Gonzaga University
School of Engineering

Final Project Report
For

WSDOT US 195 Hangman Valley City Street Network

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Sponsor
Washington State Department of Transportation

Center for Engineering Design
Project CE - 08

April 22, 2010
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WSDOT US 195 Hangman Valley City Street Network

Center for Engineering Design
Project CE-08

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April 22, 2010
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Section 1.0 Introduction

The Hangman Valley City Street Network project consists of the construction of an arterial along the western edge of the US 195 corridor. This new arterial will eliminate access points along US 195 from Interstate 90 in the north to Cheney Spokane Road in the south (Figure 1.0). By doing so, the new city street network will relieve congestion on US 195 as well as improve safety in the Hangman Valley corridor. The, presently unnamed, “new city street” (referred to herein) has already been conceptually designed but requires a detailed hydraulics report for pavement drainage and stormwater management, pavement design, and economically viable alternatives for the project.

Figure 1.0 – Project Vicinity Map
Section 1.1 Project Need

The Hangman Valley along US 195 has seen large growth in the last decade. This location is one of the largest undeveloped areas both within the urban growth boundary and is close to downtown Spokane. With this growth comes an increase in the amount of traffic traveling to and from the metropolitan area of Spokane. The addition of a new city street will allow US 195 to remain free of traffic signals, thus improving traffic flow and increasing safety.

The financial burden of the new road is yet to be fully determined. The State of Washington has existing funds in place, but these funds are not enough to cover the entire project. The rest of the necessary funds are expected to be handled through federal money, such as bonds, as well as new state and local monies. These funds would need to be assessed through new or existing taxes on the residents of Spokane or Spokane County.

Constructing the new city streets network project will increase the safety for commuters along US-195 in the Hangman Valley by decreasing local traffic commuters on US-195. The new city streets network will eliminate present access points to said highway through use of collectors, overpasses, etc.

Section 2.0 Scope of Project

The Washington State Department of Transportation (WSDOT) has a conceptual design of the alignment for the new city street shown in Figure 2.1 and Figure 2.2. Since the conceptual design has been completed, the Gonzaga engineering design team will focus their efforts on several tasks. These tasks include: an offsite hydrologic analysis, a hydrologic analysis within the project limits, and hydraulic stormwater management design. Additional work for this project will consist of determining cost estimates for the hydraulic work. A detailed hydraulic report will be completed and submitted to WSDOT as the final element of this project outlining the design methodologies incorporated within the project design.

Within the hydraulic stormwater management design for this project an evaluation of the roadway cross-section design for the typical cut and fill sections will be examined. WSDOT has provided a possible view of the appropriate lane distribution on the roadway and this will be a starting point for the cross-section design (Figure 2.3). Different options for the roadway will be the adjustment of bicycle lanes, sidewalks or pedestrian areas, swale placement and use, and aesthetic properties of the roadway.

This project will look at a couple sustainable options for treatment of stormwater runoff. One sustainable option is incorporating recycled tires into the asphalt. Another possibility is using glass aggregate in the road. In addition, the Engineering design team will investigate sustainable options related to stormwater runoff such as utilizing water in an evaporation pond for irrigation of landscaped property.
The project will incorporate the appropriate design standards for this work. The two major design standards that will guide the design for this project are the Spokane Regional Stormwater Manual (hydrologic and hydraulic analysis) and the City of Spokane Department of Engineering Services Design Standards (pavement design). Additional standards referenced will include:

- WSDOT Hydraulics Manual
- WSDOT Highway Runoff Manual
- American Association of State Highway and Transportation Officials (AASHTO)

Figure 2.1 - Proposed New City Street, I-90 to just South of Thorpe Road (WSDOT)
Figure 2.2 - Proposed New City Street, Thorpe Road to Cheney-Spokane Road (WSDOT)

Figure 2.3 – Proposed Architectural Cut-and-Fill Section (WSDOT)
Section 3.0 Project Analysis and Design

Section 3.1 Hydraulic Report

This Hydraulic Report documents the following permanent hydraulic/hydrologic improvements, permanent erosion control measures, and the other related modifications to be constructed as a result of constructing the new road system:

- Curb and gutters constructed along both sides of the roadway to convey runoff into swales.

- Swales placed along both sides of the new road and will be lined with natural grasses. These swales will aid in runoff treatment and help to convey excess water to drywells.

- Drywells are located in the swales to provide excess water disposal through infiltration.

Funding

Funding of this project is still being developed and is not available at the time of this report.

Section 3.1.1 Site Location

The site is located within Spokane County, just west of the existing Highway 195 from Interstate 90 (North) to Cheney-Spokane Road (South). The new roadbed will exist over the top of the existing Fish Lake Trail from station 140+21.161 to station 67+40 and then continue to station 20+00 at Cheney-Spokane Road.

Section 3.1.2 Vicinity Map

Figure 1.0 identifies the regional area of the project and Figures 2.1 and 2.2 provide a more detailed view.

Section 3.1.3 Scope of Work

This project involves the construction of a new roadway at the locations described in Section 1.1. The hydraulic features installed on the project site are curbs and gutters, swales along the roadway and drywells installed in the swale bottoms in various locations. The curb and gutter assemblies will occur along both sides of the entire length of the new road (approximately 2.28 miles). The curb will have curb inlets at approximately 300 foot intervals to allow for roadway drainage into the swales. The swales are designed in a trapezoidal shape and will also be on both sides of the road and along the entire length. They will allow for initial treatment of stormwater as well as conveyance of excess runoff into installed drywells. The 10 foot deep drywells will be
installed at various intervals (see Appendix A-4) in the bottom of swales to allow for additional stormwater runoff.

Section 3.2.0 Site Conditions

The existing project site is located on a hillside that slopes from the West (greater elevation) to the East (lower elevation). The road bed is approximately one quarter of the way up the hill and is higher in elevation than the existing US Highway 195. Groundwater drains from the West to the East and seeps out just east of the existing Thorpe Road and Fish Lake Trail intersection. This surface water flows through culvert #3 (Figure 3.1) under US Highway 195 where it merges with Hangman creek due East of Thorpe Road.

![Figure 3.1 – Existing Culvert Locations within Project Limits](image)

Section 3.2.1 Existing Conditions

The project site lies within the Hangman Water Resource Inventory Area (WRIA) number 56. The overall existing drainage basin for this project is the Hangman Creek Basin. This project site drains into Hangman Creek which flows into the Spokane River. Within the project limits there are no threshold discharge areas.
Section 3.2.2 Existing Hydraulic Features

The impact of the new road to the existing offsite drainage is minimal because of the swale and drywell design along the new roadway. There will not be any stormwater contribution to the culvert because the surface water flow, originating from the basin groundwater, does not come to the surface until it is downhill of the planned roadway. There are no plans for a piping network that would contribute to this culvert’s flow.

3.2.2.A EXISTING CULVERTS

There are two existing drainage feature within the project limits (see Figure 3.1). The first is a nine foot diameter culvert that carries the project basin’s groundwater underneath US Highway 195 where it is then discharged into Hangman Creek (Culvert #3). This culvert is in good working condition and there are no plans to replace or upgrade it. The culvert carries approximately 141.7 cubic feet per second peak-flow from the drainage area due to rainfall runoff.

Culvert location: Station 124+50.00

The second feature is a five foot concrete culvert that runs perpendicular to the proposed Fish Lake Road (Culvert #2). It is located just south of and parallel to Thorpe Road. This culvert is designed to carry offsite stormwater underneath Fish Lake Trail where it can then be merged with the above mentioned nine foot culvert. Upon inspection, the culvert was found to be in good working condition and did not appear to have any flow through it in many years. It was constructed in 1906 when Fish Lake Trail was an active railroad.

Culvert location: Station 125+20.00

3.2.2.B EXISTING STREAM CROSSING

There is one stream crossings on this project which consists of the nine-foot culvert location identified in Section 3.2.2.A.

3.2.2.C EXISTING DITCHES AND OPEN CHANNEL FLOW

There are no roadside ditches within the limits of this project because no roads currently exist where Fish Lake Trail lies. At the Thorpe Road intersection, there are no roadside ditches along Thorpe because of the Fish Lake Trail overpass. Also, there are no open channel systems within the limits of the project.

3.2.2.D EXISTING ENCLOSED DRAINAGE SYSTEM

There are no existing enclosed drainage systems within the limits of this project.
3.2.2.E  EXISTING BRIDGES AND BRIDGE DRAINS

There are no bridge crossings within the project limits. Currently there are two overpasses that have been integrated into Fish Lake Trail that cross at 16th Avenue and Thorpe Road. These overpasses will be removed. The existing ground where Fish Lake Trail runs will be graded down to the level of 16th Avenue and Thorpe Road at their respective intersections.

A natural grade separation is created by the active Burlington Northern Sante Fe Rail Road system that is adjacent to US 195.

3.2.2.F  EXISTING FLOOD PLAINS

The project is remote from a flood plain. The project limits are located on a hillside located outside of the designated flood plain boundaries of Hangman Creek.

3.2.2.G  EXISTING SUBSURFACE DRAINAGE

There is only one identifiable groundwater drainage feature within the limits of the project. It is located at the Thorpe Road and Fish Lake Trail intersection and carries some of the groundwater that seeps out under US 195 and merges with Hangman Creek (identified as Culvert #3 – Figure 3.1).

Section 3.2.3  Threshold Discharge Areas (TDAs)

Threshold Discharge Areas are a requirement as part of WSDOT roadway design. This proposed project is being designed for future ownership by the City of Spokane. Following discussion with WSDOT Eastern Region’s Hydraulics Engineer, TDAs are not applicable to this project.

Section 3.2.4  Soils

3.2.4.1  pH AND RESISTIVITY

pH and resistivity measurements are not required because no new pipes are proposed for installation as a result of the proposed work on this project.

3.2.4.2  SOIL BORINGS

Design soil borings have not been conducted at this time. A soil sample has been obtained from a hole at the intersection of Thorpe Road and the Fish Lake Trail crossing to a depth approximately two feet. This soil sample was taken in order to further assist in estimating the hydraulic conductivity of the soils within the project location. Additional soil boring samples are recommended before construction is to begin.

3.2.4.3  SOIL TYPES
The major soil type within project limits, as identified in the Soil Survey of Spokane County, is Marble Variant Sandy Loam. Other soil types identified within the project limits, and ranked by relative predominance are Marble Loamy Sand, and Hesseltine Stony Loam. Figure 3.2 shows the soil map with approximate limits of the project location and soils distribution.

Figure 3.2 – Hydrologic Soil Types Map
Figure 3.3 – Soils Map Legend

Table 3.1 – Hydrologic Soil Types

<table>
<thead>
<tr>
<th>Map Unit Symbol</th>
<th>Map Unit Name</th>
<th>Soil Type</th>
<th>Acre</th>
<th>Percent of Total</th>
</tr>
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<tbody>
<tr>
<td>MaC</td>
<td>Marble loamy sand</td>
<td>A</td>
<td>87.3</td>
<td>26.6%</td>
</tr>
<tr>
<td>SzE</td>
<td>Springdale gravelly loamy sand</td>
<td>A</td>
<td>32.3</td>
<td>9.8%</td>
</tr>
<tr>
<td>HhA</td>
<td>Hardesty silt loam</td>
<td>B</td>
<td>10.4</td>
<td>3.2%</td>
</tr>
<tr>
<td>HoB</td>
<td>Hesseltine silt loam</td>
<td>B</td>
<td>0.01</td>
<td>0.0%</td>
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<tr>
<td>HsB</td>
<td>Hesseltine stony loam</td>
<td>B</td>
<td>42.9</td>
<td>13.0%</td>
</tr>
<tr>
<td>McB</td>
<td>Marble variant sandy loam</td>
<td>B</td>
<td>115.6</td>
<td>35.1%</td>
</tr>
<tr>
<td>BbB</td>
<td>Bernhill silt loam</td>
<td>C</td>
<td>8.8</td>
<td>2.7%</td>
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<tr>
<td>HvC</td>
<td>Hesseltine very rocky complex</td>
<td>D</td>
<td>31.5</td>
<td>9.6%</td>
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<td><strong>328.81</strong></td>
<td><strong>100.0%</strong></td>
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(1) Output from National Soil Survey, February 28 2010
Figure 3.2 delineates an area of interest that encompasses the majority of the area within project limits. A small section of land to the north of the area of interest to Interstate 90 is within the project limits but is not included in Figure 3.2. The hydrologic soil types are not identified for this area because it is a dense residential area. A summary of the various soil types is shown in Table 3.1.

Hydrologic soil groups are based on estimates of potential runoff. Runoff potential is depicted on the rates of infiltration and ponding within different types of soils. Soil types A and B have high to moderate infiltration rates with low runoff potential. As seen from the above table, approximately 36% is soil type A, 51% is soil type B, and 12% is types C and D. The majority of the soils within the project limits is type A and B and therefore will have minimal runoff. An official geotechnical investigation is recommended prior to construction of the new city street.

3.2.4.4 SOIL INFILTRATION RATES

Sieve analyses of a sample have been performed. The sample location is slightly north of Thorpe Road and to the east of the proposed city street. The results are presented in Table 3.2 and Chart 3.1.

<table>
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<th>Sieve Size</th>
<th>Percent Finer</th>
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<td>20</td>
<td>51.1%</td>
</tr>
<tr>
<td>40</td>
<td>17.4%</td>
</tr>
<tr>
<td>200</td>
<td>0.56%</td>
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</tbody>
</table>

Using the Spokane 200 Method the percent finer of the 200 sieve was found to be 0.56%. This shows the soil has a hydraulic conductivity of 0.5 cm per second, 708 inches per hour. The
Spokane Regional Stormwater Manual requires a minimum hydraulic conductivity of 0.5 inches per hour for swales. Concluding the soils within the project limits are extremely permeable.

3.2.4.5 WELL MONITORING AND GROUNDWATER LEVELS

There are four locations within the project limits where active wells exist. These locations are logged with Washington State Department of Ecology and are identified in Table 3.3.

Table 3.3 – Individual Water Wells within Project Limits

<table>
<thead>
<tr>
<th>Figure Location</th>
<th>Owner Name</th>
<th>Log ID</th>
<th>Diameter (inches)</th>
<th>Depth (feet)</th>
<th>Completion Date</th>
</tr>
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<tr>
<td>1</td>
<td>John Fritz</td>
<td>154943</td>
<td>8</td>
<td>69</td>
<td>11/22/1983</td>
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<tr>
<td>2</td>
<td>Tom Crider</td>
<td>159587</td>
<td>6</td>
<td>200</td>
<td>5/1/1992</td>
</tr>
<tr>
<td>3</td>
<td>Mike Cody</td>
<td>156680</td>
<td>6</td>
<td>500</td>
<td>5/7/1994</td>
</tr>
<tr>
<td>4</td>
<td>Bruce Norgan</td>
<td>150106</td>
<td>6</td>
<td>80</td>
<td>2/17/1976</td>
</tr>
<tr>
<td>5</td>
<td>Freeman Bartley</td>
<td>152819</td>
<td>6</td>
<td>135</td>
<td>5/2/1979</td>
</tr>
</tbody>
</table>

The wells are located in Township 42 North, Range 42 East, Section 36 as shown in Figure 3.4. The wells are located to the west, upslope, of the proposed city street and will not be affected.
3.2.4.6 OTHER SOIL TESTING

Upon completion of a geotechnical investigation, additional testing will be conducted in order to determine if the soil is suitable for disposal using bioinfiltration swales and/or contains any hazardous materials.

Section 3.2.5 Existing Stormwater Outfalls

Existing stormwater outfalls are identified and discussed within Section 2.2.A and Section 3.4.1 of this document. These culverts have been visually inspected during a site visit to the project location and are functionally stable; further examination may be deemed necessary.

Section 3.2.6 Existing Utilities

No known utilities have been identified within the project limits. Review of as-buils obtained from the utility companies is recommended before construction to identify any conflicts as a result of the proposed drainage work.
Section 3.3.0 Design Standards

There were two major design standards referenced to complete the design of the proposed project. They are:

(1) CITY OF SPOKANE Design Standards (2007)
(2) SPOKANE REGIONAL STORMWATER MANUAL (2008)

In addition to the above design standards, the following standards were used to aid the design:

- WSDOT Design Manual, Section 800
- WSDOT Highway Hydraulic Manual (September 2008 update)
- WSDOT Standard Plans and Specifications

Section 3.3.1 Design Frequency

To size the hydraulic features on this project different design frequencies are used. A 10-year rainfall, 24-hour storm is used to size the length of street between the curb inlets. To size the swale a 50-year rainfall, 24-hour storm is used. A 100-year rainfall, 24-hour storm sized the drywells and was used to check conveyance of the swales. These design storms were chosen to meet the requirements set forth by the Spokane Regional Storm Water Manual. Snow melt was not considered in the design. Figures 3.5 – 3.10 show the isopluvial and mean annual precipitation maps used in the Spokane Regional Stormwater Manual with the project location indicated in a black rectangle. Table 3.4 summarizes the precipitation indicated in the project location.

Table 3.4 – Design Storm Precipitation within Project Area and Use within the Project

<table>
<thead>
<tr>
<th>Storm Intensity</th>
<th>Precipitation (in)</th>
<th>Use within the Project Realms</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-year</td>
<td>1.2-1.4</td>
<td></td>
</tr>
<tr>
<td>10-year</td>
<td>1.6</td>
<td>Size distance between curb inlets</td>
</tr>
<tr>
<td>25-year</td>
<td>2.2</td>
<td></td>
</tr>
<tr>
<td>50-year</td>
<td>2.2</td>
<td>Size swales</td>
</tr>
<tr>
<td>100-year</td>
<td>2.4-2.6</td>
<td>Size dry wells, check conveyance in swales</td>
</tr>
<tr>
<td>Mean annual precipitation</td>
<td>17-18</td>
<td></td>
</tr>
</tbody>
</table>
Figure 3.6 – 10 year, 24 hour design depth
Figure 3.7 – 25 year, 24 hour design depth
Figure 3.8 – 50 year, 24 hour design depth
Figure 3.9 – 100 year, 24 hour design depth
Figure 3.10 – Mean Annual Precipitation
Section 3.3.2 Stormwater Management Guidelines

MINIMUM REQUIREMENT 1 – STORMWATER PLANNING
Stormwater planning has been conducted and accomplished using the design standards listed in Section 3.0 of this document.

MINIMUM REQUIREMENT 2 – CONSTRUCTION STORMWATER POLLUTION PREVENTION
Guidelines pertaining to stormwater pollution prevention during construction have not been addressed. BMP’s will be developed prior to construction of this project.

MINIMUM REQUIREMENT 3 – SOURCE CONTROL OF POLLUTANTS
Construction and post-construction source control of the highway system will be managed through operational and structural BMPs discussed in WSDOT’s Maintenance Manual and is the responsibility of the contractor. These methods will be identified prior to construction of the project.

MINIMUM REQUIREMENT 4 – MAINTAIN THE NATURAL DRAINAGE SYSTEM
The natural drainage system will be maintained for all offsite runoff. No additional impacts to the natural drainage system will occur.

MINIMUM REQUIREMENT 5 – RUNOFF TREATMENT
Runoff treatment is provided by allowing the first half inch of rainfall from a pollution generating impervious surface to be treated in the swale, as per the Spokane Regional Stormwater Manual (pgs 6-12). Required swale size was accomplished using the geometry of the swale and equation 6-1A, which takes into account infiltration.

This project will discharge to drywells. The drywell capacities are designed for a 50 year storm event and will be 6 inches above the bottom of the swale, to allow treatment of water from pollution generating impervious surfaces. The bioinfiltration swales will allow treatment of water coming off of impervious pollution generating surfaces without requiring any further water treatment.

Drywells will be used to handle the runoff after the first half-inch of rain that does not generate so much pollution. The drywells will be 10 feet deep and will have a disposal capacity of 1 cfs, the maximum allowed. This helps achieve a minimum number of drywells within the project. Use of drywells in this project will eliminate the need for costly evaporation ponds or conveyance to city storm sewers.

On sections of the road with a slope of more than 1 percent, a berm is required to hold the water to be treated. The maximum berm height is listed as 8 inches; however, we are requesting a variance to this to construct berms of a height of 9 inches. This is to provide a head of 3 inches for the drywell inlet. A drywell inlet with a head of 3 inches can accommodate 1.0 cfs flow while a drywell inlet with a head of 2 inches can only accommodate 0.5 cfs flow. These calculations are taken from equation 5.8(a) of the Washington State Highway Manual.
**Enhanced Water Quality Treatment**

This project is exempt from enhanced water quality treatment because future stormwater discharge will not impact present offsite drainage discharge into Hangman Creek. The stormwater discharge will be treated using bioinfiltration swales and evaporation ponds.

**Oil Control**

Oil Control is not required for this project. Oil Control is not required on this project because all intersections have less than 15,000 vehicles (ADT) as shown in traffic counts provided by the City of Spokane in Appendix A-8.

**Phosphorous**

Phosphorus water quality treatment is not required on this project at this time but may be required in the future based on increased development of the area and assumed use of fertilizer in new city lawns.

**MINIMUM REQUIREMENT 6 – FLOW CONTROL**

This project employs flow control using swales to provide flow control storage above the drywell inlet. The swales are sized for the proper treatment volume, and the drywells are designed based on a 50-year rainfall event. Using the Modified Rational Method, we find there is sufficient storage. The storage spreadsheet is shown in Table 3.5 which was calculated with the procedure used in Appendix A-2.

<table>
<thead>
<tr>
<th>t min</th>
<th>t Sec</th>
<th>I in/hr</th>
<th>Q cfs</th>
<th>V in cu ft</th>
<th>V out cu ft</th>
<th>Storage cu ft</th>
<th>Provided Storage cu ft</th>
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</thead>
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<td>5</td>
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<td>419.661974</td>
<td>300</td>
<td>119.6619742</td>
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</tr>
<tr>
<td>10</td>
<td>600</td>
<td>2.474977</td>
<td>0.672233</td>
<td>540.474947</td>
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<td>626.68439</td>
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<td>0.334617</td>
<td>807.095314</td>
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<td>853.808362</td>
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<td>-1246.191638</td>
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<td>40</td>
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<td>1.026274</td>
<td>0.279748</td>
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<td>2400</td>
<td>-1503.547125</td>
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<tr>
<td>45</td>
<td>2700</td>
<td>0.952316</td>
<td>0.256664</td>
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<td>-1764.167475</td>
<td>1084.6</td>
</tr>
<tr>
<td>50</td>
<td>3000</td>
<td>0.890687</td>
<td>0.241921</td>
<td>972.522414</td>
<td>3000</td>
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<tr>
<td>55</td>
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<td>0.227714</td>
<td>1006.9501</td>
<td>3300</td>
<td>-2293.0499</td>
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<td>60</td>
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<td>0.204795</td>
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<td>4200</td>
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<td>0.193581</td>
<td>1099.6041</td>
<td>4200</td>
<td>-3100.395995</td>
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<td>-3372.353623</td>
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<td>0.179497</td>
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<td>-3645.47481</td>
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<tr>
<td>85</td>
<td>5100</td>
<td>0.635902</td>
<td>0.172718</td>
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<td>5100</td>
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<td>0.166562</td>
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<td>0.592539</td>
<td>0.160944</td>
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<td>5700</td>
<td>-4470.737357</td>
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<td>6000</td>
<td>0.57355</td>
<td>0.155783</td>
<td>1252.49375</td>
<td>6000</td>
<td>-4747.506249</td>
<td>1084.6</td>
</tr>
</tbody>
</table>
MINIMUM REQUIREMENT 7 – WETLANDS PROTECTION

Stormwater runoff discharges will not occur to wetlands, either directly or indirectly, as a result of the proposed work form this project.

MINIMUM REQUIREMENT 8 – INCORPORATIONG WATERSHED/BASIN PLANNING INTO STORMWATER MANAGEMENT

There is no known Basin or Action Plans covering the area with project limits.

MINIMUM REQUIREMENT 9 – OPERATIONS AND MAINTENANCE

Specific operations and maintenance for this project shall be determined upon approval of final design. Basic operations and maintenance for the roadway surface will include debris removal (ie sweeping of sands and gravels), re-stripping, and snow removal. Swale operations and maintenance shall include maintaining inlet opening from obstructions. In addition, vegetation planted (turf or natural) within the swales will need to be trimmed down.

Section 3.3.3 Stormwater Retrofit Analysis

Stormwater retrofit analysis was not addressed in this project. The design team is not aware of any existing structures or facilities that would need to be improved upon to meet current design requirements.

Section 3.3.4 Other Requirements

There were no additional requirements other than those listed in the Spokane County Stormwater Manual. All local requirements are based on Spokane County’s requirements.

Section 3.3.5 Pipe Alternatives

There were no applicable pipe alternatives used in the design of this project.

Section 3.3.6 Downstream Analysis

This project will have no appreciable impact to Hargman Creek or the Spokane River. All onsite stormwater treatment (infiltration) will be accomplished through the use of swales and drywells to mimic natural infiltration.

Section 3.3.7 New Stormwater Outfalls

No new stormwater outfalls will be incorporated into the design of this project.
Section 3.4.0 Developer Conditions

The City of Spokane is the developer for the new city street. The following design alternatives have been considered for the construction of the new city street.

DESIGN ALTERNATIVES

Alternative 1:

Conveyance of stormwater runoff created by the new city street to low areas. This alternative consists of installing piping or channels to convey runoff to evaporation ponds located at low points along the alignment.

Alternative 2:

Treatment of stormwater runoff using roadside infiltration swales, and drywells spaced at the calculated intervals, along the entire alignment.

DESIGN PROPOSAL

Alternative 2 is selected as the most feasible option for this project. WSDOT has directed the design to treat all stormwater runoff within the limits of the project. Treatment of stormwater runoff generated by the new roadway surface using infiltration swales and drywells is desirable in order to eliminate the need for the construction of a new stormwater sewer system. This alternative is also the most practical because treatment of this runoff will occur as soon as the runoff is removed from the roadway. This allows for treatment along the length of the roadway instead of conveying the runoff to a low point and designing treatment areas in low points to store a greater volume of water; resulting in sizing of evaporation ponds that may extend past the project limits.

Future Corridor Needs

There are no future corridor needs for this arterial.

Section 3.4.1 Drainage Basins and Existing Culverts

An offsite hydrologic analysis is used to determine if runoff from upslope will have a direct impact on the proposed project. A 100-year-24-hour storm precipitation was used to calculate the peak runoffs. This design storm has a precipitation depth of 2.26 inches (provided by WSDOT Eastern Region GIS system located at Cheney-Spokane Road) shown in Figure 3.9.

Offsite drainage basins upslope of the project are delineated and all are located south of I-90 and to the west of the active Burlington Northern Santa Fe (BNSF) railroad tracks. Three offsite drainage basins exist adjacent to the proposed city street. These drainage basins are shown below in Figure 3.11 and Figure 3.12. One additional offsite drainage basin was analyzed, adjacent to Basin C and continues south to Cheney-Spokane road. Through topographic maps and a site
visit, it is determined this drainage basin will not contribute flow to the proposed project. Runoff predictions for Drainage Basins A-C are discussed within this section.

There are presently five culverts used for the offsite runoff; the locations of these culverts are shown in Figures 3.1, 3.11, and 3.12. Two 3 feet diameter culverts under Fish Lake Trail near I-90 ("1"), two 5 feet diameter culverts run under the active railroad and Fish Lake Trail at Thorpe Road ("2") and feed one nine feet diameter culvert at Thorpe road running under US 195 ("3") discharging to Hangman Creek located to the east of US 195.

![Figure 3.11- Drainage Basin A & B](image1)
![Figure 3.12 – Drainage Basin C](image2)

**Engineering Methodologies**

To determine if the existing culverts are adequately sized for the flows produced by the design storm, the above drainage basins were analyzed using Curve Number and SCS Runoff methodologies. Lag times were calculated by using areas, composite curve numbers, excess runoff volumes, slopes and velocities, and time of concentrations and peak times. The areas for each offsite basin were delineated using topographic maps and a site visit. Initially using topographic maps, each basin was thought to have been larger due to an increasing elevation to the west. During the site visit, existing elevated roads (that have no drainage features underneath them) create clear separation lines; indicating precipitation landing outside of the areas shown for each basin would infiltrate and have no impact on the delineated basins. Composite curve numbers were estimated for each basin by approximating the percentages of different ground coverage within each basin using aerial photography. The SCS Runoff Method calculates the peak and concentration times using these inputs and estimated velocities for each basin. Once peak and concentration times are estimated, values for the lag time can be determined and input
into HEC-HMS. Table 3.6 shows the estimated values used to determine the lag time for each basin.

**Table 3.6 – Drainage Basin Calculations**

<table>
<thead>
<tr>
<th>Soil A</th>
<th>Soil B</th>
<th>Soil C</th>
<th>Soil D</th>
</tr>
</thead>
<tbody>
<tr>
<td>36</td>
<td>60</td>
<td>73</td>
<td>79</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Basin</th>
<th>Composite Curve Number</th>
<th>Estimated Velocity (ft/s)</th>
<th>Time of Concentration Length (ft)</th>
<th>Area of Basin (ac)</th>
<th>Time of Concentration (hr)</th>
<th>Lag Time (hr)</th>
<th>Time to Peak (hr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>70.7</td>
<td>0.95</td>
<td>5,561</td>
<td>363.1</td>
<td>1.63</td>
<td>0.98</td>
<td>1.09</td>
</tr>
<tr>
<td>B</td>
<td>60.3</td>
<td>0.95</td>
<td>7,818</td>
<td>410.8</td>
<td>2.29</td>
<td>1.37</td>
<td>1.53</td>
</tr>
<tr>
<td>C</td>
<td>60.4</td>
<td>0.9</td>
<td>9,706</td>
<td>856.6</td>
<td>3.45 ave</td>
<td>2.07</td>
<td>2.31</td>
</tr>
</tbody>
</table>

(1) Assumes fair soil types

The peak flows are determined using HEC-HMS. The HEC-HMS model requires the use of assumptions to calculate peak flows. No base-flows are present and no impervious areas were taken into account to estimate the composite curve numbers. The input values used for calculating peak flows are shown in Table A-3.1. Calculated peak flows using HEC-HMS are shown in Table 3.7. These values utilize a simulation of 100-year rainfall for 24-hours with 15 minute increments.

**Table 3.7 – HEC-HMS Peak Flows**

<table>
<thead>
<tr>
<th>Hydrologic Element</th>
<th>Drainage Area (sq. mi.)</th>
<th>Lag Time (min.)</th>
<th>Peak Discharge (CFS)</th>
<th>Time of Peak (hr.)</th>
<th>Volume (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basin A</td>
<td>0.567</td>
<td>59</td>
<td>97.0</td>
<td>13:00</td>
<td>0.78</td>
</tr>
<tr>
<td>Basin B</td>
<td>0.642</td>
<td>83</td>
<td>61.9</td>
<td>13:15</td>
<td>0.56</td>
</tr>
<tr>
<td>Basin C</td>
<td>1.353</td>
<td>125</td>
<td>98.7</td>
<td>14:15</td>
<td>0.55</td>
</tr>
<tr>
<td>Junction 1(1)</td>
<td>1.995</td>
<td>NA</td>
<td>152.3</td>
<td>13:45</td>
<td>0.55</td>
</tr>
</tbody>
</table>

(1) Common drainage point for Basin B & C

The peak flows calculated will be used to determine if the culverts discussed above are adequately sized. Junction 1 is a common drainage point for Basins B and C resulting in a combined peak flow from Basin B and C. The peak runoff will be seen first by the culverts at
location “2”, then location “3” (Figure 3.12), finally flowing to Hangman Creek. Graphical representations of each basin’s 100-year hydrographs are shown in the following figures (Figures 3.13- 3.15).

Figure 3.13 – HEC-HMS Basin A Hydrograph
Figure 3.14 – HEC-HMS Basin B Hydrograph

Figure 3.15 – HEC-HMS Basin C Hydrograph
Culvert Analysis

As mentioned, there are two 3 feet (36 inch) diameter culverts at which Offsite Basin A drains a peak flow of 97 cfs. Basins B and C combined to drain approximately 153 cfs to two 5 feet (60 inch) diameter culverts first under Fish Lake Trail and then continue onto a single 9 feet (108 inch) diameter culvert running under US 195, finally discharging into Hangman Creek. In order to determine if these existing culverts are adequately sized for the rainfall produced by a 100 year-24 hour storm depth of 2.26 inches, the headwater at the inlet of each culvert is needed. Figure 3.16 provides an approximation of the headwaters at each culvert.
Figure 3.16 – Headwater Calculator for Circular Culverts (Bureau of Public Roads)
By utilizing Figure 3.16, inputting the diameter and peak flows for each culvert, the headwaters are approximated. As indicated in the Hydraulics Manual, WSDOT has determined a headwater versus diameter ratio (HW/d) less than 1.25 for a 25-year storm event is adequate. Table 3.8 shows the results of the HW/d ratio for each culvert.

<table>
<thead>
<tr>
<th>Location</th>
<th>Diameter (feet)</th>
<th>Diameter (inches)</th>
<th>Peak Flow (cfs)</th>
<th>HW/d Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3</td>
<td>36</td>
<td>97</td>
<td>2.50</td>
</tr>
<tr>
<td>2</td>
<td>5</td>
<td>60</td>
<td>152</td>
<td>1.20</td>
</tr>
<tr>
<td>3</td>
<td>9</td>
<td>108</td>
<td>152</td>
<td>0.5</td>
</tr>
</tbody>
</table>

As seen from the approximated results for the HW/d ratio for each culvert, the two 3 feet diameter culverts located near I-90 at location “1” seen in Figure 3.11 exceed the maximum HW/d ratio of 1.25. WSDOT permits larger ratios if existing site conditions warrant a larger value. The rational for a larger ratio must be discussed with WSDOT Hydraulics Office. HW/d ratios shall not exceed a value of 3. As shown, the three feet diameter culverts are less than the maximum. It is recommended further examination of the existing conditions be conducted to determine if these culverts are sufficient. As shown, all other existing culverts seen at locations “2” and “3” in Figure 3.12 are adequately sized, with HW/d ratios less than 1.25, for runoff flows.

Section 3.4.2 TDAs

Threshold Discharge Areas do not apply as described in Section 3.2.3 of this document.

Section 3.5.0 Hydrologic and Hydraulic Design

There are no hydrologic or hydraulic design features within the project limits or scope other than swales and drywells.

Section 3.5.1 Calculations

The spacing methodology can be seen Section A3.3. Concrete curb and gutter will be used along the entire alignment. Table 3.9 lists locations where curb cuts will be placed; this is valid for both left and right of the center line. Sizing methodology for swales and a sample calculation is shown in Section A-3.8.
Table 3.9 Inlet Stationing and Distance

<table>
<thead>
<tr>
<th>Station</th>
<th>Distance (ft)</th>
<th>Station</th>
<th>Distance (ft)</th>
</tr>
</thead>
<tbody>
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<td>20+00</td>
<td>300</td>
<td>84+45</td>
<td>300</td>
</tr>
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<td>23+00</td>
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<td>26+00</td>
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<td>90</td>
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<td>121+92</td>
<td>300</td>
</tr>
<tr>
<td>57+45</td>
<td>300</td>
<td>124+92</td>
<td>300</td>
</tr>
<tr>
<td>60+45</td>
<td>300</td>
<td>125+32</td>
<td>40</td>
</tr>
<tr>
<td>63+45</td>
<td>300</td>
<td>128+32</td>
<td>300</td>
</tr>
<tr>
<td>66+45</td>
<td>300</td>
<td>131+32</td>
<td>300</td>
</tr>
<tr>
<td>69+45</td>
<td>300</td>
<td>132+89</td>
<td>157</td>
</tr>
<tr>
<td>72+45</td>
<td>300</td>
<td>135+89</td>
<td>300</td>
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<tr>
<td>75+45</td>
<td>300</td>
<td>138+89</td>
<td>300</td>
</tr>
<tr>
<td>78+45</td>
<td>300</td>
<td>140+25</td>
<td>136</td>
</tr>
<tr>
<td>81+45</td>
<td>300</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Section 3.6.0 Permits and Associated Reports

No determination has been made for this project to date.

Section 3.6.1 Environmental Issues, Fish and Other Endangered Habitat

There will be no additional environmental issues or impacts to fish or endangered species foreseen by the construction of this project.

3.6.1.1 RECEIVING BODIES

As identified in Section 2.1 of this document, Hangman Creek located to the east of US 195 is the only tributary that will continue to receive stormwater runoff from the offsite drainage basins discussed in Section 4.1 of this document. The post construction condition will remain unchanged.
3.6.1.2 FLOOD PLAINS

The project will not encroach on the hydraulic floodway.

3.6.1.3 STREAM CROSSINGS

Project improvements will not modify the existing stream crossings noted in Section 3.2.2 of this report.

3.6.1.4 WETLANDS

There are no wetlands identified with project limits.

3.6.1.5 STEEP SLOPES

There are no steep slopes within the limits of the project.

3.6.1.6 OTHER SENSITIVE AREAS

There is no critical habitat, sensitive areas, special basin, or action plan issues specific to the area within project limits.

There are no critical aquifer recharge areas (CARAs), wellhead protection zones, nor sole-source aquifers (SSAs) within project limits.

Wells

Wells located within the right of way of the project have been identified and discussed in Section 3.2.4.5 of this document.

3.6.1.7 FISH SURVEYS

No fish surveys are known for the proposed work within project limits.

3.6.1.8 ENDANGERED AND THREATENED SPECIES

No endangered or threatened species have yet been identified within the vicinity of this project.

3.6.1.9 FISH PASSAGE

No fish blockages exist within project limits. The streams that cross the project boundary are too small to host fish species since they are seasonal.
3.6.1.10 IMPAIRED WATER BODIES

No impaired water bodies are known. Testing of water bodies shall be conducted prior to construction to determine if impaired bodies are present.

Section 3.6.2 Permits/Approvals

Permits and approvals are not addressed as a component of this report. The necessary permits/approvals will need to be acquired prior to finalizing the design and construction of the project.

Section 3.6.3 Easements

This project will be constructed entirely within WSDOT right-of-way obtained to date. If deemed necessary for construction after final design is complete, WSDOT will obtain any additional right of way.

Section 3.6.4 Additional Reports or Studies

Not additional reports or studies have been conducted, but may be deemed necessary contingent upon approval of this project.

Section 3.7.0 Inspection and Maintenance Summary

Inspection maintenance during construction is not addressed.

Section 4.0 Pavement and Roadway Design

The initial data the team received was sent in AutoCAD along with some applicable Bentley Microstation drawings. This AutoCAD data was an indirect conversion from Microstation accomplished by WSDOT design engineers. It had to be saved in XYZ coordinate format and imported into AutoCAD. The drawings we received were WSDOT’s proposed design and did not contain the true existing ground in all areas. The area near the proposed centerline alignment of Fish Lake Road had been modified slightly to allow for what WSDOT considered to be their initial design. The Gonzaga design team used the given existing ground and continued to modify it, through the new corridor, based on the hydraulic and road design requirements. The team received the improved existing ground, Fish Lake Road centerline alignment, proposed on/off ramps at Cheney-Spokane Road, and US Highway 195 alignment properties. The basic method that the team used was taking what we needed from the original Civil3D drawing and importing them into a “new” drawing. This allowed us to keep the file size relatively small and manageable.

The existing ground was comprised of over 800,000 surface triangles and proved to be excessive for our hydraulic purposes. This was the first hurdle the team faced and work began on removing
much of the surface so that AutoCAD Civil3D could run properly. If this step was not taken, Civil3D would barely function as a result. Although this was a very time consuming process, the surface editing was accomplished by isolating regions of the surface and deleting them. The areas deleted were mostly to the East of US Highway 195 and the extreme southern and northern reaches of the project area. This allowed Civil3D to run more efficiently and further progress could be made on the design.

The next step was to import the Fish Lake Road alignment from the initial drawing. This required further modification because the Microstation to Civil3D conversion left the alignment broken up with incomplete elevation connections. Once this was finished, a design vertical profile could be created based on WSDOT’s proposal for vertical curve and road surface slope data. Once the vertical profile was applied to the Fish Lake Road alignment, work began on designing the road’s cross-sections. The initial cross-section design was based on architectural renderings provided by WSDOT that are shown in Figure 2.3. The lane widths and slopes, sidewalk and trail data, and day lighting preferences were all based on the WSDOT initial design concept. Our job was to adjust these cross-sections based on hydraulic requirements such as swale width, swale depth, and use of detention ponds as necessary. The roadway cross-sections needed to be designed for 4 distinct areas along the 2.3 mile alignment. The 4 cross-sections differed in lane widths and sidewalk/trail widths. After cross-section design, they were applied to the Fish Lake Road alignment in what is called a corridor. One of the more challenging parts of this process was getting the cross-sections to transition with one another in the corridor and make sure that there were no “gaps” in the data. These gaps usually show up as glitches in the Civil3D drawing and make items like earthwork quantities inaccurate.

Once the road cross-sections were applied to the alignment, then sample lines needed to be created. Sample lines are used to calculate material quantity; the more sample lines in the drawing, the more accurate your material quantity is. Sample lines also allowed us to develop material quantity charts for the entire corridor. Figure 4.1 shows that the total quantity of concrete (curbs, gutters, and sidewalks) will be 3,181.01 cubic yards.

![Concrete Volume Table](image)

**Figure 4.1 - Material Quantity Chart Example**

The initial design of Fish Lake Road had swales with a bottom width of 2 feet and side slopes at 2:1. This gave a total width of 6 feet which fit in nicely to the architects design. After the team performed the initial hydraulic work that included 1/2 inch of run-off from the impervious surfaces, we realized that a 6 foot wide swale at 1 foot deep was large enough in most areas for treatment but it did not provide enough storage for larger rainfall totals. Civil3D makes design changes quick and easy and the swale bottom width was changed to 5.3 feet. Since we are using
parametric software, a few clicks of the mouse changed the entire corridor, and new quantities were automatically updated. The same process was used when we found out that there were some areas that had right-of-way encroachment. The day lighting was changed from 4:1 to 2:1 and most of the issues were resolved with just a few minutes of work because the whole drawing is dynamically connected; change one parameter and all other parameters that are affected also change as indicated in Figures 4.2 and 4.3.

By far the largest hurdle the Gonzaga design team faced was learning the software. Our proposed duration for this phase of the project proved to be unreasonable because we just did not know how long it would take to convert the data from one drawing to the other. The design team enlisted the help of the Civil3D adjunct professor at Gonzaga, Matt Townsend. Although this process could not have been completed without his expertise, it became a difficult balance of having him do it versus the team doing the work with his oversight. Another issue was crashing the Civil3D software; it was a daily if not hourly occurrence because of the size of the road and the detail of the existing ground. The initial learning curve with Civil3D is very steep, but once the basics were understood, progress was quickly made and the program proved to be a nice asset in the hydraulic analysis.

Figure 4.2 – 2:1 Slope for Daylighting
Figure 4.3 – 4:1 Slope for Daylighting
# Section 5.0 Cost Estimate

## ENGINEER'S ESTIMATE

**WSDOT US 195 Hangman Valley City Street Network**

<table>
<thead>
<tr>
<th>Item No.</th>
<th>Item Description</th>
<th>Unit</th>
<th>Cost per Unit</th>
<th>Eng'rs Est (Total Amount)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td><strong>Preparation</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Mobilization</td>
<td>L.S.</td>
<td><strong>$425,065.40</strong></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Clearing and Grubbing</td>
<td>41.39 ACRE</td>
<td><strong>$1,000.00</strong></td>
<td><strong>$41,394.63</strong></td>
</tr>
<tr>
<td>3</td>
<td>Removal of Fish Lake Trail Asphalt</td>
<td>8014.00 S.Y.</td>
<td><strong>$5.00</strong></td>
<td><strong>$40,700.00</strong></td>
</tr>
<tr>
<td></td>
<td><strong>Grading</strong></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Roadway Excavation Incl. Haul</td>
<td>301834.41 C.Y.</td>
<td><strong>$4.75</strong></td>
<td><strong>$1,433,713.45</strong></td>
</tr>
<tr>
<td></td>
<td><strong>Drainage</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Curb Inlet</td>
<td>87.00 EACH</td>
<td><strong>$150.00</strong></td>
<td><strong>$13,050.00</strong></td>
</tr>
<tr>
<td>6</td>
<td>Single Stage Drywell</td>
<td>104.00 EACH</td>
<td><strong>$1,500.00</strong></td>
<td><strong>$156,000.00</strong></td>
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<tr>
<td></td>
<td><strong>Surfacing</strong></td>
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<td></td>
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</tr>
<tr>
<td>7</td>
<td>Crushed Surfacing Base Course</td>
<td>50205.26 TON</td>
<td><strong>$12.00</strong></td>
<td><strong>$722,463.14</strong></td>
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<tr>
<td>8</td>
<td>Anti-Stripping Additive</td>
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<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td><strong>Hot Mix Asphalt</strong></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>HMA CL. 1/2 IN.</td>
<td>8844.02 TON</td>
<td><strong>$60.00</strong></td>
<td><strong>$530,641.45</strong></td>
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<td></td>
<td><strong>Erosion Control and Planting</strong></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>Swale Seeding</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td><strong>Traffic</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>Cement Conc. Traffic Curb and Gutter</td>
<td>24042.40 L.F.</td>
<td><strong>$20.00</strong></td>
<td><strong>$480,846.00</strong></td>
</tr>
<tr>
<td>12</td>
<td>Paint Line (Turn Lane and Shoulder)</td>
<td>490848.0 L.F.</td>
<td><strong>$0.13</strong></td>
<td><strong>$6,251.02</strong></td>
</tr>
<tr>
<td></td>
<td><strong>Other Items</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>Cement Conc. Sidewalk</td>
<td>28794.82 S.Y.</td>
<td><strong>$28.00</strong></td>
<td><strong>$806,254.91</strong></td>
</tr>
</tbody>
</table>

**Subtotal** $4,675,719.44

Engineering Contingencies (20% of Subtotal) $935,143.89

Tax (8.7%) $468,145.11

**Contract Total** $6,099,008.43
Section 6.0 Project Outcomes
Section 6.1 - Recommendations

The hydraulic design of the proposed city street network was accomplished with the assumption the soils within the entire project limits are permeable. The soils appear to be adequate for drainage based upon information collected from the NRCS soil survey and a sieve analysis conducted on the soil sample obtained as explained in Section 3.2.4.2 through Section 3.2.4.4. Both the soil survey and sieve analysis confirm a high hydraulic conductivity concluding there will be minimal ponding occurring due to stormwater runoff created by the new roadway surface. A further geotechnical investigation is recommended.

Swale design (width, depth, etc.) for stormwater runoff was completed by identifying the section of roadway that would create the largest volume of runoff to be treated, i.e. the largest swale treatment volume required, and applying those dimensions along both sides of the roadway for the entire length of the alignment. The purpose of utilizing a consistent swale design is for continuity of aesthetics and ease of construction for the entire project. This design team recommends the use of AutoCAD Civil 3D to change the swale dimensions to match the required treatment volume along the alignment of the roadway. The use of AutoCAD will allow ease in estimating volume differences between the minimal required design and the design proposed within this document. A cost analysis can then be conducted for the two options.

Funding has not been secured to completely fund this project to date. This affects when this project is to begin the construction phase. Once both funding and construction dates have been confirmed, a further evaluation of the changing cost (labor, materials, interest rates, etc.) for the proposed design is recommended.

Section 6.2 - Conclusion

The offsite hydrologic analysis conducted concludes that the delineated offsite drainage basins presented in Section 3.4.1 are as shown. The upslope areas of these basins will not contribute any additional runoff affecting the proposed project. As explained within this document, this is believed to be accurate because clear separation boundaries are present on site. The offsite hydrologic analysis also concludes the existing drainage structures indicated in Figure 3.1 are adequate in size to accommodate the basins’ runoff flows; design of the hydraulic features for this project will not need to treat any additional stormwater runoff generated from the offsite basins.

Construction of the project will have no direct impact Hangman Creek. The incorporation of swales and drywells along the entire alignment of the roadway allows all runoff volumes generated by the roadway surface to be treated onsite. No downstream analysis is required because historical flows within Hangman Creek will not be changed.

On sections of the road with a slope of more than 1 percent, a berm is required to hold the water to be treated. Berm construction within the swales is normally permitted to be 6 or 8 inches. The maximum berm height is listed as 8 inches. Calculations show constructing a 9 inch height berm
in the swale will provide a head of 3 inches over the drywell grate inlet; accommodating 1.0 cfs flow to each drywell which is deemed necessary and explained within this document.

The right-of-way possessed by WSDOT currently is adequate for the construction of this project. There are no areas where the designed roadway cross-sections (Figure A-6.1) fall out of right-of-way limits along the alignment of the roadway. Therefore, other than evaluating the cost of construction of the swales, changing dimensions versus a consistent swale design, is not necessary.

The use of incorporating design tools, such as AutoCAD Civil 3D, has been beneficial in the design process of the roadway. AutoCAD Civil 3D has allowed the design team to verify cross-section design and apply it to the alignment of the roadway to see if there are right-of-way limits. It also calculated quantities needed to develop a cost estimate with ease. A significant advantage of designing using AutoCAD Civil 3D is it will allow the designer to make changes effortlessly to the design and see the potential impacts it will have on overall cost. An example of this could be the raising of the roadway elevation will change the quantity of cut material from the original design; resulting in reduced labor for excavation and ultimately lowering construction cost.
Section 7.0 References

*Wetland Delineation*

*Well Logs*

*Soil Data*

*General Information*

- Greg Lahti, WSDOT Hydraulic Engineer (Liaison)
- Wayne Cornwall, WSDOT Hydraulic Engineer
- WSDOT Hydraulics Manual
- WSDOT Highway Runoff Manual
- American Association of State Highway and Transportation Officials (AASHTO)

*Technological Design Aids*

HEC-HMS 3.3

HEC-RAS 4.0

AutoDesk Auto Civil 3D 2010

Microsoft Office 2007
Appendix A-1  Stormwater Design Documentation Spreadsheet

There is no need to include a Stormwater Design Documentation Spreadsheet because no TDAs exist within the project limits.

Appendix A-2  TDA Maps, Drainage Basin Maps, and Area Calculations

To find the required storage, we used the Modified Rational or “Bowstring” method. First find the time of concentration.

\[ Tc = \frac{L}{K \cdot \sqrt{S}} \]

Where
- \( Tc \) = Time of concentration, minutes
- \( K \) = Ground cover coefficient, feet/minute
- \( S \) = Slope, percent

Now find the Intensity, \( I \).

\[ I = \frac{M}{Tc^n} \]

Where
- \( I \) = Rainfall intensity, inch/hr
- \( M \) = Coefficient of rainfall intensity
- \( n \) = Coefficient of rainfall intensity
- \( Tc \) = Time of concentration, minutes

Now find the flow \( Q \) at intensity \( I \).

\[ Q = CIA \]

Where
- \( Q \) = Flow, cfs
- \( I \) = Intensity, inch/hr
- \( A \) = Area, acres

Once this is found, find the equation for flow in.

\[ Vin = 1.34 \cdot Q \cdot t \]

Where
- \( Vin \) = Flow in, cubic feet
- \( Q \) = Flow, cfs
- \( t \) = Time, seconds
Once all of these are found, it is necessary to find the flow out.

\[ V_{out} = D \cdot C_d \cdot t \]

Where
- \( V_{out} \): flow out from drywells, cubic feet
- \( D \): number of drywells in the area
- \( C_d \): capacity of each drywell, cfs
- \( t \): time, seconds

Finally, subtract \( V_{out} \) from \( V_{in} \) to find the required storage. Compare this to available storage.

\[ S_t = \left( \frac{1.5 \cdot b \cdot d + \frac{1.5 \cdot d \cdot l}{2}}{2} \right) \cdot \frac{D}{(D + W)} \]

Where
- \( S_t \): available storage, cubic feet
- \( b \): bottom width of swale, feet
- \( d \): depth of swale, feet, feet
- \( l \): length of swale before drywell, feet
- \( D \): distance between berms
- \( W \): width of berms

A spreadsheet can be constructed to compare required storage to available storage.

For a basic 300 foot length of road with 1 drywell, \( L = 300 \) feet, and 34 feet of impervious surface, with \( K = 1200 \) for impervious surfaces and slope = 7 percent:

\[ T_c = \frac{300}{1200 \cdot 2 \cdot 0.07} \]

\[ T_c = 0.944 \] minute, use 5 minutes as minimum \( T_c \).

For a 50 year storm event, \( M = 10.68 \) and \( n = 6.35 \).

\[ I = \frac{16.68}{5.635} \]

\[ I = 3.84 \text{ inch/hr} \]

For area 34\*300 ft squared and \( C = 0.9 \) for impervious surfaces,

\[ Q = 0.9 \left( \frac{(34 \cdot 300)}{43560} \right) \cdot 3.84 \]

\[ Q = 0.81 \text{ cfs} \]

Find \( V_{in} \) for \( T_c \).

\[ V_{in} = 1.34 \cdot 811 \cdot 5.60 \]

\[ V_{in} = 326 \text{ cubic feet} \]
Compare to Vout.

\[ V_{out} = 60 \cdot 5 \cdot 1 \quad \text{Vout}= 300 \text{ cubic feet.} \]

Find storage required.
\[ S = 326 - 300 \quad \text{Storage}=26 \text{ cubic feet.} \]

Calculate available storage.

\[ \text{St} = \left( \left( 1.5 \cdot 5.5 \cdot 1 + \frac{1.5 \cdot 1 \cdot 2}{2} \right) \cdot 300 \right) \left( \frac{7.07}{(7.07 + 4.56)} \right) \]

\[ \text{St}= 1460 > 26 \text{ cubic feet. There is plenty of storage here.} \]

A spreadsheet is then used to calculate the required storage of the swales along the alignment of the roadway (see Table 3.5).

**Appendix A-3 Calculations and Program Output**

**A-3.1 STORMSHED AND/OR MGS FLOOD OUTPUT REPORTS**

The values shown in Table A-3.1 were input into HEC-HMS with a precipitation depth of 2.26 inches (100-year-24-hour storm Figure 3.9)

<table>
<thead>
<tr>
<th>Basin(1)</th>
<th>Area (sq.mi.)</th>
<th>Composite Curve Number</th>
<th>Lag Time (min)</th>
<th>Impervious Area (%)</th>
<th>Add'l Initial Abstractions (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.567</td>
<td>70.7</td>
<td>59</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>B</td>
<td>0.642</td>
<td>60.3</td>
<td>83</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>C</td>
<td>1.353</td>
<td>60.4</td>
<td>125</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

(1) Loss method: SCS Curve Number;
Transform Method: SCS Unit Hydrograph;
Baseflow: None

The Hydrographs created using HEC-HMS for the simulated model can be seen in Figure 3.13 – 3.15.
Figure Appendix A-3.1.1 - HEC-HMS Model Schematic

Table Appendix A-3.1.2 - HEC-HMS Model Global Summary Table

<table>
<thead>
<tr>
<th>Hydrologic Element</th>
<th>Drainage Area (MI2)</th>
<th>Peak Discharge (CFS)</th>
<th>Time of Peak</th>
<th>Volume (IN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Freeway (Basin A)</td>
<td>0.567</td>
<td>97.0</td>
<td>02Feb2010, 13:00</td>
<td>0.78</td>
</tr>
<tr>
<td>Junction-1</td>
<td>1.995</td>
<td>152.3</td>
<td>02Feb2010, 13:45</td>
<td>0.55</td>
</tr>
<tr>
<td>Reach-1</td>
<td>0.642</td>
<td>81.8</td>
<td>02Feb2010, 13:30</td>
<td>0.56</td>
</tr>
<tr>
<td>Reach-2</td>
<td>1.353</td>
<td>98.7</td>
<td>02Feb2010, 14:15</td>
<td>0.55</td>
</tr>
<tr>
<td>Thorpe North (Basin B)</td>
<td>0.642</td>
<td>81.9</td>
<td>02Feb2010, 13:15</td>
<td>0.55</td>
</tr>
<tr>
<td>Thorpe South (Basin C)</td>
<td>1.353</td>
<td>98.7</td>
<td>02Feb2010, 14:15</td>
<td>0.55</td>
</tr>
</tbody>
</table>
A-3.2 BMPS DESIGN

As described in Sections 2.1 through 2.4, there are no conflicts with soil issues, existing features, or drainage. The swale sizing was accomplished with equation 6-1A of the Spokane Regional Stormwater Manual and with the geometry of the swale.

The volume of the swale was calculated with equation 6-1A:

\[ V = 1133 * A \]

*Equation 6-1A*

Where:
- \( V \) = volume of bioinfiltration swale (cubic feet)
- \( A \) = hydraulically connected impervious area to be treated (acres).

The bottom width of the swale was then calculated using the geometry of a swale with a base \( b \) and 3:1 side slopes, with a maximum depth of 6 inches.

\[ V = b * 0.5 + (0.5 * 1.5/2) * 2 \]

*Equation A-3.2.1*

Where:
- \( V \) = volume of bioinfiltration swale (cubic feet)
- \( b \) = bottom width of swale (feet)

For a unit width of 1 foot of roadway.

This equation can be rearranged to find the bottom width of a swale:

\[ b = 2 * V - 1.5 \quad \text{or} \quad b = 2 * 1133 * A - 1.5 \]

The bottom width required for a swale that drains an impervious area of 34\,ft^2 square feet would be:

\[ b = 2 * 1133 * (34 \, \text{ft}^2) / (43560 \, \text{ft}^2/\text{acre}) - 1.5 = 0.26 \, \text{feet} = 3 \, \text{inch bottom depth of swale}. \]

On sections where the slope is greater than 1%, berms must be placed to hold the water to a 6 inch depth to ensure treatment.

Calculation of berm frequency and dimensions on a section with a slope of 7%.
D = .5/S

Where

D = distance between berms (feet)
S = slope of road

D = .5/.07 = 7.14 feet spacing between berms. Calculate berm dimensions.

Berms rise 9 inches high at a slope of 2:1, have a 12 inch top width, and come back down at a slope of 2:1. 2*9+12+2*9=48 inches.

In 48 inches, the swale falls an additional 48*.07=3.36 inches, at a 2:1 slope this results in a total berm width of 48+3.36*2=54.7 inches.

Swale sizing with a slope and berms

Equation A-3.2.1 is not appropriate for sizing swales when there is a slope and berms. To size swales with a slope and berms, we use equation A-3.2.2.

\[ V = \frac{[1/2(0.5*b+(0.5*1.5/2)*2)](D/(D+W))}{Equation A-3.2.2} \]

Where

V = volume of bioinfiltration swale (cubic feet)
b = bottom width of swale (feet)
D = distance between berms (feet)
W = width of berm (feet)

This equation can be rearranged to provide the base width b of a swale:

\[ b = 4*V*(D+W)/(D-1.5) \]

or

\[ b = 4*1133*A^*(D+W)/(D-1.5) \]
For a 7 percent slope, our berm width is already calculated as 54.7 inches or 4.56 feet and our berm spacing is already calculated as 7.14 feet. The maximum impervious area in this slope range is 42 feet.

\[ b = \frac{(4 \times 1133 \times 42 \text{ ft}^2)}{(43560 \text{ ft}^2/\text{acre})(7.14+4.56)/7.14-1.5} = 5.66 \text{ feet bottom width.} \]

A bottom width of 5.5 feet is chosen.

**A-3.3 Gutter Design**

Concrete curb and gutter will be used along the entire alignment. Table A-3.3.1 lists locations where curb cuts will be placed; this is valid for both left and right of the center line. The gutter analysis calculations are shown in Section A-3.4.

**Table A-3.3.1-Inlet Stationing and Distance**

<table>
<thead>
<tr>
<th>Station</th>
<th>Distance (ft)</th>
<th>Station</th>
<th>Distance (ft)</th>
</tr>
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<tr>
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<td>55</td>
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<td>300</td>
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<td>29+55</td>
<td>90</td>
<td>96+45</td>
<td>300</td>
</tr>
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<td>30+45</td>
<td>300</td>
<td>98+35</td>
<td>190</td>
</tr>
<tr>
<td>33+45</td>
<td>300</td>
<td>100+92</td>
<td>257</td>
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<td>36+45</td>
<td>300</td>
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<td>39+45</td>
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<td>57+45</td>
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<td>124+92</td>
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<td>81+45</td>
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### Table A-3.4.1- INLET SPACING - CURB AND GUTTER SPREADSHEET for 10 yr Storm (ENGLISH UNITS)

<table>
<thead>
<tr>
<th>Station</th>
<th>Distance</th>
<th>Width</th>
<th>AQ</th>
<th>L</th>
<th>T</th>
<th>I</th>
<th>G.W.</th>
<th>G.L.</th>
<th>d</th>
<th>Qv</th>
<th>Vcontinuous</th>
<th>Vsides</th>
<th>Fc</th>
<th>Rs</th>
<th>E</th>
<th>Qs</th>
<th>Qp</th>
<th>Zd</th>
<th>Zd Check</th>
<th>Velocity Check</th>
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<td>6645</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Project Name:** New City Street

**Project #:**

**S.R.:**

**Designed By:** Johnny Davis, Julie Guthrie, Josh Richards and Jeremiah Woodard

**Date:**

**Updated:**
<p>| Station | Distance | Width | AQ | ΣQ | Slope L | Super T | G.W. | G.L. | d | Zd | Q&lt;sub&gt;op&lt;/sub&gt; | Vcontinuous ** | Vside ** | E&lt;sub&gt;x&lt;/sub&gt; | R&lt;sub&gt;x&lt;/sub&gt; | E | Q&lt;sub&gt;x&lt;/sub&gt; | Q&lt;sub&gt;op&lt;/sub&gt; | Zd Check | Velocity Check |
|---------|----------|-------|----|----|--------|--------|------|------|---|----|-------------|-------------|----------|--------|--------|--------|-----|--------|-------------|----------|----------------|
| 5746    | 300      | 25.83 | 0.62 | 1.40 0.07 | 735 0.02 | 0.83 | 2.00 | 0.09 | 4.50 | 0.50 | 4.97 | ALLOWABLE &gt; Zd | DESIGN | VELOCITY &lt; 5 FT/SEC |
| 5446    | 300      | 25.83 | 0.62 | 1.53 0.06 | 975 0.02 | 0.83 | 2.00 | 0.10 | 5.00 | 0.57 | 4.63 | ALLOWABLE &gt; Zd | DESIGN | VELOCITY &lt; 5 FT/SEC |
| 5146    | 300      | 25.83 | 0.62 | 1.65 0.06 | 714 0.02 | 0.83 | 2.00 | 0.10 | 5.00 | 0.61 | 4.97 | ALLOWABLE &gt; Zd | DESIGN | VELOCITY &lt; 5 FT/SEC |
| 4846    | 300      | 25.83 | 0.62 | 1.73 0.06 | 714 0.02 | 0.83 | 2.00 | 0.10 | 5.00 | 0.63 | 4.99 | ALLOWABLE &gt; Zd | DESIGN | VELOCITY &lt; 5 FT/SEC |
| 4546    | 300      | 25.83 | 0.62 | 1.78 0.04 | 955 0.02 | 0.83 | 2.00 | 0.16 | 8.00 | 0.79 | 2.12 | 1.53 | 0.25 | 0.23 | 0.43 | 0.45 | 0.60 | ALLOWABLE &gt; Zd | DESIGN | VELOCITY &lt; 5 FT/SEC |
| 4246    | 300      | 25.83 | 0.62 | 1.76 0.04 | 955 0.02 | 0.83 | 2.00 | 0.16 | 8.00 | 0.76 | 2.06 | 1.53 | 0.25 | 0.23 | 0.43 | 0.44 | 0.58 | ALLOWABLE &gt; Zd | DESIGN | VELOCITY &lt; 5 FT/SEC |
| 3946    | 300      | 25.83 | 0.62 | 1.75 0.04 | 955 0.02 | 0.83 | 2.00 | 0.16 | 8.00 | 0.75 | 2.02 | 1.53 | 0.25 | 0.23 | 0.43 | 0.43 | 0.57 | ALLOWABLE &gt; Zd | DESIGN | VELOCITY &lt; 5 FT/SEC |
| 3646    | 300      | 25.83 | 0.62 | 1.75 0.04 | 955 0.02 | 0.83 | 2.00 | 0.16 | 8.00 | 0.74 | 2.00 | 1.53 | 0.25 | 0.23 | 0.43 | 0.43 | 0.57 | ALLOWABLE &gt; Zd | DESIGN | VELOCITY &lt; 5 FT/SEC |
| 3346    | 300      | 25.83 | 0.62 | 1.74 0.04 | 955 0.02 | 0.83 | 2.00 | 0.16 | 8.00 | 0.74 | 1.99 | 1.53 | 0.25 | 0.23 | 0.43 | 0.42 | 0.56 | ALLOWABLE &gt; Zd | DESIGN | VELOCITY &lt; 5 FT/SEC |
| 3046    | 300      | 25.83 | 0.62 | 1.74 0.04 | 955 0.02 | 0.83 | 2.00 | 0.16 | 8.00 | 0.73 | 1.98 | 1.53 | 0.25 | 0.23 | 0.43 | 0.42 | 0.56 | ALLOWABLE &gt; Zd | DESIGN | VELOCITY &lt; 5 FT/SEC |
| 2955    | 91       | 25.83 | 0.19 | 1.31 0.04 | 955 0.02 | 0.83 | 2.00 | 0.14 | 7.00 | 0.49 | 1.81 | 1.40 | 0.29 | 0.26 | 0.48 | 0.33 | 0.36 | ALLOWABLE &gt; Zd | DESIGN | VELOCITY &lt; 5 FT/SEC |
| 6945    |          |       | 0.62 | 0.008 0.068 | 676 0.02 | 0.83 | 2.00 | 0.12 | 6.00 | 0.42 | 2.19 | 1.67 | | |
| 7245    | 300      | 25.83 | 0.62 | 0.50 0.008 | 676 0.02 | 0.83 | 2.00 | 0.14 | 7.00 | 0.68 | 2.48 | 1.85 | 0.33 | 0.21 | 0.47 | 0.29 | 0.33 | ALLOWABLE &gt; Zd | DESIGN | VELOCITY &lt; 5 FT/SEC |
| 7545    | 300      | 25.83 | 0.62 | 0.95 0.008 | 676 0.02 | 0.83 | 2.00 | 0.14 | 7.00 | 0.68 | 2.48 | 1.85 | 0.33 | 0.21 | 0.47 | 0.29 | 0.33 | ALLOWABLE &gt; Zd | DESIGN | VELOCITY &lt; 5 FT/SEC |
| 7845    | 300      | 25.83 | 0.62 | 1.18 0.008 | 676 0.02 | 0.83 | 2.00 | 0.15 | 7.50 | 0.86 | 2.69 | 1.93 | 0.29 | 0.18 | 0.41 | 0.39 | 0.56 | ALLOWABLE &gt; Zd | DESIGN | VELOCITY &lt; 5 FT/SEC |
| 8145    | 300      | 25.83 | 0.62 | 1.34 0.008 | 676 0.02 | 0.83 | 2.00 | 0.16 | 8.00 | 1.00 | 2.69 | 2.02 | 0.27 | 0.17 | 0.39 | 0.46 | 0.72 | ALLOWABLE &gt; Zd | DESIGN | VELOCITY &lt; 5 FT/SEC |</p>
<table>
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<th>Station</th>
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<th>AQ</th>
<th>SQ</th>
<th>Slope</th>
<th>Super</th>
<th>G.W.</th>
<th>G.I.</th>
<th>d</th>
<th>Zd</th>
<th>Qmp*</th>
<th>Vcontinuous</th>
<th>Vside</th>
<th>Eo</th>
<th>Re</th>
<th>Ep</th>
<th>Qi</th>
<th>Qhr**</th>
<th>Zd Check</th>
<th>Velocity Check</th>
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<td>4.01</td>
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<td>4.50</td>
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<td>VELOCITY &lt; 5 FT/SEC</td>
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<td>0.27</td>
<td>Zd ALLOWABLE</td>
<td>VELOCITY &lt; 5 FT/SEC</td>
</tr>
</tbody>
</table>
Variable definition for inlet spacing are as follows:

Distance = length of pavement flowing to gutter
Width = width of pavement flowing to gutter
D Q = flow from adjacent pavement going to gutter
S Q = total flow
Slope L = longitudinal pavement slope
Super T = transverse slope
G.W. = grate width
G.L. = grate width
d = depth of flow at face of gutter
Z_d = top width of flow prism
Q_{bp} = portion of the flow outside the width of the grate
V_{continuous} = velocity of flow over grate
E_0 = ratio of frontal flow to total gutter flow
R_s = ratio of side flow intercepted to total side flow
E = grate efficiency
Q_r = side flow interception capacity of grate
T_c = time of concentration
C = runoff coefficient for Rational Method
l = rainfall intensity
m = rainfall intensity
n = rainfall intensity
Allowable Z_d = allowable width of flow prism
A-3.5 SAG DESIGN

The following sag worksheets show that the inlets are adequate at the sags. There are no grates or combination grates used along this road, therefore the calculations of \( d_B \) and \( \sum Q \) are unnecessary.

**SAG INLET DESIGN WORKSHEET**

*Combination Inlet at low point*

\[
\begin{align*}
\sum Q & \quad \Box \quad Q_1 \\
& \quad \Box \quad Q_2
\end{align*}
\]

\[
L_1 \quad Q_{BP_1} \quad \text{Inlet A} \quad \text{Inlet B} \quad \text{Inlet C} \quad Q_{BP_2}
\]

Station 1806
Station 2860
Station 2895
Station 2920
Station 3046
Station 3486
Station 1805

**Transverse Slope**

\( S_z \) = 0.36

**Allowable**

\( Z_k \) = 24.36

**Allowable**

\( d_{sa} \) = 0.49

**Time of Concentration**

\( T_c \) = 5.0

**50 yr. rainfall coefficients**

\( c_1 \) = 10.94

**Rainfall Intensity**

\( i_{50} \) = 3.84

(for 5 minute duration)

**Distance between last inlet**

\( L_1 \) = 350.00

**Width of catchment area**

\( W_1 \) = 34.83

**Bypass from last inlet**

\( Q_{BP} \) = 0.61

**Discharge of catchment area**

\( Q_1 \) = 1.64

\[
\begin{align*}
Q_{BP} & = Q_{BP_1} + 1.64 \times 0.74 \times 1.14 = 3.83 \\
Q_{BP} & = 0.81 \times 1.04 \times 0.74 \times 1.14 = 3.83
\end{align*}
\]

**Combination of Grate inlet for sag**

\( P_r \) = 0

**Effective Perimeter of Grate Inlets**

\( P_e \) = 0.00

If \( Q > Q_{BP} \), the design is complete.

If \( Q = \text{allowable} \), additional inlets must be added and the process repeated.

**Notes:**

1. If using a combination inlet for the sag, the flanking grates inlets are not required except in depressed areas. (See Hydraulics Manual)
2. Formulas based on weir flow. (See Hydraulic Manual 5-5.2)
**SAG INLET DESIGN WORKSHEET**

**Combination Inlet at low point**

1. **Transverse Slope (S_t)**
   - 0.020

2. **Allowable Slope (S_a)**
   - 0.48

3. **Run of Concentration (τ_c)**
   - 5.6

4. **50-year rainfall coefficients (K_r)**
   - 0.012

5. **Rainfall Intensity (I)**
   - 2.94

6. **Diameter between last inlet (D)**
   - 50

7. **Width of catchpoint area (W_c)**
   - 23.83

8. **Bypass from last inlet (Q_b)**
   - 0.92

9. **Discharge of catchpoint area (Q_c)**
   - 1.06

**Q_inlet = Q_{bypass} + Q_b + Q_c = 3.90**

**Q_{outlet} = 0.92 + 0.39 + 1.21 = 2.52**

**Combination of inlet for last inlet (Q_{inlet})**

**Effective Perimeter of grate inlets (P_{eff})**

- **P_A**
  - 0.00

- **P_B**
  - 0.00

- **P_C**
  - 0.00

**Q_{outlet} = 0.92 + 0.39 + 1.21 = 2.52**

\[ Q_{inlet} = \frac{1}{Q_{outlet}} \times \frac{Q_{bypass}}{Q_b} + \frac{Q_b}{Q_c} \]

**Notes:**
1. If using a combination inlet for the sag, the basic gages (last) are not required except in a depressed area (See Hydraulics Manual).
2. Formulas based on other flow. See Hydraulic Manual 5.4.2.
SAG INLET DESIGN WORKSHEET
Combination Inlet at low point

\[ \Delta Q_1 + \Sigma Q + \Delta Q_2 \]

<table>
<thead>
<tr>
<th>L_1</th>
<th>Inlet A</th>
<th>Inlet B</th>
<th>Inlet C</th>
<th>Q_{BP1}</th>
<th>Q_{BP2}</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Transverse Slope \( S_{t} \) 0.020
Allowable \( Z_{a} \) 24.00
Allowable \( d_{a} \) 0.48
Time of Concentration \( T_{c} \) 5.0
50 yr. rainfall coefficients \( a \) 1.0
Raintfall Intensity \( i_{r} \) 3.84

Shoulder Width 0.83
Lane Width 24.00
\( (d_{a} = d_{c} = 0.24 \text{ ft allowable}) \)

Distance between last inlet \( L_1 \) 348.00 ft
Width of catchment area \( W_1 \) 28.83 ft
Bypass from last inlet \( Q_{byp} \) 0.40 cfs
Discharge of catchment area \( Q_{c} \) 0.71 cfs

\[ Q_{\text{total}} = Q_{\text{byp}} + Q_{1} + Q_{\text{c}} + Q_{2} \]

\[ Q_{\text{total}} = 0.46 + 0.71 + 0.23 + 1.26 = 2.68 \text{ cfs} \]

\[ \frac{Q}{2} = Q_{a} + Q_{b} + Q_{c} \]

\[ \frac{Q}{2} = C_{a}P_{a}(0.5d_{a})^{0.5} + C_{b}P_{b}(0.5d_{b})^{0.5} + C_{c}P_{c}(0.5d_{c})^{0.5} \]

\[ d_{b} = \left( \frac{2Q}{C_{a}P_{a}(0.5d_{a})^{0.5} + C_{b}P_{b}(0.5d_{b})^{0.5} + C_{c}P_{c}(0.5d_{c})^{0.5}} \right)^{0.5} \]

If \( d_{b} < \text{allowable } d_{b} \), the design is complete.

If \( d_{b} > \text{allowable } d_{b} \), additional inlets must be added\(^2\) and the process repeated.

Notes:
1. If using a combination inlet for the sag, blank grate inlets are not required except in a depressed area (See Hydraulics Manual).
2. Formulas based on weir flow. See Hydraulic Manual 5-5.2.
A-3.6 STORM DRAIN DESIGN

No enclosed drainage system will be used on this project. Also, there are no storm or sanitary sewers that will be affected by this project.

A-3.7 CULVERT DESIGN (INCLUDING FISH PASSAGE IF APPLICABLE)

New Culverts & Modification

Existing culverts modifications will not be required. Capacity of culverts is more than adequate; peak flows to the existing culverts have been determined using HEC-HMS and concluded to be minimal compared to the maximum capacity the culverts can carry.

Culvert End Sections:

Beveled end sections will not be provided for existing culverts.

A-3.8 DITCH DESIGN

Swales will be provided for water treatment at all locations along the roadway. No riprap protection is warranted in this design as there is low head in this system. The average depth of swale is 1.5 feet. Swales have berms spaced to provide 6 inches of treatment depth per the SRSM. Berms require a variance of 1 inch from the recommended maximum of 8 inches. A berm height of 9 inches is required to provide sufficient head for a drywell discharge of 1 cfs.

For water quality swale design see Appendix A-3.2 for calculations.

Berm sizing and spacing calculations:

\[
\text{Distance between berms} = \frac{\text{water depth}}{\text{swale slope}} = \frac{0.5 \text{ ft}}{0.07} = 7.143 \text{ ft}
\]

7.143 feet is the minimum distance between any of the berms as 7% grade is the steepest that the project will see.

All designed berms have 2:1 up and down slopes, a height of 9 inches, and a 12 inch buffer on top. For the steepest grade along the roadway, the width of the berm would be 51 inches. The minimum width would be 48 inches in areas with slopes less than 0.5% (see Appendix A-3.2).
A-3.9 DOWNSTREAM ANALYSIS (IF CALCULATIONS ARE REQUIRED)

The downstream analysis is not needed.

A-3.10 SPECIAL STREAM DESIGN/CHANNEL CHANGES

The project’s anticipated impact on stream stability will be negligible. There are no new stream crossings on this project therefore stream bank protection and channel changes are not required.

A-3.11 FLOOD PLAIN MITIGATION

Improvements proposed will not modify the flood plain elevation. Location of flood plains are along Hangman Creek located outside of the project limits on the east side of US 195.

A-3.12 BRIDGE SCOUR EVALUATION

No bridges exist, or will be constructed, as a part of this project; therefore a bridge scour evaluation was not performed.
### Appendix A-4 Drainage Plan Sheets and Details

<table>
<thead>
<tr>
<th>Drywell Locator</th>
<th>Left Stations</th>
<th>Right Stations</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2000</td>
<td>2000</td>
</tr>
<tr>
<td></td>
<td>2250</td>
<td>2190</td>
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<td>2380</td>
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<tr>
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<td>2570</td>
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<tr>
<td></td>
<td>3000</td>
<td>2760</td>
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<tr>
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<td>3480</td>
<td>3190</td>
</tr>
<tr>
<td></td>
<td>3720</td>
<td>3430</td>
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<tr>
<td></td>
<td>3960</td>
<td>3670</td>
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<tr>
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<td>4200</td>
<td>3910</td>
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<tr>
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<td>4150</td>
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<tr>
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<td>4680</td>
<td>4390</td>
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<tr>
<td></td>
<td>8310</td>
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</tbody>
</table>

Swales will be constructed along both sides of the new roadway for the entire length of the alignment. **Figure A-4.1** illustrates the typical design and function of the stormwater flow for the alignment. **Tables A-4.1(a) and A-4.1(b)** show the minimum and maximum swale widths.
## Table A-4.1(a) - Minimum Swale Width for Sections Less than 1% Slope

<table>
<thead>
<tr>
<th>Station 1 (ft)</th>
<th>Station 2 (ft)</th>
<th>Half Road Width (ft)</th>
<th>Max Sidewalk Width (ft)</th>
<th>Slope (%)</th>
<th>Runoff Volume (CF)</th>
<th>Min swale width (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2955</td>
<td>4651</td>
<td>26</td>
<td>8</td>
<td>0.50%</td>
<td>1.417</td>
<td>0.269</td>
</tr>
<tr>
<td>6945</td>
<td>8311</td>
<td>26</td>
<td>16</td>
<td>0.87%</td>
<td>1.750</td>
<td>0.685</td>
</tr>
<tr>
<td>9316</td>
<td>9835</td>
<td>26</td>
<td>16</td>
<td>0.90%</td>
<td>1.750</td>
<td>0.685</td>
</tr>
<tr>
<td>13290</td>
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<td>26</td>
<td>8</td>
<td>0.94%</td>
<td>1.417</td>
<td>0.269</td>
</tr>
</tbody>
</table>

## Table A-4.1(b) – Minimum Swale Width for Sections Greater than 1% Slope

<table>
<thead>
<tr>
<th>Station 1 (ft)</th>
<th>Station 2 (ft)</th>
<th>half Road width (ft)</th>
<th>max sidewalk width (ft)</th>
<th>slope (%)</th>
<th>Distance between berms (ft)</th>
<th>Berm width (ft)</th>
<th>min swale width (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2000</td>
<td>2955</td>
<td>35</td>
<td>9</td>
<td>1.52</td>
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<td>3.534</td>
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<tr>
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<td>7.44</td>
<td>4.54</td>
<td>4.195</td>
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<tr>
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<td>8</td>
<td>7.07</td>
<td>7.07</td>
<td>4.57</td>
<td>4.321</td>
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<tr>
<td>6470</td>
<td>6945</td>
<td>26</td>
<td>16</td>
<td>7.07</td>
<td>7.07</td>
<td>4.57</td>
<td>5.691</td>
</tr>
<tr>
<td>8311</td>
<td>9316</td>
<td>26</td>
<td>16</td>
<td>5.68</td>
<td>9.80</td>
<td>4.45</td>
<td>5.081</td>
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<tr>
<td>9835</td>
<td>11293</td>
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<td>16</td>
<td>4.006</td>
<td>12.48</td>
<td>4.32</td>
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<tr>
<td>11293</td>
<td>12532</td>
<td>26</td>
<td>16</td>
<td>4.575</td>
<td>10.93</td>
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<td>4.615</td>
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<tr>
<td>12532</td>
<td>13290</td>
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<td>16</td>
<td>3.63</td>
<td>13.65</td>
<td>4.31</td>
<td>4.311</td>
</tr>
</tbody>
</table>
Appendix A-5 Drainage Profile Sheets

Not applicable.
Appendix A-6  Typical Roadside Cross Sections and Profiles

Figure A-6.1 – Roadway Cross-Sections
Appendix A-7  Misc. Contract Plan Sheets

Not applicable.

Appendix A-8  Traffic Analysis Data (Design Year ADT)

The design life, as dictated by WSDOT’s Pavement Policy manual, are 50 year for Interstates, major arterials, and minor arterials/collectors with 18,000 lbs equivalent single axle loads (ESALs) greater than 100,000 per year. A 20 year design period is used for ESALs less than 100,000 per year. Average Daily Traffic counts were provided by the City of Spokane and shown in the following table:

<table>
<thead>
<tr>
<th>Location</th>
<th>Date</th>
<th>East Bound</th>
<th>West Bound</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>2500 W. 16th</td>
<td>7-May-09</td>
<td>1,687</td>
<td>1,378</td>
<td>3,065</td>
</tr>
<tr>
<td>2600 W. Thorpe</td>
<td>7-May-09</td>
<td>940</td>
<td>790</td>
<td>1,730</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Location</th>
<th>Date</th>
<th>North Bound</th>
<th>South Bound</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cheney Spokane Rd</td>
<td>14-May-09</td>
<td>3,045</td>
<td>3,300</td>
<td>6,345</td>
</tr>
<tr>
<td>1600 S. SR 195</td>
<td>11-Jun-09</td>
<td>10,983</td>
<td>10,188</td>
<td>21,171</td>
</tr>
<tr>
<td>3900 S. SR 195</td>
<td>11-Jun-09</td>
<td>10,896</td>
<td>10,363</td>
<td>21,259</td>
</tr>
</tbody>
</table>

Average of Totals for all locations 10,714

(1) Station ID (Provided by the City of Spokane)

The above ADT totals were taken for one 24-hour period at each location; additional ADT counts are suggested to confirm the design period. Assuming that half the traffic from the total average for all the locations at which the ADT counts were taken will utilize the proposed city street will classify it as a Major Arterial with ESALs greater than 100,000 per year. Therefore the design period is 50 years.

The Pavement Policy classifies principal arterials having a 85% reliability level. The Flexible Pavement Layer Thickness for New Pavements are given in Table 10 within WSDOT Pavement Policy. The effective modulus for the existing sub-grade within the project location is assumed to be average due to the lack of soil samples at this time. An official geotechnical investigation is recommended to confirm the selected layer thicknesses. Table A-8.2 shows layer thicknesses for the proposed new city street.
Table A-8.2 – Pavement Layer Thickness$^{(1)(2)(3)}$

<table>
<thead>
<tr>
<th>Material</th>
<th>Thickness (ft)</th>
<th>Thickness (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HMA</td>
<td>0.35</td>
<td>4.2</td>
</tr>
<tr>
<td>HMAB</td>
<td>0.35</td>
<td>4.2</td>
</tr>
<tr>
<td>CSBC</td>
<td>0.3</td>
<td>3.6</td>
</tr>
</tbody>
</table>

(1) Assumes Average Effective Modulus
(2) Assumes 85% Reliability
(3) Assumes Design Period of 1-5 Million ESALs

Appendix A-9  Environmental Documentation

No environmental research has been conducted.
TEMPORARY EROSION AND SEDIMENT CONTROL

To be completed as part of the bid award process and is the responsibility of the contractor.
US 195 Hangman Valley City Street Network

Plan and Profile Sheets