way limit, as shown in Section B in Figure 2.

Soil nail walls are proposed for these two walls, although the nails will extend into private property, requiring a permanent easement. Another option is to move the walls to the edge of the roadway so that the soil nails are mostly within the right-of-way; however, this would make it more difficult to widen the roadway or add a sidewalk to the east in the future. Another alternative is to use a concrete cantilever wall along the property line. This would require overexcavation or temporary shoring to construct the wall footing.

Walls E and G
These two walls are fill walls needed adjacent to wetlands to retain the roadway fill for a drainage culvert under the road. Given the short height and limited extent of the walls, rockery walls are a cost-effective choice.

Summary
A summary of the retaining walls is shown in the table below.

Table 1 – Retaining Wall Summary

<table>
<thead>
<tr>
<th>WALL ID</th>
<th>CUT/FILL</th>
<th>WALL TYPE</th>
<th>LENGTH (FT)</th>
<th>AREA (SF)</th>
<th>UNIT COST</th>
<th>COST</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>FILL</td>
<td>SE</td>
<td>355</td>
<td>7,150</td>
<td>$90</td>
<td>$643,500</td>
</tr>
<tr>
<td>B</td>
<td>FILL</td>
<td>SE</td>
<td>70</td>
<td>960</td>
<td>$50</td>
<td>$48,000</td>
</tr>
<tr>
<td>C</td>
<td>FILL</td>
<td>SE</td>
<td>300</td>
<td>3,660</td>
<td>$50</td>
<td>$183,000</td>
</tr>
<tr>
<td>D</td>
<td>CUT</td>
<td>SOIL NAIL</td>
<td>95</td>
<td>880</td>
<td>$120</td>
<td>$105,600</td>
</tr>
<tr>
<td>E</td>
<td>FILL</td>
<td>ROCKERY</td>
<td>50</td>
<td>400</td>
<td>$40</td>
<td>$16,000</td>
</tr>
<tr>
<td>F</td>
<td>CUT</td>
<td>SOIL NAIL</td>
<td>135</td>
<td>1,010</td>
<td>$120</td>
<td>$121,200</td>
</tr>
<tr>
<td>G</td>
<td>FILL</td>
<td>ROCKERY</td>
<td>15</td>
<td>150</td>
<td>$40</td>
<td>$6,000</td>
</tr>
</tbody>
</table>

14,210 $1,120,000
As the table shows, approximately half of the total wall cost of the project is for Wall A. The cost of Wall A could be reduced if the roadway alignment is shifted farther to the east into the hillside; however, this would likely have more right-of-way impacts than the current alignment. Another idea would be to shift the wall to the east several feet and cantilever a portion of the sidewalk past the face of the wall. This would reduce the overall wall height along the length of the wall, but would be offset by the cost of the cantilevered sidewalk.

The cut walls (Walls D and F) also represent a significant portion of the overall wall cost. These walls could be significantly reduced or possibly eliminated if the cut slope could extend past the Veal property limit. The feasibility of these cost reduction measures should be explored in a subsequent phase of design to optimize the overall cost of the retaining walls.
**FIGURE 2: 160TH AVENUE NE EXTENSION**

**CITY OF REDMOND**

**WALL TYPICAL SECTIONS**

1" = 5'

**JUNE 15, 2012**

---

**NOTE:**

AT STEEP SECTIONS OF EXISTING HILLSIDE, BENCH WALL INTO HILLSIDE 3'-4' MINIMUM.

EXISTING HILLSIDE, SEE NOTE

FILL RETAINING WALL (SE WALL)

EXISTING GROUND

PROPERTY LINE

CUT RETAINING WALL (NAIL WALL)

H

10'

6'

11'

11'

160TH

SIDEWALK

BIKE LANE

LANE

LANE

BIKE LANE

0.7H

0.1H, 1 MIN

0.7H

1.5:1

1' MIN

NOTE:

EXISTING HILLSIDE, BENCH WALL INTO HILLSIDE 3'-4' MINIMUM.
Evaluation of Crossing Alternatives at Wetland 3
City of Redmond 160th Avenue NE Extension
Evaluation of Crossing Alternatives at Wetland 3

PREPARED FOR: Steve Gibbs/City of Redmond
PREPARED BY: Mark Johnson/CH2M HILL
REVIEWED BY: Roger Mason/CH2M HILL
DATE: June 8, 2012

Introduction
This project consists of extending 160th Avenue NE from the current terminus at NE 99th to NE 102nd Street and the intersection with Redmond-Woodinville Road. An overall site layout, roadway profile, and typical roadway section is shown in Figure 1 of Appendix A. The purpose of this phase of work is to identify alternatives to be considered, evaluate them in enough detail so a preferred alternative can be selected, and develop budget-level costs for decision-making and budgeting.

The project includes several structures, including retaining walls and a crossing structure over a ravine and wetland. The crossing structure is located at Wetland 3 south of the PSE transmission lines. This memorandum identifies and evaluates several alternatives for the crossing structure, which are compared on a technical and cost basis, and a preferred alternative is recommended.

Crossing Structure Alternatives
The ravine located immediately south of the PSE transmission lines spans approximately 150 feet from bank to bank and is about 20 feet deep at the proposed road crossing. It contains seasonal stream flow and a wetland designated as Wetland 3. The roadway crosses the ravine on a sharp skew. One of the considerations for the crossing structure is to provide for an equestrian trail that would cross under the roadway along the north bank of the ravine and connect to the PSE trail on each end.

The soils in the ravine at the wetland appear to be soft, compressive soils of unknown depth. It is assumed that the placement of structure on these soils would require deep foundations (steel piles) to support the structure. The soils along the sides of the ravine appear to be relatively dense, competent soils capable of supporting spread footings without the need for deep foundations.

Several alternatives were identified for the ravine crossing, including bridge and culvert concepts. A culvert manufacturer was consulted to identify and evaluate cost-effective structure types. A plan and typical section of each alternative is included in Appendix A. The alternatives were compared using evaluation criteria that are shown in Appendix B. Planning-level comparative costs were developed for each alternative and are summarized in Appendix C.
Alternative 1 – Single Span Bridge
This alternative is considered the baseline option for crossing the ravine and is shown in Figures 2 and 3 of Appendix A. The relatively short crossing lends itself to a single span prestressed girder bridge with abutments on spread footings and wingwalls at the ends to retain the roadway embankment. A 100-foot long structure spans entirely over the wetlands and provides sufficient clearance for an equestrian trail under the structure, but is the most expensive alternative.

Alternative 2 – 48’ Elliptical Steel Culvert
This alternative is the longest practical steel closed-shape culvert span and is shown in Figures 4 and 5 of Appendix A. It requires over excavation of the poor, soft soils at the wetland. The advantage of this type of structure is the relatively low cost and ease of shipping and installation. However, it has significant impact to the wetland due to its relatively short span and likely does not provide enough space for an equestrian trail undercrossing. A separate culvert would be needed for the trail adjacent to the roadway crossing culvert, as shown in Figure 5.

Alternative 3 – 65’ Steel Arch Culvert
This alternative, shown in Figures 6 and 7 of Appendix A, provides a longer span steel culvert and consists of an arch-shaped structure on spread footings. The benefit over Alternative 2 is the longer span achieved, reducing wetland impacts. However, it requires the use of pile foundations and, as with the elliptical culvert, likely does not provide enough space for an equestrian trail undercrossing.

Alternative 4 – 60’ Precast Concrete Culvert
Precast concrete culvert structures were also considered for the crossing. This alternative is a 60’ precast concrete 3-sided structure, the longest span available for this type of culvert, and is shown in Figures 8 and 9 of Appendix A. Its main drawback is its limitation in accommodating the skewed crossing of the road over the ravine. As shown in Figure 8, it consists of tapered precast segments that, when assembled, form a curved structure. However, this approach requires a much wider structure than the roadway above, adding to the cost and additional wetland impacts.

Alternative 5 – 84’ Precast Concrete Arch Culvert
The other precast concrete alternative considered is an arch-shaped culvert that is shown in Figures 10 and 11 of Appendix A. The structure is made up of twin leaf precast panels approximately 4’ wide that are cast together at the crown of the structure during installation. This alternative is a significant improvement over the shorter span precast culvert, as it has sufficient length to span parallel to the roadway without a skew, minimizing the width of the structure. It also has sufficient headroom to accommodate an equestrian trail undercrossing.

The main drawback is the overall height of the arches, which require a relatively deep excavation at the northwest corner of the structure. A variation of this structure is a flatter profile precast arch, as shown in Figure 11. This flatter arch section spans up to about 100 feet and imparts significant thrust loads into the foundations. It is expected that a thrust block could be provided at each end to distribute the forces into the sides of the ravine.
Evaluation of Alternatives

The alternatives were evaluated and compared using evaluation criteria, as shown in Appendix B. Both a weighted and unweighted screening was conducted. Construction cost was considered to be the most important criterion, and was weighted the highest. As part of the evaluation, planning-level comparative costs were developed for each alternative and are summarized in Appendix C.

The precast concrete arch culvert, Alternative 5, scores highest for both weighted and unweighted screening. It is the lowest cost alternative, approximately 59% of the baseline bridge cost, and has relatively minor impacts to the wetland. It has high durability and is better able to resist corrosion and damage than the steel culvert alternatives. Due to its tall arch shape, it provides for an equestrian trail undercrossing without having to add a separate culvert structure.

Summary

The long span precast concrete arch culvert appears to be the best choice for this crossing due to its relative cost to the other alternatives and the minimal environmental impacts. Further refinement of this alternative, by considering flatter profile arch configurations, can be considered in a subsequent phase of design to optimize the overall cost of the crossing structure.
APPENDIX A

FIGURES OF CROSSING ALTERNATIVES
60' PRECAST CULVERT (CONSPAN) TYPICAL SECTION
APPENDIX B

EVALUATION OF ALTERNATIVES MATRIX
## City of Redmond 160th Avenue NE Extension Project
### Screening Matrix of Alternatives - Wetland 3 Crossing
#### Revision Date: June 8, 2012

<table>
<thead>
<tr>
<th>Evaluation Criteria</th>
<th>Weighting Factor (1 - 10)</th>
<th>Alt 1 - Bridge Crossing</th>
<th>Alt 2 - 48' Elliptical Stl Culvert</th>
<th>Alt 3 - 65' Steel Arch Culvert</th>
<th>Alt 4 - 60' Precast Culvert</th>
<th>Alt 5 - 84' Precast Arch Culvert</th>
<th>Comments</th>
</tr>
</thead>
</table>
| Construction Cost   | 10                       | 1                       | 3                                 | 4                             | 2                           | 4                             | Alt 1: Highest cost.  
Alt 2: 65% of bridge cost.  
Alt 3: 57% of bridge cost.  
Alt 4: 91% of bridge cost.  
Alt 5: 59% of bridge cost. |
| Impacts to Wetlands | 7                        | 4                       | 1                                 | 2                             | 1                           | 3                             | Alt 1: No wetland impacts.  
Alt 2: Impacts 1,400 SF of wetland.  
Alt 3: Impacts 600 SF of wetland.  
Alt 4: Impacts 800 SF of wetland. Structure is low and much longer than other alternatives - much less light under str.  
Alt 5: Impacts 600 SF of wetland. Structure provides alot of light underneath. |
| Provides for Equestrian Trail | 7               | 3                       | 1                                 | 1                             | 1                           | 2                             | Alt 1: Provides for equestrian trail.  
Alt 2 - 4: Equestrian trail does not fit.  
Alt 5: Equestrian trail likely fits. |
| Construction Impacts | 4                        | 2                       | 3                                 | 1                             | 2                           | 2                             | Alt 1: Hauling/placement of 100' long girders.  
Alt 2: Some overexcavation needed.  
Alt 3: Pile driving likely needed for foundations, wide structural backfill area.  
Alt 4: Pile driving likely needed for foundations.  
Alt 5: Requires deep excavation for footings. |
| Constructability    | 4                        | 2                       | 4                                 | 4                             | 3                           | 3                             | Alt 1: Requires large cranes for girder erection.  
Alt 2 - 3: Easy to ship, assembled onsite with small cranes.  
Alt 4: Heavy pieces to ship and erect.  
Alt 5: Heavy pieces to ship and erect. |
| Maintenance and Inspection | 6                   | 3                       | 2                                 | 2                             | 4                           | 4                             | Alt 1: Bridge requires biannual inspection, deck requires maintenance.  
Alt 2 - 3: Steel surfaces require maintenance.  
Alt 4 - 5: Minimal inspection and maintenance requirements. |
| Design Life         | 7                        | 3                       | 2                                 | 2                             | 3                           | 3                             | Alt 1: Prestressed concrete long-lasting, exposed bridge deck vulnerable.  
Alt 2 - 3: Galvanized steel culvert shorter design life.  
Alt 4 - 5: Concrete long lasting. |
| Provides for Utilities | 2                   | 4                       | 2                                 | 2                             | 3                           | 2                             | Alt 1: Can hang utilities off superstructure.  
Alt 2 - 3: Can place utilities above culvert.  
Alt 4: More space for utilities above culvert.  
Alt 5: Can place utilities above culvert. |
| Right-of-Way        | 3                        | 3                       | 2                                 | 2                             | 1                           | 3                             | Alt 1: Structure confined to roadway area.  
Alt 2 - 3: Have walls on each end of culvert.  
Alt 4: Structure much wider than roadway.  
Alt 5: Structure confined to roadway area. |

| Total Score (unweighted) | 25 | 20 | 20 | 20 | 26 | Alternative 5 has highest score. |
| Total Score (weighted)    | 131 | 108 | 117 | 108 | 153 | Alternative 5 has highest score. |

**Performance Ratings:**

1 - Poor, performs worse than other alternatives  
2 - Good  
3 - Better  
4 - Best, outperforms other alternatives
## Summary of Alternatives

<table>
<thead>
<tr>
<th>No.</th>
<th>Description</th>
<th>Cost</th>
<th>% of Bridge</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Prestressed Girder Bridge</td>
<td>$1,390,000</td>
<td>100%</td>
<td>Assume piles not needed.</td>
</tr>
<tr>
<td>2</td>
<td>48' Elliptical Steel Culvert</td>
<td>$910,000</td>
<td>65%</td>
<td>Assume some overexcavation.</td>
</tr>
<tr>
<td>3</td>
<td>65' Steel Arch Culvert</td>
<td>$790,000</td>
<td>57%</td>
<td>Assume supported by steel piles.</td>
</tr>
<tr>
<td>4</td>
<td>60' Precast Culvert</td>
<td>$1,260,000</td>
<td>91%</td>
<td>Assume supported by steel piles.</td>
</tr>
<tr>
<td>5</td>
<td>84' Precast Arch Culvert</td>
<td>$820,000</td>
<td>59%</td>
<td>Assume piles not needed.</td>
</tr>
</tbody>
</table>

### Costs with Equestrian Underpass Trail

<table>
<thead>
<tr>
<th>No.</th>
<th>Description</th>
<th>Cost</th>
<th>Equestr. Culvert</th>
<th>Total Cost</th>
<th>% of Bridge</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Prestressed Girder Bridge</td>
<td>$1,390,000</td>
<td>$ -</td>
<td>$1,390,000</td>
<td>100%</td>
<td>Trail fits under bridge.</td>
</tr>
<tr>
<td>2</td>
<td>48' Elliptical Steel Culvert</td>
<td>$910,000</td>
<td>$190,000</td>
<td>$1,100,000</td>
<td>79%</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>65' Steel Arch Culvert</td>
<td>$790,000</td>
<td>$190,000</td>
<td>$980,000</td>
<td>71%</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>60' Precast Culvert</td>
<td>$1,260,000</td>
<td>$190,000</td>
<td>$1,450,000</td>
<td>104%</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>84' Precast Arch Culvert</td>
<td>$820,000</td>
<td>$ -</td>
<td>$820,000</td>
<td>59%</td>
<td>Trail likely fits under culvert.</td>
</tr>
</tbody>
</table>

### Alternative 1 - Prestressed Concrete Girder Bridge Cost (Spread Footings):

**Bridge**
- Bridge Length: 100 FT
- Bridge Width: 52 FT
- Bridge Deck Area: 5,200 SF
- Square Foot Cost: $200

**Bridge Cost:** $1,040,000

**Retaining Walls**

<table>
<thead>
<tr>
<th>Location</th>
<th>Max Ht</th>
<th>Length</th>
<th>Area (SF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SE Corner</td>
<td>14</td>
<td>12</td>
<td>101</td>
</tr>
<tr>
<td>SW Corner</td>
<td>18</td>
<td>24</td>
<td>259</td>
</tr>
<tr>
<td>NE Corner</td>
<td>15</td>
<td>30</td>
<td>270</td>
</tr>
<tr>
<td>NW Corner</td>
<td>8</td>
<td>12</td>
<td>58</td>
</tr>
</tbody>
</table>

**Total Wall Area:** 688
**Square Foot Cost:** $100
**Wall Cost:** $69,000

**Subtotal:** $1,110,000
**Design Contingency:** 25%
**Total Structure Cost:** $1,390,000

### Alternative 2 - 48' Elliptical Steel Culvert (28' Rise)

<table>
<thead>
<tr>
<th>No.</th>
<th>Item</th>
<th>Unit</th>
<th>Quantity</th>
<th>Unit Price</th>
<th>Amount</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>STRUCTURE EXCAV CL A INCL. HAUL</td>
<td>C.Y.</td>
<td>5,500</td>
<td>$20</td>
<td>$110,000</td>
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</tr>
<tr>
<td>2</td>
<td>ELLIPTICAL STEEL CULVERT</td>
<td>L.F.</td>
<td>75</td>
<td>$4,500</td>
<td>$337,500</td>
<td>Installed cost</td>
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<tr>
<td>3</td>
<td>CULVERT HEADWALLS</td>
<td>C.Y.</td>
<td>22</td>
<td>$600</td>
<td>$13,200</td>
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</tr>
<tr>
<td>4</td>
<td>SELECT STRUCTURAL BACKFILL</td>
<td>C.Y.</td>
<td>3,600</td>
<td>$25</td>
<td>$90,000</td>
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</tr>
<tr>
<td>5</td>
<td>RETAINING WALLS</td>
<td>S.F.</td>
<td>1,500</td>
<td>$100</td>
<td>$150,000</td>
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</tr>
<tr>
<td>6</td>
<td>ROADWAY EMBANKMENT</td>
<td>C.Y.</td>
<td>2,500</td>
<td>$10</td>
<td>$25,000</td>
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</tr>
</tbody>
</table>

**Subtotal:** $726,000
**Design Contingency:** 25%
**Total Structure Cost:** $910,000

### Alternative 3 - 65' Steel Arch Culvert (19' Rise)

<table>
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<tr>
<th>No.</th>
<th>Item</th>
<th>Unit</th>
<th>Quantity</th>
<th>Unit Price</th>
<th>Amount</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>STRUCTURE EXCAV CL A INCL. HAUL</td>
<td>C.Y.</td>
<td>3,400</td>
<td>$20</td>
<td>$68,000</td>
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</tr>
<tr>
<td>2</td>
<td>STEEL ARCH CULVERT</td>
<td>L.F.</td>
<td>75</td>
<td>$3,300</td>
<td>$247,500</td>
<td>Installed cost</td>
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<tr>
<td>3</td>
<td>CULVERT HEADWALLS</td>
<td>C.Y.</td>
<td>16</td>
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<tr>
<td>4</td>
<td>CONCRETE CLASS 4000 FOR FOOTING</td>
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<td>67</td>
<td>$400</td>
<td>$26,800</td>
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<tr>
<td>5</td>
<td>FABRICATING STEEL H-PILES</td>
<td>L.F.</td>
<td>480</td>
<td>$50</td>
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City of Redmond 160th Avenue NE Extension Project  
Comparative Cost Evaluation of Alternatives - Wetland 3 Crossing  
Revision Date: June 8, 2012

<table>
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<tr>
<th>No.</th>
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<th>Amount</th>
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</thead>
<tbody>
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<td>6</td>
<td>DRIVING STEEL H-PILES</td>
<td>EA.</td>
<td>12</td>
<td>$2,500</td>
<td>$30,000</td>
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<td>7</td>
<td>SELECT STRUCTURAL BACKFILL</td>
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<td>4,100</td>
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<td>S.F.</td>
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ALTERNATIVE 4 - 60' PRECAST CULVERT (12' RISE)

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<td>STRUCTURE EXCAV CL A INCL. HAUL</td>
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<td>CULVERT HEADWALLS</td>
<td>C.Y.</td>
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<td>$1,260,000</td>
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ALTERNATIVE 5 - 84' PRECAST ARCH CULVERT (22' RISE)

<table>
<thead>
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<th>Amount</th>
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<tr>
<td>7</td>
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<td>Design Contingency</td>
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STEEL CULVERT FOR EQUESTRIAN UNDERPASS TRAIL

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<tr>
<th>No.</th>
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<tr>
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<td>Total Structure Cost</td>
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<td></td>
<td></td>
<td>$190,000</td>
</tr>
</tbody>
</table>

Notes:
1. The estimate does not include sales tax, escalation, or owner costs such as engineering, administrative, construction management, legal, or permitting.
2. This cost opinion is in June 2012 dollars. It does not include future escalation or unusual material cost increases. No potential hazardous material mitigation is included.

The cost opinions shown have been prepared for guidance in project evaluation from the information available at the time of preparation. The final costs of the project will depend on actual labor and material costs, actual site conditions, productivity, competitive market conditions, final project scope, final project schedule, and other variable factors. As a result, the final project costs will vary from the costs presented above. Because of these factors, funding needs to be carefully reviewed prior to making specific financial decisions or establishing final budgets.
Geotechnical Summary of Findings Memorandum
City of Redmond 160th Avenue NE Extension
Geotechnical Summary of Findings Memorandum and Preliminary Evaluation of Geotechnical Conditions

PREPARED FOR: Steve Gibbs/City of Redmond
PREPARED BY: Ken Green/CH2M Hill
REVIEWED BY: Roger Mason, Mark Johnson/CH2M Hill
DATE: August 4, 2012

Introduction
This project consists of extending 160th Avenue NE from the current terminus at NE 99th to NE 102nd Street and the intersection with Redmond-Woodinville Road. An overall site layout, roadway profile, and typical roadway section is shown in Figures 1A and 1B of this report. The purpose of this project is to identify alternative roadway alignments and options to be considered, evaluate them in enough detail so that a preferred alternative can be selected, and to develop budget-level costs for decision-making and budgeting for future planning and advancement of the project.

This memorandum provides a geotechnical summary of findings and provides preliminary evaluation of geotechnical conditions along the proposed alignment. This memo also provides recommendations for the anticipated work associated with design and construction of the new roadway and discusses areas where additional subsurface investigations will be necessary.

No additional geotechnical field work was completed for this phase of the project; but instead prior geotechnical work conducted by others along with careful field observation of site conditions were noted as a basis for the decisions that form the basis of our recommendations contained in the phase of the work. As this project moves forward in the planning and design process, additional detailed field investigations will be required to confirm foundation conditions along selected areas of the alignment as required to develop final recommendations for types of structures, foundation conditions and preparation, and other important information necessary for design development.

Existing Reports
Existing reports and information pertinent to this project include the following:

- USDA Natural Resources Conservation Service (NRCS), Soil Survey Data, version 7, Jul 2, 2012
- Preliminary Geotechnical Engineering Services, 160th Avenue Northeast Extension Route Location Study, Redmond, Washington, for City of Redmond, May 23, 1995, GeoEngineers. This site evaluation included field inspections, 7 test pits, and 3 hand auger borings, and hand probes in ravines. This report is attached to this memorandum. Field activities were accomplished between May 4 and May 16, 1995. Laboratory testing consisted of testing field moisture contents of a total of 20 samples obtained from field explorations and are provided in the attached report.
- Selected Hand Auger Hole Logs from Report of Geotechnical Engineering Services Proposed Redmond 74 Residential Development, city of Redmond File No. PPL90-002, Redmond,
Washington, by GeoEngineers, Inc. (presumed to be about 1990). Attachment includes a total of 6 hand auger borings.


**Background Information**

Figure 1A provides a plan view of the proposed roadway alignment illustrating the transition point at the south end of the project where the proposed roadway will connect to existing City of Redmond streets at 160th Avenue NE. The connection at the north end is at the Redmond-Woodinville road and is slightly north of the portion of the alignment shown on Figure 1A. The new roadway will occupy the existing alignment of 156th Avenue NE and will require only slight modification between station 40+00 shown on Figure 1A and a new intersection that is proposed to be constructed near Redmond-Woodinville Road NE at the north end of the corridor.

Figure 1B illustrates the conditions along the alignment in profile. Figures 1A and 1B also illustrate the generally location of existing utilities, wetlands, trail, and other existing features and provides a general location of retaining walls culverts and other structures that will be required for development of this segment of the proposed roadway. Extensive grading and new construction is required along about 1,400 lf of the alignment starting at about station 26+00 on the south end and extending to about station 40+00 on the north end. Additional road improvements and modifications are required at the north end where the new roadway will connect to Redmond-Woodinville Road NE making the length of the entire proposed roadway improvements about 2,400 lf in length.

Several roadway alignments have been evaluated as part of this study and are discussed in another technical memo. The roadway generally begins at the toe of the slope next to the Sammamish River Valley at 160th Avenue NE on the south end of the corridor, and then gradually climbs up the east valley wall, crossing existing incised ravines and utility corridors along the way. The north end of the proposed corridor terminates on a broad, gently sloping plateau. Construction of road improvements will require extensive earthwork including sidehill cuts into the hillside on the east side of the new road, cuts through the high ridges that separate the ravines, and fills in many areas along the west edge of the new roadway. These cuts and fills will require the use of retaining walls in numerous areas including walls on the east side to limit the width of disturbance into private property, walls on the west side to limit the extent of filling into the wetland and onto steep slopes below the roadway alignment. Some combination of filling and use of bridges and culverts will be required to cross each of the ravines across which the new road will traverse. The roadway will need to avoid the existing overhead powerlines, poles and towers to the extent possible, and will need to avoid disturbance to the existing 54” diameter Tolt Water Supply Pipeline located near station 35+40.

Fills along the west side of the road in areas near the Sammamish Valley floor are expected to range in height from a few feet to more than 20 feet. The largest fills and associated retaining walls are expected to occur between about stations 27+00 and 30+00. Fills and possible retaining walls will also likely be
required at ravine crossings near wetlands 2 and 3. Small walls may be needed at wetland 4 to limit filling in the wetland and wetland buffer areas.

Cuts of 10 feet or more will likely be required along the east side. These cuts, when they occur in hard silt or dense sand and gravel can be formed at 2H:1V. Where the cuts would extend beyond the property lines, small retaining walls will be required to retain a portion of the cut to control the width of the excavation.

**Site Conditions**

The proposed project extends along the east edge of the Sammamish River Valley which is characterized by its broad north-south trending orientation. The valley in this area is quite flat and generally at or slightly below elevation 30 feet (mean sea level) in the vicinity of the proposed road corridor. The geologic conditions differ significantly along the length of the proposed road corridor. The proposed roadway starts at 160th avenue NE near the base of the slopes that define the edges of the Sammamish Valley. From there, the proposed road alignment extends to the northeast crossing along the side of steep slopes of the Valley’s sidewall and across a portion of a broad upland known locally as Education Hill.

This corridor crosses three distinctly different physiographic areas each having uniquely different geologic conditions. The deeper soils along the corridor are the result of glacial activities that occurred during the advancement and retreat of ice sheets during the last period of glaciation, about 10,000 to 20,000 years ago. Shallow soils in the area of the corridor are the result of erosion and deposition that occurred from meltwater streams and lakes during the retreat of glaciers; and this erosion process continues to this day. The soils in the physiographic areas are described as follows starting from oldest to youngest:

- **Transitional beds** are the oldest soil formation, consisting primarily of silt and sand, some gravel, with minor amounts of clay. These soils can be found in the sidehill slopes of the valley and under the broad plateau that caps the upland area. These soils were deposited in lakes and streams ahead of the advancing glacial ice. These soil deposits were later buried under advance outwash sand and gravel deposited by streams and rivers in front of the advancing Vashon glacier. The advance outwash is mapped as underlying much of the upland area west of Woodinville-Redmond Road. As the ice covered the region, it deposited Vashon glacial till over the older glacial and nonglacial deposits that currently mantle much of the higher elevations of the plateau. Till consists of a non-sorted, well graded mixture of clay, silt, sand, and gravel, is very dense and generally possess a low permeability. The till was overridden with great thicknesses of ice, compressing and compacting it as well as all underlying soils to a dense, generally high-strength condition. Although till was not observed along the proposed road corridor, it is known to mantle many areas at the higher elevations of Education Hill. It is likely that till once mantled the project area also but has since been removed by extensive scouring from glacial activities and continuing erosion, exposing the underlying outwash sand and gravel and transitional beds. This scouring and erosion has also formed stream channels and ravines which are presently being incised into the valley sidewalls forming the existing channels that are occupied by small streams that drain water from the upland areas of Education Hill. It is estimated by others that the contact between the top of the transitional beds and the overlying outwash sand and gravel occurs at about elevation 80.

Outwash sand and gravel and the underlying transitional beds are the soils that most predominantly exist along the sidewalls of the Sammamish River Valley extending from about road station 27+50 to 40+00; and may extend as far as the north end of the proposed project at Redmond-Woodinville road. Because of the great length of this segment of the alignment, these soil conditions represent the predominant soils that are expected to be encountered along the proposed length of the project.

Soils encountered in test pit explorations in this zone (generally below elevation 80 in the transitional beds), consists of hard silt with varying amounts of silt and gravel. The silt was encountered at depths ranging from 3.3 to 10 feet in several of the explorations. Groundwater seepage was observed in several of the test pits.
ranging from 7 to 10.5 feet below the ground surface. These depths were generally found to correspond with the top elevation of the transitional beds at about elevation 80 where observed. Test pits near the tops or edges of ridges did not encounter groundwater probably because these areas are flanked by steep slopes of the ravines that allow drainage in these areas. In several areas, seepage and wet conditions can be observed in the bottom of ravines and near the base of the steep slopes and at the heads of the ravines. The general elevation at which the seepage appears to be occurring seems to correspond to the stratigraphic contact between the transitional beds and the overlying more pervious outwash sand and gravel.

It appears that similar soil conditions may be encountered from station 40+00 to the north end of the project at Redmond-Woodinville Road. Previous explorations in this area indicate that the upland portion of the corridor is mantled by about 0.5 to 1.5 feet of topsoil which is underlain by medium dense to dense sand and silty sand and stiff to hard silt with varying amounts of sand and gravel, and extended to the maximum depth explored, about 11.5 feet deep. Perched groundwater was encountered in a few of test pits at a depth as shallow as about 2 to 3 feet.

- The Sammamish River Valley was also formed largely during the glacial activities of the last glacial period and represents the next younger physiographic area of the project. The valley shape was formed by the thick ice sheets that occupied the area, scouring the older soil and rock and depositing soils from streams and rivers associated with glacial runoff. The Sammamish River Valley was subsequently filled with unconsolidated soils, primarily soft soils consisting of alluvial deposits of peat, silt, and sand. Other post glacial deposits include slopewash in moderately to steeply sloping areas next to the valley sidewalls and deposition from deltas that formed related stream channels and ravines that enter from the sides of the valley.

These soil conditions affect the southern end of the project from about station 25+00 to 27+50 as the proposed corridor transitions from the valley bottom to the valley sidewall conditions. Soils in this area are expected to consist of topsoil underlain by deep deposits of soft silt and sand, with peat in some areas, overlying loose to dense granular soils and hard silt at depth. The thickness and extent of these surficial soils varies along this segment of the corridor. Explorations by others in this segment have encountered a topsoil layer that ranges from 0.5 to 1.5 feet in thickness, underlain by soft to medium stiff silt and peat (peat ranging from 1 to 7 feet in thickness). Next to the hillside, slopewash has resulted in soft or loose colluvial soils that mantle the older soils near the ground surface. Test pits in the area slightly south of the end of this project encountered soft surficial soils generally extending to depths of 4 to 12 feet deep and were underlain by loose to dense sand and silty sand with varying amounts of gravel and hard silt. The hard silt unit appears to be part of the transitional beds and was encountered only in the explorations located close to the valley sidewall. It is anticipated that peat and soft soils may be encountered near station 27+50 where the proposed alignment exists at its closest point to the Sammamish River Valley wetlands (shown as wetland 1A on figure 1A). Groundwater seepage was observed at depths ranging from 3.5 to 12 feet below the existing ground surface in existing explorations in this area. Groundwater levels along the east side of the valley wall are expected to generally be closer to the ground surface at locations near the base of the slope. The water levels are expected to vary in response to seasonal levels of the Sammamish river, and to the amount of surface and subsurface water originating from the outwash and transitional beds along the valley sidewall.

- The youngest soils to be encountered along the proposed corridor are found in areas of recent erosion, sedimentation and slopewash. These areas exist primarily near the bottom of several incised ravines that contain small streams. In these areas, the soils consist of topsoil and organic silt and sand. Slope wash consisting of topsoil and colluvial silt and sand also exist near the base of steep side slopes of the ravines. In the stream bottom areas, the depth of soft organic sediments is unknown but is estimated to range from less than a foot to 6 feet or more in some areas where more substantial sedimentation has occurred. Wetlands have formed and are generally associated with these small stream deposits as shown on Figures 1A and 1B as
wetland 2, wetland 3, and wetland 4. Portions of wetland 1 have likely developed from similar conditions along a small stream that exists above wetland 1.

At the base of the steep slopes and along the hillsides, slopewash has occurred. Several feet of loose topsoil and organic silt and sand may exist in some areas. At the mouth of the ravines containing small streams, alluvial fans have typically formed consisting primarily of loose to medium dense sand with silt and gravel.

The ravines and streams that occupy the incised ravines along the valley sidewall have formed from runoff from the upper plateau of Education Hill east of the Redmond. Because much of the soils that cap the higher elevations of Education Hill plateau consist of dense till, the permeability is low. This leads to relatively low rate of infiltration resulting in increased runoff to these small streams that drain from the plateau. Since more pervious sand and gravel is sandwiched between the base of the till and the top of the transitional beds, infiltration in this zone is higher. Groundwater appears to generally be perched however at the top of the dense, low permeability transitional beds resulting in a predominate line of seepage near this contact line. This seepage is expressed by supplying water to the wetland soils in the ravines.

Figure 1B shows a profile view longitudinally along most of the proposed corridor. This figure illustrates the location of incised ravines along the alignment, wetland areas, and the various physiographic regions that have been described previously. Also shown is a rough interpretation of soil types that may exist along the corridor in these various areas as described above.

Conclusions and Recommendations

In general, we believe that the subsurface conditions along most of the corridor are favorable for construction of the proposed roadway. Based on review of site topographic and geotechnical conditions observed in the field throughout the corridor and from existing reports, it appears that several elements of the site conditions near the south end of the project could lead to increased construction cost in this area. These conditions occur primarily in the vicinity of station 27+00 to 30+00. Several elements of the layout and design contribute to issues that must be dealt with near this location:

- Presence of soft compressible soils in areas where high retaining walls are required for example under the western portion of the new road in the vicinity of Wetland 1A.
- Width of the roadway overlaid on steep transverse slopes of the hillside will lead to increased height of retaining walls in this area because of the topography.
- Hillside stability issues although not evident in the field along this portion of the corridor, should be investigated. This concern is triggered by observation that a thick rather massive retaining wall may have been used along east side of 160th Ave NE immediately south of the beginning of the new corridor.
- Seepage and groundwater are expected to be encountered in excavations near the Wetland 1A because of the relatively low elevation; and possible seepage in portions of the ravine located immediate east of wetland 1A.

The above considerations within the south end of the corridor as well as other important considerations throughout the corridor are discussed in more detail later in this memorandum. These are areas that require additional investigation and more detailed understanding as the project moves forward, either because of unknown site conditions or because of potential construction cost implications to the project within these areas of the project.

Soft foundation conditions are expected to be encountered in the bottom of the ravines through which the road will pass. Overexcavation and replacement with suitable foundation soils is anticipated in most locations. The ravine at Wetland 3 is expected to be the location containing the most significant widths and depths of soft sediments that will require overexcavation.
It appears that soils encountered below anticipated depths of natural topsoil and slope wash in the remaining portions of the alignment is generally expected to be medium dense to very dense and should possess good foundation characteristics for construction of roadway and structures.

Conventional practices can be used in the construction of fill embankments, cut slopes, stream and trail crossing structure foundations, retaining walls, drainage and erosion control facilities and roadway pavement. Because of the proximity of steep slopes along the corridor, erosion and surface water control practices will have to be carefully constructed and monitored during construction. Removal of unsuitable soils will be required in the areas that have been described as having soft or undesirable foundation conditions. Nearly all of the soils encountered on the project are considered to be highly moisture sensitive and will require care during construction to perform work generally during non-rainy periods.

**Erosion Control**

Because of the sensitivity of soils to moisture and the erosive nature of the natural materials, erosion control during construction will be important. Best practices should be followed for all areas where vegetation is removed and areas are disturbed by the construction. Erosion control methods that should be used during construction include efficient channeling of surface water runoff, minimizing the extent of disturbed areas, use of erosion control slope cover such as straw and/or jute matting. Channel linings should be used to catch sediment and control downcutting and surface erosion; energy dissipaters should be used for trenches as required for controlling the velocity along the flow channels. The impacts of erosion should be mitigated to the extent possible by accomplishing the earthwork activities during normally dry seasons in July through early October.

Long-term erosion should be mitigated by accomplishing grading to limit the concentration of runoff onto fill, cut or natural slopes and other erosion-sensitive areas, installing permanent sedimentation basins and a storm drain system. Adequate seeding to establish grass and other vegetative cover at the appropriate time of the season for planting should be used to provide permanent erosion control. Careful observation with increased monitoring and maintenance should be applied for the 1st year or until the newly graded and seeded areas become fully established. Disturbance to natural drainage courses adjacent to the alignment should be kept to a minimum so that existing erosion, sedimentation, channel stability, and flow conditions will not be appreciably changed. Undisturbed natural vegetative buffers should be protected and along natural drainage courses to help reduce impacts to the drainage courses and reduce sedimentation.

**Site Earthwork Activities**

The roadway construction will require substantial cuts and fills as discuss previously. Construction of all earthwork components of the project should be planned for only the dry summer periods, generally July through early October. It is likely that earthwork accomplished during this period will allow some re-use of excavated materials so long at the characteristics of the soils are suitable including the natural moisture content. Convention excavation and earth moving equipment can be used for the project.

Imported fill will be required for construction of many of the fills. Imported materials should consist of well graded granular materials with generally less than about 10% passing the No. 200 sieve if the work is completed in the dry and away from areas of seepage. Where seepage is present or where the material could become moistened for any reason, the fill material should consist of well graded sand and gravel with less than 5 percent fines.

Excavations into the natural subgrade are anticipated to encountered seepage and excess moisture in some areas. Where seepage is encountered, the water should be controlled to the degree possible by providing drains to safely allow the water to discharge from the work areas.

It is recommended that a blanket drain is placed directly on the prepared roadway foundations to assure that water is drained away from the embankments. All seepage should be intercepted with such drainage blankets or
chimney drains to allow free discharge from the embankment and avoid building up of water pressure behind the embankment fill. If particularly wet areas are encountered, foundation drain pipes should be incorporated in the drainage layers to assure that water can be safely discharged by gravity from the foundations. The blanket and chimney drain should consist of free-draining, well graded sand and gravel or crushed rock having less than 3 percent passing the number 200 sieve.

Embankments should be placed in horizontal lifts in all areas, starting at the lowest point and working upward. Embankment lifts should tie into drainage layer or into the native soils if drainage zone is not provided. Each lift should spread to a uniform depth, not more than 10-inches in thickness prior to compaction. Where small hand-operated compaction equipment is required, lift thickness should be limited to 4-inches prior to compaction. Following the spreading of material to a uniform thickness, the soil should then be thoroughly compacted at a moisture content within 2 percent of optimum as defined by ASTM D1557. Embankment compaction should achieve a density not less than 95 percent of the maximum density in accordance with ASTM D1557. In areas where global stability is not a concern, the portion of the embankment deeper than 36 inches can be reduced to a maximum of 90 percent of the maximum density in accordance with ASTM D1557. Embankments should be kept crowned slightly at all times to prevent the ponding of water on the embankment surfaces. If soil is placed for compaction, the lift should be immediately compacted to the required density and not left exposed to absorb moisture.

Foundations for the proposed structures should be carried to a depth necessary to achieve a dense firm condition. Foundations should be proof rolled with a loaded dump truck if access if achievable and subgrade is relatively dry. In other conditions, subgrade should be tested for density and probed to determine if soft spots exist.

If the subgrade condition is wet and or soft, all excavation should be accomplished using equipment working from outside the limits of the excavation. Soft areas should be removed and disposed of. As the excavation is completed and acceptable uniform foundation conditions are achieved, the foundation should be immediately covered with the specified overlying material to protect the subgrade from further disturbance.

**Retaining Wall Types**

Retaining walls will be required at selected locations throughout the project. For cost reasons, mechanically stabilized earth (MSE) walls are proposed where site conditions allow construction on a stable foundation and where global stability requirements can be met. These walls types generally result in lower unit costs for construction, are commonly designed and used by contractors, and are tolerate of minor settlement and deflections from static and seismic conditions.

Other wall types that have been considered at this stage include cantilever soldier pile walls with lagging and soldier pile walls with tiebacks where improved global stability is necessary. These options are discussed and illustrated in much more detail in Technical Memorandum A-1 in Appendix A.

**Foundations for Structures**

Limited structures will be required for the proposed project. It appears from our initial assessment that foundations for most structures can be developed on spread footings. This includes the proposed arch culvert located at wetland 3. Further exploration will be required however, in order to identify the vertical and horizontal limits of soft organic materials in at this structure. Geologic evidence suggests that the depth of the soft undesirable materials are limited at this location and excavations carried to a reasonable depth below and outside the existing slopes will likely encounter competent soils consisting of outwash sand and gravel or transitional beds of hard silt and dense sand. It is important to investigate these foundation conditions well in advance since all soft and compressible materials will need to be removed and the geometry of the excavation has a significant impact on the type size and location of this structure. Memorandum A-3 in Appendix A provides additional discussion on this structure and assumed foundation configuration.
Other structures throughout the project corridor will require similar assessment of foundation conditions; however, conditions appear to generally be favorable for spread foundations in most areas. Structures located close to wetlands and on steep sideslopes where organic soils and where slope wash has occurred are areas where additional investigation and evaluation may be necessary to confirm foundation conditions.

**Areas of Additional Site Investigation and Analyses**

As stated previously, we in general, believe that the subsurface conditions along most of the corridor are favorable for construction of the proposed roadway. The south end of the project corridor is expected to require additional investigation and analysis to further define and allow consideration of the most practical options for roadway layout, configuration, and selection of design components such as types of retaining walls to be used. Without further investigation it is difficult to suggest in detail what may be the best options for the design; however, we believe that these options will become clear after conducting further investigation and analyses. Although some conditions may affect the project costs to some degree, none of these issues appear to affect the feasibility of the project.

**Issues at the South End of Corridor.** The most difficult area geotechnically will likely be near the south end of the project. Several elements of the layout and design contribute to issues that must be dealt with near this location. These include satisfying global stability of the roadway section with 20 foot or higher retaining walls near the south end of the project where soft compressible soils are suspected to exist near wetland 1A. The underlying soils under the west side of the proposed road corridor in this area are likely to be wet, soft, and compressible and will likely possess low strength. Further analyses will be required to determine foundation conditions and options along this portion of the alignment. Options may include overexcavation to dense competent foundation soils and replacement with densely compacted, well graded granular soils placed as foundation and backfill behind (MSE) walls. Other options include cantilever soldier piles and lagging, tied-back soldier piles and lagging, and others.

Global stability could also be an issue along the east side of this corridor at this location where steep transverse slopes will require deep cuts to achieve the width and grade for the road. Because of the steep slopes, the width of the roadway will lead to greater physical impact to terrain in this area than any other because of the steep existing slopes. This could lead to increased need for retaining walls as well as increased retaining wall height. Further alignment adjustments (vertically and horizontally) will help to optimize the solutions in this area of the alignment once the foundation conditions are carefully characterized in this area.

The use of a massive retaining wall immediately south of the south end of the proposed corridor suggests that a careful examination is needed of the adjacent area as well as the south end of the proposed corridor to assure stability of the hillside in this area. The construction of this wall along the east side of 160th Avenue NE, immediately south of the start of this proposed project appears like it could have been because of a concern for stability of the hillside and the need to provide lateral restraint from unstable soils that may exist upslope from the wall. Obtaining further information about the design of this wall would be quite helpful in understanding the concerns and conditions in this area. Further investigation and analyses will be required near the south end of the proposed alignment to determine depth to competent soils capable of supporting walls and/or cut slopes for the proposed roadway.

**Issues and/or Unknown Conditions in Other Areas of the Project that Require Additional Investigation and Analysis.** Seepage is presently occurring in certain areas of the corridor. These seeps appear to be concentrated near the contact between the top of the transitional beds (dense fine grained, low permeability soils) and outwash sand and gravel located in contact with the top of this formation. Prior investigations suggested that this contact may be at about elevation 80. The outwash is well graded and typically contains silt fines but is much more pervious than the underlying silt. These zones of seepage should be carefully identified and mapped throughout the project area. The seepage from these areas could lead to construction difficulties because of the characteristics of the moisture sensitive soils that surround these areas. Seepage if not controlled, also leads to
increased erosion and increased incidence of required overexcavation of foundations during construction. Excessive seepage in sensitive areas can often lead to lower factors of safety of surrounding slopes.

Seepage from natural groundwater can also rob the project of increased long term potential to infiltrate surface water resulting from runoff from the new roadway unless carefully collected and diverted away from these sensitive areas. For all of these reasons, it is important to further identify the zones of seepage and its seasonal variation where these conditions could affect the design and construction of new facilities. Plans should identify methods of collection and diverting the seepage to reduce the impacts to all phases of the work.

Construction in each of the ravines is expected to encounter increased thickness of soft organic sediments. It appears that ravines associated with or located near with Wetlands 1 and 2 may result in minimal depth to competent foundations and associated overexcavation of these undesirable soils. However the ravine associated with Wetland 3 is more extensively underlain with these undesirable soils. It has been our assumption however, that the required overexcavation is limited to reasonable depths where dense competent foundations will be encountered. Additional exploration will be required to further identify these conditions especially at the location of proposed footings for new arch culvert structure and trail undercrossing. It is assumed that overexcavation and placement of densely compacted granular fill will provide suitable foundations for these structures. Seepage in the vicinity of these structure foundation is likely to be an issue and will need to be controlled. Additional explorations should be used to help identify the water table and seepage conditions in these areas.

Care fill be required in selecting the alignment in the vicinity of the existing 54” Tolt Eastside Water Supply Pipeline. Coordination will be required with Seattle Public Utilities to assure that their future requirements for maintenance and operations are met.

The depth of soft compressible soils and required overexcavation for structures at the stream associated with wetland 4 is unknown. Additional investigation is required to further define and confirm conditions in this area; however, it is likely that the depth is shallow and is probably not of significant geotechnical concern.

Characteristics of the native soils including gradation and permeability should be carefully characterized throughout the length of the project, particularly at the north end. This should help in assessing areas of seepage, groundwater levels and variation, and opportunities for infiltration of stormwater. Better definition of the locations of the contact between the top of the transitional beds and the overlying granular outwash soils at selected areas of the corridor will be helpful in further understanding site conditions that will influence the design and construction.

Developing adequate capacity to handle and treat stormwater runoff is an important element of the design in order to reduce impact to wetlands and reduce the cost of the project. Additional investigation should target areas where the opportunity to infiltrate water for disposal can be effective. Making efficient use of suitable soil areas for stormwater infiltration will greatly reduce the total impact to wetlands, will help reduce otherwise expensive infrastructure needed for controlling stormwater, is more environmentally friendly, and should help reduce the overall project costs. Likewise, areas where stormwater infiltration should not be considered should be identified from stability analyses and monitoring of existing groundwater conditions. Since application of additional stormwater to these areas could reduce the stability of slopes, these critical areas should be identified and application of additional stormwater to these areas should be avoided.
NRCS Soils Map and Legends

Appendix A
MAP LEGEND

Area of Interest (AOI)

Soils

Special Point Features
- Blowout
- Borrow Pit
- Clay Spot
- Closed Depression
- Gravel Pit
- Gravelly Spot
- Landfill
- Lava Flow
- Marsh or swamp
- Mine or Quarry
- Miscellaneous Water
- Perennial Water
- Rock Outcrop
- Saline Spot
- Sandy Spot
- Severely Eroded Spot
- Sinkhole
- Slide or Slip
- Sodic Spot
- Spoil Area
- Stony Spot

Very Stony Spot
Wet Spot
Other

Special Line Features
- Gully
- Short Steep Slope
- Other

Political Features
- Cities

Water Features
- Streams and Canals

Transportation
- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

MAP INFORMATION

Map Scale: 1:10,700 if printed on A size (8.5" x 11") sheet.
The soil surveys that comprise your AOI were mapped at 1:24,000.

Warning: Soil Map may not be valid at this scale.
Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for accurate map measurements.

Source of Map: Natural Resources Conservation Service
Coordinate System: UTM Zone 10N NAD83

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: King County Area, Washington
Survey Area Data: Version 7, Jul 2, 2012
Date(s) aerial images were photographed: 7/24/2006

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.
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Existing Reports and Laboratory Analysis

Appendix B
Report
Preliminary Geotechnical Engineering Services
160th Avenue Northeast Extension Route Location Study
Redmond, Washington

May 23, 1995

For
City of Redmond
May 23, 1995

Parametrix, Inc.
5808 Lake Washington Boulevard Northeast
Suite 200
Kirkland, Washington 98033-7350

Attention: Mr. Rex Knight

We are pleased to submit three copies of our "Report of Preliminary Geotechnical Engineering Services, 160th Avenue Northeast Extension Route Location Study, Redmond, Washington, for the City of Redmond". The scope of services for our study has been developed in discussions with you and is described in the scope of services included in your agreement with the city dated April 17, 1995. Preliminary results of our study have been discussed with you as our findings were developed.

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

Yours very truly,

GeoEngineers, Inc.

James B. Thompson, P.E.
Principal

HRP:JBT:vvl
Document ID: 0500079.R
File No. 0500-079-R05
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APPENDIX A FIGURES

Soil Classification System
Logs of Test Pits
Logs of Hand Auger Holes
Moisture Content Data

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Appendix B - Selected Hand Auger Holes and Test Pits
REPORT
PRELIMINARY GEOTECHNICAL ENGINEERING SERVICES
160TH AVENUE NORTHEAST EXTENSION ROUTE LOCATION STUDY
REDMOND, WASHINGTON
FOR THE
CITY OF REDMOND

INTRODUCTION

This report presents the results of our preliminary geotechnical engineering services for the proposed 160th Avenue Northeast extension route location study. A vicinity map showing the general location of the project is shown in Figure 1. The roadway extension will be constructed between the present terminus of 160th Avenue Northeast north of Northeast 90th Street and Woodinville-Redmond Road Northeast south of the intersection with Northeast 109th Street. A corridor including three roadway alignments (A-, B- and C-Lines) is currently being considered; these alignments are shown in Figures 2 and 3.

The new roadway will be nearly one mile long and will cross three distinct physiographic areas: (1) the floor of the Sammamish River valley, (2) the east valley wall, and (3) a portion of an upland area known locally as Education Hill. The three alignments are nearly identical from the southern terminus of the project (approximate B-Line Station 8+00) up to about Station 20+00 (all lines). From this point northward, the A- and C-Lines gradually climb up the east valley wall, crossing a utility corridor between approximate Stations 32+00 to 35+50, and reach the edge of the upland at about Station 41+00. The B-Line remains on the edge of the valley floor until about Station 29+50, then climbs up the valley wall and reaches the edge of the upland, again at about Station 41+00. The three alignments converge north of Station 41+00 and extend to the northern terminus of the project at about Station 54+00. We understand from our discussions with you that the A- and C-Lines are preferred over the B-Line, with the B-Line having a low probability of being selected as the final alignment.

Construction of any of the alternative alignments will require extensive earthwork in the valley, valley wall and upland areas to satisfy roadway grade and width requirements. In addition, the roadway will cross over the existing 54-inch Tolt River water transmission pipeline and an existing recreational trail, both within the utility corridor. Smaller bridges or culverts will likely be needed to cross several small streams in the valley wall segment.

In the valley floor segment of the alignments, fill embankments will range in height from about 5 feet above existing site grade near the southern terminus to possibly more than 20 feet where the alignments begin to climb the valley wall. Roadway construction in the valley wall segment might require cuts of over 10 feet in depth and fills of 12 feet or more in height where the roadway must cross small ravines, one of which will include the future trail crossing. At
various points along the valley wall segment, sidehill construction will be necessary where the uphill side of the roadway is in cut and the downhill side in fill. Cuts and fills in the upland area will probably not exceed a few feet in height.

SCOPE

The purpose of our services is to characterize the soil and ground water conditions along the project corridor as a basis for developing preliminary geotechnical recommendations for earthwork, retaining structures and other geotechnical issues. The specific scope of our services includes the following tasks:

1. Review available plans, topographic information, geotechnical reports and published geologic information for the project corridor.
2. Conduct a site reconnaissance to observe site features including topographic relief, seepage zones along the valley wall, and features that might indicate slope instability.
3. Excavate seven test pits with a track-mounted backhoe and accomplish three hand auger borings, along with limited hand probing at various points along the corridor.
4. Accomplish a limited laboratory testing program on the samples obtained from the explorations.
5. Provide preliminary earthwork recommendations including removal of unsuitable soils, cut and fill slopes, fill type and compaction criteria, and effects of weather on construction activities.
6. Complete a preliminary evaluation of the stability of slopes within the corridor.
7. Identify options for retaining wall systems, as appropriate.
8. Provide preliminary geotechnical recommendations for the trail crossing and the crossing of the Toll River pipeline.
9. Evaluate surface and ground water conditions, and provide preliminary drainage recommendations.
10. Attend up to two meetings with the project team, as appropriate.
11. Prepare a written report containing our preliminary conclusions and recommendations along with the supporting field and laboratory data.

SITE DESCRIPTION

SURFACE CONDITIONS

The project corridor extends along the east edge of the Sammamish River valley floor northwestward to and up the valley wall and across a portion of a broad upland known locally as Education Hill. Physiographic and geologic conditions are markedly different for the valley floor, valley wall and upland areas, as described below.

The Sammamish River valley is a broad, north-south trending valley that is generally at or slightly below Elevation 30 feet (mean sea level datum) in the corridor area. Topography on the valley floor is essentially flat, although near the east edge small alluvial fans built up by streams
discharging from the valley wall exist. One of these fans is located in the vicinity of Station 20+00 (all lines) and has relief of about 6 feet. An older topographic quadrangle map of the Redmond area shows that two structures formerly occupied this fan area. Near the southern terminus of the corridor, on-going grading activities associated with a condominium development are modifying the terrain. A shallow depression identified as a wetland exists near the edge of the valley wall at about Station 29+00 (B-Line).

The valley wall is characterized by a series of steep-sided ridges and small ravines oriented generally in an east-west direction. Some of these ravines contain small streams. The approximate locations where the corridor crosses these ravines or their outlets near the bottom of the valley wall are Stations 24+00 (C-Line), 26+50 (C-Line), 29+00 (A- and C-Lines), 31+00 (all lines), 33+50 (all lines), and 37+50 (all lines). Slopes within the valley wall and associated ravines and ridges generally range between 20 and 70 percent.

The ravine bottoms are typically mantled with a layer of slopewash consisting of soft and wet soils. Some seepage was observed emanating from the ravine walls, primarily in the lower portions. Stream channels have relatively gentle gradients near the heads of the ravines but then steepen below about Elevation 80. Several small slump blocks with volumes of generally a few cubic yards were observed in a few of the ravines near their outlets. The largest such slump block we observed has a surface area of about 15 feet by 25 feet and occurs near the mouth of the ravine at Station 31+00 (B-Line).

The utility corridor includes one of the ravines (near Station 33+50, all lines) and is oriented in a northeast-southwest direction. The corridor includes steel tower and wood pole power lines, and the 54-inch diameter Tolt River pipeline. The steel tower and pipeline are located near the north edge of the utility corridor, while the wood pole line is located near the south edge. A gravel-surfaced recreational trail also exists within the northern portion of the corridor. Numerous fences exist adjacent to the corridor. We understand that the pipeline has been constructed with a deliberate downward bend where it crosses the C-Line at about Station 35+50.

The upland portion of the proposed roadway corridor is characterized by gently to moderately sloping terrain with the ground surface generally sloping less than 15 percent. Elevations in the upland portion range from about 80 to 180 feet. The terrain in the upland area apparently has been modified locally by grading related to agricultural use.

Vegetation along the corridor is quite variable and ranges from tall grass and wet area type vegetation in the valley bottom, mixed conifer and deciduous forest with dense underbrush along the valley wall, and tall grass and scattered small trees in the upland portion.

SITE GEOLOGY

Geologic conditions in the corridor area are primarily the result of several regional glaciations, the most recent being the Vashon ice advance of the Fraser glaciation. The nonglacial interval prior to the Vashon glacial advance was characterized by a climate similar to
present conditions. Erosion of previous glacial deposits and deposition occurred during such nonglacial intervals. Erosion and deposition during and following the Vashon glaciation have resulted in the modern topography of the Sammamish River valley and adjacent Education Hill.

In the corridor area, the most significant pre-Vashon deposits are deposits mapped as transitional beds of silt, sand and clay deposited in lakes and streams ahead of the advancing glacial ice. These transitional deposits were eventually buried under advance outwash sand and gravel deposited by streams and rivers in front of the advancing Vashon glacier. The advance outwash is mapped as underlying much of the upland area west of Woodinville-Redmond Road. As the ice covered the area, it deposited Vashon glacial till over the older glacial and nonglacial deposits. Vashon till was not mapped or observed along the corridor, although it is mapped east of Woodinville-Redmond Road.

Localized deposits of recessional drift consisting of sand, gravel and silt exist within the valley wall portion of the corridor. These deposits include material laid down by meltwater in direct contact with the ice or in lowlying areas freshly exposed by ice melting. Along the valley wall portion of the corridor, it appears that the contact between the transitional beds and the overlying advance outwash or recessional deposits occurs at about Elevation 80.

During and following ice melting, the upland and valley wall topography was extensively modified by erosion. Glacial scouring formed the Sammamish River valley. Runoff from upgradient areas has formed stream channels and ravines which are being incised into the valley wall. The Sammamish River valley has also been filled with alluvial deposits consisting of unconsolidated peat, silt, sand and gravel. Other post-glacial deposits include slopewash in moderately to steeply sloping areas such as the valley walls and related stream channels and ravines, and a minor amount of fill in the valley bottom and upland areas resulting from past agricultural and construction activities.

**SUBSURFACE CONDITIONS**

We evaluated subsurface conditions along the roadway corridor by reviewing logs of previous explorations together with excavating 7 test pits and 3 hand auger borings. Our recent explorations and pertinent previous exploration locations are shown in Figures 2 and 3. A description of the field and laboratory testing programs along with the logs of our current explorations are presented in Appendix A. Pertinent logs of explorations from the previous studies are included in Appendix B.

The previous studies we reviewed include the following:

Subsurface conditions for various segments of the corridor are described in the following sections.

Station 8 + 00 to Station 22 + 00 (A- and C-Lines) and Station 29 + 00 (B-Line)

Previous and current explorations accomplished within this segment of the corridor include our test pits TP-95-1 through TP-95-3, and numerous test pits for the condominium development (Townhomes on River Trail) by others. General subsurface conditions encountered by these explorations include surficial layers of topsoil, silt and peat overlying loose to dense granular soils and hard silt. The thickness and extent of the surficial soils varies along the corridor.

The topsoil layer generally ranges from 0.5 to 1.5 feet in thickness. Soft to medium stiff silt (including a layer of diatomaceous earth) and peat generally underlie the topsoil. The peat ranges in thickness from 1 to 7 feet along this segment of the corridor. Test pit TP-95-2 encountered a one-foot thick layer of slopewash and a thin layer of silt fill over a concrete slab. The slab was encountered at a depth of 2.5 to 3.0 feet, and is probably related to the former structures that occupied this location.

The soft surficial soils generally extend to depths of 4 to 12 feet along the corridor and are underlain by loose to dense sand and silty sand with varying amounts of gravel and hard silt. The hard silt unit appears to be part of the transitional beds and was encountered only in the explorations located close to the valley wall.

Ground water seepage was observed at depths ranging from 3.5 to 12 feet below the existing ground surface at the time of the various explorations. Ground water levels along the edge of the valley wall are expected to vary in response to seasonal levels of the Sammamish River, and to the amount of surface and subsurface water originating from the adjacent valley wall.

Station 22 + 00 (A- and C-Lines) and Station 29 + 00 (B-Line) to Station 41 + 00 (All lines)

Subsurface conditions along this segment of the corridor were explored by excavating test pits TP-95-4 through TP-95-7, and hand auger holes HA-95-1 through HA-95-3. In addition, we reviewed logs of previous explorations by GeoEngineers and others near the northern end of this corridor segment and observed exposures of soils in several of the ravines.

In general the explorations and exposures indicate that the valley wall portion of the corridor is mantled by a layer of topsoil or forest duff about 0.5 feet thick. The topsoil is underlain by a unit of either recessional or advance glacial outwash. The outwash consists of
medium dense sand and silty sand with varying amounts of gravel and cobbles. The surficial soils adjacent to stream channels and in the bottom of ravines consist primarily of loose silty sand and soft sandy silt. The loose and soft soils are generally 2 to 4 feet thick, based on limited hand probes. A thin layer of slopewash mantles some of the steeper valley wall slopes and ravine side slopes.

The predominant soil type underlying the outwash consists of hard silt with varying amounts of silt and gravel (transitional beds). The silt was encountered at depths ranging from 3.3 to 10 feet in several of the explorations. We also observed the contact between the silt and the overlying outwash in several of the ravines adjacent to and within the utility corridor.

Ground water seepage was observed in test pits TP-95-5 and TP-95-6 at depths ranging from 7 to 10.5 feet below the ground surface. These depths generally correspond to the top of the transitional beds. Ground water seepage was not observed in test pit TP-95-7 or in any of the current hand auger holes because these explorations were located near the ends of the ridges which are flanked by ravines. However, we did observe wet ground at the bottom of these ravines and in some cases seepage near the heads of the ravines. The general elevation at which seepage is occurring appears to correspond to the previously mentioned stratigraphic contact.

Station 41 + 00 to Station 54 + 00

The previous explorations for the Redmond 74 project indicate that the upland portion of the corridor is mantled by about 0.5 to 1.5 feet of topsoil which is underlain by medium dense to dense sand and silty sand and stiff to hard silt with varying amounts of sand and gravel. These soils extend to the maximum depth explored, 11.5 feet. Ground water seepage was encountered in a few of the previous test pits at depths of about 2 to 3 feet.

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

GENERAL

We evaluated geotechnical conditions in light of the proposed roadway construction. Based on our review of available information, reconnaissance and explorations, we conclude that geotechnical conditions within the corridor are generally favorable for roadway construction. Route conditions are such that Lines A and C are somewhat preferable over Line B from a slope stability standpoint. Also, Lines A and C largely avoid an identified wetland in the valley bottom near Station 29 + 00 (B-Line).

Conventional practices can be used in the construction of fill embankments, cut slopes, stream and trail crossing structure foundations, retaining walls, drainage and erosion control facilities and roadway pavement. Because of the rolling and locally steep terrain and erosion sensitivity of the soils along much of the corridor, precautionary measures will be required for erosion and surface water control during and following construction.
Removal of unsuitable soils should be anticipated along the valley bottom segment of the final route. The surficial soils along the corridor are sensitive to moisture. We recommend accomplishing the earthwork and foundation construction activities during periods of extended dry weather conditions.

General conclusions and preliminary recommendations for geotechnical aspects of roadway construction are presented in the following sections. Detailed design criteria can be developed during a design phase study which might include additional explorations at key locations along the final alignment.

SITE PREPARATION

The site preparation recommendations presented in this report are based on our understanding that all unsuitable soil (peat, diatomaceous earth, slopewash and other loose or soft soils) will be removed from the right-of-way. This has been standard city of Redmond practice for road projects in the vicinity of the site.

The majority of the soils along the alignments contain a relatively high fines (silt and clay) content. These soils are highly moisture-sensitive and will be difficult to properly compact when wet or when placed in wet areas. In general, these soils will also be erosion-sensitive during and following construction. Earthwork on the site will be most efficient during the normally dry period from May to October when some of the on-site soils may be usable as structural fill and when equipment traffiability will be easiest.

We recommend that removal of unsuitable soils be accomplished down to the surface of the firm native soils encountered in our recent explorations and the previous explorations accomplished by others. The depth to removal will vary along the alignments, and is expected to range from a few feet in upland and valley wall stream and ravine crossings, to as much as 12 feet in the valley bottom, depending on the thickness of any surficial soils (including fill) over the unsuitable soils. We recommend that temporary cut slopes for removal of soft soils be inclined no steeper than 1:1 (horizontal to vertical). Swales or drainage ditches should be installed around the perimeter of excavations as necessary to collect and control surface runoff.

We recommend that roadway embankment and crossing structure areas in the upland and valley wall segments of the alignments be cleared of vegetation and stripped of forest duff. Forest duff acts as a protective layer against disturbance of underlying soils during wet weather and should be removed only if necessary. Topsoil should also be stripped from areas where existing grades are within 3 feet of final grade in fill areas. Topsoil should also be stripped from fill areas where existing slopes are steeper than 5:1, regardless of fill height. It is acceptable to leave topsoil in place where fill heights exceed 3 feet and where embankment subgrade slopes are 5:1 or flatter.

We expect that the depth of stripping will range between 6 to 12 inches during dry weather conditions unless excessive disturbance is caused by clearing operations, in which case a greater depth of stripping may be required. Stripping to a greater depth can also be expected if clearing is done during wet weather.
Trees and stumps which are removed should be overturned so that most of the roots are also removed. Depressions from tree stump removal should be backfilled with compacted clean granular fill.

We recommend that exposed subgrades in roadway segments outside areas where soft soils are excavated be proofrolled with heavy compaction equipment. Proofrolling should be accomplished in dry weather only. In wet weather conditions, the exposed subgrade should be evaluated by probing. The purpose of the proofrolling and probing is to detect areas which are not, or cannot be, adequately compacted in their existing condition. Loose or soft areas detected during proofrolling should be excavated and replaced with compacted structural fill. If excavation deeper than 2 feet appears warranted, other measures can be developed to provide a stable subgrade.

Complete removal of soft soils should also be accomplished where embankment fill will be placed across ravines and stream channels, particularly for the recreational trail crossing. If difficulties are encountered in preparing a suitably stable subgrade for new embankment fill because of wet or soft soil conditions, a geotextile could be used between the natural subgrade and roadway embankment fill.

**EROSION CONTROL**

Cut and fill slopes will be subject to erosion and ravelling during construction, particularly if earthwork is accomplished during wet weather conditions. Runoff from the construction areas will increase as a result of the required removal of the vegetative cover.

Erosion control methods that should be used during construction include efficient channeling of surface water runoff, minimizing the extent of disturbed areas, erosion-preventing slope cover such as straw, channel liners and energy dissipators for trenches, and diversion ditches or levees. Other measures that should be considered include liberal use of straw bales or geotextiles, and temporary sedimentation basins. The impacts of erosion will be mitigated by accomplishing earthwork during the normally dry seasons of the year.

The erosion control measures must be carefully designed and installed for specific locations and purposes. Some measures must be in place before any general site preparation work such as clearing and grading is started. Other measures may be installed as the work progresses. These measures can best be developed in conjunction with the project grading plans.

Long-term erosion should be mitigated by accomplishing grading to limit the concentration of runoff onto fill, cut or natural slopes and other erosion-sensitive areas, installing permanent sedimentation basins and a storm drain system, and by establishing an adequate grass or vegetative cover concurrent with, and promptly after, earthwork completion. Disturbance to natural drainage courses adjacent to the alignment must be kept to a minimum so that existing erosion, sedimentation, channel stability, and flow conditions will not be appreciably changed. Undisturbed vegetative buffers should be provided along natural drainage courses to help reduce sedimentation.
Maintenance of all erosion control facilities is a key element of their successful performance. It is imperative that all erosion control measures be maintained and repaired as frequently as is needed to preserve their function.

EARTHWORK

Roadway construction will require substantial fills in the valley bottom and large cuts and fills in the valley wall. In addition, a minor amount of cutting and filling will be required in the upland segment of the corridor. Sidehill construction consisting of cuts on the uphill side and fills on the downhill side will be necessary along much of the valley wall segment. Cuts to 10 feet or more in depth and fills to 20 feet or more in height will be required for the new roadway.

Earthwork should be accomplished during the normally dry period from May to October when some of the on-site soils might be able to be used as compacted fill, and when erosion and sedimentation activity will be at a seasonal low. Erosion protection must be planned, installed and maintained properly for maximum effectiveness, as described above.

Most of the on-site soils can be excavated with conventional light to heavy power equipment. Excavation in hard glacially consolidated silt might require the use of rippers on dozers or backhoes.

New structural fill for roadway construction should be free of debris, organic contaminants, and rock fragments larger than 6 inches. The suitability of material for use as structural fill will depend on the gradation and moisture content of the soil. As the amount of fines (material passing the No. 200 sieve) increases, soil becomes increasingly more sensitive to small changes in moisture content and adequate compaction becomes more difficult to achieve. We recommend that fill soil contain no more than 5 percent fines (relative to the fraction passing the 3/4-inch sieve) for placement in wet weather. The percent fines can be higher for placement in dry weather, providing that the fill soil is moisture-conditioned as necessary for proper compaction.

Cuts along the alignments will remove a substantial volume of recessional and advance outwash and underlying fine-grained glacial soils. These materials contain a substantial percentage of silt, and should be used as fill only if placed on dry surfaces during periods of prolonged dry weather, and only if these soils can be properly moisture-conditioned for compaction. The soft and loose soils removed from the valley bottom and the bottom of ravines should not be reused as fill and should be wasted off site or used for landscaping.

Structural fill should be placed in lifts not exceeding 10 inches in loose thickness and mechanically compacted to a firm, nonyielding condition. The fill must be compacted to at least 95 percent of the maximum dry density determined in accordance with ASTM D-1557 to a depth of 2 feet below the finished roadway subgrade, and to at least 90 percent below this level. The moisture content of the fill soil must be adjusted as necessary for proper compaction. Fill surfaces should be crowned at all times to prevent the ponding of surface water.
If difficulties are encountered with surface and ground water in particular work areas and this water cannot be diverted from the work area, we recommend that geotextile and a layer of 4-inch minus quarry spalls be used. The initial lift of embankment fill may be as thick as 18 to 24 inches in these areas and in other areas where soft subgrade soils may be encountered.

Fill placed on existing slopes which are steeper than 5:1 (horizontal to vertical) should be properly keyed into the existing slope. This can be accomplished by cutting a series of 5- to 8-foot-wide horizontal benches into the existing slope as the fill is being placed.

Seepage zones will likely be encountered in cuts made within the valley wall segment of the alignments. It is important that steps be taken to maintain drainage from the slopes, both during and following construction. In our opinion, this can best be done by constructing a drainage blanket at the base of fill embankments and by installing a collection system for water which seeps from cuts or which enters the drainage blanket. Handling of seepage from cut slopes is discussed in a subsequent section of this report.

The drainage blanket should be constructed with well-graded sand and gravel containing less than 3 percent fines based on the minus 3/4-inch fraction. We recommend that the blanket have a minimum effective thickness of 18 inches. Installation of the blanket might be complicated by the need to key new fill into the existing slope. The volume of select material required to construct the drainage blanket will have to be increased accordingly.

We recommend that a member of our staff observe the placement and compaction of embankment fill. An adequate number of in-place density tests should be accomplished in the fill as it is being placed to determine if the required degree of compaction is being achieved.

Provided that all unsuitable soils are removed from roadway fill embankments, we expect that settlements due to compression of the native subgrade soils will be about 1/2 to 1 inch and should occur rapidly as fill is placed. Some additional settlement will occur within the fill itself, particularly where the fill thickness is greater than 5 feet. We estimate that the maximum amount of settlement within the fill will be no more than 1 percent of the fill thickness. Thus, for a 10-foot fill embankment, settlement on the order of 1 to 1-1/2 inches will occur. We recommend placing the final roadway pavement at least 2 to 4 weeks after placement of the fill to full height where the fill thickness is greater than 10 feet to allow for the major portion of these settlements to take place.

We recommend that the entrance and exit points of culverts passing streams below the roadway be protected from erosion by placing a blanket of 2- to 6-inch quarry spalls around all sides of the culvert. We recommend that the quarry spall blanket be at least 12 inches thick.

**CUT/FILL SLOPES AND SLOPE STABILITY**

Temporary cuts over 4 feet in height should be sloped no steeper than 1:1, unless appropriate shoring is provided. If persistent seepage is encountered in cut slopes, it might be necessary to flatten the slopes locally or to provide finger drains to reduce the potential for instability. In wet weather, cut slopes should be covered with properly secured plastic or some other impermeable membrane to keep water off and to help reduce erosion or sloughing.
In general, unretained permanent cut slopes should be inclined at 2:1 or flatter for excavations in recessional or advance outwash and hard silt to provide an acceptable factor of safety against sliding and minimize slope maintenance efforts. In areas where seepage occurs, cut slopes may need to be flattened to mitigate potential instability.

Embarkment and other structural fill slopes should be inclined no steeper than 2:1. Fill embankments must be properly keyed into existing sloped areas, as previously recommended, so that the potential for instability is mitigated.

Newly constructed cut and fill slopes will be susceptible to erosion until they are protected. Runoff is likely to increase from these slopes. Therefore, we recommend that all slope surfaces be planted as soon as practical to mitigate the potential for erosion. The use of hydroseeding might be considered for this purpose. A temporary covering should be placed on the slope face where needed until the vegetation can take affect. Silt fences should be installed and maintained at strategic locations until the vegetation is re-established.

No signs of deep-seated slope instability were observed along the corridor. However, small scale slumps were observed near the outlets of several ravines in the valley wall. Removal of these slumps combined with general removal of any soft and wet soils in the ravine bottoms should be accomplished to minimize the potential for instability of fill embankments placed across the ravines.

RETAINING WALLS

Retaining walls will be used along cut and fill segments of the roadway, particularly within the valley wall segment. The walls could exceed 15 feet in height. Several options available for retaining these cuts and fills include the following:

1. Cantilever soldier pile and lagging retaining wall.
2. Reinforced concrete retaining wall.
3. Mechanically stabilized earth retaining wall.
4. Gabion wall.
5. Rockery wall.

A cantilever soldier pile and lagging wall has the advantage of limiting disturbance to adjacent property and limiting the extent of excavation. The piles would extend into dense glacial soils to derive sufficient lateral resistance. The lagging could consist of precast concrete panels or treated timber faced with cast-in-place concrete.

A conventional reinforced concrete cantilevered wall system could be used for both cut and fill portions of the roadway, but would require more excavation than for the soldier pile wall. The wall footing would need to be founded on competent native or fill soil. Backfill behind the wall would need to be free-draining in order to prevent the buildup of excessive lateral pressures.
A wide variety of mechanically stabilized earth walls, including the Reinforced Earth system and geotextile walls with facing, could be used primarily for retaining fill embankments, particularly at the edge of the wetland near Station 29+00 (B-Line). These systems use a series of facing elements such as precast concrete panels or blocks which are attached to metal or geotextile reinforcing elements that extend horizontally into the backfill. Backfill is placed in layers over the horizontal elements, forming a reinforced soil mass. The wall panels are available with a variety of architectural finishes. Some excavation into native soils might be required in order to provide the minimum embedment length for the horizontal reinforcing elements.

Gabion walls could be used for either cut or fill sections. This type of gravity wall features galvanized wire baskets which are filled with small quarry rock and backfilled with free-draining soil. Adequate foundation support and drainage are key to the successful performance of gabion walls.

Rockery walls could be used to protect roadway cut slopes from erosion and sloughing. These walls consist of large pieces of rock stacked in a configuration that attempts to maximize contact between the pieces. However, it should be recognized that rockeries are not structural retaining walls and that rockery performance is highly dependent on slope geology, the quality of rock used and the skill exercised by the rockery constructor. We recommend that rockery heights be limited to 6 to 10 feet depending on the back slope. If it is necessary to construct a higher rockery, it should be done in multiple tiers.

Common to each wall type is the need for adequate drainage behind the wall. Typically, this would consist of a zone of free-draining backfill behind the wall and a perforated collector pipe at or below the base of the wall. Lateral loads, including surcharge loads, and passive pressure values for the selected wall type will be determined during final design.

**TRAIL CROSSING STRUCTURE SUPPORT AND PIPELINE CROSSING**

We envision that the trail crossing will consist either of a large metal culvert bolted to precast concrete foundation slabs or a precast concrete structure. Shallow foundation support for the crossing structure will be feasible, particularly if soft soils can be completely removed and replaced with structural fill below spread footings.

Spread footings for support of the crossing structure should be founded on dense native soils or on a zone of structural fill which replaces the upper unsuitable soils and extends down to the native soils. Ground water might be a concern in the construction of spread footings. If ground water is present in the base of the excavation for this crossing, we recommend that a geotextile filter fabric be placed and covered with at least 2 feet of 2- to 6-inch quarry rock or clean pit run sand and gravel containing no more than 3 percent fines. It is important that this drainage zone be placed to an elevation lower than the floor of the crossing structure.

We recommend that continuous and isolated square footings have minimum widths of 16 and 24 inches, respectively. Footings should be embedded at least 18 inches below the lowest adjacent finished grade. For preliminary design, an allowable soil bearing pressure of 2,500 pounds per square foot may be used for footings designed and constructed as
recommended. This allowable soil bearing value applies to the sum of all dead and long-term live loads, exclusive of the weight of the footing and any backfill above the footing. A one-third increase in the recommended value may be used for wind or seismic loads.

We recommend that the condition of all footing excavations be observed by a representative from our firm prior to placement of concrete to conform that the bearing soils are undisturbed and consistent with the recommendations in this report.

We estimate the total settlements of footings prepared in accordance with our recommendations will not exceed 1/2 inch. Differential settlement should not be significant provided support conditions for adjacent footings are similar.

The crossing structure will also need to be designed to resist lateral soil pressures. We can provide specific lateral soil pressure and base friction values as well as backfill criteria and compaction once the structure type is selected during final design.

We recommend that the final roadway alignment be located such that it crosses over the previously-constructed downward bend in the Tolt River pipeline, if possible. If this is not possible, we recommend that the finished grade for the roadway be such as to provide adequate cover over the top of the pipeline to minimize the influence of wheel loads on the pipeline.

**DRAINAGE**

In the valley wall segment, roadway cuts and fills will likely encounter seepage zones near the contact between the hard silt (transitional beds) and the overlying outwash soils. Any such seepage zones should be handled by installing slot drains into cut slopes and collector drains at the base of slopes. In addition, drainage blankets should be placed at the base of embankments, as discussed previously. Appropriately-sized culverts should be provided under the roadway where it crosses existing stream channels and ravines.

Temporary seepage control measures could include bench cuts, finger or slot drains, ditching and pumping. Determination of the extent of permanent and temporary collection systems will be made during final design.

Provisions for roadway drainage should include interceptor drains on uphill (cut) edges of roadway segments, pavement edge drains, lined drainage ditches and/or a free-draining subbase layer under all new pavement areas. The interceptor drain could consist of a perforated pipe embedded in pea gravel or washed rock. The drain should be covered with an asphalt-lined ditch to remove water draining from the pavement surface and from seepage zones above the roadway. The pavement surface should be sloped to drain away from the center of the roadway.

**ROADWAY PAVEMENT**

We anticipate that the subgrade for the new roadway will consist of either new fill placed and compacted as recommended under "Site Preparation and Earthwork" or on competent native soils. A CBR value of 15 is appropriate for determining the thickness of the various components of the new pavement section.
We recommend that the roadway subgrade, including embankment fill, be proofrolled using a loaded dump truck prior to placing any granular subbase, ATB or crushed rock base course soil. Soft, loose or otherwise unsuitable zones identified during proofrolling should be repaired. We recommend that a member of our staff observe the proofrolling of roadway subgrade areas and the repair of unsuitable zones.

The standard city of Redmond design pavement section for principal arterial streets consists of 4 inches of Class B asphalt concrete over 5 inches of ATB (asphalt-treated base) over 3 inches of crushed rock base course. On a preliminary basis, it is our opinion that this standard section will likely be satisfactory for the expected traffic conditions. We recommend that a subbase layer of at least 6 inches of clean, free-draining sand and gravel containing no more than 3 percent fines relative to the fraction passing the 3/4-inch sieve be included in the pavement section for the purpose of providing adequate drainage, as discussed in a previous section of this report. This subbase layer should be placed on the subgrade prepared as recommended above.

All new pavement should be Class B asphalt concrete conforming to WSDOT 1994 Standard Specifications. ATB should meet WSDOT Specification 4-06. All crushed surfacing should be top course conforming to WSDOT 1994 Standard Specification 9-03.9(3). The crushed rock top course and sand and gravel subbase should both be compacted to at least 95 percent of the maximum dry density as determined by the ASTM D-1557 test procedure.

The layer of ATB will likely be used as a temporary wearing surface during construction. This ATB could potentially be subjected to heavy construction and general traffic for several months, depending on the construction schedule and weather conditions. The design pavement section is not intended to support heavy construction equipment. Some distress of the ATB might occur, particularly if very heavily loaded haul trucks travel on the roadway. All distressed areas must be repaired prior to placing the final asphalt concrete surface.

**USE OF REPORT**

We have prepared this report for use by Parametrix, Inc. and the city of Redmond for preliminary design of a portion of this project. Design details are not known at the time of preparation of this report. As your design develops, we expect that additional explorations and consultation will be necessary to provide specific geotechnical criteria for individual elements of the project.

The explorations for this study are widely spaced, and there may be variations in subsurface conditions between the locations of the explorations and also with time. Our report, preliminary conclusions and interpretations should not be construed as a warranty of the subsurface conditions. A contingency for unanticipated conditions should be included in the project budget and schedule. Sufficient monitoring, testing and consultation should be provided by our firm during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether earthwork and foundation installation activities comply with contract plans and specifications.
When the design is finalized, we recommend that the design and specifications be reviewed by our firm to see that our recommendations have been interpreted and implemented as intended. Our scope does not include services related to construction safety precautions. Our recommendations are not intended to direct the contractor’s methods, techniques sequences or procedures, except as specifically described in our report for consideration in design.

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

We appreciate the opportunity to serve you on this interesting project. If there are any questions concerning this report or if we can provide additional services, please call.

Respectfully submitted,

GeoEngineers, Inc.

Herbert R. Pschunder, P.E.
Senior Engineer

James B. Thompson, P.E.
Principal
Note: Contours and elevations in meters.

EXPLANATION

TP-95-5  TEST PIT FOR CURRENT STUDY BY GEOENGINEERS, INC.
HA-95-1  HAND AUGER HOLE FOR CURRENT STUDY BY GEOENGINEERS, INC.
TP-4  TEST PIT FOR 1988 STUDY BY EARTH CONSULTANTS, INC.
HA-6  HAND AUGER HOLE FOR 1994 STUDY BY GEOENGINEERS, INC.

Reference: Undated, untitled preliminary site plan by Parametric, Inc.
APPENDIX A
LABORATORY TESTING

Soil samples obtained from the test pits and hand auger holes were brought to our laboratory for further evaluation. The samples were tested to determine moisture contents. Moisture content data is summarized in Figure A-9.
<table>
<thead>
<tr>
<th>MAJOR DIVISIONS</th>
<th>GROUP SYMBOL</th>
<th>GROUP NAME</th>
</tr>
</thead>
<tbody>
<tr>
<td>COARSE GRAINED SOILS</td>
<td>GRAY</td>
<td>WELL-GRADED GRAVEL, FINE TO COARSE GRAVEL</td>
</tr>
<tr>
<td>More Than 50% Retained on No. 200 Sieve</td>
<td>HW</td>
<td>POORLY-GRADED GRAVEL</td>
</tr>
<tr>
<td>More Than 50% of Coarse Fraction Retained on No. 4 Sieve</td>
<td>GM</td>
<td>SILTY GRAVEL</td>
</tr>
<tr>
<td>More Than 50% of Coarse Fraction Passes No. 4 Sieve</td>
<td>GC</td>
<td>CLAYEY GRAVEL</td>
</tr>
<tr>
<td>SAND</td>
<td>SW</td>
<td>WELL-GRADED SAND, FINE TO COARSE SAND</td>
</tr>
<tr>
<td>More Than 50% of Coarse Fraction Passes No. 4 Sieve</td>
<td>SP</td>
<td>POORLY-GRADED SAND</td>
</tr>
<tr>
<td>SAND WITH FINES</td>
<td>SM</td>
<td>SILTY SAND</td>
</tr>
<tr>
<td>FINE GRAINED SOILS</td>
<td>SC</td>
<td>CLAYEY SAND</td>
</tr>
<tr>
<td>SILT AND CLAY</td>
<td>ML</td>
<td>SILT</td>
</tr>
<tr>
<td>INORGANIC</td>
<td>CL</td>
<td>CLAY</td>
</tr>
<tr>
<td>ORGANIC</td>
<td>OL</td>
<td>ORGANIC SILT, ORGANIC CLAY</td>
</tr>
<tr>
<td>More Than 50% Passes No. 200 Sieve</td>
<td>MH</td>
<td>SILT OF HIGH PLASTICITY, ELASTIC SILT</td>
</tr>
<tr>
<td>SILT AND CLAY</td>
<td>CH</td>
<td>CLAY OF HIGH PLASTICITY, FAT CLAY</td>
</tr>
<tr>
<td>INORGANIC</td>
<td>OH</td>
<td>ORGANIC CLAY, ORGANIC SILT</td>
</tr>
<tr>
<td>ORGANIC</td>
<td>PT</td>
<td>PEAT</td>
</tr>
</tbody>
</table>

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

SOIL MOISTURE MODIFIERS:
- Dry - Absence of moisture, dusty, dry to the touch
- Moist - Damp, but no visible water
- Wet - Visible free water or saturated, usually soil is obtained from below water table
APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATIONS

Surface and subsurface conditions along the roadway alignments were explored by conducting a geologic reconnaissance, excavating 7 test pits and accomplishing 3 hand auger borings. We also accomplished a series of hand probes in the vicinity of the small creek which crosses the alignments in the vicinity of Station 37+50 (A-line) as well as in some of the ravines. These field activities were accomplished between May 4 and May 16, 1995.

Most of the geologic reconnaissance of the alignments was accomplished before starting the test pit and hand auger explorations so that the exploration locations and related logistics could be established prior to mobilizing equipment to the site. Additional reconnaissance was accomplished during the exploration program to observe small scale features associated with the ravines that might be indicative of slope instability.

Figures 2 and 3 show the locations of our explorations. Exploration locations were determined by pacing from existing site features. Existing ground surface elevations were determined by interpolation from contour lines shown on a topographic survey plan for the alignments prepared by Parametrix. Figures 2 and 3 also show the approximate locations of selected test pits and hand auger holes accomplished by GeoEngineers, Inc. and others during the period from 1981 to 1994.

The test pits were excavated using a large track-mounted backhoe and were intended to provide subsurface information in segments of the alignments not previously explored as a part of other studies. A total of seven test pits (TP-95-1 through 95-7) were excavated on May 4, 1995. Test pit depths varied from 10 to 15 feet. The backhoe was unable to approach an eighth test pit location in the vicinity of Station 30+00 (A and C lines) because of wet ground, dense vegetation and a mechanical problem with the backhoe. Two hand auger holes were attempted in the place of this test pit.

Three hand auger holes (HA-95-1 through 95-3) were excavated at locations inaccessible to the backhoe. In addition, a series of hand probes using a small-diameter steel rod were accomplished in the vicinity of the creek crossing at about Station 37+50 (A-line), as well as in some of the ravines. The purpose of the probing was to identify the thickness and extent of soft soils in areas of future roadway embankments near the creek and in the ravines.

The explorations were monitored by an engineering geologist from our staff who classified the soils encountered, obtained representative samples, observed ground water seepage conditions, excavated the hand auger holes, and prepared a detailed log of each test pit and hand auger hole. Soils encountered were classified in general accordance with the system described in Figure A-1. Test pit logs are presented in Figures A-2 through A-6. Hand auger hole logs are presented in Figures A-7 and A-8.
# LOG OF TEST PIT

<table>
<thead>
<tr>
<th>DEPTH BELOW GROUND SURFACE (FEET)</th>
<th>SOIL GROUP CLASSIFICATION SYMBOL</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 - 0.5</td>
<td>ML</td>
<td>Topsoil zone, 6 inches thick</td>
</tr>
<tr>
<td>0.5 - 1.5</td>
<td>ML</td>
<td>Dark brownish gray silt with roots and other organic matter (soft, moist)</td>
</tr>
<tr>
<td>1.5 - 3.0</td>
<td>ML</td>
<td>Tan silt with organic matter (medium stiff, moist) (diatomaceous earth)</td>
</tr>
<tr>
<td>3.0 - 4.0</td>
<td>PT</td>
<td>Brown and black peat (soft, moist)</td>
</tr>
<tr>
<td>4.0 - 9.0</td>
<td>SM/ML</td>
<td>Brown and gray silty fine to coarse sand with gravel interbedded with brown silt with sand and occasional gravel (medium dense/stiff, wet)</td>
</tr>
<tr>
<td>9.0 - 10.0</td>
<td>ML</td>
<td>Gray silt with fine sand and occasional gravel (hard, wet)</td>
</tr>
</tbody>
</table>

**TEST PIT TP-95-1**

Approximate station: 15+90 (A-line)

Approximate ground surface elevation: 28 feet

Test pit completed at 10.0 feet on 05/04/95

Rapid ground water seepage observed at 4.0 feet

Severe caving observed at 5.0 feet

Disturbed soil samples obtained at 0.5, 0.8, 2.5, 3.5, 6.0 and 9.0 feet

The depths on the test pit logs, although shown to 0.1 foot, are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.
<table>
<thead>
<tr>
<th>DEPTH BELOW GROUND SURFACE (FEET)</th>
<th>SOIL GROUP CLASSIFICATION SYMBOL</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 - 1.0</td>
<td></td>
<td>Topsoil zone, 12 inches thick</td>
</tr>
<tr>
<td>1.0 - 2.0</td>
<td>SM</td>
<td>Light brown silty fine to medium sand (loose, moist) (slope wash)</td>
</tr>
<tr>
<td>2.0 - 2.5</td>
<td>ML</td>
<td>Brown and black silt with sand, roots, organic matter and wood debris (soft, moist) (fill)</td>
</tr>
<tr>
<td>2.5 - 3.0</td>
<td></td>
<td>Concrete slab</td>
</tr>
<tr>
<td>3.0 - 4.0</td>
<td>ML</td>
<td>Dark brown silt with fine sand, gravel and organic matter (soft, moist)</td>
</tr>
<tr>
<td>4.0 - 4.5</td>
<td>OL</td>
<td>Black organic silt with fine sand and occasional gravel (soft, moist)</td>
</tr>
<tr>
<td>4.5 - 12.0</td>
<td>ML</td>
<td>Brown and gray silt with fine sand, occasional gravel and organic matter (soft, moist)</td>
</tr>
<tr>
<td>12.0 - 14.0</td>
<td>SM/ML</td>
<td>Brownish gray silty fine to coarse sand with gravel interbedded with silt with sand and occasional gravel (medium dense/stiff, wet)</td>
</tr>
<tr>
<td>14.0 - 15.0</td>
<td>ML</td>
<td>Gray silt with fine sand and occasional gravel (hard, wet)</td>
</tr>
</tbody>
</table>

**TEST PIT TP-95-2**

Approximate station: 19+40 (A-line)
Approximate ground surface elevation: 38.0 feet

- Test pit completed at 15.0 feet on 05/04/95
- Moderate ground water seepage observed at 12.0 feet
- Minor caving observed at 12.0 feet
- Disturbed soil sample obtained at 3.5 feet

The depths on the test pit logs, although shown to 0.1 foot, are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.
# LOG OF TEST PIT

<table>
<thead>
<tr>
<th>DEPTH BELOW GROUND SURFACE (FEET)</th>
<th>SOIL GROUP CLASSIFICATION SYMBOL</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 - 1.0</td>
<td></td>
<td>Topsoil zone, 12 inches thick</td>
</tr>
<tr>
<td>1.0 - 3.0</td>
<td>ML</td>
<td>Tannish gray silt with fine sand and occasional gravel and cobbles (soft, moist)</td>
</tr>
<tr>
<td>3.0 - 10.5</td>
<td>PT</td>
<td>Dark brown peat (soft, wet)</td>
</tr>
<tr>
<td>10.5 - 14.5</td>
<td>SM/ML</td>
<td>Gray silty fine to coarse sand with gravel interbedded with brown silt with sand (medium dense/stiff, wet)</td>
</tr>
<tr>
<td>14.5 - 15.0</td>
<td>ML</td>
<td>Gray silt with sand and occasional gravel (hard, moist)</td>
</tr>
</tbody>
</table>

**TEST PIT TP-95-3**

- Approximate station: 22+90 (A-line)
- Approximate ground surface elevation: 29.0 feet
- Test pit completed at 15.0 feet on 05/04/95
- Moderate ground water seepage observed at 3.0 feet
- Rapid ground water seepage observed at 10.5 feet
- Moderate caving observed at 10.5 feet
- Disturbed soil samples obtained at 1.5, 7.0 and 12.0 feet

The depths on the test pit logs, although shown to 0.1 foot, are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.
## LOG OF TEST PIT

<table>
<thead>
<tr>
<th>DEPTH BELOW GROUND SURFACE (FEET)</th>
<th>SOIL GROUP CLASSIFICATION SYMBOL</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 - 1.0</td>
<td></td>
<td>Topsoil zone, 12 inches thick</td>
</tr>
<tr>
<td>1.0 - 1.5</td>
<td>ML</td>
<td>Tan organic silt with sand and organic matter (soft, moist)</td>
</tr>
<tr>
<td>1.5 - 3.5</td>
<td>ML</td>
<td>Light brown silt with sand and organic matter (medium stiff, moist to wet)</td>
</tr>
<tr>
<td>3.5 - 11.0</td>
<td>ML</td>
<td>Gray silt with sand and gravel (very stiff to hard, moist)</td>
</tr>
</tbody>
</table>

**TEST PIT TP-95-4**

Approximate station: 26+30 (C-line)

Approximate ground surface elevation: 40.0 feet

Test pit completed at 11.0 feet on 05/04/95

Slow ground water seepage observed at 3.5 feet

Minor caving observed at 3.5 feet

Disturbed soil samples obtained at 2.5 and 4.0 feet

<table>
<thead>
<tr>
<th>DEPTH BELOW GROUND SURFACE (FEET)</th>
<th>SOIL GROUP CLASSIFICATION SYMBOL</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 - 0.5</td>
<td></td>
<td>Topsoil zone, 6 inches thick</td>
</tr>
<tr>
<td>0.5 - 4.0</td>
<td>ML</td>
<td>Light brown and gray silt with sand and gravel (medium stiff, moist)</td>
</tr>
<tr>
<td>4.0 - 10.0</td>
<td>SM</td>
<td>Brown silty fine to coarse sand with gravel and occasional cobbles (medium dense, moist to wet)</td>
</tr>
<tr>
<td>10.0 - 12.5</td>
<td>SM/ML</td>
<td>Brown silty fine to coarse sand with gravel interbedded with sandy silt (medium dense/very stiff, wet)</td>
</tr>
</tbody>
</table>

**TEST PIT TP-95-5**

Approximate station: 35+30 (C-line)

Approximate ground surface elevation: 100.0 feet

Test pit completed at 12.5 feet on 05/04/95

Slow ground water seepage observed at 7.0 and 10.5 feet

Minor caving observed at 7.0 and 10.5 feet

Disturbed soil sample obtained at 8.0 feet

---

THE DEPTHS ON THE TEST PIT LOGS, ALTHOUGH SHOWN TO 0.1 FOOT, ARE BASED ON AN AVERAGE OF MEASUREMENTS ACROSS THE TEST PIT AND SHOULD BE CONSIDERED ACCURATE TO 0.5 FOOT.
## LOG OF TEST PIT

<table>
<thead>
<tr>
<th>DEPTH BELOW GROUND SURFACE (FEET)</th>
<th>SOIL GROUP CLASSIFICATION SYMBOL</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 - 0.5</td>
<td>ML</td>
<td>Topsoil zone, 6 inches thick</td>
</tr>
<tr>
<td>0.5 - 1.5</td>
<td>SM</td>
<td>Light brown and gray silt with sand, organic matter and occasional gravel (soft, moist)</td>
</tr>
<tr>
<td>1.5 - 9.5</td>
<td>SM</td>
<td>Brown silty fine to coarse sand with gravel (medium dense, moist) Grades with interbedded silt with fine sand at 7.0 feet</td>
</tr>
<tr>
<td>9.5 - 12.0</td>
<td>ML</td>
<td>Light brown silt with sand (very stiff to hard, wet) Test pit completed at 12.0 feet on 05/04/95 Slow ground water seepage observed at 10.0 feet Minor caving observed at 8.0 feet Disturbed soil sample obtained at 10.0 feet</td>
</tr>
</tbody>
</table>

**TEST PIT TP-95-7**  
Approximate station: 31+70 (C-line)  
Approximate ground surface elevation: 90.0 feet  
| 0.0 - 0.5                         | SM                               | Forest duff and topsoil zone, 6 inches thick |
| 0.5 - 5.0                         | SM                               | Orangish brown silty fine sand with occasional gravel and organic matter (medium dense, moist) |
| 5.0 - 8.0                         | SM                               | Brown silty fine sand with fine to coarse gravel (medium dense, moist) |
| 8.0 - 13.0                        | ML                               | Light grayish brown silt with fine sand (hard, moist) Test pit completed at 13.0 feet on 05/04/95 No ground water seepage observed No caving observed Disturbed soil samples obtained at 2.5, 5.0 and 10.0 feet |

The depths on the test pit logs, although shown to 0.1 foot, are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.
# LOG OF HAND AUGER HOLE

<table>
<thead>
<tr>
<th>DEPTH BELOW GROUND SURFACE (FEET)</th>
<th>SOIL GROUP CLASSIFICATION SYMBOL</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 - 0.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.5 - 2.0</td>
<td>SM</td>
<td></td>
</tr>
<tr>
<td>0.0 - 0.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.3 - 0.7</td>
<td>SM</td>
<td></td>
</tr>
<tr>
<td>0.7 - 3.3</td>
<td>SM</td>
<td></td>
</tr>
<tr>
<td>3.3 - 5.0</td>
<td>ML</td>
<td></td>
</tr>
</tbody>
</table>

**HAND AUGER HOLE HA-95-1**

- Approximate station: 30+40 (C-line)
- Approximate ground surface elevation: 82.0 feet
- Forest duff and topsoil zone, 6 inches thick
- Brown silty fine sand with occasional gravel and organic matter (loose, moist)
- Hand auger hole completed at 2.0 feet on 05/04/95
- Refusal on gravel or cobble at 2.0 feet
- No ground water seepage observed
- Disturbed soil sample obtained at 1.0 foot

**HAND AUGER HOLE HA-95-2**

- Approximate station: 29+60 (C-line)
- Approximate ground surface elevation: 80.0 feet
- Forest duff and topsoil zone, 3 inches thick
- Brown silty fine sand with occasional gravel and organic matter (loose, moist)
- Hand auger hole completed at 5.0 feet on 05/04/95
- No ground water seepage observed
- Disturbed soil samples obtained at 1.8 and 3.5 feet

The depths on the hand auger hole logs, although shown to 0.1 foot, are based on an average of measurements across the hand auger hole and should be considered accurate to 0.5 foot.
# Log of Hand Auger Hole

<table>
<thead>
<tr>
<th>Depth Below Ground Surface (Feet)</th>
<th>Soil Group Classification Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 - 0.3</td>
<td></td>
<td>Forest duff and topsoil zone, 3 inches thick</td>
</tr>
<tr>
<td>0.3 - 4.0</td>
<td>SM</td>
<td>Brown silty fine sand with gravel and a trace of organic matter (loose to medium dense, moist)</td>
</tr>
<tr>
<td>4.0 - 6.5</td>
<td>SP</td>
<td>Brownish gray fine to medium sand with gravel (medium dense, moist)</td>
</tr>
</tbody>
</table>

**Hand Auger Hole HA-95-3**

- Approximate Station: 30+00 (C-line)
- Approximate ground surface elevation: 84.0 feet
- Hand auger hole completed at 6.5 feet on 05/16/95
- No ground water seepage observed
- Minor caving observed at 5.0 feet

The depths on the Hand Auger Hole logs, although shown to 0.1 foot, are based on an average of measurements across the Hand Auger Hole and should be considered accurate to 0.5 foot.
<table>
<thead>
<tr>
<th>Exploration Number</th>
<th>Depth of Sample (feet)</th>
<th>Soil Classification</th>
<th>Moisture Content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP-95-1</td>
<td>0.5</td>
<td>ML</td>
<td>64</td>
</tr>
<tr>
<td>TP-95-1</td>
<td>0.8</td>
<td>ML</td>
<td>81</td>
</tr>
<tr>
<td>TP-95-1</td>
<td>2.5</td>
<td>ML</td>
<td>98</td>
</tr>
<tr>
<td>TP-95-1</td>
<td>3.5</td>
<td>PT</td>
<td>175</td>
</tr>
<tr>
<td>TP-95-1</td>
<td>6.0</td>
<td>SM</td>
<td>14</td>
</tr>
<tr>
<td>TP-95-1</td>
<td>9.0</td>
<td>ML</td>
<td>15</td>
</tr>
<tr>
<td>TP-95-2</td>
<td>3.5</td>
<td>ML</td>
<td>22</td>
</tr>
<tr>
<td>TP-95-3</td>
<td>1.5</td>
<td>ML</td>
<td>27</td>
</tr>
<tr>
<td>TP-95-3</td>
<td>7.0</td>
<td>PT</td>
<td>191</td>
</tr>
<tr>
<td>TP-95-3</td>
<td>12.0</td>
<td>SM</td>
<td>10</td>
</tr>
<tr>
<td>TP-95-4</td>
<td>2.5</td>
<td>ML</td>
<td>11</td>
</tr>
<tr>
<td>TP-95-4</td>
<td>4.0</td>
<td>ML</td>
<td>19</td>
</tr>
<tr>
<td>TP-95-5</td>
<td>8.0</td>
<td>SM</td>
<td>16</td>
</tr>
<tr>
<td>TP-95-6</td>
<td>10.0</td>
<td>ML</td>
<td>24</td>
</tr>
<tr>
<td>TP-95-7</td>
<td>2.5</td>
<td>SM</td>
<td>8</td>
</tr>
<tr>
<td>TP-95-7</td>
<td>5.0</td>
<td>SM</td>
<td>15</td>
</tr>
<tr>
<td>TP-95-7</td>
<td>10.0</td>
<td>ML</td>
<td>19</td>
</tr>
<tr>
<td>HA-95-1</td>
<td>1.0</td>
<td>SM</td>
<td>19</td>
</tr>
<tr>
<td>HA-95-2</td>
<td>1.8</td>
<td>SM</td>
<td>10</td>
</tr>
<tr>
<td>HA-95-2</td>
<td>3.5</td>
<td>ML</td>
<td>17</td>
</tr>
</tbody>
</table>
SELECTED HAND AUGER HOLE LOGS

FROM

REPORT OF GEOTECHNICAL ENGINEERING SERVICES
PROPOSED REDMOND 74 RESIDENTIAL DEVELOPMENT
CITY OF REDMOND FILE NO. PPL90-002
REDMOND, WASHINGTON

BY GEOENGINEERS, INC.
# LOG OF HAND-AUGER BORING

<table>
<thead>
<tr>
<th>DEPTH BELOW GROUND SURFACE (FEET)</th>
<th>SOIL GROUP CLASSIFICATION SYMBOL</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 - 3.5</td>
<td>ML</td>
<td>Light brown silt with organics (stiff, moist)</td>
</tr>
<tr>
<td>3.5 - 6.0</td>
<td>OL</td>
<td>Dark brown organic silt (loose, moist)</td>
</tr>
<tr>
<td>6.0 - 7.5</td>
<td>ML</td>
<td>Brown and gray silt with organics and a trace of fine sand and occasional charcoal fragments (very soft, moist)</td>
</tr>
<tr>
<td>7.5 - 10.0</td>
<td>SP</td>
<td>Gray fine to medium sand with silt (medium dense to dense, moist)</td>
</tr>
</tbody>
</table>

**HAND-AUGER BORING 4**
Approximate Ground Surface Elevation: 27 feet
Hand-auger boring completed at 10.0 feet on 01/12/94
Slight ground water seepage observed at 10.0 feet
Disturbed soil samples obtained at 7.0 and 10.0 feet

| 0.0 - 3.5                         | OL                              | Dark brown organic silt (very soft, moist) |
| 3.5 - 7.5                         | PT                              | Dark brown peat (very soft, wet) |

**HAND-AUGER BORING 5**
Approximate Ground Surface Elevation: 30 feet
Hand-auger boring completed at 7.0 feet on 01/12/94
Ground water seepage observed at 2.5 feet

| 0.0 - 3.5                         | SM                              | Brown silty fine sand with occasional organics and charcoal fragments (loose, moist) (fill) |
| 3.5 - 4.5                         | ML                              | Gray silt with trace of fine sand (stiff, moist) |
| 4.5 - 6.0                         | SM                              | Brown silty fine to medium sand (medium dense, moist) |
| 6.0 - 6.5                         | SP-SM                           | Brown fine to medium sand with silt (medium dense, wet) |
| 6.5 - 7.5                         | ML                              | Gray silt with fine to medium sand (stiff, wet) |

**HAND-AUGER BORING 6**
Approximate Ground Surface Elevation: 135 feet
Hand-auger boring completed at 7.5 feet due to refusal on large rock on 01/15/94
Ground water seepage observed at 6.0 feet

The depths on the test pit logs, although shown to 0.1 foot, are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.
### LOG OF HAND-AUGER BORING

<table>
<thead>
<tr>
<th>DEPTH BELOW GROUND SURFACE (FEET)</th>
<th>SOIL GROUP CLASSIFICATION SYMBOL</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 - 4.4</td>
<td>SM</td>
<td>Brown silty fine to medium sand with occasional gravel and trace of organics (medium dense, moist) (fill)</td>
</tr>
<tr>
<td>4.4 - 4.6</td>
<td>SM</td>
<td>Light brown silty fine to medium sand with gravel (very dense, moist)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Hand-auger boring completed at 4.6 feet due to refusal on large rock on 01/15/94</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No ground water seepage observed</td>
</tr>
</tbody>
</table>

**HAND-AUGER BORING 8**

**Approximate Ground Surface Elevation:** 34 feet

<table>
<thead>
<tr>
<th>DEPTH BELOW GROUND SURFACE (FEET)</th>
<th>SOIL GROUP CLASSIFICATION SYMBOL</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 - 1.5</td>
<td>SM</td>
<td>Dark brown silt with fine sand and organics (soft, moist)</td>
</tr>
<tr>
<td>1.5 - 4.0</td>
<td>SM</td>
<td>Gray silty fine sand with occasional organics (loose, moist)</td>
</tr>
<tr>
<td>4.0 - 7.0</td>
<td>ML</td>
<td>Gray silt with fine sand (medium stiff to stiff, moist)</td>
</tr>
<tr>
<td>7.0 - 8.0</td>
<td>OL</td>
<td>Dark brown organic silt (very soft, moist)</td>
</tr>
<tr>
<td>8.0 - 10.2</td>
<td>SM</td>
<td>Gray silty fine sand with occasional fine gravel (medium dense, wet)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Hand-auger boring completed at 10.3 feet on 01/15/94</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Slight ground water seepage observed at 9.0 feet</td>
</tr>
</tbody>
</table>

**HAND-AUGER BORING 9**

**Approximate Ground Surface Elevation:** 140 feet

<table>
<thead>
<tr>
<th>DEPTH BELOW GROUND SURFACE (FEET)</th>
<th>SOIL GROUP CLASSIFICATION SYMBOL</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 - 3.0</td>
<td>SM</td>
<td>Brown silty fine to medium sand with occasional gravel and trace of organics (medium dense, moist) (till)</td>
</tr>
<tr>
<td>3.0 - 4.0</td>
<td>SM</td>
<td>Light brown silty fine to medium sand with gravel (dense, moist)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Hand auger boring completed at 4.0 feet on 01/15/94</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No ground water seepage observed</td>
</tr>
</tbody>
</table>

*The depths on the test pit logs, although shown to 0.1 foot, are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.*
SELECTED TEST PIT LOGS
FROM
REPORT TO INTRAWEST, RESULTS OF THE GEOTECHNICAL STUDY PROPOSED TOWNHOMES ON THE RIVER TRAIL
REDMOND, WASHINGTON
1994
BY GOLDER ASSOCIATES, INC.
TEST PIT 15

Date: 9/20/89  Depth to Water: 11.0 feet  Test Pit Depth: 12.0 feet
Surface Elevation: 27.0 feet

0.0 to 2.0 feet  Soft, yellow-brown, fibrous, fine, sandy, silty PEAT and ASH, with shrinkage cracks (Pt)
2.0 to 5.5 feet  Soft, dusky brown, silty PEAT (Pt)
5.5 to 8.0 feet  Firm, dark greenish gray CLAYEY SILT, trace fine sand (ML)
8.0 to 10.0 feet  Soft, dusky brown, amorphous, silty PEAT, trace wood fragments (Pt)
10.0 to 12.0 feet  Compact, gray-brown, fine to coarse SAND, and GRAVEL, trace cobbles (SP)

TEST PIT 16

Date: 9/20/89  Depth to Water: 8.0 feet  Test Pit Depth: 10.0 feet
Surface Elevation: 28.3 feet

0.0 to 3.0 feet  Soft, pale yellow-brown, fibrous, silty PEAT and ASH, some very fine sand, large
shrinkage cracks (Pt)
3.0 to 6.0 feet  Soft, dusky brown, silty PEAT, scattered fibers (Pt)
6.0 to 8.0 feet  Soft, dusky brown, silty PEAT, with 6-inch thick firm, greenish gray SILT, little to some
clay, with occasional 2-inch thick fine sand layers (Pt)
8.0 to 10.0 feet  Compact, gray-brown, fine to coarse SAND, and COBBLES, some gravel (SP)

TEST PIT 17

Date: 9/20/89  Depth to Water: 8.0 feet  Test Pit Depth: 9.0 feet
Surface Elevation: 27.1 feet

0.0 to 3.0 feet  Soft, dusky brown, fibrous, silty PEAT (Pt)
3.0 to 6.0 feet  Soft, dusky yellow-brown, silty PEAT and ASH (Pt), with layers of firm, greenish gray SILT,
little clay, trace fine sand (ML)
6.0 to 9.0 feet  Loose to compact, gray-brown, gravelly SAND, some well-rounded cobbles, caves easily
(SP)

TEST PIT 18

Date: 9/20/89  Depth to Water: 10.0 feet  Test Pit Depth: 12.0 feet
Surface Elevation: 28.0 feet

0.0 to 2.0 feet  Soft, dusky brown, fibrous, silty PFAT (Pt)
2.0 to 4.0 feet  Loose, greenish gray, silty fine SAND (SM)
4.0 to 8.0 feet  Firm to stiff, greenish gray SILT and CLAY, grades to fine sand (ML)
8.0 to 12.0 feet  Loose to compact, gray-brown, medium to coarse SAND and COBBLES (well rounded) (SP)

TEST PIT 19

Date: 9/20/89  Depth to Water: 10.0 feet  Test Pit Depth: 12.0 feet
Surface Elevation: 28.0 feet

0.0 to 3.0 feet  Soft, pale yellowish brown to dusky brown, fibrous, silty PEAT and ASH, little fine sand, 3"
to 6" long shrinkage cracks (Pt)
3.0 to 5.0 feet  Compact, greenish gray, fine sandy CLAYEY SILT, laminated (ML)
5.0 to 12.0 feet  Loose to compact, gray-brown, fine to coarse SAND, some fine gravels, trace cobbles,
caves easily (SP)

Golder Associates
TEST PIT 35

Date: 1/31/94
Surface Elevation: 28.0 feet

Depth to Water: Unknown
Test Pit Depth: 4.3 feet

0.0 to 4.3 feet Compact, moderate yellow brown, fine to coarse SAND and fine to coarse GRAVEL, no caving (SW-GW)

TEST PIT 36

Date: 1/31/94
Surface Elevation: 28.0 feet

Depth to Water: 3.8 feet
Test Pit Depth: 6.0 feet

0.0 to 1.8 feet Stiff, mottled tan olive gray CLAYEY SILT (CL-ML) (TOPSOIL)
1.8 to 3.8 feet Very stiff, dark brown, PEAT (Pt)
3.8 to 4.3 feet Compact, olive gray, fine SAND and SILT, massive (SM)
4.3 to 6.0 feet Compact, moderate yellow brown, fine to coarse SAND, little fine to coarse GRAVEL (SW)

TEST PIT 37

Date: 2/1/94
Surface Elevation: 30.5 feet

Depth to Water: 12.0 feet
Test Pit Depth: 12.0 feet

0.0 to 1.0 feet Soft, dark brown SILT, abundant roots and fibers (ML) (TOPSOIL)
1.0 to 6.0 feet Stiff to very stiff, light olive gray, SILT to CLAYEY SILT (ML) (ASH)
6.0 to 7.5 feet Firm to stiff, dark brown, PEAT to ORGANIC SILT (Pt-OL)
7.5 to 11.0 feet Compact, moderate yellow brown, gravelly, fine to coarse SAND, little cobble, no caving (SW)

TEST PIT 38

Date: 2/1/94
Surface Elevation: 28.9 feet

Depth to Water: 10.0 feet
Test Pit Depth: 10.0 feet

0.0 to 1.0 feet Soft to stiff, dark brown SILT to ORGANIC SILT (ML-OL) (TOPSOIL)
1.0 to 5.0 feet Stiff to very stiff, light olive gray, SILT to CLAYEY SILT (ML) (ASH)
5.0 to 7.0 feet Soft to stiff, dark brown, PEAT to ORGANIC SILT (Pt-OL)
7.0 to 10.0 feet Compact, moderate yellow brown, fine to coarse SAND and fine to coarse GRAVEL, little cobble, no caving (SW-GW)

TEST PIT 39

Date: 2/1/94
Surface Elevation: 28.3 feet

Depth to Water: 10.0 feet
Test Pit Depth: 10.0 feet

0.0 to 0.3 feet Soft, light olive gray to medium brown ORGANIC SILT, abundance of roots and fibers (OL) (TOPSOIL)
0.3 to 3.5 feet Stiff to very stiff, light olive gray, SILT to CLAYEY SILT, abundance of vertical desiccation cracks, blocky (ML) (ASH)
3.5 to 8.0 feet Soft to firm, dark brown, PEAT (Pt)
8.0 to 8.3 feet Very stiff to hard, light olive gray to tan, SILT (ML) (ASH)
8.3 to 9.0 feet Soft to firm, dark brown, PEAT (Pt)
9.0 to 10.0 feet Compact, medium gray, fine to medium SAND (SP)
TEST PIT 45

Date: 2/1/94  
Surface Elevation: 29.0 feet  
Depth to Water: Unknown  
Test Pit Depth: 7.0 feet

0.0 to 1.0 feet  Soft to firm, light brown ORGANIC SILT (OL) (TOPSOIL)
1.0 to 2.0 feet  Stiff to very stiff, light olive gray, SILT to CLAYEY SILT (ML)
2.0 to 3.0 feet  Soft to stiff, dark brown, PEAT (Pt)
3.0 to 4.0 feet  Compact, light olive gray, silty, fine SAND (SM)
4.0 to 7.0 feet  Compact, moderate yellow brown, gravelly, fine to coarse SAND, little cobble, trace silt, no caving (SW)

TEST PIT 46

Date: 2/1/94  
Surface Elevation: 35.0 feet  
Depth to Water: 3.5 feet  
Test Pit Depth: 7.0 feet

0.0 to 0.5 feet  Soft, dark brown ORGANIC SILT, trace to little gravel (OL) (TOPSOIL)
0.5 to 7.0 feet  Compact to dense, moderate yellow brown, silty, fine to medium SAND and fine to coarse GRAVEL, no caving (SM-GM)

TEST PIT 47

Date: 2/1/94  
Surface Elevation: 28.0 feet  
Depth to Water: Unknown  
Test Pit Depth: 7.0 feet

0.0 to 1.0 feet  Soft to firm, dark brown ORGANIC SILT (OL) (TOPSOIL)
1.0 to 2.0 feet  Stiff, light olive gray to tan, SILT to CLAYEY SILT (ML)
2.0 to 3.5 feet  Soft, dark brown, PEAT; interbedded with firm to stiff, light olive gray CLAYEY SILT (Pt-ML)
3.5 to 5.5 feet  Compact, light olive gray, silty, fine SAND, trace cobble (SM)
5.5 to 7.0 feet  Compact, moderate yellow brown, gravelly, fine to medium SAND, trace to little cobble, trace silt, no caving (SW)
SELECTED TEST PIT LOGS

FROM

PRELIMINARY GEOTECHNICAL ENGINEERING STUDY
REDMOND 74-ACRE PARCEL
WOODINVILLE-REDMOND ROAD NORTHEAST
REDMOND, WASHINGTON FOR
TRIAD ASSOCIATES
1988

BY EARTH CONSULTANTS, INC.
### TEST PIT NO. 103

Logged By: **KAL**  
Date: **10-13-88**  
Elev.: **105'**

<table>
<thead>
<tr>
<th>Depth (ft.)</th>
<th>USCS</th>
<th>Soil Description</th>
<th>W (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>sm</td>
<td>Reddish brown sandy SILT, moist, medium dense, mottled</td>
<td></td>
</tr>
<tr>
<td></td>
<td>SM</td>
<td>Tan silty SAND with gravel, damp, very dense</td>
<td>11</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- cobbles below 9 feet</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>- becomes heavily mottled at 11 feet</td>
<td></td>
</tr>
</tbody>
</table>
| Test pit terminated at 12.5 feet below existing grade.  
No groundwater seepage encountered during excavation.

### TEST PIT NO. 104

Logged By: **KAL**  
Date: **10-13-88**  
Elev.: **130'**

<table>
<thead>
<tr>
<th>Depth (ft.)</th>
<th>USCS</th>
<th>Soil Description</th>
<th>W (%)</th>
<th>PL</th>
<th>LL</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>sm</td>
<td>Tan silty SAND moist, medium dense</td>
<td>14</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>CL</td>
<td>Tan SILT moist, medium stiff to stiff, lamenated, slightly plastic</td>
<td>35</td>
<td>28</td>
<td>54</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- gravel at 7.5 feet and becomes mottled</td>
<td>31</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>SM</td>
<td>Brown fine silty SAND, moist, dense</td>
<td>15</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>- becomes coarser at 10 feet</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>- becomes mottled at 11 feet</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
| Test pit terminated at 11 feet below existing grade.  
No groundwater seepage encountered during excavation.

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis, and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

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**Earth Consultants Inc.**  
Geotechnical Engineering and Geology  

**TEST PIT LOGS**  
REDMOND 74 ACRE PARCEL  
REDMOND, WASHINGTON

Proj. No. 4085  
Date Oct'88  
Plate 5
TEST PIT NO. 105

Logged By: KAL
Date: 10-13-88

Elev.: 120' +

<table>
<thead>
<tr>
<th>Depth (ft.)</th>
<th>USCS</th>
<th>Soil Description</th>
<th>W (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>SM</td>
<td>Redish brown silty SAND with gravel, moist, dense</td>
<td>11</td>
</tr>
<tr>
<td>5</td>
<td>sm</td>
<td>Tan silty SAND with gravel, moist, dense</td>
<td>8</td>
</tr>
<tr>
<td>10</td>
<td>ML</td>
<td>cobbles below 6 feet</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>ML</td>
<td>Tan SILT, moist, stiff, mottled</td>
<td>26</td>
</tr>
<tr>
<td>15</td>
<td>sm</td>
<td>Thin layer of silty SAND at 11 feet</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>ml</td>
<td>Tan fine silty SAND heavily mottled, interbedded</td>
<td>37</td>
</tr>
<tr>
<td>15</td>
<td>sm</td>
<td>With tan SILT, moist, dense to stiff</td>
<td>26</td>
</tr>
<tr>
<td>15</td>
<td></td>
<td>Gray sandy SILT, moist, very stiff</td>
<td></td>
</tr>
</tbody>
</table>

Test pit terminated at 13 feet below existing grade.
No groundwater seepage encountered during excavation.

TEST PIT NO. 106

Logged By: KAL
Date: 10-13-88

Elev.: 75' +

<table>
<thead>
<tr>
<th>Depth (ft.)</th>
<th>USCS</th>
<th>Soil Description</th>
<th>W (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>sm</td>
<td>Tan silty SAND with gravel, moist, medium dense</td>
<td>8</td>
</tr>
<tr>
<td>5</td>
<td>GP-GM</td>
<td>organic debris at 3 feet</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>GP-GM</td>
<td>Tan GRAVEL with sand and silt, moist, dense</td>
<td>6</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>cobbles at 7 feet</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>becomes very dense at 8 feet</td>
<td>6</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>becomes wet at 9.5 feet</td>
<td></td>
</tr>
</tbody>
</table>

Test pit terminated at 11 feet below existing grade.
No groundwater seepage encountered during excavation.

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis, and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.
### Log of Test Pit 3

<table>
<thead>
<tr>
<th>Depth (ft.)</th>
<th>USCS</th>
<th>Soil Description</th>
<th>w</th>
<th>Lab Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>ML</td>
<td>Black TOPSOIL, loose, moist.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>MH</td>
<td>Brown, sandy SILT, medium dense, moist.</td>
<td>16</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>ML</td>
<td>Gray, clayey SILT to silty CLAY, very stiff, moist.</td>
<td>41</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>SM</td>
<td>Gray, sandy SILT, medium dense, grades to silty SAND.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Test pit terminated at 12.5 feet on 11/9/78. No groundwater seepage encountered.

### Log of Test Pit 4

<table>
<thead>
<tr>
<th>Depth (ft.)</th>
<th>USCS</th>
<th>Soil Description</th>
<th>w</th>
<th>Lab Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>SM</td>
<td>Black TOPSOIL and SOD, loose, moist.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>ML</td>
<td>Brown, silty SAND, medium dense, moist to damp.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>SM</td>
<td>Brown, sandy SILT to clayey sandy SILT, medium dense to dense, moist.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>SM</td>
<td>Blue-gray, sandy SILT with some gravel, very dense, moist.</td>
<td>17</td>
<td></td>
</tr>
</tbody>
</table>

Test pit terminated at 11 feet on 11/9/78. Seepage encountered from 4 to 9 feet.
## TEST PIT LOGS

### Log of Test Pit 5

<table>
<thead>
<tr>
<th>Depth (ft.)</th>
<th>USCS</th>
<th>Soil Description</th>
<th>Lab Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>SM</td>
<td>Black TOPSOIL and SOD, loose, moist.</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>SM</td>
<td>Brown, silty gravelly SAND, medium dense, wet.</td>
<td>11</td>
</tr>
</tbody>
</table>

Test hole terminated due to very rapid seepage at 6 feet on 11/10/78.

### Log of Test Pit 6

<table>
<thead>
<tr>
<th>Depth (ft.)</th>
<th>USCS</th>
<th>Soil Description</th>
<th>Lab Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>SM</td>
<td>TOPSOIL and SOD</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>SM</td>
<td>Brown, silty gravelly SAND with some cobbles and boulders, medium dense, moist. Becomes gray in color.</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>GM</td>
<td>Gray, gravelly silty SAND, dense, moist to damp. (Sandy till)</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>GP</td>
<td>Brown, sandy GRAVEL with some SILT, dense, wet.</td>
<td></td>
</tr>
</tbody>
</table>

Test pit terminated at 10 feet on 11/9/78. Groundwater seepage not encountered.
### Log of Test Pit 8

<table>
<thead>
<tr>
<th>Depth</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>6&quot; TOPSOIL, loose, moist.</td>
</tr>
<tr>
<td>5</td>
<td>Brown-gray, sandy SILT with gravel and sand lenses, medium dense, moist to damp.</td>
</tr>
<tr>
<td>10</td>
<td>Grades to silty SAND.</td>
</tr>
<tr>
<td>15</td>
<td>Test pit terminated at 11.2 feet on 11/9/78. Moderate seepage from 3 feet.</td>
</tr>
</tbody>
</table>

Elev. 148+
## Test Pit Logs

**Log of Test Pit 9**

<table>
<thead>
<tr>
<th>Depth (ft.)</th>
<th>USCS</th>
<th>Soil Description</th>
<th>Lab Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>ML</td>
<td>Black, sandy TOPSOIL, loose, moist.</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>ML</td>
<td>Gray, sandy SILT to clayey SILT with some GRAVEL, medium dense, moist.</td>
<td>21</td>
</tr>
<tr>
<td>10</td>
<td>SM</td>
<td>Brown, slightly silty gravelly SAND, dense, wet.</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td></td>
<td>Test pit terminated on 11/9/78. Groundwater seepage at 2 feet.</td>
<td></td>
</tr>
</tbody>
</table>

**Log of Test Pit 10**

<table>
<thead>
<tr>
<th>Depth (ft.)</th>
<th>USCS</th>
<th>Soil Description</th>
<th>Lab Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>SII</td>
<td>Black TOPSOIL</td>
<td>18</td>
</tr>
<tr>
<td>5</td>
<td>ML</td>
<td>Reddish-brown, silty SAND with gravel, medium dense, wet.</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>ML</td>
<td>Gray, sandy SILT, medium dense, moist.</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>ML</td>
<td>Blue, clayey SILT with fine SAND, medium dense, moist.</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td></td>
<td>Test pit terminated at 11.4 feet on 11/9/78. No groundwater seepage encountered.</td>
<td></td>
</tr>
</tbody>
</table>
### Log of Test Pit II

<table>
<thead>
<tr>
<th>Depth (ft.)</th>
<th>USCS</th>
<th>Soil Description</th>
<th>Lab Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td>Black TOPSOIL.</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>ML</td>
<td>Brown, fine sandy SILT, medium dense, moist.</td>
<td>26</td>
</tr>
<tr>
<td>10</td>
<td>SM</td>
<td>Brown, fine to medium SAND, medium dense, moist.</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td></td>
<td>Test pit terminated at 10.5 feet on 11/9/78. No groundwater seepage encountered.</td>
<td></td>
</tr>
</tbody>
</table>

### Log of Test Pit I2

<table>
<thead>
<tr>
<th>Depth (ft.)</th>
<th>USCS</th>
<th>Soil Description</th>
<th>Lab Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td>Black TOPSOIL and SOD</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>SM</td>
<td>Brown, gravelly silty SAND to fine sandy SILT, medium dense, moist.</td>
<td>20</td>
</tr>
<tr>
<td>10</td>
<td>ML</td>
<td>Gray-brown, fine sandy SILT, medium dense, moist.</td>
<td>35</td>
</tr>
<tr>
<td>15</td>
<td></td>
<td>Test pit terminated at 10 feet on 11/9/78. No groundwater seepage encountered.</td>
<td></td>
</tr>
</tbody>
</table>
### Log of Test Pit 13

<table>
<thead>
<tr>
<th>Depth (ft.)</th>
<th>USCS</th>
<th>Soil Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>ML</td>
<td>Black TOPSOIL and SOD</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Gray, fine sandy SILT, medium dense, moist.</td>
</tr>
<tr>
<td>5</td>
<td>SM</td>
<td>Brown, gravelly silty SAND, medium dense, moist.</td>
</tr>
<tr>
<td>10</td>
<td>ML</td>
<td>Blue, sandy SILT with gravel, dense, moist.</td>
</tr>
</tbody>
</table>

Test pit terminated at 10 feet on 11/9/78. Minor groundwater seepage from 1.5 feet.

### Log of Test Pit 14

<table>
<thead>
<tr>
<th>Depth (ft.)</th>
<th>USCS</th>
<th>Soil Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td>Black TOPSOIL, loose, moist.</td>
</tr>
<tr>
<td></td>
<td>SM</td>
<td>Brown, gravelly silty SAND, medium dense, moist.</td>
</tr>
<tr>
<td>5</td>
<td>SM</td>
<td>Brown, silty fine to medium SAND, dense, moist.</td>
</tr>
</tbody>
</table>

Test pit terminated at 10 feet on 11/9/78. Groundwater seepage not encountered.
SELECTED TEST PIT LOGS

FROM

REPORT OF PRELIMINARY GEOTECHNICAL ENGINEERING STUDY
MAINGATE SHOPPING CENTER
REDMOND, WASHINGTON
1981

BY EARTH CONSULTANTS, INC.
### TEST PIT NO. 23

Logged By: GK  
Date: 1/16/81  
Elev.: 28±

<table>
<thead>
<tr>
<th>Depth (ft.)</th>
<th>USCS</th>
<th>Soil Description</th>
<th>W (%)</th>
<th>Lab Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>NH</td>
<td>(10&quot; SOD &amp; TOPSOIL) Light tan clayey SILT, very stiff, wet.</td>
<td>88.9</td>
<td>qu=3.75 tsf</td>
</tr>
<tr>
<td>5</td>
<td>Pt</td>
<td>Brown PEAT, firm, wet.</td>
<td>180</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>CH</td>
<td>Gray silty CLAY with sand seams, soft, wet.</td>
<td>33.2</td>
<td>qu=0.25 tsf</td>
</tr>
<tr>
<td>15</td>
<td>GP</td>
<td>Gray clean SAND with gravel, loose to medium dense, saturated.</td>
<td>17.7</td>
<td></td>
</tr>
</tbody>
</table>

Test pit terminated at 11 feet.  
Moderate seepage encountered from 3.5 to 6 feet, heavy seepage from 8.5 to 11 feet.  
Caving encountered from 6 to 11 feet.

### TEST PIT NO. 24

Logged By: GK  
Date: 1/16/81  
Elev.: 27±

<table>
<thead>
<tr>
<th>Depth (ft.)</th>
<th>USCS</th>
<th>Soil Description</th>
<th>W (%)</th>
<th>Lab Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>NH</td>
<td>(8&quot; SOD &amp; TOPSOIL) Light tan clayey SILT, very stiff, wet.</td>
<td>127</td>
<td>qu=2.75 tsf</td>
</tr>
<tr>
<td>5</td>
<td>Pt</td>
<td>Brown PEAT, firm, wet.</td>
<td>254</td>
<td></td>
</tr>
<tr>
<td>7.5</td>
<td>SP</td>
<td>Tan grading gray gravelly clean SAND, wet, to saturated, loose to medium dense.</td>
<td>18.9</td>
<td></td>
</tr>
</tbody>
</table>

Test pit terminated at 10 feet.  
Heavy seepage encountered from 8 to 10 feet.  
Caving encountered from 6 to 10 feet.
## TEST PIT NO. 25

Logged By: GK  
Date: 1/16/81  
Elev.: 50±

<table>
<thead>
<tr>
<th>Depth (ft.)</th>
<th>USCS</th>
<th>Soil Description</th>
<th>W (%)</th>
<th>Lab Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>GM</td>
<td>(8&quot; DUFF &amp; TOPSOIL) Red-tan silty sandy GRAVEL, medium dense with roots.</td>
<td>8.3</td>
<td></td>
</tr>
</tbody>
</table>
| 5          | HL   | Tan mottled SILT, hard, moist.                            | 24.4  | qu=4.5  
            |      |                                                           |       | tsf      
            |      |                                                           |       | LL=31    
            |      |                                                           |       | PL=27    |
| 10         | SH   | Red-tan silty fine SAND, dense, wet.                      | 23.8  |          |
| 15         |      | Test pit terminated at 11 feet.                          |       |          |
|            |      | Slight seepage encountered at 5 feet.                     |       |          |

## TEST PIT NO. 26

Logged By: GK  
Date: 1/16/81  
Elev.: 30±

<table>
<thead>
<tr>
<th>Depth (ft.)</th>
<th>USCS</th>
<th>Soil Description</th>
<th>W (%)</th>
<th>Lab Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>MH</td>
<td>(14&quot; SOD &amp; TOPSOIL) Light tan clayey SILT, very stiff, wet.</td>
<td>142</td>
<td>qu=3tsf</td>
</tr>
<tr>
<td>5</td>
<td>Pt</td>
<td>Brown PEAT, firm, wet.</td>
<td>267</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>SH</td>
<td>Gray silty fine SAND, medium dense, wet.</td>
<td>22.3</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>SP</td>
<td>Gray gravelly clean SAND, loose to medium dense, saturated.</td>
<td>11.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Test pit terminated at 11 feet.</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Slight seepage encountered at 6 feet.</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Heavy seepage encountered from 9 to 11 feet.</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cavino encountered from 9 to 11 feet.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### TEST PIT NO. 27

Logged By: GK  
Date: 1/16/81  
Elev. 29±

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>USCS</th>
<th>Soil Description</th>
<th>W (%)</th>
<th>Lab Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>MH</td>
<td>(8&quot; SOD &amp; TOPSOIL) Light tan clayey SILT, hard, wet.</td>
<td>89.2</td>
<td>qu=4.25 tsf</td>
</tr>
<tr>
<td>5</td>
<td>Pt</td>
<td>Brown PEAT, firm, wet.</td>
<td>170</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>SP S1</td>
<td>Tan gravelly SAND with silt, medium dense, moist.</td>
<td>9.3</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>SP</td>
<td>Gray gravelly clean SAND, loose to medium dense, saturated.</td>
<td>10.0</td>
<td></td>
</tr>
</tbody>
</table>

Test pit terminated at 11 feet.  
Heavy seepage encountered from 9.5 to 11 feet.  
Caving encountered from 9 to 11 feet.

### TEST PIT NO. 28

Logged By: GK  
Date: 1/16/81  
Elev. 28±

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>USCS</th>
<th>Soil Description</th>
<th>W (%)</th>
<th>Lab Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>MH</td>
<td>(12&quot; SOD &amp; TOPSOIL) Light tan clayey SILT, very stiff, wet.</td>
<td>88.3</td>
<td>qu=3.75 tsf</td>
</tr>
<tr>
<td>5</td>
<td>Pt</td>
<td>Brown PEAT, firm, wet.</td>
<td>287</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>GP</td>
<td>Red-tan sandy GRAVEL, loose to medium dense, moist grading saturated.</td>
<td>4.0</td>
<td></td>
</tr>
</tbody>
</table>

Test pit terminated at 10 feet.  
Heavy seepage encountered at 9 feet.  
Caving from 9 to 10 feet.
# TEST PIT NO. 29

Logged By: GK  
Date: 1/16/81  
Elev.: 20±

<table>
<thead>
<tr>
<th>Depth (ft.)</th>
<th>USCS</th>
<th>Soil Description</th>
<th>W (%)</th>
<th>Lab Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>MH</td>
<td>(7&quot; SOD &amp; TOPSOIL) Light brown clayey SILT with roots, very stiff, wet.</td>
<td>92.5</td>
<td>qu=3.75 tsf</td>
</tr>
<tr>
<td>5</td>
<td>OL</td>
<td>Brown peaty organic SILT, stiff, wet.</td>
<td>188</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>GP</td>
<td>Red-tan sandy GRAVEL, loose to medium dense, moist grading saturated.</td>
<td>7.4</td>
<td></td>
</tr>
</tbody>
</table>
| Test pit terminated at 9 feet.  
Heavy seepage encountered from 8 to 9 feet.  
Caving encountered from 8 to 9 feet. |

---

# TEST PIT NO. 30

Logged By: GK  
Date: 1/16/81  
Elev.: 23±

<table>
<thead>
<tr>
<th>Depth (ft.)</th>
<th>USCS</th>
<th>Soil Description</th>
<th>W (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>GP</td>
<td>(18&quot; SOD &amp; TOPSOIL)</td>
<td>92.2</td>
</tr>
</tbody>
</table>

Gray sandy GRAVEL, loose to medium dense, moist grading wet.

8.1

Test pit terminated at 7 feet.  
Heavy seepage encountered from 5 to 7 feet.  
Caving encountered from 5 to 7 feet.
Introduction

This memorandum describes the general criteria, preliminary analyses and recommendations for the proposed stormwater facilities and low-impact-development features for the 160th Avenue NE Extension Project. The purposes of the memorandum are to identify drainage design criteria to be used for the development of the preliminary design and summarize preliminary stormwater design strategies and basis for the feasibility and cost of stormwater for the project.

This project consists of extending 160th Avenue NE from the current terminus at NE 99th to NE 102nd Street and the intersection with Red-Wood Rd. The purpose of this phase of work is to identify the alternatives to be considered, evaluate the alternatives in enough detail so a preferred alternative can be selected, and develop budget level costs for the preferred alternative to be used for decision making and budgeting. Preliminary stormwater analysis will include determining the type, size and location of alternatives for flow control, water quality treatment, and conveyance facilities. Additionally, the feasibility of low-impact development techniques on the project site will be reviewed and a conceptual plan will be developed that outlines the suite of practices suitable to the site and locations where LID techniques are feasible.

Stormwater Regulations

Federal

Federal stormwater regulations are contained in the Clean Water Act and typically are promulgated through local stormwater requirements. Federal stormwater-related requirements and approvals for the Renton project will need to meet the requirements of Section 7 of the Endangered Species Act (ESA), which is regulated by the U.S. Department of Interior, National Marine Fisheries Service (NMFS) and U.S. Fish and Wildlife Service (USFWS).
**State**

If a project (new development or redevelopment on a single or multiple parcel site) has disturbed area exceed 1 acre, the project owner is required to file a Notice of Intent with the WSDOE for coverage under the National Pollutant Discharge Elimination System (NPDES) program’s General Permit for Stormwater Discharges Associated with Construction Activities. These filings would likely require the project to provide erosion control measures consistent with Ecology’s SMMWW.

Hydraulic Permit Approvals from the Washington Department of Fish and Wildlife (WDFW) may be necessary for in-water work required.

**City**


**Minimum Technical Requirements**

The 2005 Ecology Manual contains 9 minimum requirements. As a new roadway extension, this project is defined as new development, large project and anticipated to exceed the thresholds requiring compliance with Minimum Requirements #1 through #9 for all new plus replaced impervious surfaces. The following paragraphs list each requirement with a discussion of how it will be addressed for this project.

**Minimum Requirement 1 - Stormwater Site Plans**

A Stormwater Site Plan following the outline provided in Chapter 3 of Volume 1 of the 2005 Ecology Manual. The Site Plan shall document the BMP selection process for the project.

**Minimum Requirement 2 - Construction Stormwater Pollution Prevention Plan (CSWPPP)**

The project will be required to provide a Construction Stormwater Pollution Prevention Plan (SWPPP) as part of the Stormwater Site Plan. The SWPPP shall be implemented beginning with initial soil disturbance and until final stabilization. The SWPPP will be developed at later phases of the project.

**Minimum Requirement 3 - Source Control of Pollution**

Source control BMPs selected, designed and maintained in accordance with Volume IV of the Ecology Manual will be required.

**Minimum Requirement 4 - Preservation of Natural Drainage Systems and Outfalls**

The proposed project crosses 3 Class IV and 1 Class III (currently listed as Class IV but proposed to be re-classified) stream and will be required to maintain existing drainage patterns to the maximum extent practicable. Outfalls will require energy dissipation.
Minimum Requirement 5 - On-Site Stormwater Management
Projects are required to implement on-site stormwater management BMPs to infiltrate, disperse, and retain stormwater runoff on-site to the maximum extent feasible without causing flooding, groundwater contamination, or erosion impacts. All post-construction landscaped areas within the project area are required to have compost amended soils. The project is encouraged to use runoff reduction/on-site stormwater management techniques to meet flow control requirements. A site assessment to determine the applicability and feasibility of runoff reduction techniques is required.

Groundwater Protection
The majority of the project area is within Wellhead Protection Zone 4, therefore, runoff from pollution generating impervious surfaces can be infiltrated without treatment provided the soil profile provides treatment per Chapter 3.3 of Volume III of the 2005 Ecology Manual. The southern extent of the project where the roadway ties into 160th Avenue is within Wellhead Protection Zone 3. Therefore, runoff from pollution generating impervious surfaces can be infiltrated with treatment prior to infiltration based on land use (see Minimum Requirement #6). Native soils cannot be assumed to provide treatment.

Minimum Requirement 6 - Runoff Treatment
The project will require stormwater treatment of all pollution-generating impervious surfaces. Where discharge is to the ground, see provisions under Minimum Requirement #5 for treatment prior to discharge to native soils.

Treatment Level
Oil Control: Not Applicable

The project will not consist of any high-use intersections, therefore Oil Control will not be required.

Phosphorus Control: No Applicable

The project discharges downstream of Lake Sammamish

Enhanced Treatment: Applies (assumed)

The project discharges within a ¼-mile of a fish bearing stream (Sammamish River). Enhanced treatment for reduction of dissolved metals is required if the project AADT is exceeds 7,500.

Basic Treatment: Applies

Basic treatment is not necessary where the project infiltrates to native soils meeting treatment requirements in Wellhead Zone #4.

Minimum Requirement 7 - Flow Control
The project currently discharges to the Sammamish River within a ¼-mile downstream of the site. The currently downstream flowpath is, however, not completely manmade as the discharges flow via streams and/or wetlands prior to discharging via a culvert beneath the Sammamish River Trails which discharges to the River. Flow control is not required if the project discharges directly, or indirectly, to the Sammamish River subject to the following conditions:
• Direct discharge does not divert drainage from any perennial streams classified as Class 1, 2, 3 or 4 or from any Category I, II, or III wetland;

• Flow splitting devices are provided to route natural runoff volumes to any Class 4 intermittent stream or Category IV wetland, defined as matching the durations ranging from 50% of the 2-year to the 50-year peak flow as determined using a continuous hydrologic model.

• The conveyance system must be manmade and extend to the ordinary high water line of the Sammamish River; and

• The conveyance system has adequate capacity and erodible elements of the conveyance system are adequately stabilized to prevent erosion.

**Flow Control Levels:**
If the requirements for direct discharge and flow control exemption, as described above, cannot be met, then the project will be subject to providing flow control to the following levels.

**Flow Control Duration Standard – Forested Conditions**
The study area is currently mapped as a forested pre-condition. Stormwater discharges shall match the developed discharge durations to forested durations for the range of pre-development rates from 50% of the 2-year peak up to the full 50-year peak flow.

**Alternative Flow Control – Regional Facility Areas**
Regional Stormwater Facility 360P is currently mapped downstream of a portion of the study area. However, per the City of Redmond, this is a potential future facility and therefore will have no impact on the project.

**Minimum Requirement 8 – Wetlands Protection**
Much of the project site discharges to existing wetlands and will be required to meet the wetlands protection standard. The standard requires that discharges to the wetland shall maintain the hydrologic conditions, hydrophytic vegetation, and substrates necessary to support the existing and designated uses. The hydrologic analysis shall use the existing land cover condition to determine the existing hydrologic conditions unless otherwise directed. The wetlands themselves may be considered for hydrologic modification and/or stormwater treatment in accordance with Guide Sheet 1B in Appendix I-D.

**Wetland Buffers**
Stormwater treatment and flow control facilities shall not be built within a natural vegetated buffer, except for:

• Necessary conveyance systems or as allowed in wetlands approved for hydrologic modification and/or treatment.

**Minimum Requirement 9 – Operation and Maintenance**
Final design for the project will require the preparation of an operations and maintenance manual.
**Approach**

Due to limited data and development of the preferred project alternative, the preliminary stormwater analysis focuses on identifying the site constraints and opportunities for stormwater management and recommends a tiered approach to managing stormwater as the design is developed further.

**Site Analysis**

The site is characterized by numerous critical areas that will drive the feasibility of various stormwater BMPs.

**Slopes**

The project area is dominated by steep slopes that form the Sammamish River valley wall.

**Soils**

Preliminary geotechnical investigations characterize the general physiographic and soil conditions as follows.

- Sammamish Valley Wall. The existing site soils on upland areas consist primarily of highly permeable, but erosive, glacial outwash soils. It may be possible to infiltrate runoff within these soils, however, care shall be taken to minimize directing runoff that may create erosion or slope stability issues. The underlying soils or lowland portions of the site (approximately below elevation 80) on the site are characterized by a native soil consisting of transitional beds which are dominated by silts and may contain perched groundwater.

- Drainage Crossings and Wetlands. Drainages crossed by the project consist primarily of:

- Sammamish Valley Floor. Soils in this area at the southern extents of the project are expected to consist of topsoil underlain by deep deposits of soft silt and sand, with peat in some areas, overlying loose to dense granular soils and hard silt at depth.

**Streams and Wetlands**

Four drainages, all Class IV streams, and four wetlands, either Category III or Category IV, were identified within the study area. It will be critical to minimize disturbance to these resources, maintain existing site hydrology and provide upstream controls to minimize the discharge of sediment, excess flow and pollutants to these resources.

**Preliminary Recommendations**

**Erosion Control**

Due to the steep slopes and highly erosive nature of the existing site soils, erosion control during construction will be critical. To the extent practical, the limits of site disturbance shall be kept to the minimum extents necessary to complete the construction of the proposed roadway extension.

- **Preserve Vegetation/Mark Clearing Limits.** Existing site vegetation shall be maintained and limits of work be clearly marked to minimize excessive removal of vegetation from the site.

- **Establish Construction Access.** Due to the surrounding critical areas and difficulty of accessing the site, access to the project should be limited to discrete entrance and exit points to preserve the native soils, vegetation and critical areas surrounding the project. Construction staging should be carefully planned to minimize the necessary disturbance to accomplish site work. The access points to public roads at the north and south ends of the project should be protected with stabilized entrances, wheel washing and street sweeping as necessary.
Control Flow Rates. Construction flows from the project site should be controlled in such a manner that they do not discharge excessive flows to natural water features downstream of the project and create the risk of erosion and turbidity.

Install Sediment Controls. All runoff from the construction site shall be directed through an approved sediment control BMP (e.g., filters, ponds, traps, silt fences, etc.).

Stabilize Soils. As noted above the site soils are highly erosive and shall be stabilized if disturbed and left unworked for an extended period. Due to the sensitivity of the site, work during the wet season should be limited.

Protect Slopes. The project will require extensive grading of site slopes. Grading work and retaining walls shall be designed to minimize erosion by directing runoff away from slopes or disturbed areas. Slope cover such as straw and/or jute matting, erosion control blankets, interceptor dikes and swales and vegetation should be used.

Protect Drain Inlets. Existing drainage inlets primarily exist at the south end of the project in NE 160th Avenue. As this is the most downstream portion of the site, it is important to provide inlet protection at these locations.

Stabilize Channel and Outlets. Channel linings should be used to catch sediment and control downcutting and surface erosion; energy dissipaters should be used for trenches as required for controlling the velocity along the flow channels. All concentrated stormwater discharge points should be stabilized using rock pads or dispersion trenches to minimize point erosion.

Control Pollutants. Construction should design, install, implement and maintain effective pollution prevention measures to minimize the discharge of pollutants.

Control De-Watering. Due to the presence of groundwater seeps, dewatering is to be expected on this project. Dewatering water shall be controlled to discharge only clean, non-turbid de-watering water and otherwise handle and treat turbid water in accordance with standards.

Maintain BMPs. Proper erosion control requires on-going maintenance of both temporary and permanent erosion control BMPs.

Manage the Project. Erosion control on the project shall be proactively managed by maintaining an updated construction SWPPP, on-going inspections by a CESCL and continually manage and adapt erosion control on the project.

Water Quality

All stormwater runoff from pollution-generating surfaces shall be treated prior to discharge. The tiered approach below places a priority on green infrastructure or low-impact development techniques that not only provide water quality but provide additional runoff reduction and provide additional landscape benefits to the roadway.

Tier 1: Where possible, final roadway sections should consider separating sidewalk and non-pollution surfaces from pollution-generating surfaces by provide onset crowns or reverse slopes. Additionally, porous sidewalks may be considered to directly infiltrate runoff into the underlying road subgrade (note: due to the steep longitudinal slope of the proposed roadway, check dams may be necessary to prevent lateral flow in the subgrade).

Tier 2: The preliminary stormwater concept consists primarily of bioretention planters within the planter strips to filter and to the extent practical infiltrate runoff directly below the planters or through connection to infiltration trenches beneath the roadway subgrade.

Tier 3: Where the above techniques are not feasible, the stormwater runoff may be mitigated by directing flow through Filterras or other approved proprietary tree/planter box filtration devices.

Tier 4: As a last resort, runoff may be treated via underground filter cartridge vaults, catch basins or other methods.
Flow Control

The proposed project will at a minimum maintain existing site hydrology to drainage crossing and adjacent wetlands. Where feasible, the project may directly discharge to the Sammamish River without flow control, provided it can be demonstrated that the conveyance system to the River is either completely manmade or has negligible impacts to the hydroperiod of existing wetlands.

Tier 1: The stormwater will infiltrate runoff to the maximum extent feasible within the project area through either rain gardens or infiltration trenches located at the base of structures.

Tier 2: Where infiltration alone cannot meet the flow control requirements, the project may consider modification or modeling of the existing downstream wetlands and conveyance system to demonstrate negligible hydrologic impact to the wetlands and suitable conveyance capacity to meet the combined requirements for direct discharge and wetlands protection. Further study for this option is required in future design development stages.

Tier 3: Where infiltration and discharge to wetlands is not possible, flow control will be provided via subsurface vaults or adjacent detention ponds.

References

City of Redmond, Municipal Code


Purpose

The purpose of this technical memorandum is to conduct a planning-level assessment of existing wetland and stream conditions for the preliminary design and feasibility of extending 160th Avenue NE from its current terminus at NE 99th northward to NE 102nd Street. This technical memorandum does not provide any information on potential impacts to wetlands or wetland buffers, or any associated mitigation. A complete wetland and stream delineation report will be prepared at a later phase of the proposed project as well as information on the impacts to wetlands, wetland buffers, streams, and stream buffers, and associated mitigation.

Summary

The work completed as part of this assessment did not include a complete wetland delineation. Only the boundaries of the wetlands within the study area were identified to help guide project engineers to avoid and minimize potential impacts to wetlands and streams. The study area is defined by an approximately 200-foot-wide corridor centered on the proposed roadway alignment.

On March 14, 2012, biologists from CH2M HILL assessed the streams and wetlands within the project study area. Randy Whitman assessed the streams. Hans Ehlert and Megan Karl assessed the wetlands, and flagged the approximate boundaries of Wetlands 1A, 2A, and 4A. ESA Adolfson had previously delineated Wetland A (see Reid Middleton 2009), which was renamed Wetland 3A for consistency with this memo. Four drainages, all Class IV streams, and four wetlands, either Category III or Category IV, were identified within the study area. Three of the wetlands are narrow slope wetlands located within the drainages adjacent to streams. The fourth wetland is a depressional wetland that is part of the larger system located on the historical floodplain of the Sammamish River valley. Figure A provides information on the streams and wetlands identified in the study area.
FIGURE A
REDMOND 160TH AVE EXTENSION - EXISTING WETLAND LOCATIONS
Previous Studies

Previous studies of wetlands in the study area were used as a starting point for the current assessment. The previous studies included:

- **Final Supplemental Environmental Impact Statement, 160th Avenue NE Road Extension prepared for the City of Redmond by Parametrix in 1999 (Parametrix 1999).** This was a planning-level report and did not include detailed information that could be used for design and the information was developed more than 13 years ago. Residential developments have since been constructed at the north and south ends of the proposed new road extension and have likely modified the existing site conditions described in the report.

- **PSE Phase 3 Trail Improvement Project Sammamish River Trail to SR 202, Design Report prepared for the City of Redmond by Reid Middleton in 2009 (Reid Middleton 2009).** This study includes detailed information for a portion of the project study area, such as wetland delineation, wetland rating, and stream classification within the PSE power line easement.

Streams

The following stream descriptions are organized geographically, starting from the south end of the alignment and proceeding northward. The four streams in the study area are summarized in Table A, and illustrated in Figure A.

**Stream 1**

Stream 1 is a very small high gradient, intermittent watercourse. It is a Class IV stream and non-fish bearing. The watercourse drains from Redmond Woodinville Road NE westward to a drain next to 160th Avenue NE. Under extreme flow conditions, it appears that some of the flow could bypass the drain and continue flow down the hill slope where it would empty into a wetland and infiltrate into the Sammamish River floodplain. Upstream of 160th Avenue NE, the watercourse is buried under leaves and other organic debris and vegetation.

Construction of 160th Avenue NE and the nearby residential development appears to have diverted the natural drainage into a pipe system that flows to an infiltration pit in the floodplain wetland at the bottom of the slope. An overflow pipe appears to have been constructed within the intermittent watercourse.

**Stream 2**

Stream 2 is a small, high gradient, intermittent, Class IV stream and is non-fish bearing. During the stream evaluation on March 14, 2012, flow was visually estimated to be approximately 0.1 cubic feet per second (cfs) with a wetted channel width of approximately 1.0 foot. The bankfull width was estimated to be approximately 2.0 feet wide near the approximate alignment centerline. The approximate centerline of the alignment is situated at a distinct break in channel gradient, with a shallower slope immediately upstream and
steeper slope downstream. The channel also becomes confined in a ravine to the downstream side. Channel substrate consists of gravel, sand, and organic materials. Riparian vegetation consists of reed canarygrass and blackberry upstream of the centerline, and Douglas-fir, western red cedar, big leaf maple, blackberry, and salmonberry to the downstream side.

Stream 3
Stream 3 is located in the ravine immediately to the south of the PSE powerline trail. It is a high gradient, Class IV stream and is non-fish bearing. The flow regime (intermittent vs. perennial) could not be determined at the time of the site visit. The riparian vegetation in the ravine consists of Douglas-fir, western red cedar, red alder, big leaf maple, and salmonberry. During the stream evaluation on March 14, 2012, the wetted channel width was approximately 2.0 feet with a flow estimated to be approximately 0.1 cfs. The bankfull width was estimated to be approximately 3.0 feet. Substrate consists of gravel and sand.

Stream 4
Stream 4 is the largest of the four water courses within the proposed 160th Avenue NE extension alignment. It is a high gradient, Class IV stream and is non-fish bearing. The flow regime (intermittent vs. perennial) could not be determined at the time of the site visit. During the stream evaluation on March 14, 2012, the flow was approximately 0.5 cfs, having a wetted width of approximately 5 - 6 feet. Bankfull width was approximately 7.0 feet at the location examined, which was within the alignment but uncertain with regard to the centerline. Depth ranged from 2 to 6 inches deep. Substrate consists of cobbles, boulders, gravel, and sand. Sand loading appeared to be excessive. The channel had abundant small woody debris. The riparian vegetation consists of western red cedar, red alder, big leaf maple, sword fern, and salmonberry.

<table>
<thead>
<tr>
<th>Stream Name</th>
<th>Stream Class</th>
<th>Flow Regime</th>
<th>Stream Buffer Width (feet)</th>
<th>Approximate Bankfull Width (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stream 1</td>
<td>Class IV</td>
<td>Intermittent</td>
<td>25</td>
<td>-</td>
</tr>
<tr>
<td>Stream 2</td>
<td>Class IV</td>
<td>Intermittent</td>
<td>25</td>
<td>2.0</td>
</tr>
<tr>
<td>Stream 3</td>
<td>Class IV</td>
<td>Indeterminant (Intermittent or Perennial)</td>
<td>25-36</td>
<td>3.0</td>
</tr>
<tr>
<td>Stream 4</td>
<td>Class IV</td>
<td>Indeterminant (Intermittent or Perennial)</td>
<td>25-36</td>
<td>7.0</td>
</tr>
</tbody>
</table>

Source: City of Redmond 2011

* Stream buffer width required by RZC Article IV Environmental Regulations.
Wetlands

The following wetland descriptions are organized geographically, starting from the south end of the alignment and proceeding northward. The wetlands are summarized in Table B, and shown on Figure A. Table C provides information on the wetland replacement ratios for the wetlands in the study area. Additional information on wetland impacts and mitigation related to replacement for any impacts will be performed at a later phase of the proposed project. The intent of this memorandum is to provide an assessment of the existing condition of wetlands in the study area.

Appendix A: Wetland Photos provides photos of the four wetlands, and Appendix B: Wetland Rating Forms provides information on the wetland work completed as part of the site visit. In addition, the following available information was collected and reviewed prior to visiting the site:

- **Topography.** Topography slopes to the west. The areas of lowest elevation onsite are located in the western portion of the study area.

- **Soils.** Soils mapped onsite are Alderwood gravelly sandy loam, 15-30% slopes (AgD), Kitsap silt loam, 2-8% slopes (KpB), and Kitsap silt loam, 8-15% slopes (KpD). None of these are listed as hydric soils.

- **City of Redmond Municipal Code Chapter 21.64, Critical Areas.** Chapter provides information on streams and wetlands in the City of Redmond including classification and buffer requirements.

- **City of Redmond Municipal Code Chapter 21.64, Critical Areas, Map 64.4 Wetlands.** The map shows the entire study area as either “mixed wetland/upland” or “wetland”.

- **Previous studies.** Parametrix (1999) shows wetlands associated with the four drainages onsite. Reid Middleton (2009) includes a delineation of the wetland within the Puget Sound Energy (PSE) power line easement, which bisects the study area.

- **Aerial Photography with available basemap** (showing project alignment, study area limits, drainage improvements, and previously mapped streams and wetlands).

**Wetland 1A**

Wetland 1A is located at the southwestern portion of the proposed alignment (Figure A). It is a relatively large Category III depressional wetland that is located on the historical floodplain of the Sammamish River Valley. Wetland 1A becomes inundated from late winter through spring and drains via a culvert to the Sammamish River. Wetland 1A is dominated by emergent and scrub-shrub vegetation such as reed canarygrass, willow, dogwood, and red alder (see Appendix A: Wetland Photos). Vegetation within the within the PSE powerline easement is regularly disturbed by PSE vegetation management. This wetland is bounded by the Sammamish River Trail to the west, the PSE Powerline Trail to the north, the hillside to the east, and an apartment complex to the south. The portion of the wetland buffer that is within the study corridor is relatively well vegetated hillslope with various native trees and shrubs, and invasive blackberry vines.
Wetland 2A

Wetland 2A is located just south of the center portion of the proposed alignment (Figure A). It is a relatively small (around 0.25 acre) Category IV slope wetland that is located on a slope and drains into Wetland 1A which is in the floodplain of the Sammamish River Valley. Wetland 2A is dominated by emergent and scrub-shrub vegetation such as reed canarygrass and salmon berry (see Appendix A: Wetland Photos). Numerous sedge plants and skunk cabbage were observed as well. This wetland is bounded by draining into Wetland 1A to the southwest, the PSE Powerline Trail to the north, and the hillside to the east. The portion of the wetland buffer that is within the study corridor is relatively well vegetated hillslope with various native trees and shrubs in the north, south and west, and invasive blackberry vines to the east.

Wetland 3A

The delineation and wetland rating performed by ESA Adolfson appears to be reasonably correct and was adopted for purposes of this wetland assessment. According to ESA Adolfson data, Wetland 3A is located just north of the center portion of the proposed alignment just south of the PSE Powerline Trail (Figure A). It is a relatively small (less than 1 acre) Category III slope wetland that is located on a slope and drains into Wetland 1A which is in the floodplain of the Sammamish River Valley. Wetland 3A is dominated by emergent and scrub-shrub vegetation such as alder, willow, reed canarygrass, salmon berry, bulrush, and stinging nettle (see Appendix A: Wetland Photos). This wetland is bounded by draining into Wetland 1A to the south, the PSE Powerline Trail to the north, and the hillside to the west and east. The portion of the wetland buffer that is within the study corridor is relatively well vegetated hillslope with various native trees and shrubs.

Wetland 4A

Wetland 4A is located in the northeastern portion of the proposed alignment (Figure A). It is a relatively small (less than 0.25 acre) Category III slope wetland that is located on a slope and has Stream 4 running through it. Wetland 4A is dominated by emergent and scrub-shrub vegetation such as reed canarygrass, alder, willow, ladyfern, and salmon berry (see Appendix A: Wetland Photos). This wetland is bounded by a disturbed area and the southern edge of 156th Ave NE to the north, the PSE Powerline Trail to the south, and the hillside to the east and west. The portion of the wetland buffer that is within the study corridor is relatively well vegetated hillslope with various native trees and shrubs in the east, south, and west, and invasive blackberry vines to the north.
### TABLE B
Cowardin Class, HGM Class, Category, and Buffer Width of Wetlands Located in the Study Area

<table>
<thead>
<tr>
<th>Wetland ID</th>
<th>Cowardin Class</th>
<th>HGM Class</th>
<th>Category</th>
<th>Wetland Buffer Width (feet)</th>
<th>Stream Present</th>
<th>Source of Wetland Boundary Information</th>
<th>Wetland Rated by</th>
</tr>
</thead>
<tbody>
<tr>
<td>1A</td>
<td>PEM/PSS</td>
<td>Depressional</td>
<td>3</td>
<td>80</td>
<td>No</td>
<td>CH2M HILL 2012</td>
<td>CH2M HILL 2012</td>
</tr>
<tr>
<td>2A</td>
<td>PEM/PSS</td>
<td>Slope</td>
<td>4</td>
<td>50</td>
<td>Yes</td>
<td>CH2M HILL 2012</td>
<td>CH2M HILL 2012</td>
</tr>
<tr>
<td>3A</td>
<td>PEM/PSS/PFO</td>
<td>Slope</td>
<td>3</td>
<td>80</td>
<td>Yes</td>
<td>Reid Middleton 2009</td>
<td>Reid Middleton 2009</td>
</tr>
<tr>
<td>4A</td>
<td>PEM/PSS/PFO</td>
<td>Slope</td>
<td>3</td>
<td>80</td>
<td>Yes</td>
<td>CH2M HILL 2012</td>
<td>CH2M HILL 2012</td>
</tr>
</tbody>
</table>

* Category is based on Ecology’s rating system (Hruby, 2004), which the City of Redmond adopted without modification.

* Buffer width required by Critical Areas Ordinance for City of Redmond. Assumes the proposed roadway is considered a high-impact land use, which has the highest buffer width. Source: City of Redmond 2011.

PEM = palustrine emergent marsh; PSS = palustrine scrub-shrub; PFO= palustrine forested

### TABLE C
Wetland Replacement Ratios for Wetlands in the Study Area

<table>
<thead>
<tr>
<th>Wetland Category</th>
<th>Creation or Re-establishment</th>
<th>Rehabilitation (Restoration)</th>
<th>Re-establishment or Creation (R/C) and Enhancement</th>
<th>Enhancement Only</th>
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</thead>
<tbody>
<tr>
<td>III</td>
<td>2:1</td>
<td>4:1</td>
<td>1:1 R/C and 2:1 E</td>
<td>8:1</td>
</tr>
<tr>
<td>IV</td>
<td>1.5:1</td>
<td>3:1</td>
<td>1:1 R/C and 2:1 E</td>
<td>6:1</td>
</tr>
</tbody>
</table>

Source: City of Redmond 2011
References


City of Redmond. 2011. Critical Areas Map 64.1 Fish and Wildlife Habitat Conservation Areas.

City of Redmond. 2011. Critical Areas Map 64.3 Streams Classification.

City of Redmond. 2011. Critical Areas Map 64.4 Wetlands.


