Volume III

Model Designs

Volume III

The objective of example design section of this Guidance Document is to provide a detailed analysis of each of the chosen alternatives from the selection guidance matrix for each of the design locations. There are five locations that have multiple options for improving the Jordan River. The first location, focused on site controls evaluated bioretention, rainwater harvesting, and permeable pavement designs for implementation in a parking lot example located at the ORP on the University of Utah campus. A bioswale, sand filter, and gutter filter were designed for implementation for a roadway site example located at 4500 South and 600 West. For a developed outfall example evaluating end of pipe controls, a settling basin, cartridge filters, and a Continuous Deflective Separation treatment system were designed for a location at 1300 South 900 West. The constructed wetlands example consists of an extended stormwater wetland and subsurface gravel wetland system located in Bluffdale near 14600 south Redwood Road. Finally, the stream restoration examples that were designed were an in-line detention basin as well as an off-line detention basin. Each of the designs is supplemented with drawings and cost estimates to help evaluate the feasibility of each alternative. The alternatives were then placed into a decision analysis and weighed against social, economic, and environmental criteria to quantitatively provide a recommendation. A map and summary of each location and design alternatives are shown in Figure 4.1 and Table 4.1 below.

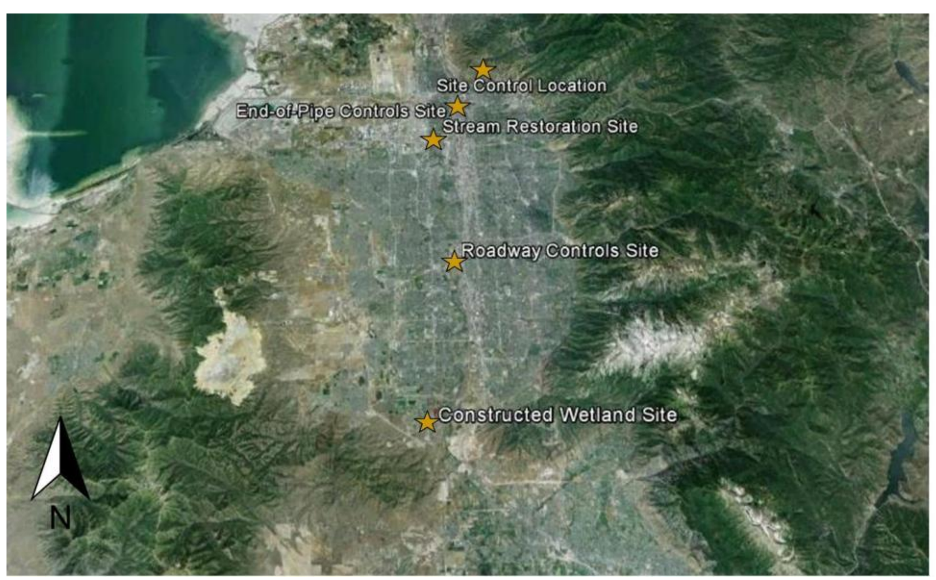

Figure 3.1: Map of Design Example Locations (Source: www.earth.google.com)

3.1 Parking Lot Design Example

The following is an example of a bioretention designed to treat discharge from a parking lot. This design is intended as an illustrative example, rather than a ready-to-build design. Because of this, some factors have been idealized for simplicity.

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3.1.1 Overview

In 2012, the United States EPA is holding the “Campus Rainworks Challenge”, in which “student teams are invited to create an innovative green infrastructure design for a site on their campus showing how managing stormwater at its source can benefit the campus community and the environment” [1]. Students at the University of Utah created a team to enter into this contest and chose to create designs for the University’s Outdoor Recreation Program (ORP) parking lot. SEA felt that this parking lot was a good opportunity to showcase a bioretention design and will design an example bioretention system for the southern portion of the parking lot.

3.1.1.1 Location

The selected site for this design example is the Outdoor Recreation Program (ORP) parking lot. The ORP parking lot is located just off Stover St and Connor Rd at the University of Utah (*Figure* 3.1). The drainage area considered for this design is the southern portion of the parking lot (*Figure* 3.1). Note that any rainfall north of this area drains to a separate location and does not contribute to erosion south of the parking lot.

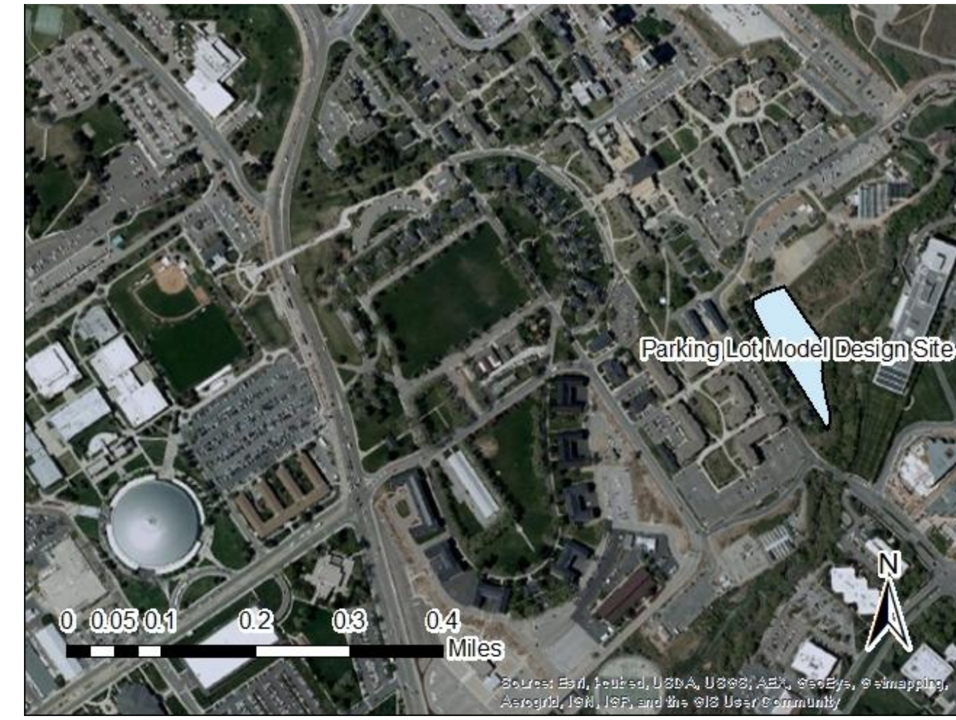

Figure 3.1: Site Location Arial View (Source: www.arcgis.com)

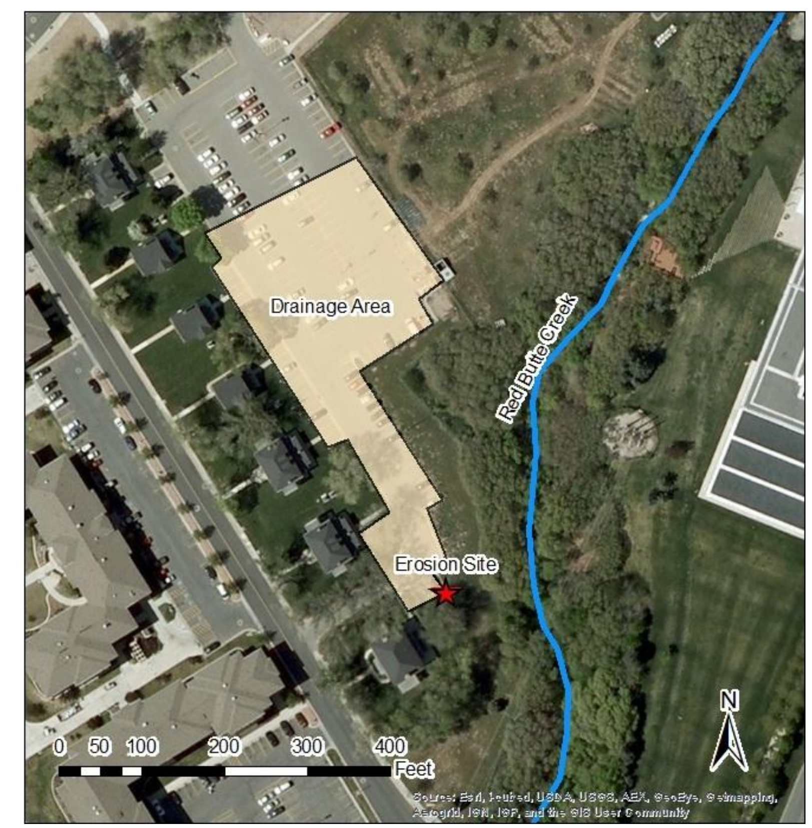

Figure 3.2: Site Location Arial View (Source: www.arcgis.com)

3.1.1.2 Benefits

The current drainage system at the southeast corner of the parking lot is not adequate. Visual inspection of the site reveals erosion along the hillside from the drain inlet to Red Butte Creek. SEA has concluded that during a storm event, the drain overflows. The excess water is discharged down the slope towards Red Butte Creek, creating erosion. This discharge already contains pollution from the parking lot and collects a large amount of sediment while it erodes the hillside. A control measure is needed both to eliminate erosion and mitigate the settlement entering Red Butte Creek.

Figure 3.3: Erosion South of ORP Parking Lot

3.1.2 Description of Solution

The designer is encouraged to review the fact sheets in Volume II and the selection guidance in Volume III to determine which stormwater control measure best fits their location. For this parking lot, a bioretention system was chosen because it has a low life-cycle cost, while providing a high level of pollutant removal and flood control. Additionally, because bioretention only requires limited space, the recommended design will not reduce the capacity of the parking lot. Finally, bioretention is an aesthetically appealing control measure, as it imitates the natural habitat of the area.

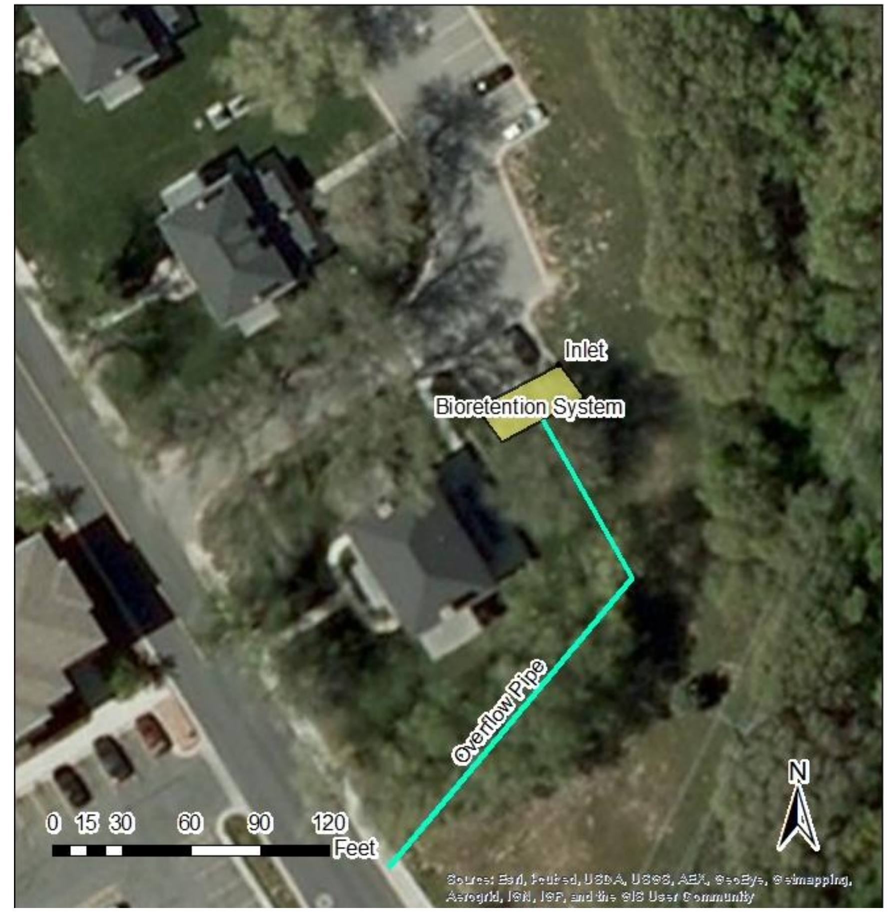

Figure 3.4: Bioretention System Location (Source: www.arcgis.com)

3.1.3 Basis of Design

The following describes the conditions that the engineer must either determine or assume for the design of a bioretention system. Note that this is not an exhaustive list and the required information will vary based on factors such as climate and local regulations.

3.1.3.1 Drainage Area

It is recommended that the designer determine the drainage area based on a site survey, GIS data, or both. For this example design, a site visit determined the boundaries of the drainage area. Following the site visit, the boundaries were approximated on a GIS map. Using GIS, the drainage area was calculated as 0.66 acres.

3.1.3.2 In Situ Soil

The engineer is advised to obtain a soils report from a qualified geotechnical engineer. Parameters that should be obtained from this report may include in situ soil porosity, hydraulic conductivity, depth to groundwater table, etc. For this design example, a soils report was not obtained. Rather, in situ soil data was approximated based on information from the Natural Resources Conservation Service (NRCS). From this data it was estimated that the in situ soil is Bingham Gravelly Loam [2]. It should be noted that this method is not recommended for design of an actual bioretention system.

3.1.3.2.1 Porosity

As mentioned previously, the engineer is advised to obtain a soils report from a qualified geotechnical engineer. For this example, the porosity of the soil was estimated as 40%. This value was chosen because it was given by Ley et. al. as an average value for gravelly loam [3].

3.1.3.3 Storage Volume

The designer should look to local regulations to determine the required storage volume for bioretention systems. If local regulations do not specify this volume, the minimum recommended storage volume for bioretention design is the Water Quality Capture Volume (WQCV). The WQCV was initially calculated to optimize the pollutant removal effectiveness of detention basins with respect to costs [4]. While the theory behind this volume may not be directly applicable to bioretention systems, SEA believes that the WQCV will effectively treat the first flush. The engineer may also choose to design the system for a volume larger than the WQCV. The engineer should understand that in cases where first flush loading is not a concern, increasing the storage volume will increase costs without greatly increasing pollution removal. For this design example, the WQCV was calculated to be 1110 ft^3 (see section 3.1.4). This volume was considered acceptable for this example because of the limited space available south of the parking lot.

3.1.3.4 Storage Media

It is recommended that the engineer select an aggregate with a high porosity for the storage media. For this design example, Utelite medium aggregate was chosen because it is made by a local company and has a porosity of 47% [5].

3.1.3.5 Vegetation

Because the ability of a plant to thrive varies greatly from climate to climate, the designer is advised to obtain a listing of vegetation native to the area. Because regular irrigation of the bioretention system is not practical, it is recommended that drought-resistant plants are chosen in arid climates. Additionally, the vegetation chosen should be host to arbuscular mycorrhizal fungi (AMF) because this increases the ability of the plant to thrive in the first year while also improving the hydraulic properties of the soil [6]. For plant quantity, it is recommended that a 12 inch buffer is between vegetation and the edge of the bioretention [7]. For this design example, a recommended list of vegetation was obtained from Houdeshel et. al. Plants chosen were blue grama grass, rubber rabbitbrush, and fireweed. The number of plants required was calculated using the open source website, Landscape Calculator. Using this calculator, it was estimated that 699 #1 size plants were needed for this bioretention system [7].

3.1.4 Calculations

The following sections guide the designer through the minimum calculations that must be made for design of a bioretention design. The engineer may take two approaches to determine bioretention size:

1. Determine bioretention area then calculate the required soil depths.
2. Determine the desired soil layer depths then calculate the required bioretention area.

The first approach is recommended when there are space constraints on the bioretention. Because space south of the parking lot is limited, the first approach was chosen for this design example.

The engineer will notice that this design example assumes that the entire storm volume is captured before any infiltration occurs. This may result in an overly conservative design and, if given time, the engineer should perform infiltration calculations to reduce the size of the bioretention system. Given little time the calculations that follow are suitable for design.

3.1.4.1 Required Storage Volume

As mentioned previously, the minimum recommended storage volume is the water quality capture volume. If space and money permits, the bioretention may be designed for a larger volume. This design example illustrates the calculation of WQCV, the 2-year 24-hour storm, and the 10-year 24-hour storm. The calculated volumes are summarized in **Error! Reference source not found.**.

3.1.4.1.1 Water Quality Capture Volume

The water quality capture volume was created to optimize the sizing of stormwater control measures that work through sedimentation (i.e. detention basins) [4]. SEA, however, believes that this volume will be effective at treating the first flush and suggests that the designer may use this volume to size bioretention systems where first flush is of primary concern. These calculations are based on the Water Environment Federation (WEF) Design of Urban Stormwater Controls Manual of Practice 87 (MOP 87). The WQCV is captured as follows:

$$WQCV = P_0 A \qquad (3.1.1)$$

$WQCV = Water\ Quality\ Capture\ Volume\ (ft^3)$
$P_0 = Captured\ Precipitation\ Depth\ (feet)$
$A = Drainage\ Area\ (ft^2)$

$$P_0 = aCP_{mean} \qquad (3.1.2)$$

$a = Drawdown\ time\ coefficient\ (Unitless)$
$C = Runoff\ Coefficient\ (Unitless)$
$P_{mean} = Mean\ precipitation\ (feet)$

The drainage area was previously determined. The designer may obtain the drawdown time coefficient from Table 3.1.1. It is recommended that a draw-down time of 24 hours is used. If water is pooled in the bioretention system for an extended period of time, adverse effects may occur, such as mosquito breeding.

Table 3.1.1: Drawdown time coefficients [8].

Drawdown Time	Drawdown time coefficient (a)
12	1.109
24	1.299
48	1.545

The runoff coefficient, C, may be obtained from the WEF MOP 87. For parking lots, the runoff coefficient is 0.95 [8].

The mean precipitation is also obtained from the WEF MOP 87. For Salt Lake City, the mean precipitation is 3.7 inches.

3.1.4.2 Soil Layer Depth

The designer must design the soil layers to allow the specified drainage volume to be stored within the bioretention. The recommended top soil layer depth is 1'-8" so as to allow the vegetation to thrive in the first year after planting. It is also recommended that the storage layer is kept around 2' thick to allow vegetation to root through the layer [6]. This layer may be deeper or shallower based on the vegetation being used. The designer is advised to determine obtain rooting information about the vegetation that is used. The maximum recommended ponding depth is 2' to eliminate safety concerns. For this design example, a ponding depth of 6", a top soil depth of 1'-8", and a storage layer depth of 2' were chosen.

$$V_{storage} = A_{bio}(d_p + n_{TS}d_{TS} + n_{stor}d_{stor}) \qquad (3.1.3)$$

$V_{storage} = the\ storage\ capacity\ of\ the\ bioretention\ system\ (ft^3)$
$A_{bio} = the\ area\ of\ the\ bioretention\ system\ (ft^2)$
$d_p = the\ surface\ ponding\ depth\ of\ the\ bioretention\ system\ (ft)$
$n_{TS} = the\ porosity\ of\ the\ bioretention\ top\ soil\ (Unitless)$
$d_{TS} = the\ depth\ of\ the\ top\ soil\ layer\ (ft)$
$n_{stor} = the\ porosity\ of\ the\ bioretention\ storage\ layer\ (Unitless)$
$d_{stor} = the\ depth\ of\ the\ storage\ layer\ (ft)$

3.1.5 Life-Cycle Cost Estimates

Table 3.1.2 contains the estimated life-cycle cost for the recommended bioretention system. It should be noted that the estimated cost is based partially on average costs in the United States and partially on average costs in the state of Utah. Costs may vary greatly even within the state of Utah and over time.

Table 3.1.2: Bioretention Cost Estimates

Installation	Unit	Quantity	Unit cost	Cost	Source	Notes
3/4" Decorative Rock	CY	5	$75.00	$375.00	Wholesale Landscape Supply, UT	Typical local cost; 2" depth
Utelite Medium Aggregate	CY	55	$46.37	$2,550.35	Utelite Corporation	
Blue Grama Grass	#1	350	$9.09	$3,181.50	Glover Nursery	Local nursery; assumes 12" plant spacing and 12" buffer at all edges of bioretention
Fireweed	#1	175	$5.60	$980.00	Glover Nursery	Local nursery; assumes 12" plant spacing and 12" buffer at all edges of bioretention
Rubber Rabbitbrush	#1	175	$6.50	$1,137.50	Glover Nursery	Local nursery; assumes 12" plant spacing and 12" buffer at all edges of bioretention
Labor	HR	32	$40.00	$1,280.00	Utah Workforce Services	Labor only for placement of soil; soil cannot be placed with backhoe and must be placed by hand
1-1/4 CY Backhoe	CY	100	$5.96	$596.00	Walker's Building Reference Estimators Book	Includes foreman, operator, and equipment; includes all excavation and soil placement
Tree Removal	Each	1	$987.00	$987.00	Utah NRCS	Average cost of tree removal in Utah; includes all equipment
Pipe Placement	Linear Foot	250	$58.50	$14,625.00	Utah NRCS	Black Steel Pipe - Plain end, Welded - 12" - Includes labor and machinery for assembly but not excavation costs or shipping
Pipeline Excavation	CY	42	$5.50	$231.00	Utah NRCS	Pipeline Installation, Excavated BCY, Loam, Sandy Clay, Sand, or Gravel - Includes Labor
20 CY Rear Dump Trucks	CY	60	$9.79	$587.40	Walker's Building Reference Estimators Book	Inludes truck rental, gas, and dissposal ; for removal of excavated soil
Site Maintenance	**Unit**	**Quantity**	**Unit cost**	**Cost**	**Source**	**Notes**
Labor	HR	48	$40.00	$1,920.00	Utah Workforce Services	On average companies charge $40.00/hr/worker
Recurring Maintenance	**Unit**	**Quantity**	**Unit cost**	**Cost**	**Source**	**Notes**
Topsoil	CY	45	$35.00	$1,800.00	Wholesale Landscape Supply, UT	20 year average replacement
Utelite Medium Aggregate	CY	55	$46.37	$2,200.00	Utelite Corporation	20 year average replacement
1-1/4 CY Backhoe	CY	100	$5.96	$596.00	Walker's Building Reference Estimators Book	Includes foreman, operator, and equipment
20 CY Rear Dump Trucks	CY	100	$9.79	$979.00	Walker's Building Reference Estimators Book	Inludes truck rental, gas, and dissposal
	Total Installation Cost			$26,530.75		
	Annual Site Maintenance Cost			$1,920.00		
	20 Year Recurring Maintenance Cost			$5,575.00		
	Life-Cycle Cost			$62,646.62		

3.1.6 Construction Drawings

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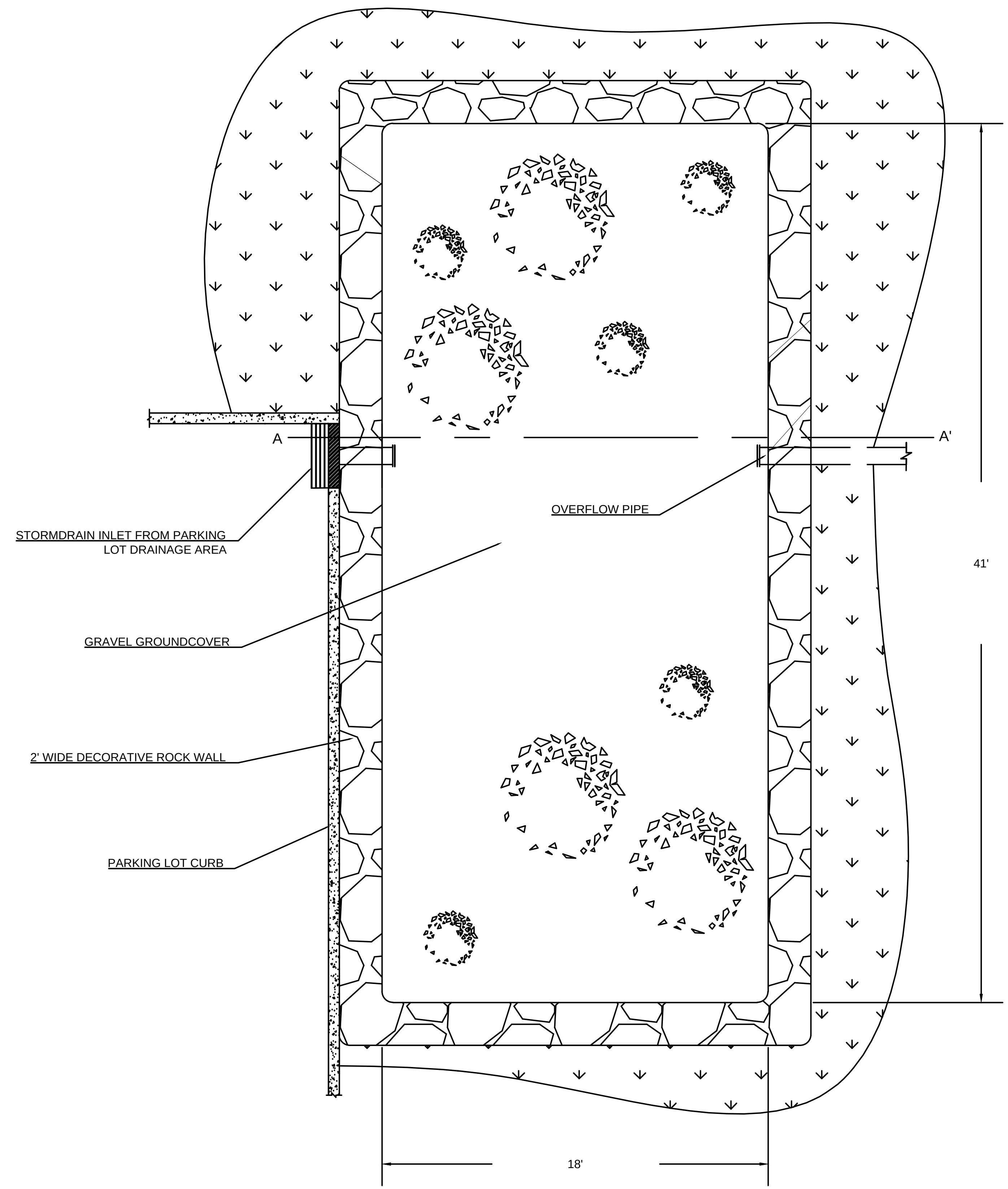

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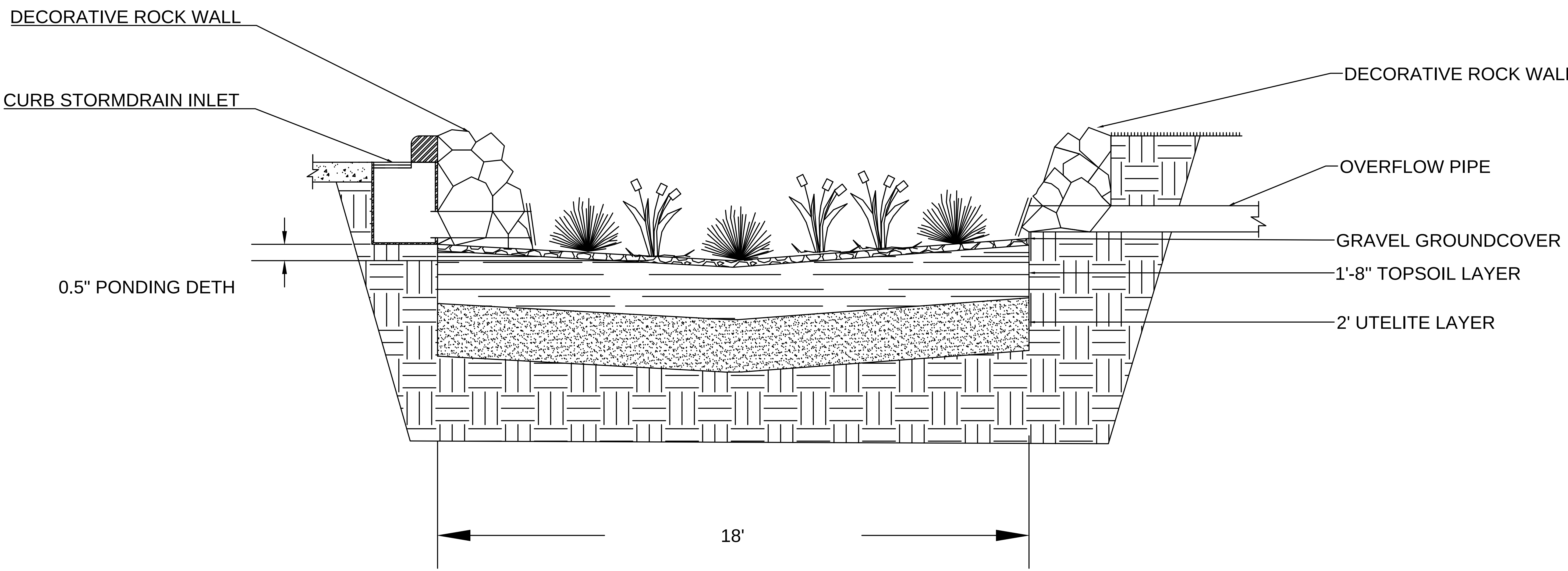

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3.1.7 Works Cited

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3.2 Roadway Site Example

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3.2.1 Overview

The main emphasis of roadway controls is runoff pollution associated with stormwater that runs off bridges and roads. As stormwater flows over these surfaces it picks up dirt, dust, automotive liquids, heavy metals, organic matter, bacteria, and debris. These contaminants are then carried to the nearest body of water. The overall goal of the JRC and SEA is to improve the water quality and habitat surrounding the Jordan River. In order for this goal to be met, roadway controls must be implemented. The following sections will address the recommended stormwater control measure and its pertaining calculations.

3.2.1.1 Location

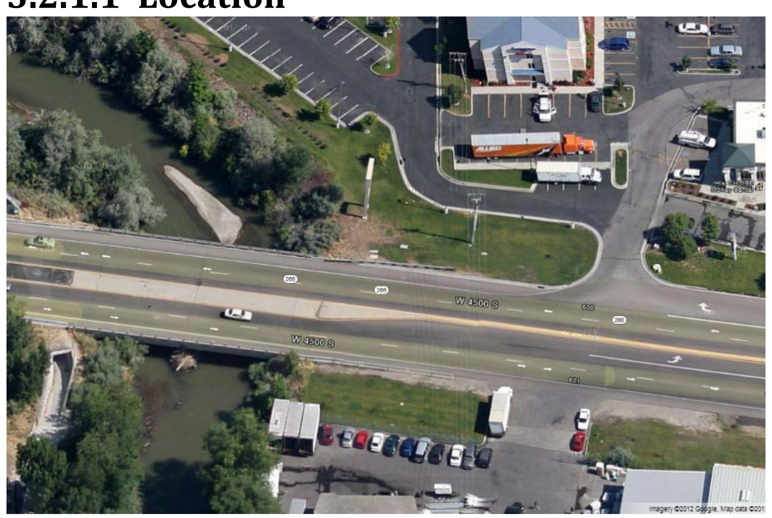


Figure 3.2 1. 4500 South 600 West Roadway Site

The selected site is a grass filled sloped area located north of 4500 south and about 600 west, as seen in Figure 3.2.1. The site is directly east of the Jordan River and is pre-fitted with a grated drain inlet. The drain outlet is located under the bridge and discharges directly into the river. This site was chosen due to its close proximity to the Jordan River and the heavy traffic on 4500 south. The combination of a high traffic volume and close proximity to the Jordan River provides a great opportunity to treat and remove a large amount of contaminants before they enter the river.

3.2.1.3 Benefits

The Jordan River and its stakeholders will benefit from the implementation of this stormwater solution because it will significantly decrease the amount of TSS, nutrients, roadway deicing salts, heavy metals, and oils and grease that are discharged from the adjacent roadway into the Jordan River [1]. By reducing these pollutants and sediments the water in the Jordan River will become less turbid and chemically cleaner. The cleaner river will promote environmental, residential, and commercial growth along its banks.

3.2.1.4 Description of Solution

Bioswales are landscape features designed to remove silt and pollution from stormwater. They usually consist of a drainage course with sides that slope gently downward. The drainage course can have any arrangement of vegetation, compost, and stones. If there is adequate space, the swale is designed to meander in order to maximize time that water is in the swale. Some bioswales are simply rain gardens that do not have a storm drain because the flows do not require drainage in addition to the ground [1].

3.2.2 Basis of Design

The bioswale that has been designed for the location East of the bridge at 4500 S and 600 W, Murray, UT, will be of the drainage course variety. Gently sloped, grass covered sides will take water from the roadway and direct it into the bioswale. The treatment area of the bioswale will consist of medium sized stones that will slow the velocity of the water and retain it longer, allowing treatment to be more effective. Among the stones will be a variety of bushes, shrubs, tall grasses, and medium sized trees planted in topsoil that is located beneath the stones. The purpose of the flora in the bioswale is to reduce the velocity of the stormwater runoff and remove pollutants through the process of root uptake as well as provide additional aesthetic quality. Some pollutants in roadway stormwater are beneficial to plants. The layer of topsoil beneath the stones will provide trickle-through filtration that will remove TSS, heavy metals, and oils and grease as the water travels through the pores in the soil [1]. After the water has traveled through vegetation it is discharged through the grated inlet at the site location. This grated inlet will then output stormwater into the Jordan River.

- Design Flow

The bioswale channel is designed to handle the 10-year, 24-hour storm. This storm produces a peak run-off rate of 1.62 ft^3/s. The stormwater will flow at a normal depth of 4.62 inches.

- Maintenance

The primary maintenance required for a swale is removing debris from the swale and mowing the grass. The grass should not be cut shorter than the design flow depth of the bioswale channel. On annual basis the swale should have sod replace in areas that have eroded.

3.2.2.1 Calculations

The bioswale channel has been designed for the 10-year, 24-hour storm [2]. To calculate the peak run-off rate [eq. 3.2.2], intensity data was used from the National Oceanic and Atmospheric Administration. The bioswale channel was designed with 3:1 grass lined side slopes and longitudinal slope of 1%. When calculating the normal flow depth it was necessary to compute the Manning roughness coefficient [eq. 3.2.3] for the bioswale channel. This was done by using empirical retardance factor for a grass-legume mixture [2]. The normal depth was then back calculated using the Manning's Equation [eq. 3.2.1].

Manning's Equation:

$$Q = \frac{1.49}{n} R^{2/3} S_f^{1/2}$$

[3.2.1]

Where:

Q = Flow rate (cfs)
A = Cross sectional area of flow (ft²)
R = Hydraulic radius (ft)
S = Bottom Slope of Channel (ft/ft)
n = Manning's roughness coefficient

Peak Runoff Rate:

$$Q = CiA$$

[3.2.2]

$$Q = (0.9)\left(\frac{0.267\ in.}{5\ min.}\right)(.56\ acres) = 1.62\ ft^3/s$$

Where:

Q = Peak Runoff Rate (ft^3/s)
C= Coefficient of Runoff
i = Rainfall Intensity (ft/s)
A= Runoff Area (ft^2)

Manning Roughness Coefficient:

$$n = \frac{(RK_v)^{1/6}}{C_n + 19.97\log[(RK_v)^{1.4} S_o^{0.4}}$$

[3.2.3]

$$n = \frac{(0.31876ft \times 1.0)^{1/6}}{30.2 + 19.97\log[(0.31876ft \times 1.0)^{1.4} 0.01ft/ft^{0.4}]}$$

$$n = 0.06823$$

Where:

R = Hydraulic Radius (ft)
K_v = Unit Conversion Factor (ft^{-1})
C_n = Dimensionless Retardance
S_o = Bottom Slope of Channel (ft/ft)

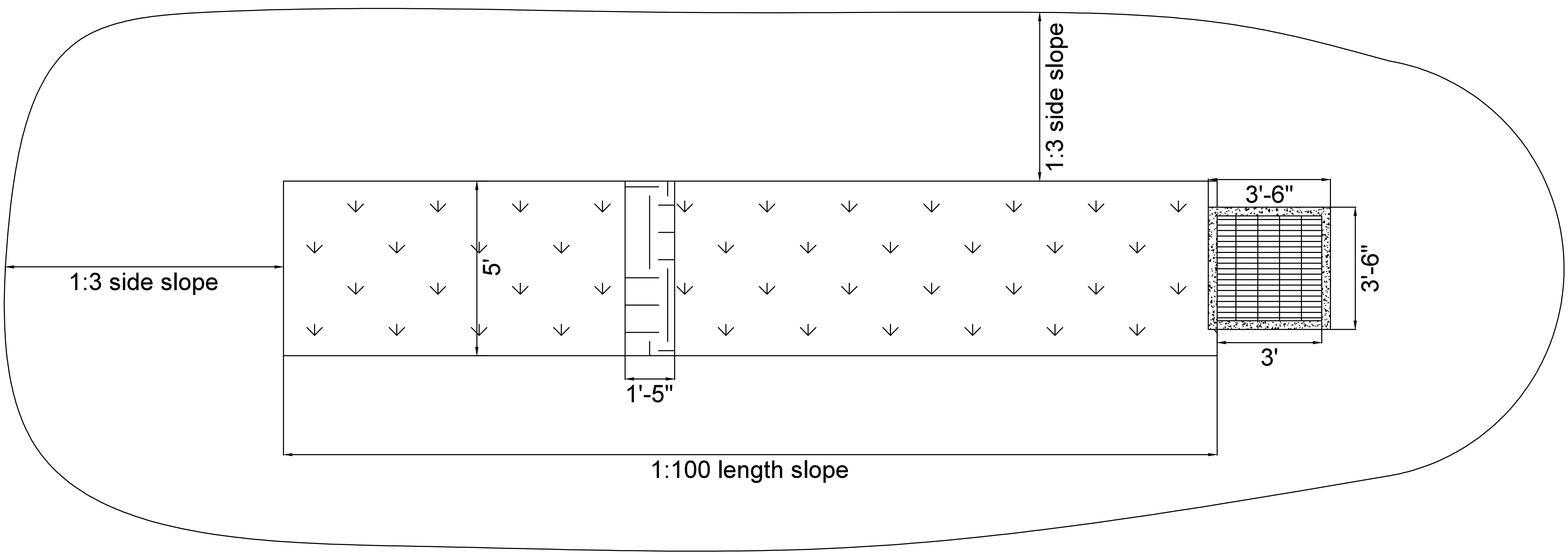
3.2.3 Cost Estimates

The cost estimation was based on pricing obtained from local retailers [2], [3]. This cost estimate is intended to be used as rough estimation of the installation cost as well as the routine maintenance. The cost estimate located in Table 3.2.1 is based off of the material, labor, excavation and maintenance costs. The costs should be considered an estimate and susceptible to change, as the design varies.

Table 3.2.1. Example of Cost Estimate

Installation	Unit	Quantity	Unit cost	Cost	Source	Notes
Plants and Vegetation	Plant	150	$20.00	$3,000	J & L Garden Center, UT	This is just an average unit cost of plants
Underdrain Pipe	FT	500	$10.00	$5,000	Double E trucking	This is an average price per foot
Mulch	CY	33	$35.00	$1,155	J & L Garden Center, UT	This is just an average unit cost of mulch
Grass Seed	SF	250	$20.00	$5,000	J & L Garden Center, UT	This is just an average unit cost of grass seed
Dump Truck, 12 CY Capacity	HR	24	$65.00	$1,560	Utah NRCS	For hauling ecavated material and mulch
Site Maintenance	**Unit**	**Quantity**	**Unit cost**	**Cost**	**Source**	**Notes**
Labor	HR	48	$40.00	$1,920	Utah Workforce Services	On average companies charge $40.00/hr/worker
Plants and Vegetation	Plant	50	$20.00	$1,000	J & L Garden Center, UT	This is just an average unit cost of plants
Recurring Maintenance	**Unit**	**Quantity**	**Unit cost**	**Cost**	**Source**	**Notes**
Grass Seed	SF	125	$20.00	$2,500	J & L Garden Center, UT	20 year average replacement
Mulch	CY	33	$35.00	$1,155	J & L Garden Center, UT	20 year average replacement
Plants and Vegetation	Plant	150	$20.00	$3,000	J & L Garden Center, UT	20 year average replacement
	Total Installation Cost			$15,715		
	Annual Site Maintenance Cost			$2,920		
	20 Year Recurring Maintenance Cost			$6,655		

3.2.4 Construction Drawings



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Roadway Example

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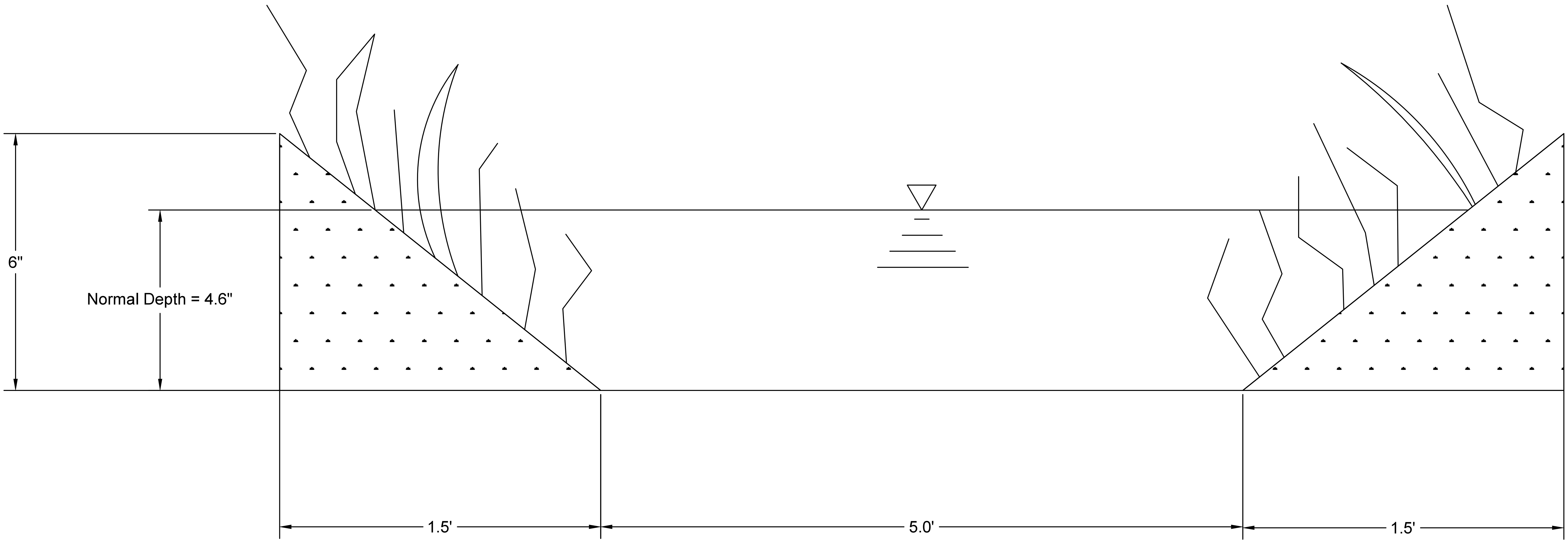

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Roadway Example

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Cross Section

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3.2.2

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3.3 Developed Outfall Example

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3.3.1 Overview

A mechanical treatment facility can be used as an effective developed outfall structure for the improvement of stormwater quality prior to discharge into the Jordan River. Stormwater quality can be improved through the removal of TSS, organic matter, nutrients, pollutants, and debris. Mechanical treatment facilities typically require small areas of land and can be retrofitted to an existing stormwater sewer system, which makes them ideal for developed urbanized areas.

This example design is a basic design outline to be used for the selected urbanized location. However, mechanical treatment facilities can be used for a variety of different applications. Mechanical treatment facilities are commonly used for the treatment of stormwater collected upon parking lots and industrial sites prior to introducing the stormwater into the storm sewer system.

3.3.1.1 Location

The site selection for the developed outfall example is located at 1300 South 900 West in Salt Lake City, Utah. The objective for this site is to design a SCM to improve the quality of stormwater prior to discharge into the Jordan River. The 1300 South outfall structure discharges the stormwater that is collected from Red Butte Canyon, Emigration Canyon, Parleys Canyon, and the urban runoff that is collected in the storm sewers along 1300 South. For this site location it would be unfeasible to construct a stormwater treatment facility with enough capacity to treat the total annual discharge due to space constraints.

The 1300 South site has two outfall structures that discharge into the Jordan River. There is a large rectangular stormwater outfall structure that has a cross section of approximately fifteen by five feet, and a 48 inch diameter storm sewer pipe. A mechanical treatment facility can be implemented at this location by intersecting the 48 inch storm sewer pipe and diverting a percentage of the stormwater flow into the mechanical treatment facility. During smaller storm events the treatment facility will have the capacity to filter all of the stormwater discharge. During the spring runoff and large storm events the high stormwater flow rates will bypass the treatment facility and be discharged out of the 48 inch storm sewer pipe into the Jordan River. Figure 3.3.1 displays the location for the developed outfall example.

Figure 3.3.1: Mechanical treatment facility site location (Source: www.google.earth.com)

3.3.1.2 Benefits

The addition of a mechanical treatment facility to an existing stormwater outfall structure will improve the stormwater quality prior to discharge into the Jordan River. Treatment facilities are capable of reducing the contaminants generated during the first flush. By reducing the organic load within the stormwater it will help to improve the dissolved oxygen content of the Jordan River. Mechanical treatment facilities can also efficiently remove TSS, trash, and oil and grease. The reduction of contaminates in the stormwater will improve the aesthetics of the Jordan River by the removal of turbidity and trash that collects in the storm sewer system.

3.3.2 Description of Solution

The End-of-Pipe stormwater contaminant removal system chosen for the developed outfall example is a Continuous Deflective Separator (CDS). This technology screens, separates, and traps debris, sediment, organic material, and oil and grease from stormwater runoff. The CDS units are designed to achieve an 80 percent annual solids load reduction based on an average particle size of 125-microns [1]. Inline systems can treat up to 6 cfs and offline system can be designed to treat 1-300 cfs. The unit chosen for the 1300 S 900 W location is an offline system designed for 7.5 cfs.

The design for this system will consist of the CDS unit and an upstream stormwater flow diverter. The CDS unit is a precast concrete cylinder 8 feet in diameter and 16 feet tall with an inlet, outlet, separation cylinder, debris sump, and manhole access cover. The connections will be to 24" pipe that discharges to the Jordan River downstream and connects to the diverter upstream. The upstream diverter will be a cast-in-place concrete box in line with the existing pipe that splits the flow of water continuously supplying the CDS and sending water through the existing pipe when flow exceeds 7.5 cfs. The upstream diverter was sized and designed to replace a single eight foot section of the existing 48 inch storm sewer pipe. During the excavation for the upstream diverter, care should be given not to disturb or damage the existing storm sewer pipe.

3.3.2.1 Basis of Design

The CDS unit will be designed to improve the quality of stormwater runoff that enters the Jordan River, therefore improving the overall water quality of the Jordan River. The mechanical treatment system will remove TSS, organic matter, debris, and other pollutants by filtration and hydrodynamic separation.

The site provided as an example for a retrofit project is the outlet located at 1300 South 900 West in Salt Lake City, UT.

- Water Source Analysis

The outlet into the Jordan River located at 1300 South 900 West discharges a combined runoff from Emigration Canyon, Red Butte Canyon, and Parley's Canyon watersheds. The exact nominal and maximum discharge rates are unknown at this point, however it is clear that the flow rates are much higher than the nominal capacity of most mechanical systems. The exact levels of contamination are also unknown.

- Design Flow

The CDS system will be designed to handle a maximum flow of 7.5 cfs with an internal bypass of 30 cfs. The internal bypass allows flow rates greater than the design flow rate to circumvent the filtration chamber without the re-suspension of previously trapped contaminants.

- Maintenance

All treatment systems will require inspection and maintenance after every major storm event, at monthly intervals for the first year in order to develop a yearly maintenance schedule. At minimum, bi-annual maintenance and inspection shall be conducted.

- CDS System Parameters

The CDS System alternative will use treatment screens, separation cylinder, and oil baffle for contaminant removal. Recommended maintenance and removal of solids should occur when the unit storage capacity reaches 75 percent, and as suggested above.

- Sizing

The standard size for the CDS system is an eight foot diameter cylindrical concrete vault.

- Service Connection Data

All connections shall be sealed with a joint sealing compound in order to prevent seepage. Piping connections will be made to the eight foot diameter cylindrical concrete vault, upstream diverter, and the 24 inch & 48 inch storm sewer piping.

- Water Treatment

1. Minimum removal of 80 percent TSS with average particle size of 63 microns [1].
2. Capturing and retaining 100 percent of pollutants greater than or equal to 2.4 mm [1].
3. Capturing and retaining 85 percent of petroleum hydrocarbons [1].

3.3.3 Calculations

The CDS treatment system has the capacity to treat 7.5 cfs of stormwater with an internal flow bypass of 30 cfs. An upstream diversion unit needs to be installed into the existing 48" storm sewer pipe in order to divert a maximum flow rate of 7.5 cfs into the CDS treatment system. Manning's equation was used to verify if a 24" storm sewer pipe flowing at capacity will allow the design flow rate to enter the CDS treatment system. A slope of 0.001 ft/ft and a Manning's roughness coefficient of 0.013 were assumed for the calculations. This assumption was verified by back calculating the flow if the velocity was within 2-3 ft/s. A velocity in this range will prevent sediment deposition or scouring of the pipe [2]. Any variation to the slope of the storm sewer pipe will greatly affect the flow rate. If the site location requires a steeper slope, the designer should select a smaller pipe size in order to regulate the flow rate.

Manning's Equation:

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

3.3.1

Where:

Q = Flow rate (cfs)
A = Cross sectional area of flow (ft^2)
R = Hydraulic radius (ft)
S = Slope of the storm sewer pipe (ft/ft)
n = Manning's roughness coefficient

Flow area equation:

$$A = \frac{\pi d^2}{4}$$

3.3.2

Where:

d = diameter of concrete sewer pipe (ft)

Wetted perimeter equation:

$$WP = \pi d$$

3.3.3

WP = Wetted perimeter for pipe flowing full (ft)

Hydraulic radius equation:

$$R = \frac{A}{WP}$$

3.3.4

For a 24 inch concrete pipe (n = 0.013) flowing full and slope (S = 0.001 ft/ft):

$$A = \frac{\pi (2\ ft)^2}{4} = 3.14\ ft^2$$

$$R = \frac{A}{WP} = \frac{3.14\ ft^2}{\pi (2\ ft)} = 0.500\ ft$$

$$Q = \frac{1.49}{0.013} (3.14\ ft^2)(0.5)^{\frac{2}{3}} (0.001)^{\frac{1}{2}} = 7.17\ cfs$$

Check flow velocity, V (ft/s):

$$V = \frac{Q}{A} = \frac{7.17\ cfs}{3.14\ ft^2} = 2.28\ ft/s$$

The design flow rate for the 24 inch storm sewer pipe was determined to be 7.2 cfs, which slightly lower than the capacity for CDS treatment unit. Minimal amount of sediment will be collected in the 24 inch storm sewer line due to the flow velocity was calculated to be 2.29 ft/s, which is within acceptable range of 2-3 ft/s.

3.3.4 Cost Estimates

The cost estimation was based on pricing obtained from Contech Engineered Solutions sales representative for the purchase and installation price of the CDS 4045 model number. Pricing for the excavation and installation for the concrete storm sewer pipe was obtained from cost estimation handbooks [3] and online resources. A detailed cost estimate is shown in Table 3.3.1, howerver prices are susceptible to change due to variation in design. This cost estimate was created to provide an example for the designer for the pricing of a mechanical treatment facility.

Table 3.3.1: Cost estimate for the CDS treatment system

Item	Unit	Quantity	Unit cost	Extended cost	Source	Notes
Capital Cost						
CDS:						
CDS 4045	EA	1	$99,000	$99,000	Sales representative from Contech Engineering Solutions	All Capital Cost includes material and installation
24" Storm Sewer Pipe	LF	40	$58.10	$2,324	fortpecktribes.org/asrwss	
Diversion Unit:						
Reinforced Concrete	CY	6	$185	$1,110	Walker's Building Estimator's Reference Book: 28th Edition	
Steel Rebar	LB	510	$1.85	$944	Walker's Building Estimator's Reference Book: 28th Edition	
36" Sewer Lid	EA	1	$2,000	$2,000	rockvillemd.gov	
Outfall Structure:						
Reinforced Concrete	CY	4	$185.00	$749	Walker's Building Estimator's Reference Book: 28th Edition	
Steel Rebar	LB	430	$1.85	$796	Walker's Building Estimator's Reference Book: 28th Edition	
24" Storm Sewer Pipe	LF	40	$58.10	$2,324	fortpecktribes.org/asrwss	
Steel Grating (4'x4')	EA	1	$1,200	$1,200	usbr.gov/pmts	
Annual Maintenance						
Upkeep	Yearly	1	$1,400	$1,400	scvurppp-w2k.com/permit	
Inspection & Cleanout	Monthly	12	$320	$3,840	simplyhired.com/jobsalaries	assuming $40/hr rate for 8 hrs
Vacuum Truck	Monthly	12	$495	$5,940	dohenysupplies.com/rentals	
Recurring Maintenance						
Replacement of filter screens	EA	1	$12,500	$12,500	Sales representative from Contech Engineering Solutions	10 yr average replacement
		Total Capital Cost		$110,446		
		Total Annual Maintenance		$9,780		
		Total Recurring Maintenance		$12,500		

3.3.5 Construction Drawings

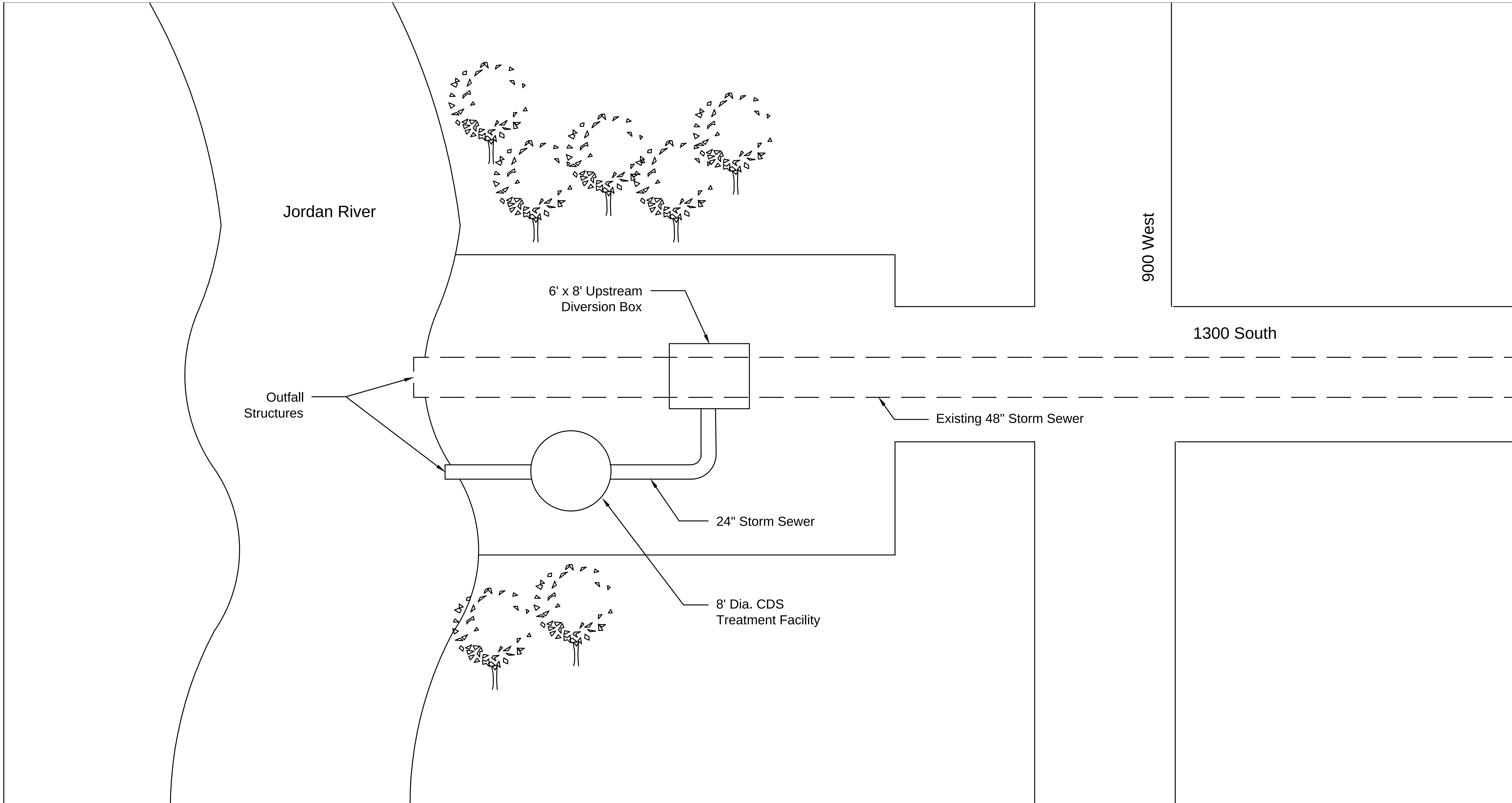
List of Drawings

3.3.1 – Plan View

3.3.2 – Upstream Diversion Box

3.3.3 – CDS Treatment System

3.3.4 – Outfall Structure



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UNIVERSITY OF UTAH

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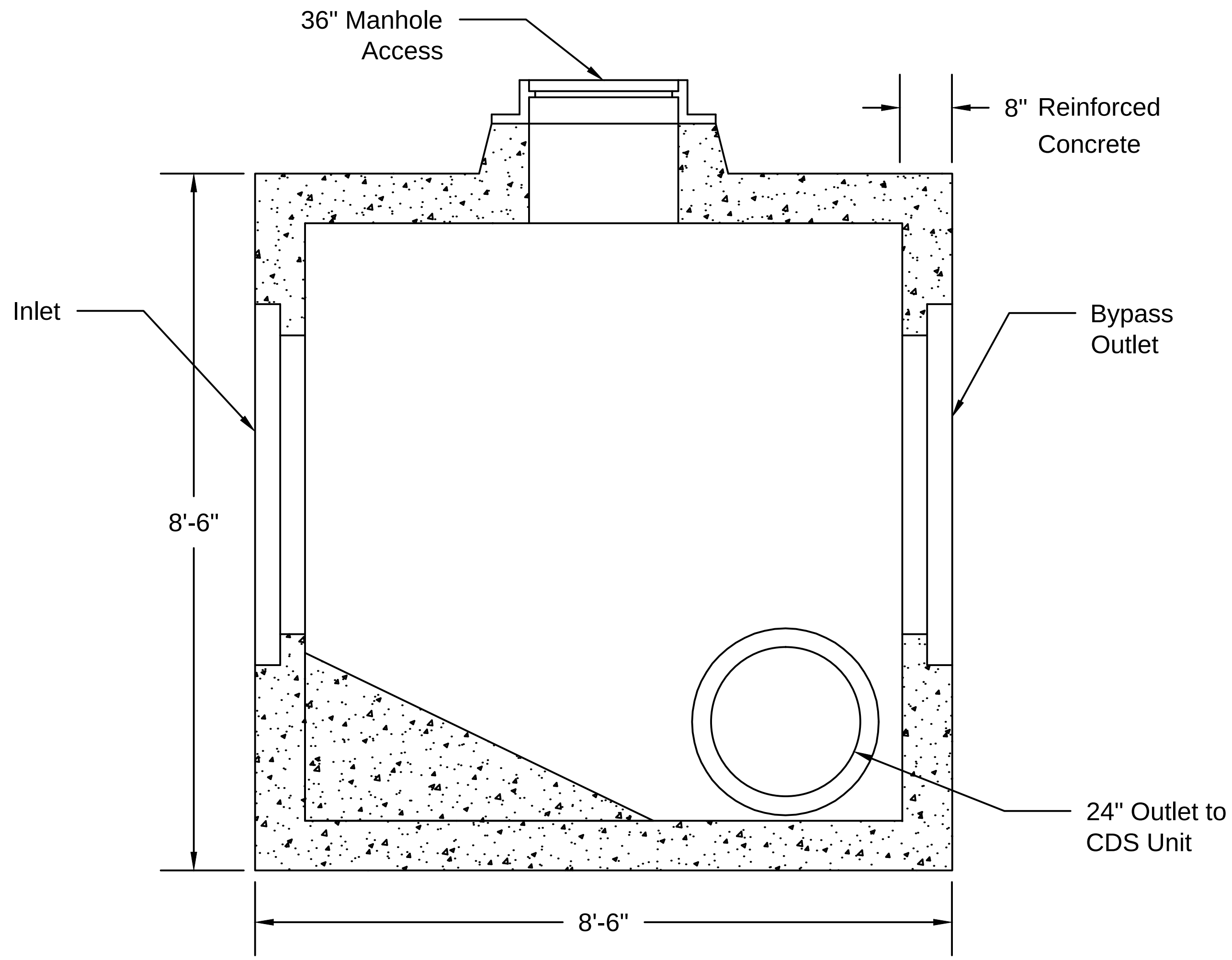
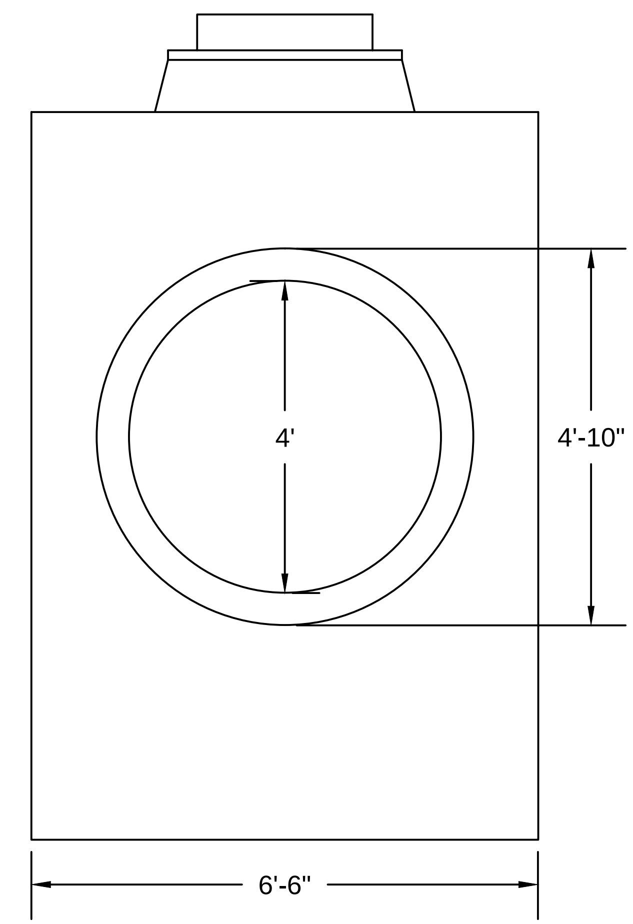
TITLE OF DRAWING:
Plan View

PLAN NO.
1

SHEET NO.
3.3.1





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2

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Upstream Diverter

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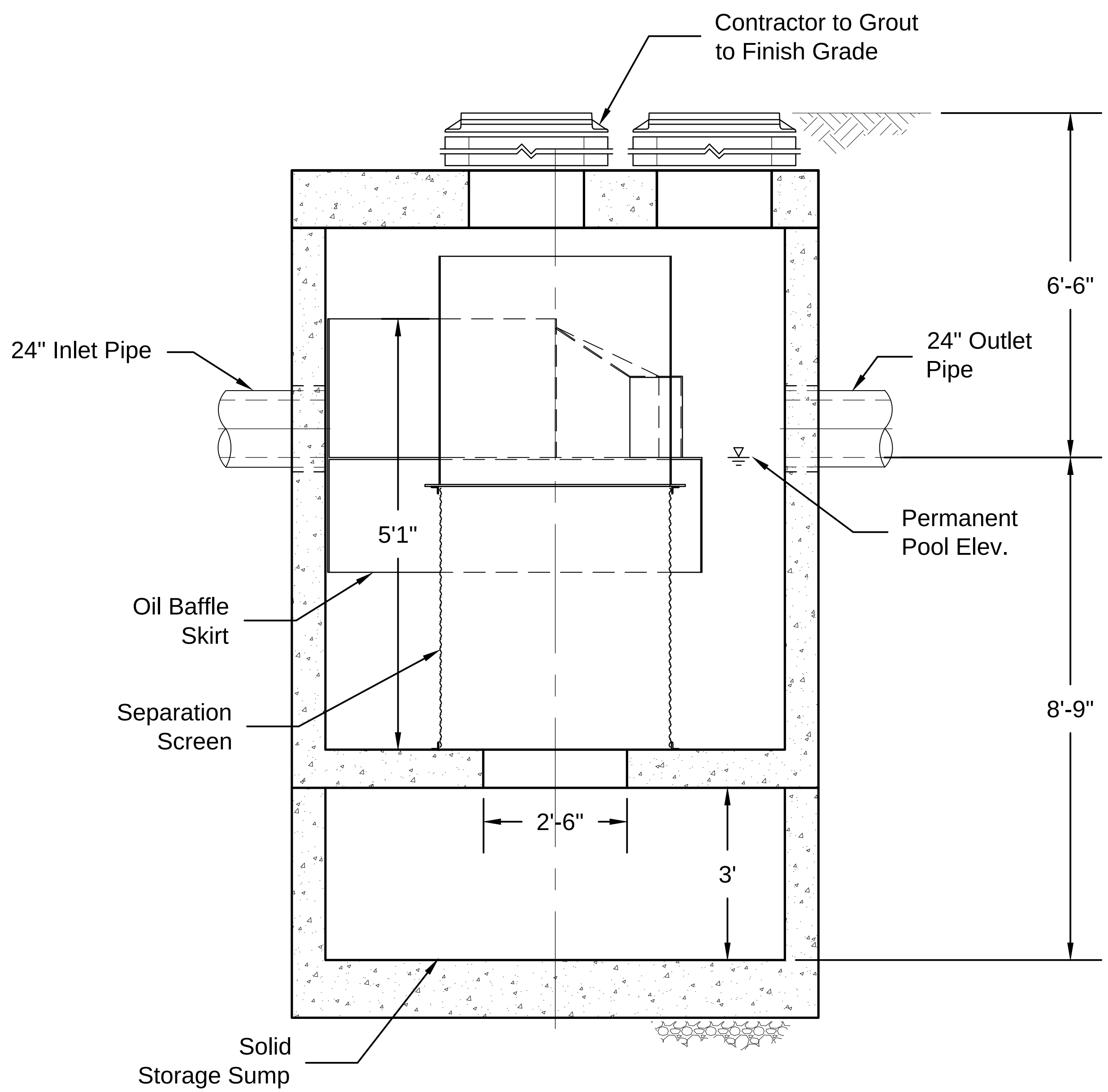
3.3.2



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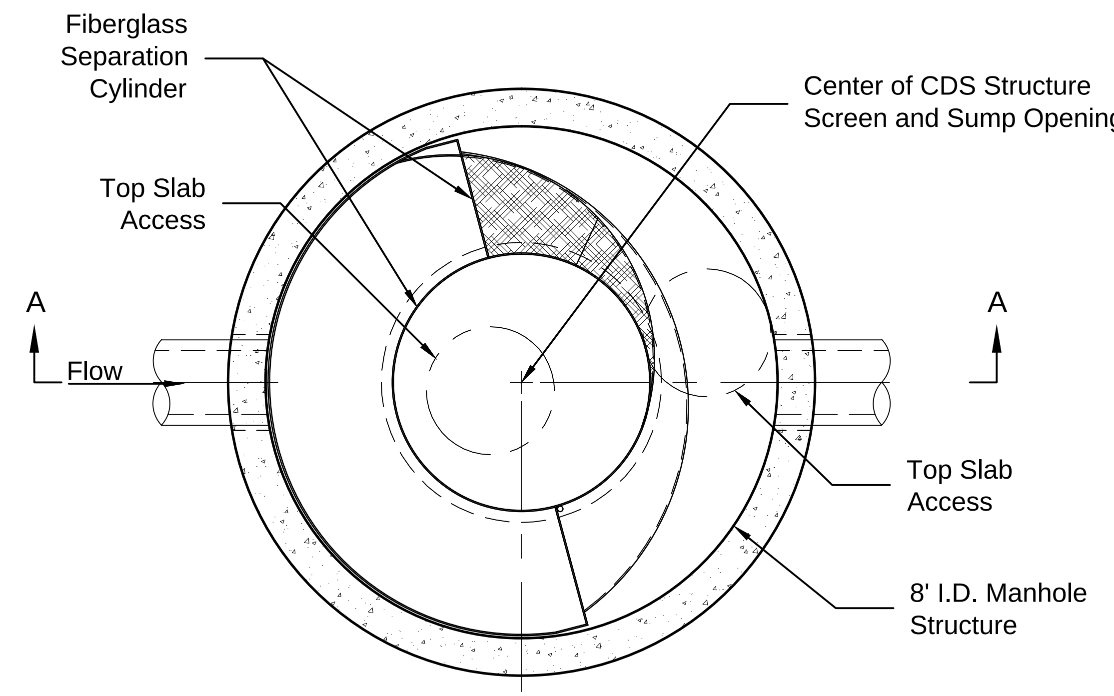

Plan View
CDS 4045

THIS PRODUCT MAY BE PROTECTED BY ONE OR MORE OF THE FOLLOWING U.S. PATENTS: 5,788,848; 6,641,720; 6,511,595; 6,581,783; RELATED FOREIGN PATENTS, OR OTHER PATENTS PENDING.

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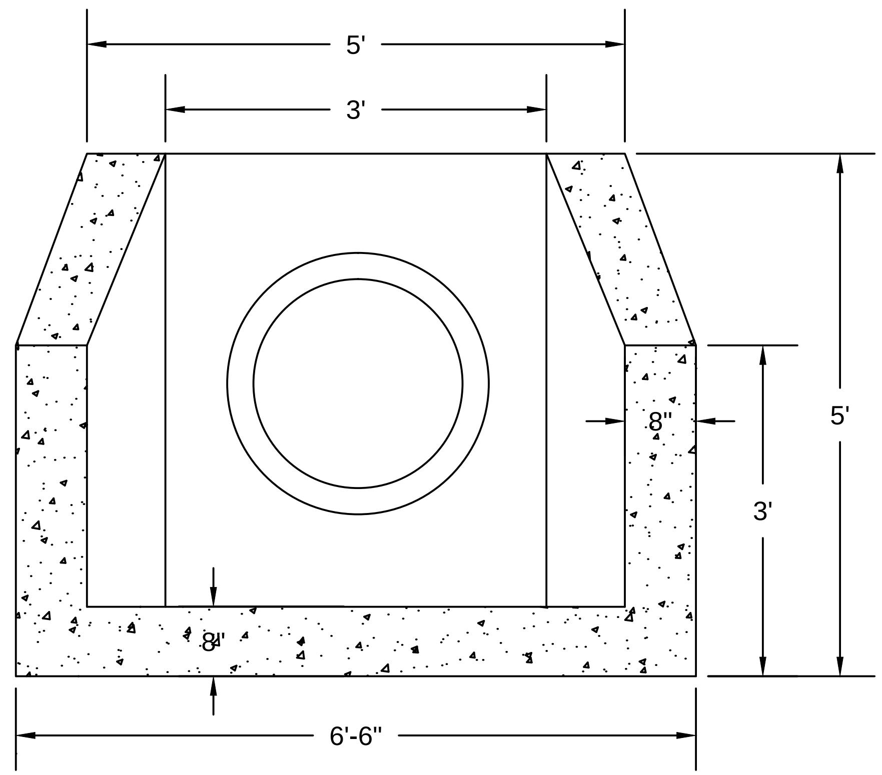
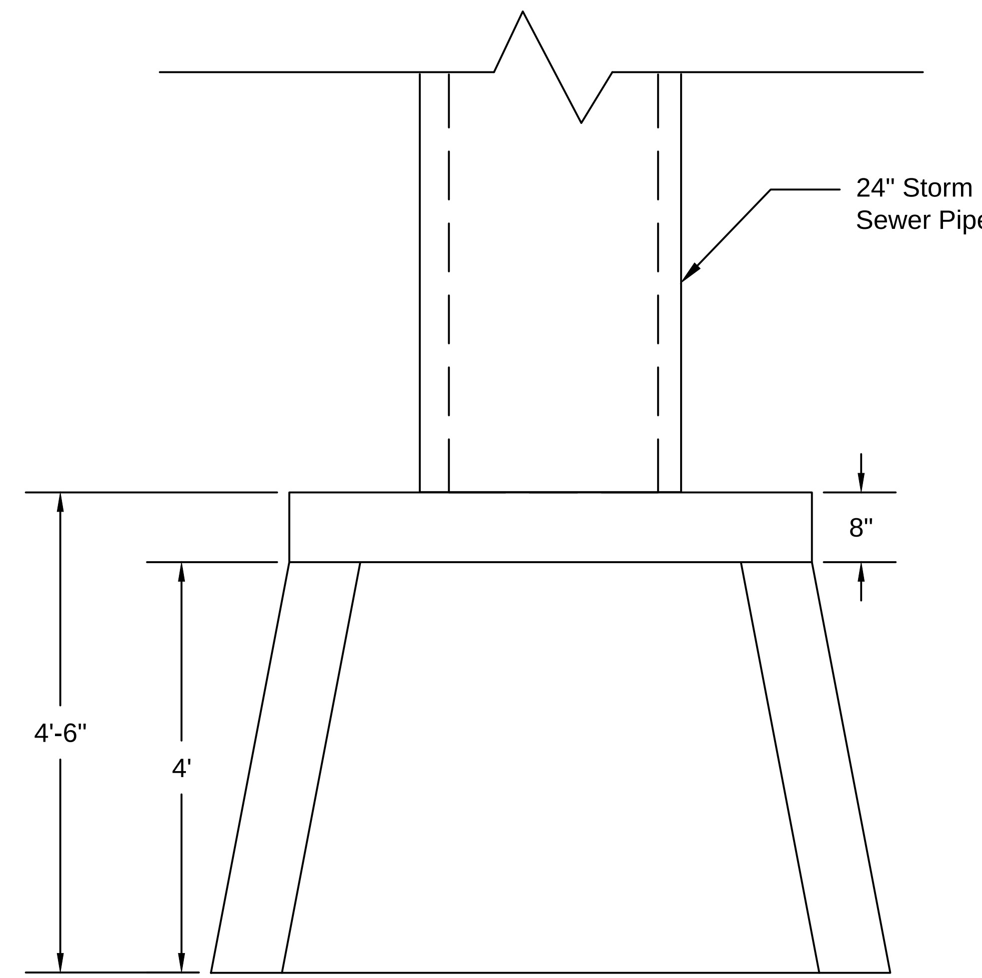
DESISGN NAME: Developed Example
PLAN NO.: 3
TITLE OF DRAWING: CDS Treatment System
SHEET NO.: 3.3.3



Front View



Plan View

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PLAN NO. 4

TITLE OF DRAWING: Outfall Structure

SHEET NO. 3.3.4

3.3.6 Works Cited

[1] Contech, "CDS Guide to Operation, Design, Performance and Maintenance," 2008. [Online]. Available: http://www.contechstormwater.com. [Accessed 12 October 2012].

[2] WEF/ASCE, Design & Construction of Urban Stormwater Magament Systems, ASCE, 1992.

[3] J. Ratner, Building Estimator's Refference Handbook, Frank R. Walker Company, 2002.

3.4 Stream Restoration Example

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3.4.1Overview

There are many tributary streams that enter into the Jordan River. Restoration projects along the banks of the Jordan River will not improve water quality if tributary streams are ignored. Most stormwater runoff, considered a tributary stream, enters into the Jordan River through piping systems. By daylighting these pipes, SCMs can be implemented to improve stormwater quality. This example can be applied to any site where a tributary stream is being restored along the Jordan River. The design will consist of the SCM that will be implemented into the tributary stream restoration.

3.4.1.1 Description of Solution

The purpose of this example design is to show how a detention basin can be implemented into a tributary stream restoration project. At the 900 South Oxbow location, SEA has interacted with the stakeholders for the project to better understand the vision for the site. Dr. Ty Harrison, ecological restoration consultant, heads the group with Ray Wheeler, Dan Potts, Leslie Chan and Emy Maloutas participating as well. Their vision is to create a constructed wetland at the site to improve habitat and water quality. As a part of their vision, they want the stormwater to be as clean as possible when it enters into the wetland area.

Using the EPC selection guidance matrix, it was decided that the best alternative was the implementation of a detention basin alongside the tributary stormwater stream. Further analysis was needed to decide whether the detention basin should be constructed off-line or in-line with the tributary stormwater stream. Using a decision matrix, the in-line detention basin proved best for the site. A cage will be placed at the outlet pipe in order to prevent the first flush from entering into the wetland. The detention basin will reduce the flow and allow the pollutants to settle and allow clean water to enter the wetland. In order to follow the vision of the stakeholders, vegetation will be used to improve aesthetic appeal as well as habitat quality.

The detention basin will treat contaminants in stormwater discharge that affect the dissolved oxygen content of the Jordan River. The major contaminants include nutrients, organic matter, and trash which all affect the wildlife and quality of the water flowing through the Jordan River. Water will be treated throughout the detention basin before it discharges into the Jordan River, thereby increasing the dissolved oxygen content and improving the water quality.

3.4.1.2 Site Location

The location that will be used to implement this example is the 900 South Oxbow located at approximately 900 South and 1100 West in Salt Lake City, UT. The 900 South Oxbow Restoration and Enhancement Project (900 South Oxbow) received $382,322 through Chevron's funding, to the Utah Department of Environmental Quality, after the 2010 Red Butte Creek oil release [1]. The goal of the project is to improve the water quality and habitat and enhance access to the 9 Line trail. The 9 Line trail is a paved trail that runs from Redwood road to 700 West along the old 900 South railroad corridor. The 900 South Oxbow's *Red Butte Creek Project Proposal*, submitted to the Utah Division of Water Quality, stated that the project site is located along the Jordan River Parkway in Salt Lake City between 800 South and 1000 South at the intersection of the 9 Line trail and will enhance 5.48 acres along 4,640 linear feet of the steam bank.

The 900 South Oxbow site location was selected based on the design criteria. The site selection criteria specified that the design site must be a tributary to the Jordan River that can be restored to provide stormwater treatment and improve habitat functions. The 900 South Oxbow project is large but contains a smaller component that is meant to improve stormwater quality.

The proposed diverted stormwater flow path will have an inflow at the stormwater outlet drain, which is indicated in Figure 3.4.1. If stormwater is not treated, the polluted water will enter the Jordan River and will introduce more pollutants to the river. The stormwater must be treated prior to being introduced into Jordan River in order to maximize the benefits and the overall goals of the 900 South Oxbow restoration project. Stream restoration projects must contain integrated stormwater management practices to improve the overall quality of the stream.

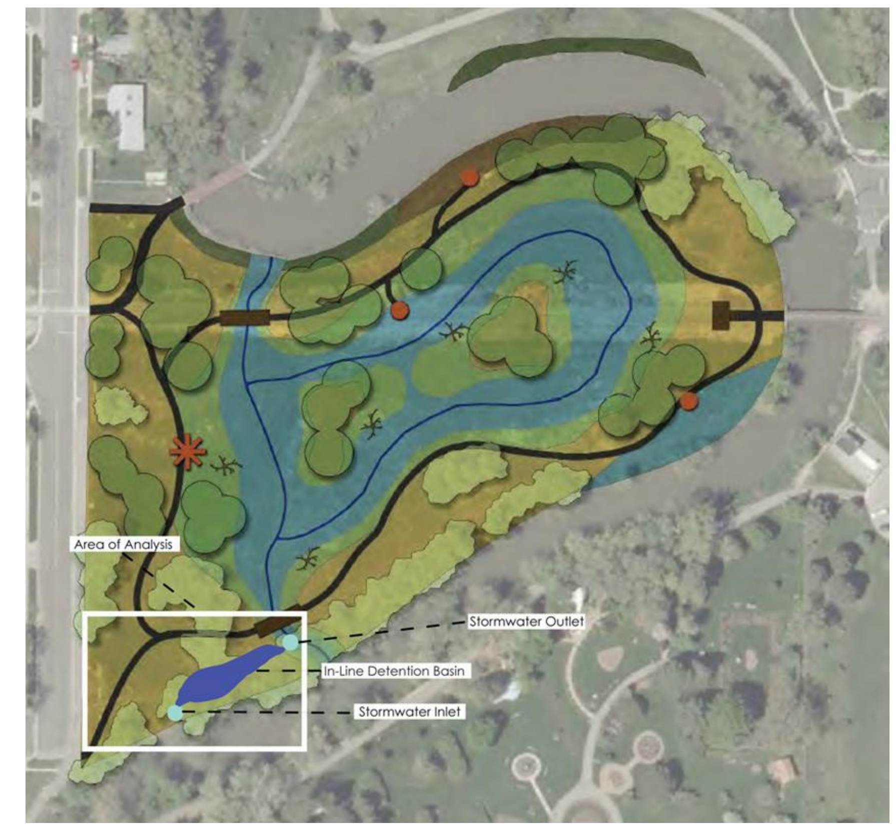


Figure 3.4.1: 900 South Oxbow Site. [1]

The existing conditions allow stormwater to flow directly from the neighborhood to the east into the Jordan River. A stream will be created to divert the water northward, through a detention basin where stormwater management practices will be implemented to improve stormwater quality before entering into the wetland area.

The EPC Selection Guidance Matrix was used to select the best alternatives for the site. Using the selection guidance matrix, the alternative that would best fit the site is a detention basin. A detention basin will be used, but since it is involved with a stream restoration project it will appear as a small constructed wetland.

3.4.1.3 Benefits

Introducing SCMs before allowing the water to enter into the wetland area has many benefits. Allowing the sediments and pollutants settle in the detention basin will allow the wetland to further filter the stormwater. You can think of it as letting the water pass through two filters instead of one. The detention basin will act as a filter for larger sediments while the wetland area will filter out the smaller, finer sediments and pollutants from the stormwater. This allows for very clean water entering into the Jordan River.

Using the detention basin will decrease the discharge rate entering into the wetland area. As the area became developed, the flow rate entering into the Jordan River has increased. As the stormwater fills the detention basin, it discharges into the wetland at pre-development flow rates. This gives the wetland more time to filter out the stormwater.

Since the detention basin will include vegetation, it becomes a natural way to improve filtration and discharge rates of incoming stormwater. This brings an aesthetic appeal for those who frequent the area as well as improve habitat for native species.

3.4.2 Basis of Design

For this example project, the stormwater needs to be filtered before entering into the wetland area. A cage at the stormdrain outfall will be installed to catch the first flush and a detention basin will be used to allow pollutants to settle. The detention basin will be sized to hold a 10 year, 24 hour storm. Any storm of larger magnitude will overflow the basin directly into the wetland area. The detention basin outlet structure will be designed to reduce the flow to 7.7cfs. This allows the wetland to take on pre-development flow characteristics. The basis of design for the detention basin is summarized in Table 3.4.1.

Table 3.4.1: 900 South Oxbow Design Criteria

Basis of Design	
Surface Description	Dry Grass
Drainage Area	18.68 acres
10-yr. 24-hr storm precipitation	1.88 in
Pre-Development Maximum Discharge	7.7 cfs
Post-Development Maximum Discharge	15.8 cfs

3.4.3 Calculations

Using the characteristics of the watershed, the maximum flow rates of both the pre and post-development watersheds can be calculated using HEC-HMS. HEC-HMS is a computer program created by the Army Corps of Engineers. Once the flow rates are found, a detention basin and outlet structure can be designed within the program. Once inputted into the program, trial and error is used to determine the size of both the detention basin as well as the outlet structure. Design is complete when the outflow discharging from the detention basin equals the pre-development discharge rate. Full details pertaining to the following calculations are found in Appendix F.

3.4.3.1 Watershed Analysis

The area of analysis was approximated to be 18.68 acres. In order to find this number, the area was observed using Google Earth. Estimation for the streets that feed into the outlet pipe is shown in Figure 3.4.2. The area was found using a Google Maps online area calculator.

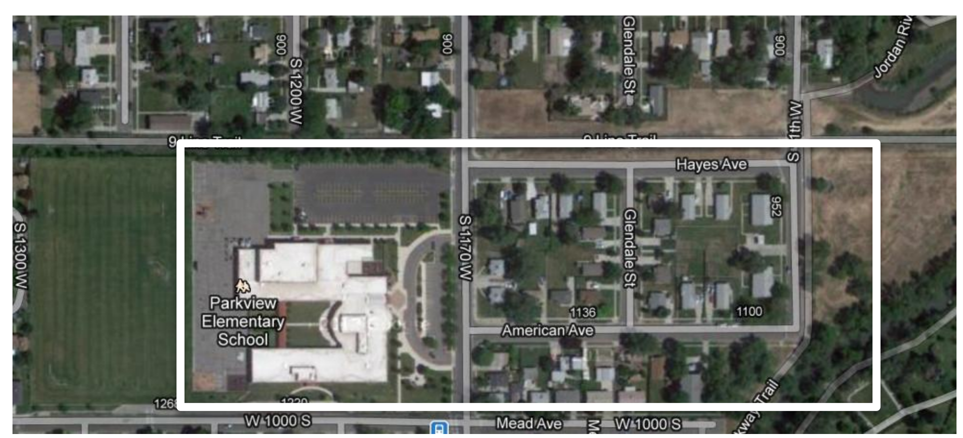


Figure 3.4.2: Watershed Area (Source: www.earth.google.com and SEA)

Since space is limited, a goal was made to use approximately 0.5 acres of space on the 900 S Oxbow site. Also, since the water will flow into the wetland for further treatment, the design storm for the detention basin will be the 10 year, 24 hour storm.

PDS-based precipitation frequency es

Duration	Ave			
	1	2	5	10
5-min	0.122 (0.107-0.141)	0.154 (0.135-0.178)	0.211 (0.184-0.245)	0.263 (0.226-0.307)
10-min	0.185 (0.163-0.214)	0.234 (0.206-0.271)	0.320 (0.280-0.373)	0.400 (0.343-0.467)
15-min	0.230 (0.202-0.265)	0.290 (0.255-0.335)	0.397 (0.347-0.462)	0.496 (0.426-0.579)
30-min	0.310 (0.272-0.357)	0.391 (0.343-0.451)	0.535 (0.467-0.623)	0.667 (0.573-0.779)
60-min	0.383 (0.337-0.442)	0.484 (0.425-0.559)	0.662 (0.578-0.770)	0.826 (0.709-0.965)
2-hr	0.499 (0.449-0.565)	0.619 (0.556-0.696)	0.804 (0.715-0.912)	0.978 (0.862-1.11)
3-hr	0.585 (0.533-0.648)	0.718 (0.657-0.798)	0.898 (0.813-0.998)	1.[illegible]6 (0.959-1.19)
6-hr	0.758 (0.704-0.825)	0.926 (0.857-1.01)	1.12 (1.03-1.22)	1.[illegible]0 (1.18-1.41)
12-hr	0.952 (0.879-1.03)	1.16 (1.07-1.26)	1.40 (1.29-1.52)	1.[illegible]0 (1.4[illegible].75)
24-hr	1.14 (1.04-1.24)	1.39 (1.27-1.52)	1.66 (1.51-1.81)	1.88 (1.71-2.05)

Figure 3.4.3: NOAA Atlas 14 Rainfall Data

Using NOAA Atlas 14 [2], shown in Figure 3.4.1, the rainfall intensity over the watershed area during the 10 year, 24 hour storm is 1.88in. Using this value, the total volume of water during the storm can be calculated by multiplying both the watershed area and the detention basin area by the intensity of the storm. This is given in Equation 3.4.1:

$$V_w = (A_w * i + A_{DB} * i)$$

3.4.1

Where:

V_w= Total Volume of Water (ft^3)

i = Intensity of 10-yr, 24-hr storm (in)

A_w = Area of Watershed (ft^2)

A_{DB} = Area of Detention Basin (ft^2)

The total amount of water that will fall over the area is 131,209 ft^3.

3.4.3.2 Flow Analysis

The flow analysis was conducted using the HEC-HMS computer program. In order to compute the maximum flow rate, the curve number must be found. Curve numbers are used as a constant to measure the amount of abstractions as well as infiltration of an area. The ground type is important in order to determine the curve number. The curve numbers for this analysis were found using a document prepared by the United States Department of Agriculture called TR-55 [2]. Curve numbers are used in HEC-HMS to incorporate infiltration. The curve number for the pre-development watershed is 86. Running the analysis, the maximum flow rate flowing from the area is 7.7cfs. The data must be compared to the post-development data. In the post-development watershed, it was approximated that 75% of the area was impervious surfaces. With this information, the curve number for the impervious surfaces is 98 and the residential area is 74. A factored curve number can be calculated using Equation 3.4.2.

$$CN_{new} = \frac{(A_W * 0.75 * CN_P) + (A_W * 0.25 * CN_N)}{A_W}$$

3.4.2

Where:
CN_{new} = Compiled Curve Number
A_W = Area of Watershed (ft^2)
CN_P = Curve Number of Pavement
CN_N = Curve Number of Neighborhood

The combined curve number used was 92. Using this curve number, the analysis in HEC-HMS showed maximum flow rate is 15.8cfs. The flow data is summarized in Table 3.4.2. The analysis necessary to compute these values is explained in section 3.4.3.4

Table 3.4.2: Maximum Discharge Values

Maximum Discharge (cfs)	
Pre-Development Watershed	7.7
Post-Development Watershed	15.8

3.4.3.3 Detention Basin Design

The purpose of the detention basin is to reduce the post-development discharge rates. For the 900 S. Oxbow site, these flows will be reduced so the water discharging into the wetland area will enter at pre-development discharge rates. This will be done by allowing water to fill in the detention basin while slowly releasing into the wetland through an outlet structure.

3.4.3.3.1 Detention Basin Shape

The detention basin will be shaped according to Figure 3.4.4.

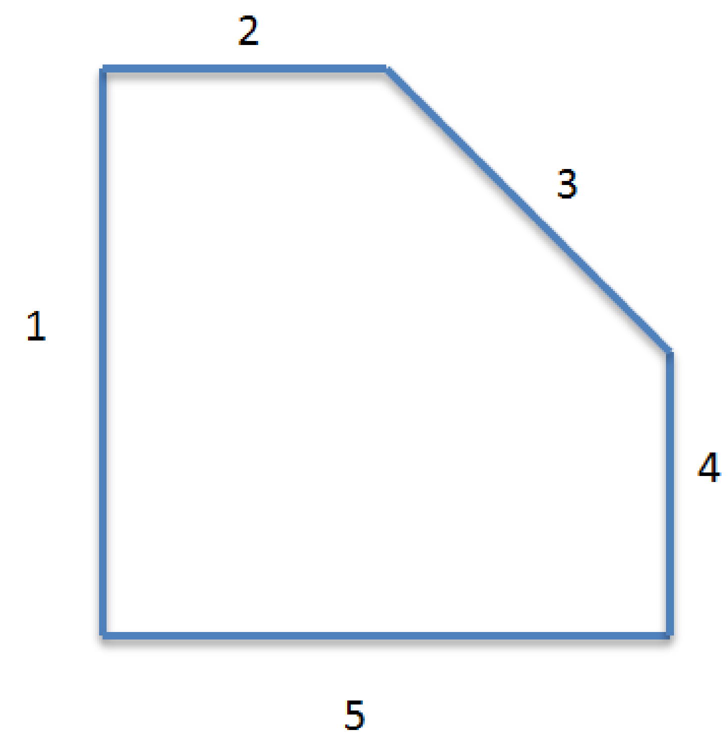


Figure 3.4.4: Detention Basin Shape and Side Numbers

The numbering on the diagram will help for calculating the area of the basin. Sides 1 and 5 will be the same length as well as 2 and 4. This will ease design calculations of the basin. The length of side 3 is calculated using the Pythagorean Theorem shown in Equation 3.4.3. The value of side 3 is expressed in Table 3.4.3.

$$Side\ 3 = \sqrt{(5-2)^2 + (1-4)^2}$$

3.4.3

Where:
Side 3 = Diagonal Side (ft)
1,2,4,5 = Side Number (ft)

The area of the basin is calculated using Equation 3.4.4. It is important to minimize the size of the detention basin area to allow more space for the proposed wetland area.

$$A = (2 * 1) + [4 * (5-2)] + \left[\frac{(1-4)(5-2)}{2}\right]$$

3.4.4

Where:
A = Area (ft^2)
1,2,4,5 = Side Number (ft)

In order to minimize the amount of land used for the detention basin, a goal was made to keep the detention basin within 0.500 acres. With the dimensions given in Table 3.4.3, the area of the detention basin is 0.200 acres.

Table 3.4.3: Side Lengths

Side	Length (ft)
1	100
2	50
3	70.83
4	50
5	100

To ensure safety and aesthetic appeal, the slope of the basin will be 4:1. This is for every 1ft elevation drop, there will be 4ft of horizontal distance change. In order to calculate the depth of the basin that will accommodate the total amount of water, small increments of volume will be calculated to approximate the total volume. For this calculation, an increment of 0.1 feet will be used. This means that for every 0.1 feet of drop, 0.4 feet of horizontal distance will be used. For sides 1 and 5, 0.8 will be subtracted and for sides 2 and 4, 0.4 will be subtracted.

3.4.3.3.2 Detention Basin Volume

The volume will be calculated using Microsoft Excel. will be used to compute the areas at each sub section. Since the step is small, the volume between steps is calculated by averaging the areas of the two steps and multiplying by the step shown in Equation 3.4.5. The volumes are added up to calculate the total volume.

$$V_{sub-section} = 0.1 * \left(\frac{A_1 + A_2}{2}\right)$$

3.4.5

Where:
$V_{sub-section}$ = Volume of the Sub-section (ft^3)
A_1= Area of Area 1 (ft^2)
A_2= Area of Area 2 (ft^2)

Using this method, the total volume after 6.5 feet is 32,427ft^3. HEC-HMS will be used to verify if the detention basin size is sufficient for the 10 year, 24 hour storm event. See. The detention basin sizing and volume is summarized in Table 3.4.4.

Table 3.4.4: Detention Basin Summarization

Detention Basin	
Height (ft)	6.50
Volume (ft^3)	32426.94
Top Area (ft^2)	8750.00
Bottom Area (ft^2)	2016.00
m (ft/ft)	4.00
Step Size (ft)	0.10
Outfall Q (cfs)	7.50

3.4.3.4 HEC-HMS Analysis

The system can be set up in HEC-HMS to verify if the detention basin will reduce the maximum post-development discharge to the pre-development discharge. In order to accomplish this, an outlet structure needs to be designed to slowly release water from the detention basin into the wetland. By running an analysis in HEC-HMS, the position of the outlets can be adjusted until the desired discharge is met. Through trial and error, the desired outlet structure will have an outlet at the base of the structure and a spillway at the top. The design values are shown in Table 3.4.5. The outlet at the base will allow smaller storm flows to flow like a river into the wetland. As the storm size increases, the detention basin will fill up until it begins to flow over the spillway into the wetland.

Table 3.4.5: Outlet Structure Design Summarization

Outlet Structure	
Spillway	
Location (ft)	5.00
Height (ft)	1.50
Width (ft)	1.00
Area (ft^2)	1.50
Outlet	
Location (ft)	0.25
Area (ft^2)	0.30
Width (in)	7.20
Height (in)	6.00

With the outlet structure in place, the detention basin will be able to accommodate a 10 year, 24 hour storm event. The results of the HEC-HMS analysis are shown in Figure 3.4.5 and the hydrograph is shown in Figure 3.4.6.

Project: Post-Development
Simulation Run: Post Development 10-yr 24-hr Reservoir: Detention Basin

Start of Run:	23Nov2010, 13:00	Basin Model:	Basin 1
End of Run:	25Nov2010, 00:00	Meteorologic Model:	10-yr
Compute Time:	15Nov2012, 20:52:18	Control Specifications:	10-yr_24-hr

Volume Units: (●) IN ( ) AC-FT

Computed Results

Peak Inflow :	15.8 (CFS)	Date/Time of Peak Inflow :	24Nov2010, 01:15
Peak Outflow :	7.5 (CFS)	Date/Time of Peak Outflow :	24Nov2010, 02:00
Total Inflow :	1.69 (IN)	Peak Storage :	0.6 (AC-FT)
Total Outflow :	1.69 (IN)	Peak Elevation :	6.0 (FT)

Figure 3.4.5: HEC-HMS Analysis Results

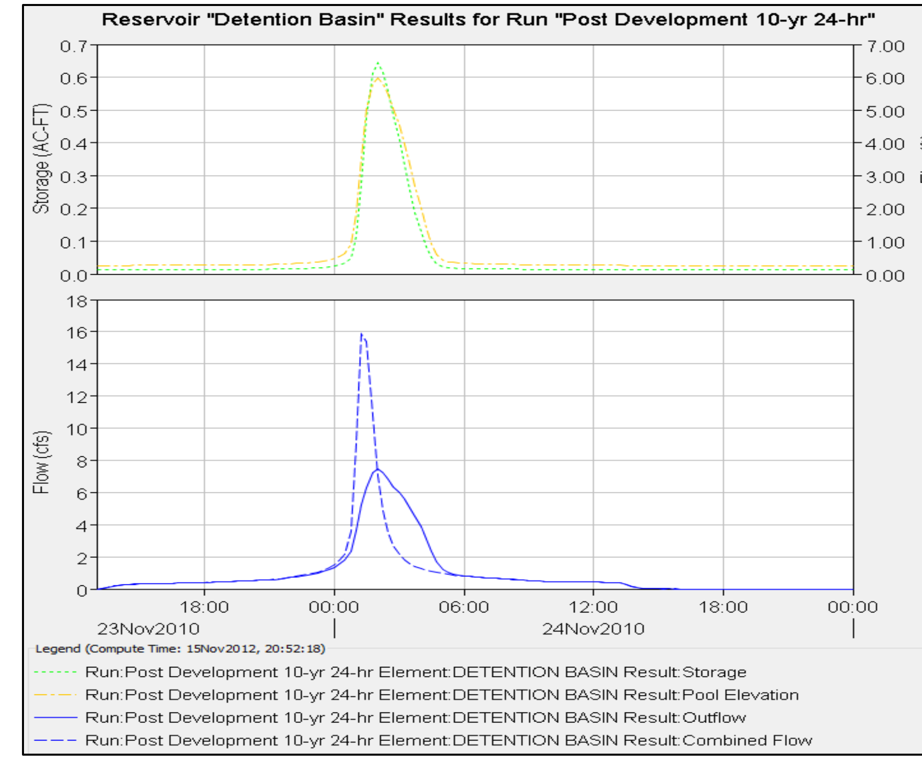


Figure 3.4.6: Post-Development Hydrograph with Outlet Structure

The hydrograph shown in the lower region Figure 3.4.6 shows the amount of runoff flowing through the outlet structure. The dashed line shows the flows as if the outlet structure is no there and the water is able to flow freely into the wetland area. The upper half of Figure 3.4.6 shows the elevation of water inside the detention basin. During the peak hours of the 10 year, 24 hour storm event, the detention basin will approximately fill to capacity. The detention basin is designed for this storm event and larger storm events will overflow the detention basin and will lead directly into the wetland area.

3.4.4 Cost Analysis

The cost estimation was based on prices found from local concrete companies, local nurseries, estimation handbooks, and other online resources. The cost estimation is meant to serve as a guide to show how typical costs are calculated. However, values are susceptible to change based on pricing and design. See for an example cost estimate. The vegetation that is planted should be of native Utah species as this will help restore the looks of the area to their natural state. Concrete will be used only for control structures or weirs that affect the flow throughout the basin. Costs not associated with this estimate are any signage or other educational props that visitors can read. A summary of the example cost estimate is shown in Figure 3.4.7.

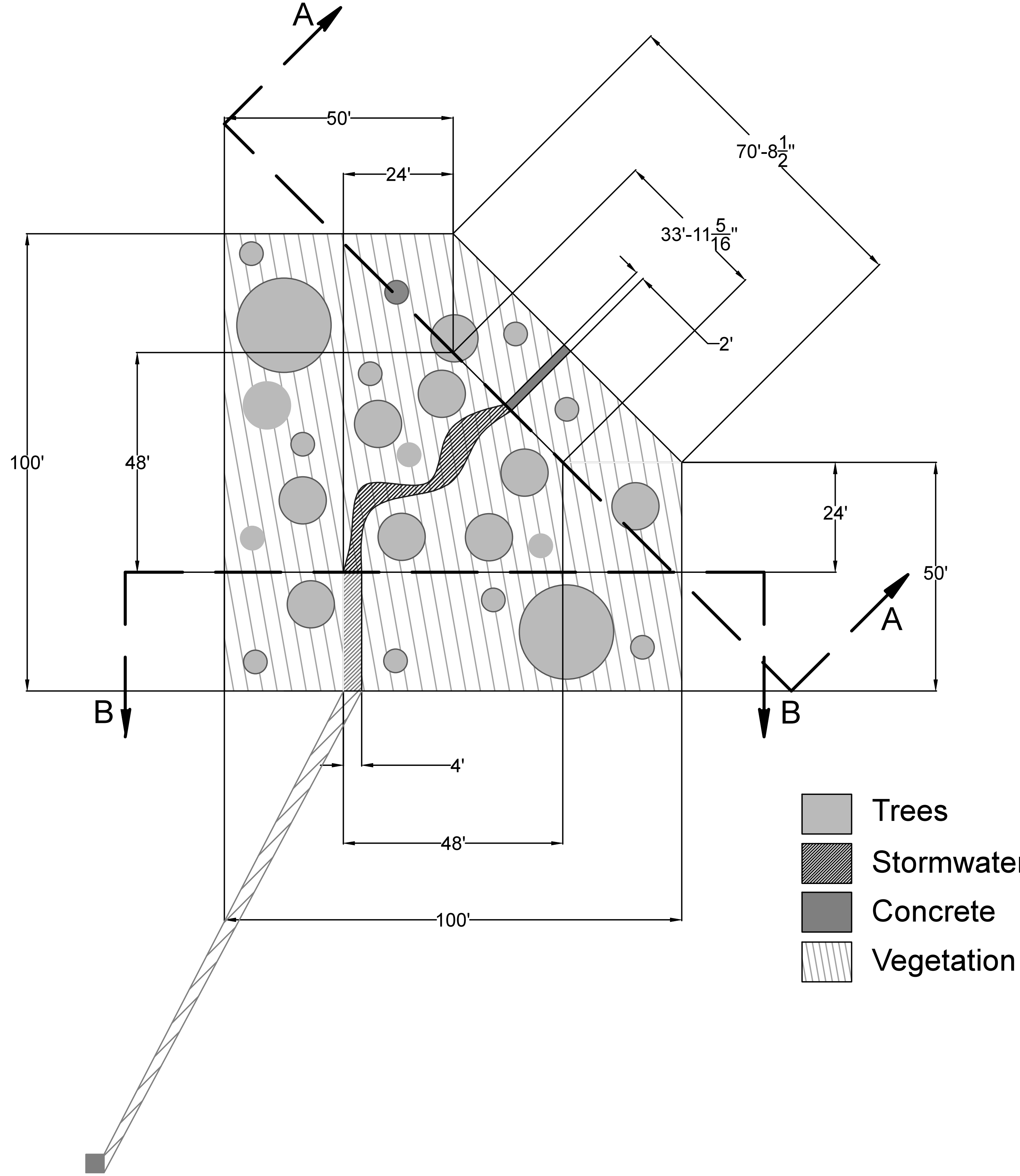
Figure 3.4.7: Detention Basin Cost Estimate

Installation		Unit	Quantity	Unit cost	Cost	Source	Notes
Soil						Wholesale Landscape Supply, UT	Typical local cost
	Soil Fill	CY	2,643	$16.00	$42,282.24		
	Vegetation Soil	CY	27	$18.00	$490.63		
	Ground Cover (wood chips, bark	CY	27	$14.00	$381.60		
	Compaction Soil						
Vegetation						J & L Garden Center, UT	
	Tree	Tree	12	$75.00	$900.00		
	Shrub	Shrub	13	$35.00	$455.00		
	Wetland Grass	SF	23,784	$0.34	$8,086.48		
Concrete		CY	3	$85.00	$245.56	Altaview Concrete	Typical local cost
1-1/4 CY Backhoe		CY	1,350	$5.96	$8,046.00	Walker's Building Reference Estimators Book	Includes foreman, operator, and equipment
20 CY Rear Dump Trucks		CY	68	$91.79	$6,195.83	Walker's Building Reference Estimators Book	Includes truck rental, gas, and disposal
Site Maintenance		**Unit**	**Quantity**	**Unit cost**	**Cost**	**Source**	**Notes**
Labor		HR				Utah Workforce Services	On average companies charge $40.00/hr/worker. This is a yearly estimate.
	Landscaping Maintenance		40	$40.00	$1,600		
	Vactruck		40	$40.00	$1,600		
	Janitorial (if needed)		10	$40.00	$400		
	Irrigation Cost (if applicable)	10,000 SY	2	$12.00	$2,968.21		This is a yearly estimate
Recurring Maintenance		**Unit**	**Quantity**	**Unit cost**	**Cost**	**Source**	**Notes**

Total Installation Cost	$67,083.32
Total Site Maintenance Cost	$6,568.21
Total Recurring Maintenance Cost	$0.00
Total Cost	$73,651.53

3.4.5 Construction Drawings

List of Drawings:



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DESIGN NAME
Detention Basin
PLAN NO.
1
TITLE OF DRAWING:
Plan View
SHEET NO.
3.4.1

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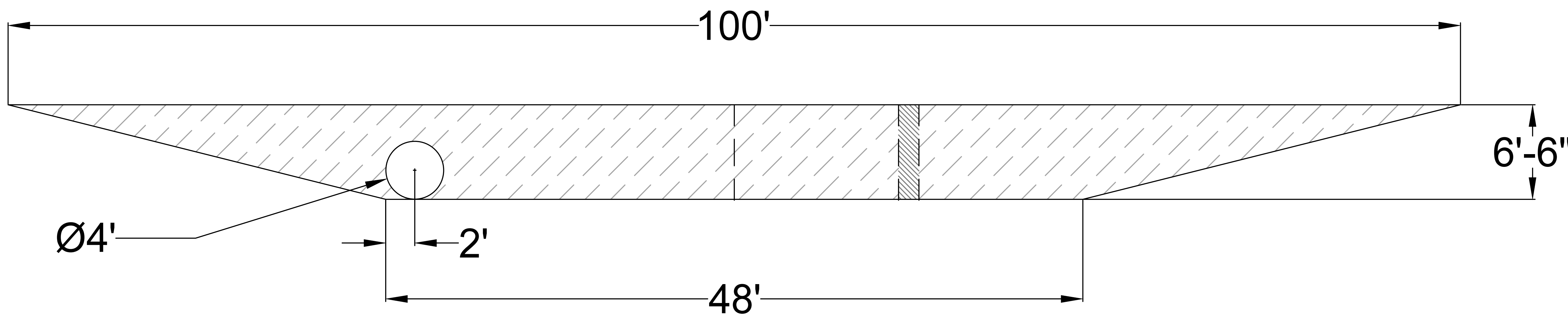

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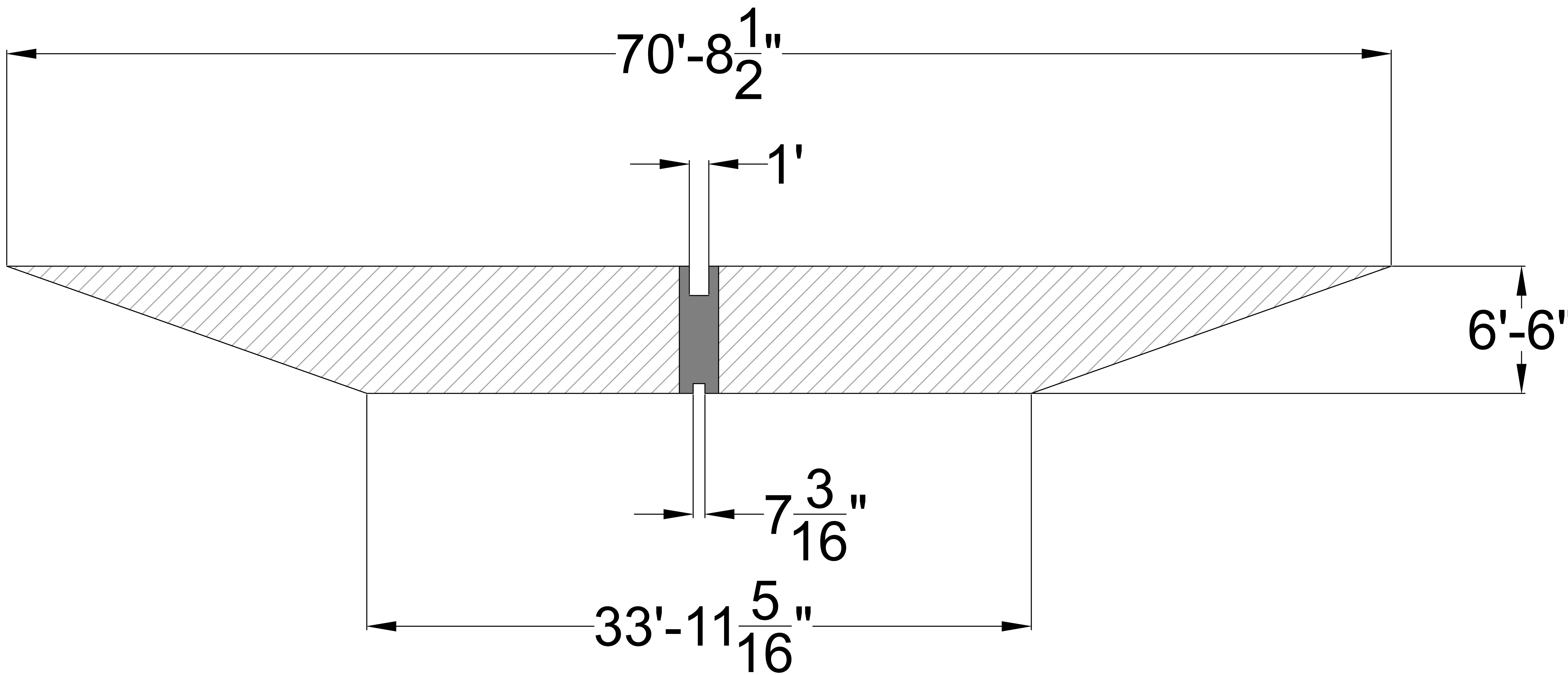
DESIGN NAME
Detention Basin
PLAN NO.
2
TITLE OF DRAWING:
Elevation View
SHEET NO.
3.4.2



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PLAN NO.
3
TITLE OF DRAWING:
Section A-A
SHEET NO.
3.4.3

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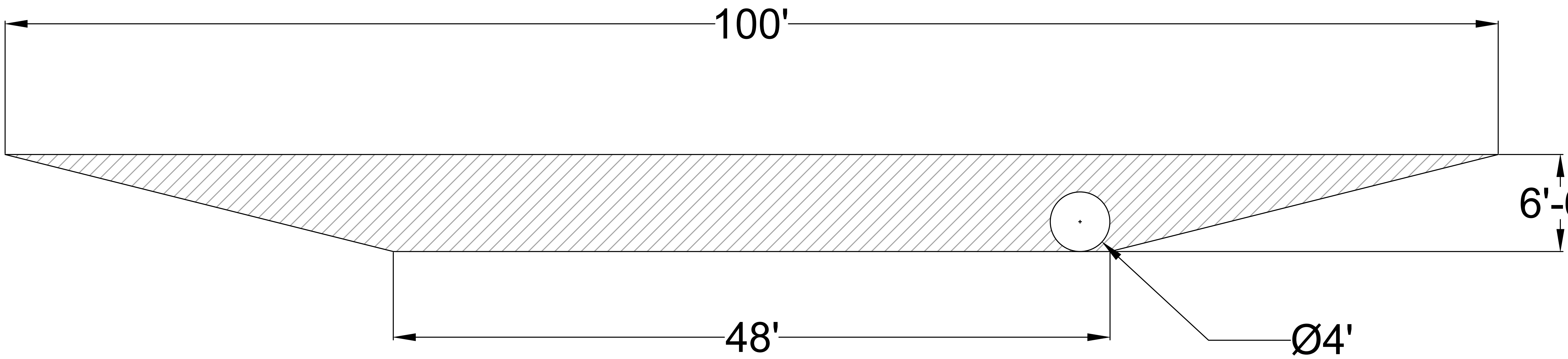

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DESIGN NAME: Detention Basin

PLAN NO: 4

TITLE OF DRAWING: Section B-B

SHEET NO: 3.4.4

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3.4.6 Works Cited

[1] Salt Lake City Parks and Public Lands, "Red Butte Creek Mitigation Fund Restoration Projects," Salt Lake City Parks and Public Lands, 1 August 2010. [Online]. Available: http://www.slcclassic.com/publicservices/parks/RedButteCreekMitigation.htm. [Accessed 25 November 2012].

[2] N. O. a. A. Administration. [Online]. Available: http://www.noaa.gov. [Accessed 24 11 2012].

[3] United States Department of Agriculture, "Urban Hydrology for Small Watersheds, TR-55," Conservation Engineering Division, Washington D.C, 1986.

3.5 Undeveloped Outfall Example

3.5.1Overview

The selected area is 20 acres and is privately owned. The owner has agreed to sell the land to the city to create a wetland environment and preserve its natural looks. Approximately three quarters of the land is home to native grasses and trees, with some invasive species. The remaining land closer to the river is abnormally dry and lacks vegetation. The area is currently overrun with invasive species; most notably phragmites, tamarisk, and Russian olive. It also has a small spring and irrigation feed, which provides a small wetland type environment. This has brought hope that the area could be expanded to receive stormwater from 14600 south and adjacent Bluffdale roadways before discharging into the Jordan River. However, the residents want to maintain and enhance the natural feel with the meandering stream and native species.

3.5.1.1Site Location

The proposed constructed wetland is located in Bluffdale, just north of 14600 South and east of Redwood Road, as seen in Figure 3.5.1. The area is undeveloped except for a barn and a few small farm buildings.

Figure 3.5.1: Bluffdale Wetland Location Highlighted in Red.

3.5.1.2 Benefits

Constructed wetlands serve many purposes. They are designed to simulate a natural wetland, and are used to treat stormwater and catch floodwater. They help remove nutrients and pollutants that would otherwise be harmful to animal and plant life within the water's ecosystem. Constructed wetlands also offer educational value. They are often easily accessible and can be useful in demonstrating how the nutrient removal process works. In designing a constructed wetland, there is also an opportunity to make the system aesthetically pleasing. This will help the JRC meet their goal of providing a more socially welcoming environment.

3.5.2 Description of Solution

The chosen design to be implemented is an extended wetland. This particular wetland is home to high and low marshy regions. There are deep pools amongst the shallower wetlands. The high and low marshes are designed to aid in contaminant removal but also allow different plant and wildlife to thrive in one common area. This wetland is designed to withstand any rapid inflows and then equalize and go back to normal levels within twenty-four hours of the storm [1]. This would be beneficial for an area such as the Jordan River corridor, where it is common to receive large amounts of snowfall and warm temperatures. The stormwater is pooled in the detention zone for as long as 24 hours before moving on [1].

Extended Stormwater Wetlands typically have a Pollutant removal as follows: 65-90 percent for suspended solids, 15-75 percent for phosphorous, and up to 55 percent for nitrogen [1]. Metals such as copper and zinc have removal efficiencies between 20-65 percent [1]. This design will be friendly as a waypoint for migrating birds along the Jordan River. The deeper pooled sections and outer edges of the wetland create an environment that will keep certain invasive species such as phragmites away. Detailed schematics of this design can be seen in Figure 3.5.2.

Some important things to note in designing and implementing an extended stormwater is that if the in-flow decreases to where the pond could freeze, the stormwater could pass over the ice, rendering the filtering system ineffective. This is where the constant spring and irrigation feed would be vital to this design. There will be maintenance costs as low as $750 and as much as $1500 per year associated with the extended stormwater wetland. Table 3.5.1 has a detailed cost analysis. This includes debris removal, dredging the pretreatment areas, invasive species monitoring, and vegetation rehabilitation [1].

3.5.3 Basis of Design

Flow Rate

The Bluffdale wetland was designed to accommodate the 100 year, 24 hour storm event and extended periods of dryness. The wetland was sized to accommodate extreme rainfall, while the natural spring will provide a base flow during extended dry periods. This was done based on parameters specific to the Bluffdale site. The size accommodation was based on the amount of land available. A smaller plot of land would likely be designed to accommodate a 2 year, 24 hour storm [2].

Peak Flow

The wetland was designed to provide adequate stormwater detention for nutrient removal while reducing the peak flow. The weir design was created such that the peak outflow will not exceed 6.45 cubic feet per second.

Influent Water Quality Improvement

The wetland will be designed with adequate vegetation and stormwater detention time to maximize nutrient removal and solids filtration.

Future Community Education Opportunities

A stormwater wetland provides an excellent educational opportunity to teach the benefits of stormwater treatment. The wetland thus was designed to be accessible and could be visited by school children and nature enthusiasts.

Aesthetically Appealing

The wetland was designed with native plant species. This will encourage the presence of native wildlife. This is significantly more appealing than dense Phragmites and Russian Olive infestations.

Construction Cost

The wetland was designed to minimize construction costs. This was done by selecting the wetland area with the least amount of excavation, imported materials, and maintenance required. The natural topography of the selected region helped achieve this.

3.5.4Cost Estimation

The largest costs are accrued in the landscaping and concrete reinforcement for the outfall and weir. The landscaping will require significant attention to construct the wetland to appear as natural as possible. It will also be crucial in properly landscaping to achieve the best nutrient and contaminant removal. The concrete will be laid in such a manner that it will be as discrete as possible. Finally, purchasing and maintaining the plants will be the next most expensive undertaking.

The following paragraph details one viable option to help offset the costs of the Bluffdale wetland. The EPA offers a Five Star Restoration Program, which provides grants for community-based wetland restoration projects. Funding levels vary, but range from $5,000 to $20,000, with $10,000 as the average amount awarded per project [3]. To apply for such grants, the EPA requires a detailed project description, a detailed line item budget which identifies matching and requested funds, maps, diagrams, and GPS coordinates for the proposed project, and identification of project partnerships and applicant expertise [3]. The EPA then reviews the proposal, and decides whether or not to fund the project based on its feasibility and impact on the environment.

Table 3.5.1:Ccost Analysis for Extended Stormwater Wetland

Installation	Unit	Quantity	Unit cost	Cost	Source	Notes
Labor	HR	32	$40.00	$1,280	Utah Workforce Services	On average companies charge $40.00/hr/worker
Wetland Excavation	CY	833	$5.00	$4,165	Spring Brook, ND WWTP Project	
Import Fill Placement	CY	208	$5.00	$1,040	Spring Brook, ND WWTP Project	
Landscaping	SY	208	$5.00	$1,040	Spring Brook, ND WWTP Project	
Reinforced Concrete Installation	CY	15	$3,350.00	$50,250	Spring Brook, ND WWTP Project	
Weir Gate Installation	EA	2	$1,000.00	$2,000	Riverton, UT ULDC Pump Station Project	
Import Engineered Fill	CY	208	$5.00	$1,040	Spring Brook, ND WWTP Project	
Weir Gate		2	$1,000.00	$2,000	Riverton, UT ULDC Pump Station Project	
Reinforced Concrete	CY	15	$350.00	$5,250	Spring Brook, ND WWTP Project	
Vegetation	SY	208	$5.00	$1,040		
Excavation	CY	833	$5.00	$4,165	Spring Brook, ND WWTP Project	
Dewatering	LS	1	$1,000.00	$1,000	Spring Brook, ND WWTP Project	
Maintenance	**Unit**	**Quantity**	**Unit cost**	**Cost**	**Source**	**Notes**
Labor	HR	32	$40.00	$1,280	Utah Workforce Services	On average companies charge $40.00/hr/worker
Debris Removal	Yearly	1	$250.00	$40	Spring Brook, ND WWTP Project	As needed
Pretreatment Area Dredging	Yearly	1	$250.00	$40	Spring Brook, ND WWTP Project	
Invasive Species Removal	Yearly	1	$500.00	$40		As needed
Seasonal Planting	Yearly	1	$200.00	$40		
Vegetation	Yearly	1	$50.00	$40		
Dredging	Yearly	1	$250.00	$40	Spring Brook, ND WWTP Project	
	Total Installation Cost			$74,270		
	Total Recurring Maintenance Cost			$1,520		
	Total Cost			$75,790		

3.5.5 Calculations

The following calculations are based upon HEC-HMS models that were developed. This is a sample of how a wetland could be constructed for the Bluffdale or similar undeveloped location. The main requirements were the wetland and weir sizing.

3.5.6.1 Wetland Sizing

A hydraulic model was constructed using the following parameters:

Project: Wetland   Simulation Run: Run 1

Start of Run: 17Nov2012, 13:00   Basin Model: Basin 1
End of Run: 18Nov2012, 14:00   Meteorologic Model: 100 yr storm
Compute Time: 18Nov2012, 15:15:08   Control Specifications: Control 1

Show Elements: All Elements   Volume Units: (•) IN ( ) AC-FT   Sorting: Hydrologi

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
14600 SOUTH	0.03224	26.1866	18Nov2012, 01:15	3.07
Spring	0.00000	0.0223	17Nov2012, 13:00	
JUNCTION	0.03224	26.2089	18Nov2012, 01:15	3.09
WETLAND	0.03224	6.4559	18Nov2012, 02:00	2.96

Figure 3.5.3: Hydraulic Model Parameters

HEC-HMS calculated the following volume of water for the inflow of the wetland:

Project: Wetland
Simulation Run: Run 1   Junction: JUNCTION

Start of Run: 17Nov2012, 13:00   Basin Model: Basin 1
End of Run: 18Nov2012, 14:00   Meteorologic Model: 100 yr storm
Compute Time: 18Nov2012, 15:15:08   Control Specifications: Control 1

Volume Units: ( ) IN (•) AC-FT

Computed Results

Peak Outflow : 26.2089 (CFS)   Date/Time of Peak Outflow : 18Nov2012, 01:15
Total Outflow : 5.3 (AC-FT)

Figure 3.5.4: Peak Inflow of Constructed Wetland for 100 Year Storm Event

This data shows that a total of 5.3 Ac-ft of water will enter the wetland during the 24 hour storm event. The wetland can now be sized with this information.

This particular wetland will be sized to be 3 feet deep. This will provide enough volume for stormwater detention, but also keep construction costs relatively low. The wetland area was calculated as follows (Equation 3.5.1):

$$\frac{V}{D} = \text{A (1.77 Acres or 76,956 ft}^2\text{)}$$

3.5.1

Where:
V=Volume (acre-ft.)
D=Depth (ft.)
A=Area (ft^2)

The wetland shape will be square to minimize the footprint. The length will be the entire length of Mr. Jones' property (1,154 ft) since that land would be required to connect the stormwater to the river anyways. The area was calculated above as 76,956 ft^2. Then, using the equation for the area of a rectangle, the width was calculated as follows:

$$L*W = \text{Area}$$

3.5.2

Where:
L=Length (ft.)
W=Width (ft.)
A=Area (ft^2)

Therefore, the dimensions of the rectangular wetland shall be:
L = 1,154 ft
W = 67 ft

3.5.6.2 Weir Sizing

Basin Name: Basin 1
Element Name: WETLAND
Method: Orifice Outlet
Direction: Main
Number Barrels: 1
*Center Elevation (FT): 1
*Area (FT2): 2
*Coefficient: 0.5

Figure 3.5.5: Wetland Outlet Dimensions

The weir was to be sized with an opening area of 2.0 ft^2 . The opening in this case is designed to be square shaped, although a v-shaped or any other weir shape would do. The center of the opening is 1.0 ft above the ground, with anything below being retained.

The following is a summary of the wetland flow and storage data used to construct the wetland:

Project: Wetland
Simulation Run: Run 1   Reservoir: WETLAND

Start of Run: 17Nov2012, 13:00   Basin Model: Basin 1
End of Run: 18Nov2012, 14:00   Meteorologic Model: 100 yr storm
Compute Time: 04Dec2012, 09:29:55   Control Specifications: Control 1

Volume Units: (•) IN ( ) AC-FT

Computed Results

Peak Inflow : 26.2089 (CFS)   Date/Time of Peak Inflow : 18Nov2012, 01:15
Peak Outflow : 6.4559 (CFS)   Date/Time of Peak Outflow : 18Nov2012, 02:00
Total Inflow : 3.09 (IN)   Peak Storage : 3.3 (AC-FT)
Total Outflow : 2.96 (IN)   Peak Elevation : 1.6 (FT)

Figure 3.5.6: Wetland Flow and Storage Data

3.5.6Construction Drawings

List of drawings:

3.5.1- Weir Profile

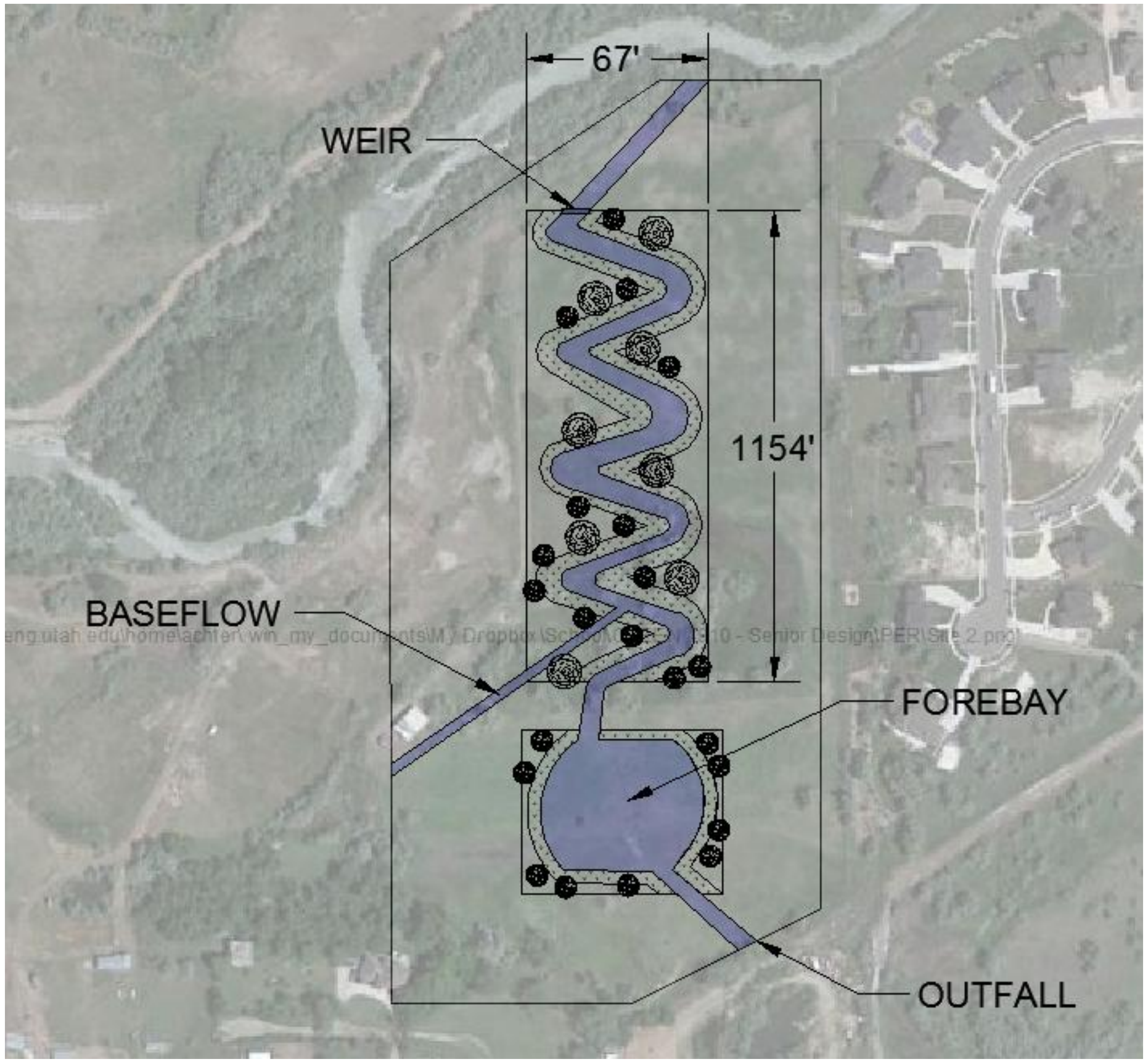
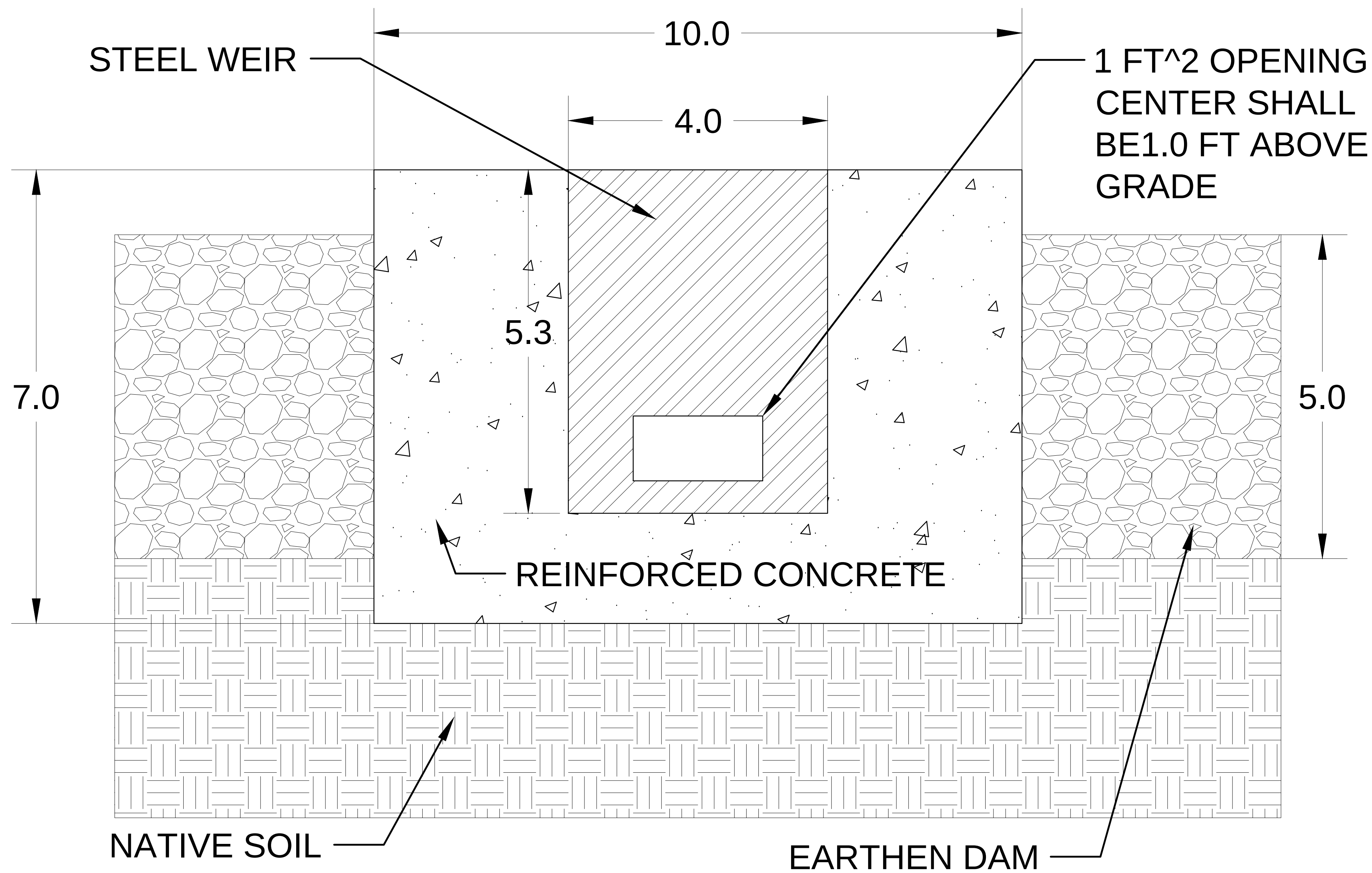


Figure 3.5.2: Extended Wetland Model Design



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STUDENT ENGINEERING ASSOCIATES
SEA

DESIGN NAME: Undeveloped Outfall

TITLE OF DRAWING: Weir Profile

PLAN NO. 1

SHEET NO. 3.5.1

3.5.7Work Cited

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