# Cherokee Heights Culvert Analysis and Erosion Control Feasibility Study

Prepared for
Lower Mississippi River Watershed Management Organization

April 8, 2015

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## April 8, 2015

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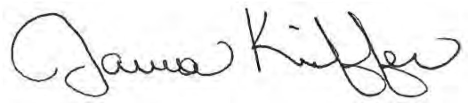
## Certifications

I hereby certify that this report was prepared by me or under my direct supervision and that I am a duly licensed Professional Engineer under the laws of the State of Minnesota.



April 8, 2015

Janna Kieffer, PE
PE #: 43571

Date

I hereby certify that the geotechnical section of this report was prepared by me or under my direct supervision and that I am a duly licensed Professional Engineer under the laws of the State of Minnesota.

April 8, 2015

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Date

# Executive Summary

## Background and Problem Description

The 60-inch diameter culvert is located under Cherokee Heights Boulevard in St. Paul, approximately 300 feet north of Annapolis Street. The drainage area tributary to this culvert is approximately 47 acres of residential and park land, and encompasses portions of St. Paul, Mendota Heights, and West St. Paul. The Cherokee Heights ravine is located downstream of the Cherokee Heights culvert within the Brickyard Area of Lilydale Regional Park, which is characterized by steep slopes, intermittent streams and seeps, and trails and ravines that convey stormwater from the direct and upland tributary areas. High flow rates and velocities through the culvert have caused erosion problems on the upstream and downstream ends of the culvert and in the downstream Cherokee Heights ravine. Erosion of the ravine has contributed to instability of adjacent banks and decreased water quality in downstream Pickerel Lake.

Slope stability and erosion concerns in the Brickyard Area, including in the ravine downstream of the Cherokee Heights culvert, prompted the City of St. Paul to initiate two separate but related studies. The *Cherokee Heights Culvert Analysis and Erosion Control Feasibility Study* (Cherokee Heights Feasibility Study), which is the subject of this summary report, was initiated by the City of St. Paul but undertaken by the Lower Mississippi River Watershed Management Organization (LMRWMO) due to the inter-community drainage to the culvert and downstream ravine. The objectives of the Cherokee Heights feasibility study were to identify and evaluate options for stabilizing the culvert and slopes downstream approximately 300 feet to improve stability of the adjacent banks, reduce erosion, and improve downstream water quality. The *Brickyard Area of Lilydale Regional Park Stormwater Management and Slope-Stability Study* (Brickyard Study) also conducted by Barr Engineering Co. (Barr, 2015), was undertaken by the City of St. Paul Department of Parks and Recreation, evaluated erosion and slope-stability issues in the larger Brickyard Area, with the primary objective of developing concept-level stormwater management, erosion-control, and slope-stability recommendations for the park area.

## Improvement Alternatives

Several improvement options were evaluated for stabilizing the Cherokee Heights culvert and downstream slope, including: (1) downstream channel stabilization; (2) upstream modifications; and (3) downstream piped conveyance.

### Downstream Channel Stabilization

To minimize erosion of the channel and side slopes and reduce the instability of adjacent banks within the ravine, regrading and stabilizing the channel is recommended. Stabilization should include armoring the channel with rip-rap and a properly graded filter material to prevent migration of underlying fine-grained soils through the rip-rap. Throughout portions of the ravine, the channel should be raised and the side slopes regraded to a more stable slope to reduce flow velocities and provide increased buttressing of the channel side slopes.

## Upstream Modifications

Installation of a tiered outlet upstream of the existing culvert to reduce peak flows and expansion of the storage area to accommodate temporary, short duration ponding/storage of runoff was evaluated as an improvement alternative. Because results of the geotechnical analysis indicate that increased ponding that promotes infiltration in the area upstream of the Cherokee Heights ravine can reduce the stability of downstream slopes, increased upstream infiltration and/or construction of a permanent, unlined stormwater detention pond to reduce peak flows were not considered further.

Due to the depth and steep side slopes of the low area/ravine directly upstream of the Cherokee Heights culvert, a substantial amount of excavation would be required within the ravine or in the upland park area, resulting in the loss of many highly-valued trees and significant change to the aesthetic character of the park. While upstream outlet modifications and storage expansion would be effective in reducing peak flows for most storm events, modeling results indicate that the 100-year peak flow is reduced only by 30%, so the design and associated costs of the downstream channel stabilization would not be significantly reduced by implementing upstream modifications.

## Downstream Piped Conveyance

The third alternative evaluated to help stabilize the channel downstream of the Cherokee Heights culvert was to install an underground pipe system down the entire bluff to convey runoff from the Cherokee Heights culvert to Pickerel Lake. While this pipe would effectively reduce flows and velocities in the Cherokee Heights ravine, the project has high construction costs, significant construction-related impacts, and some level of channel stabilization would still be necessary in the Cherokee Heights ravine for flows that exceed the capacity of the underground pipe.

# Opinions of Probable Costs

Planning-level opinions of construction costs for the three evaluated improvement alternatives are summarized in Table EX-1. The planning-level opinions of construction cost are intended to provide assistance in evaluating and comparing alternatives and should not be assumed as absolute values for given alternatives.

**Table EX-1 Planning-level opinions of construction costs for improvement alternatives**

| Improvement Alternative | Estimated Cost | Estimated Cost Range (-30%/+50%) |
|---|---|---|
| Downstream Channel Stabilization | $400,000 | $280,000 - $600,000 |
| Upstream Culvert Modifications and Ravine Expansion | $350,000 | $250,000 - $530,000 |
| Downstream Piped Conveyance | $2,100,000 | $1,500,000 - $3,200,000 |

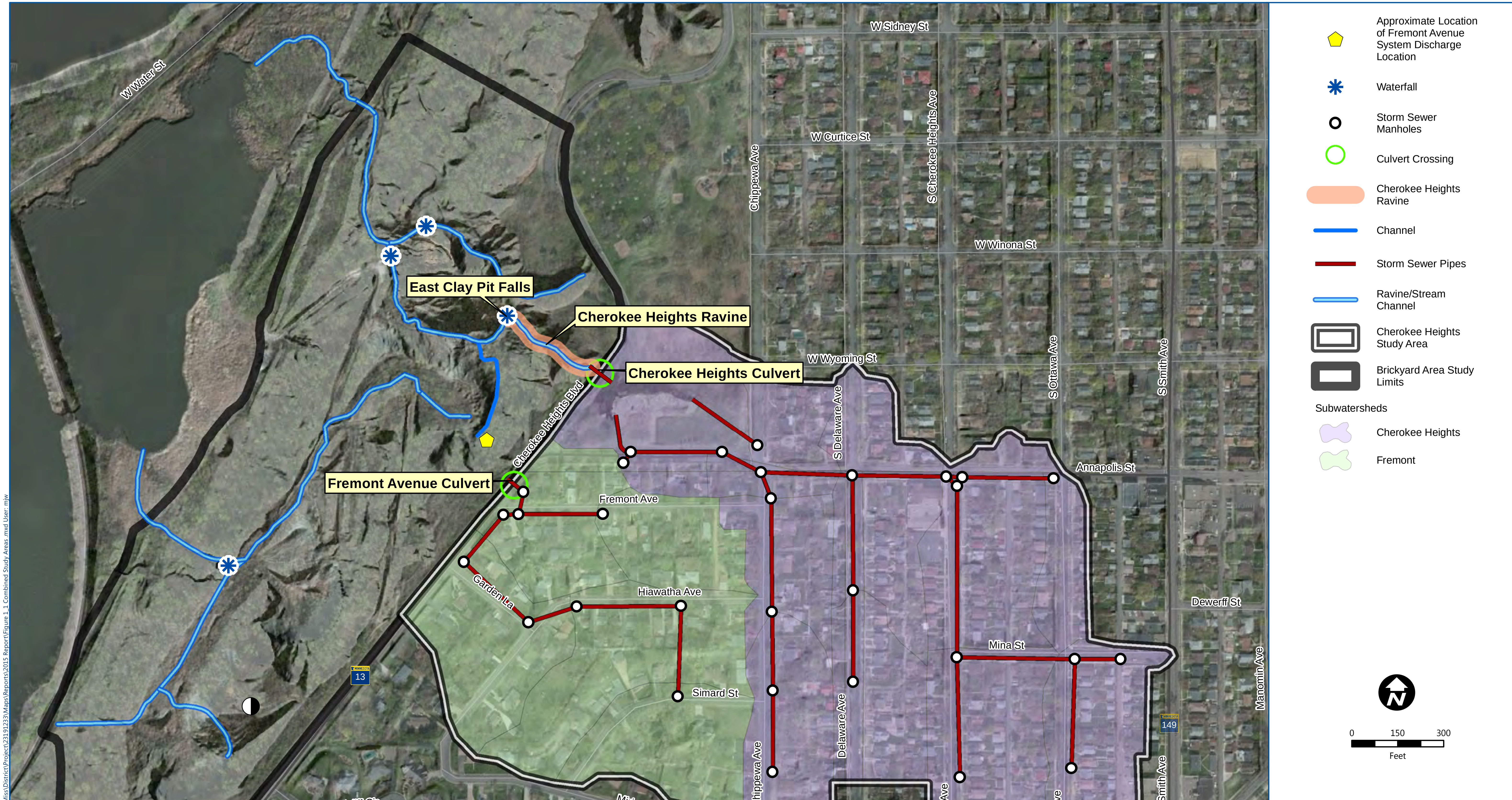
# 1.0 Background and Problem Description

## 1.1 Background

Drainage from portions of Cherokee Heights Regional Park and the adjacent residential area discharges through a 60-inch-diameter reinforced-concrete culvert (Cherokee Heights culvert) that extends underneath Cherokee Heights Boulevard and into a steep ravine. The Cherokee Heights culvert is located in St. Paul, approximately 300 feet north of Annapolis Street. The drainage area tributary to this culvert is approximately 47 acres of residential and park land, and encompasses portions of St. Paul, Mendota Heights, and West St. Paul. High flow rates and velocities through the culvert have caused erosion problems on the upstream and downstream ends of the culvert and in the downstream ravine, which conveys runoff down the bluff and eventually to Pickerel Lake.

The ravine downstream of the Cherokee Heights culvert is located within the Brickyard Area of Lilydale Regional Park, which is an area along the Mississippi River bluff that was used as a clay-mining and brick-making site during the 1890s to the 1970s. The Brickyard Area is characterized by steep slopes, intermittent streams and seeps, and trails and ravines that convey stormwater from the direct and upland tributary areas. Erosion of the ravines has contributed to instability of adjacent banks and decreased water quality in downstream Pickerel Lake.

Slope stability and erosion concerns in the Brickyard Area, including in the ravine downstream of the Cherokee Heights culvert, prompted the City of St. Paul to initiate two separate but related studies. The *Cherokee Heights Culvert Analysis and Erosion Control Feasibility Study* (Cherokee Heights Feasibility Study), which is the subject of this summary report, was initiated by the City of St. Paul but undertaken by the Lower Mississippi River Watershed Management Organization (LMRWMO) due to the inter-community drainage to the culvert and downstream ravine. The objectives of the Cherokee Heights feasibility study were to identify and evaluate options for stabilizing the culvert and slopes downstream approximately 300 feet to improve stability of the adjacent banks, reduce erosion, and improve downstream water quality. The *Brickyard Area of Lilydale Regional Park Stormwater Management and Slope-Stability Study* (Brickyard Study) also conducted by Barr Engineering Co. (Barr, 2015), was undertaken by the City of St. Paul Department of Parks and Recreation. This study evaluated erosion and slope-stability issues in the larger Brickyard Area, with the primary objective of developing concept-level stormwater management, erosion-control, and slope-stability recommendations for the park area. Figure 1-1 shows the study areas for both studies.




COMBINED STUDY AREAS
CHEROKEE HEIGHTS CULVERT ANALYSIS AND EROSION CONTROL FEASIBILITY STUDY

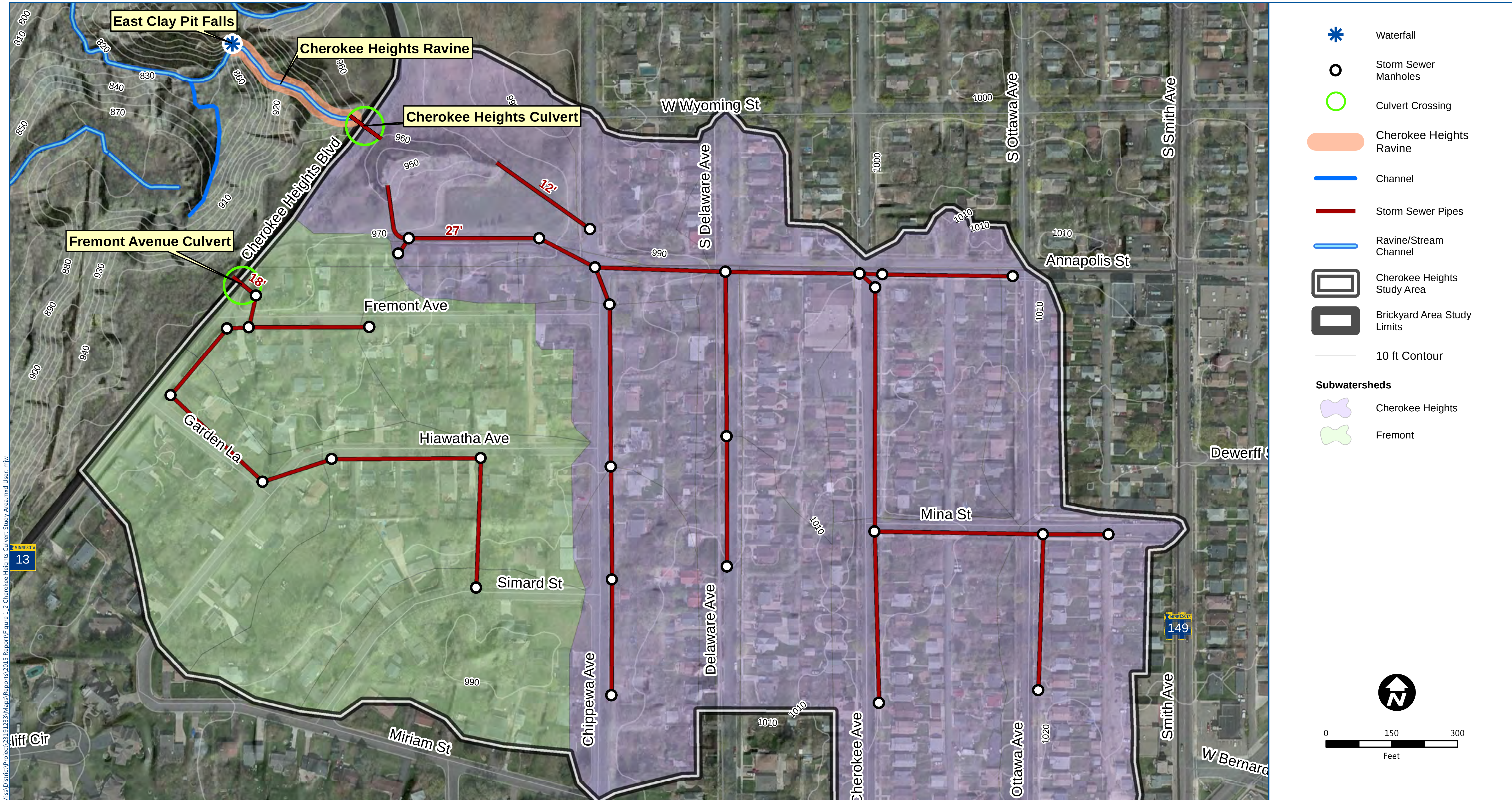
FIGURE 1-1

Figure 1-2 shows the area tributary to the 60-inch diameter Cherokee Heights culvert. As shown in the figure, there is a low area/ravine on the upstream side of the culvert, with significant elevation change from the nearby roadway and adjacent parkland. A 12-inch storm sewer from Chippewa Avenue discharges into the far northeast side of the low area/ravine. A 27-inch storm sewer system along Annapolis Street discharges to the low area/ravine on the southeast side. High velocities from this system have historically caused erosion issues at the downstream end of the culvert, as stormwater reflects off the existing energy dissipation structure and erodes the nearby bank (Photo 1-1). In 2013, the City of St. Paul conducted repairs to the existing system, which included reconstruction of the flared end section on the upstream side of the 60-inch culvert and installation of additional rip-rap protection, as well as placement of additional rip-rap at the outfall to reduce the erosion impacts. The 2013 City of St. Paul project also included installation of an 18-inch storm sewer and catch basins to convey runoff from Cherokee Heights Boulevard to the low area/ravine upstream of the 60-inch culvert and bulk heading of the storm sewer outfalls to the downstream ravine.

Figure 1-2 also shows an 18-inch corrugated metal pipe under Cherokee Heights/Highway 13 between Fremont Avenue and Annapolis Street (Fremont Avenue culvert). This pipe is part of a pipe system that extends approximately 150 feet down the bluff, with discharge then flowing through a channel and eventually joining the larger Cherokee Heights ravine downstream of the East Clay Pit Falls. Detailed information regarding the pipes downstream of the Fremont Avenue culvert was not available; therefore, these pipes are not shown in Figure 1-2. However, the approximate discharge location is identified based on a field visit by City of Mendota Heights staff. The Fremont Avenue system, owned and maintained by the Minnesota Department of Transportation (Mn/DOT), receives drainage from portions of Highway 13 and City of Mendota Heights storm sewer along Fremont Avenue. During large, intense rainfall events, the Fremont Avenue system also receives flow-by from Annapolis Street. When the runoff exceeds the capacity of the Fremont Avenue culvert, stormwater pools in a low area between the culvert and Annapolis Street, then flows in a northeasterly direction along Cherokee Heights Boulevard to the low area adjacent to the 60-inch Cherokee Heights culvert.

Based on Barr's field observations and email correspondence from Mn/DOT to City of St. Paul staff, it is our understanding that a pipe failure occurred within the existing Fremont Avenue culvert system in late-June 2014. As a result of a pipe separation, a large cavity developed along the westbound shoulder of Highway 13. The existing 18-inch corrugated metal pipe was replaced by Mn/DOT with a 30-inch corrugated plastic pipe that discharges to a riprap lined basin for energy dissipation. The newly-installed 30-inch plastic pipe discharges approximately 60 feet from the Highway 13 right-of-way, whereas the previous Fremont Avenue culvert system discharged down the bluff approximately 150 feet from the right-of-way into an existing channel. In recent correspondence between Mn/DOT and City of St. Paul staff, Mn/DOT indicated that installation of the 30-inch plastic pipe discharging to the riprap lined basin was an emergency fix, and that a permanent solution involving routing the drainage to a more stabilized outlet would be scoped for an upcoming Highway 13 resurfacing project. With the current outfall located mid-way down the bluff, discharge from the 30-inch Fremont Avenue replacement system has a high potential to cause increased erosion of the bluff.

Since the stormwater modeling had already been completed prior to receiving information on temporary replacement of the Fremont Avenue culvert, the study conditions and modeling results are based on the 18-inch Fremont Avenue culvert in place prior to the pipe failure.




CHEROKEE HEIGHTS CULVERT STUDY AREA
CHEROKEE HEIGHTS CULVERT ANALYSIS AND EROSION CONTROL FEASIBILITY STUDY

Figure 1-2

## 1.2 Problem Description

High flow rates and velocities through the ravine downstream of the Cherokee Heights culvert, in combination with sandy, erodible soils have caused erosion that is contributing to the instability of the adjacent banks and delivering sediment to downstream Pickerel Lake, thereby degrading its water quality. The area directly downstream of the Cherokee Heights culvert was observed to be highly eroded, with significant scouring around the flared end section (Photos 1-2 and 1-3). The Cherokee Heights Ravine channel is steep, with about 30 feet of drop in the approximately 300 feet between the culvert and East Clay Pit Falls (see Figure 1-1). Observation of the channel revealed significant erosion along the channel bottom and side slopes, with the channel bottom scoured down to the underlying Decorah Shale bedrock at the downstream end of the ravine near the East Clay Pit Falls.

The channel within the Cherokee Heights Ravine is fairly narrow and meanders slightly between the culvert and the East Clay Pit Falls. The ravine side slopes are steep and unstable, with several active failures of the adjacent banks observed in the ravine during the summer of 2014, most notably near the culvert outlet and approximately midway to the East Clay Pit Falls. Continued downcutting of the channel is aggravating the situation. As the channel down cuts, it erodes the bottom of the adjacent side slopes (toe of the slope); the bottom of the slopes becomes much steeper than above, leading to an unstable condition and resultant failures of the ravine walls. Several large, mature trees have been lost within the ravine due to the undercutting of side slopes, as shown in Photo 1-4. Loss of large, mature trees, and their root structure, can further destabilize the ravine slopes.

Just above the East Clay Pit Falls, a berm of soil directs the flow path of the stream roughly parallel to the edge of the East Clay Pit wall. Several sections of broken pipe were observed in this area; two sections appear to be held in place by the roots of a mature tree above the waterfall and parallel to the stream flow. Although the original purpose and use of these pipe sections is unknown, they no longer convey flow and water spills over the falls to the downstream channel.

**Photo 1-1 Energy dissipation structure downstream of the 27-inch outfall and erosion of adjacent side slope (May 2014 site visit)**

**Photo 1-2** **Erosion adjacent to the storm sewer outlet in Cherokee Heights ravine (May 2014 site visit)**

**Photo 1-3** **Erosion and scouring at the outfall of the Cherokee Heights culvert (May 2014 site visit)**

**Photo 1-4 Cherokee Heights ravine slope failure (July 2014 site visit)**

## 1.3 Project Objective and Approach

The objective of this study was to evaluate options to reduce erosion by stabilizing the approximately 300 feet of channel between the Cherokee Heights Culvert and the East Clay Pit Falls and reducing peak flow rates and velocities, as feasible. Due to the concern regarding the stability of the existing slopes in the Brickyard Area downstream of the Cherokee Heights Culvert, the project approach included a geotechnical analysis. Of specific interest were the effects of potential changes in water content/saturation on downstream slope stability, as options to reduce flows include upstream ponding. A stormwater model of the study area was also developed to understand flowrates and velocities through the culvert and downstream ravine. The geotechnical and stormwater analyses for the Cherokee Heights Feasibility Study were conducted in conjunction with the Brickyard Study.

# 2.0 Geotechnical Analysis

A combined geotechnical analysis was conducted as part of this study and the Brickyard Area study to evaluate the stability of the existing slopes in the Brickyard Area, including the ravine downstream of the Cherokee Heights culvert. The geotechnical analysis also included evaluation of the potential impacts of increased stormwater infiltration in the upland area on downstream slope stability. The study limits of the geotechnical analysis are shown in Figure 2-1. A summary of the geotechnical analysis is provided in the following sections. Additional information is available in Appendix C of the *Brickyard Area of Lilydale Regional Park Stormwater Management and Slope-Stability Study* (Barr, 2015).

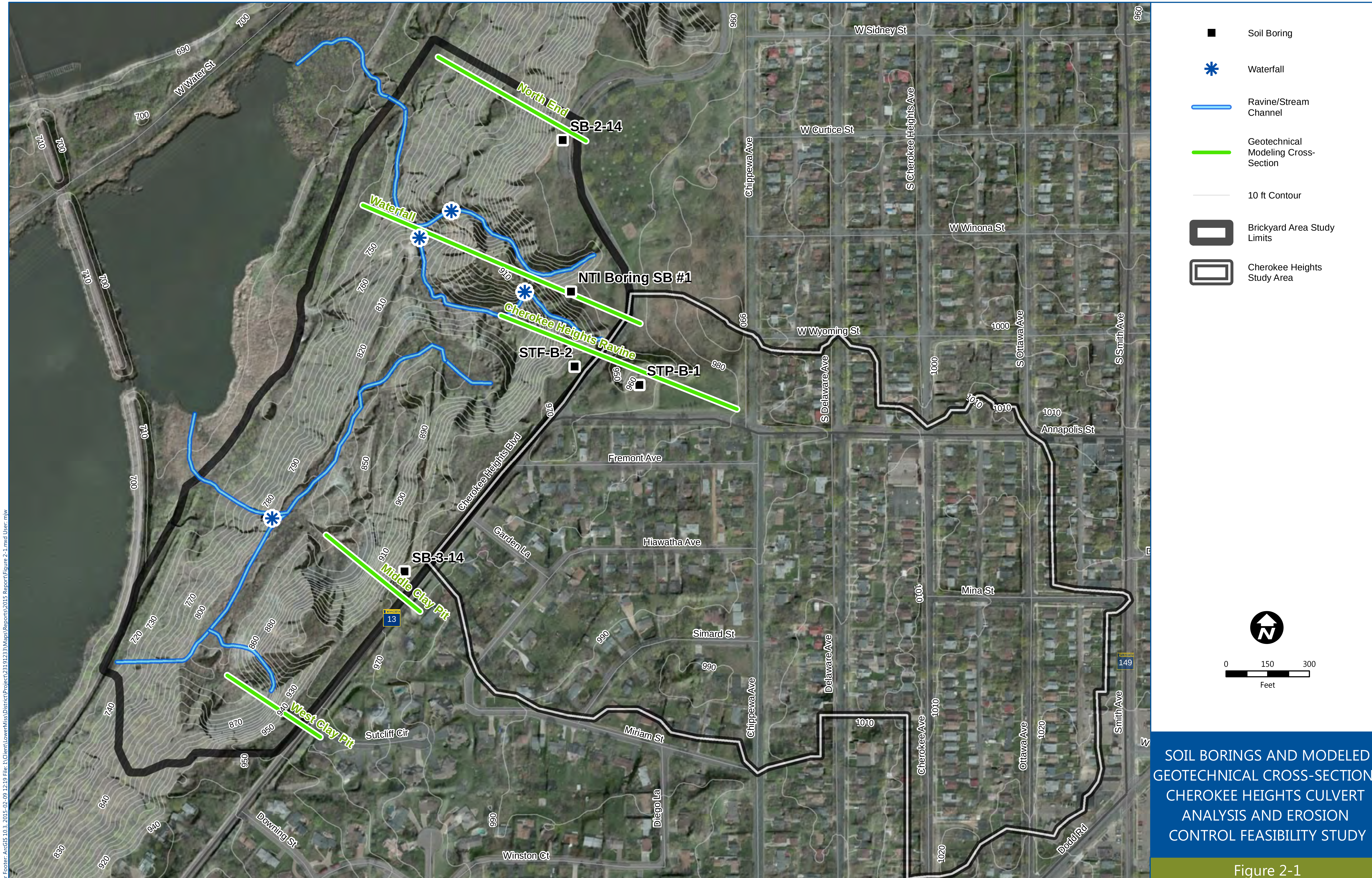
## 2.1 Field Investigation—Soil Borings and Lab Analysis

To evaluate the stability of the existing slopes and the effects of potential changes in water content/saturation, the physical properties of the soil and rock need to be understood. These properties consist of the following:

- Stratigraphy of the soils in the area of interest
- Natural moisture content of the soils
- Unit weight of the soils and rock
- Plasticity of the clay soils/weathered rock
- Grain size of the soils
- Strength of the soils (both undrained/drained and saturated/unsaturated, as appropriate)
- Presence of weak soil/rock layers
- Permeability of the soils

A total of five soil borings were completed (one as part of the previous NTI study of the Brickyard area, four by Barr). Boring locations are shown on Figure 2-1 and described in the Brickyard Area Study report. The termination depths of the borings ranged from approximately 50 to 104 feet below existing grade, with most of the borings reaching about 100 feet below existing grade.

Soil samples were transported to Soil Engineering Testing (SET) of Richfield, Minnesota, for laboratory analysis. Results from the laboratory analysis are included in the Brickyard Area Study report.




SOIL BORINGS AND MODELED GEOTECHNICAL CROSS-SECTIONS
CHEROKEE HEIGHTS CULVERT ANALYSIS AND EROSION CONTROL FEASIBILITY STUDY

Figure 2-1

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### 2.1.1 General Site Geology

The bedrock in the geotechnical analysis study areas was formed in Cambrian and Ordovician times, when Minnesota was located in a tropical climate near the equator.

The upper bedrock encountered in the geotechnical study area is the lower portion of the Galena Group. The Galena Limestone, a hard, buff-colored limestone rock, is mapped as the top bedrock unit near the park. Based on soil borings performed for this study, the Galena Limestone was very thin to absent. The basal member of the Galena Group is the Decorah Shale, a grayish-green shale rock with a high concentration of fossils encountered below the site soils (Minnesota Geological Survey, 1999). This is the primary bedrock unit in the park and forms the walls of the three clay pits within the Brickyard Area of Lilydale Regional Park.

### 2.1.2 Stratigraphy

The stratigraphy (rock and soil layers) of the geotechnical study area generally consists of sandy, glacially derived soils of variable thickness overlying shale, then sandstone bedrock, as described in the site geology section of Appendix C of the *Brickyard Area of Lilydale Regional Park Stormwater Management and Slope-Stability Study Report*. Occasional clay seams were encountered in the soils and interbedded limestone layers were seen in the Decorah Shale.

Cross sections interpreted from the boring logs are provided in the *Brickyard Area of Lilydale Regional Park Stormwater Management and Slope-Stability Study Report* to illustrate the inferred subsurface conditions. As an example, Figure 2-2 shows the stratigraphy for the Middle Clay Pit. The other cross sections are similar, but with soil layers varying in order and thickness. For modeling purposes the presence of the limestone layers inter-bedded with the shale was not included. Additional information on the stratigraphy of each cross section can be found in Appendix C of the Brickyard Area Study report.

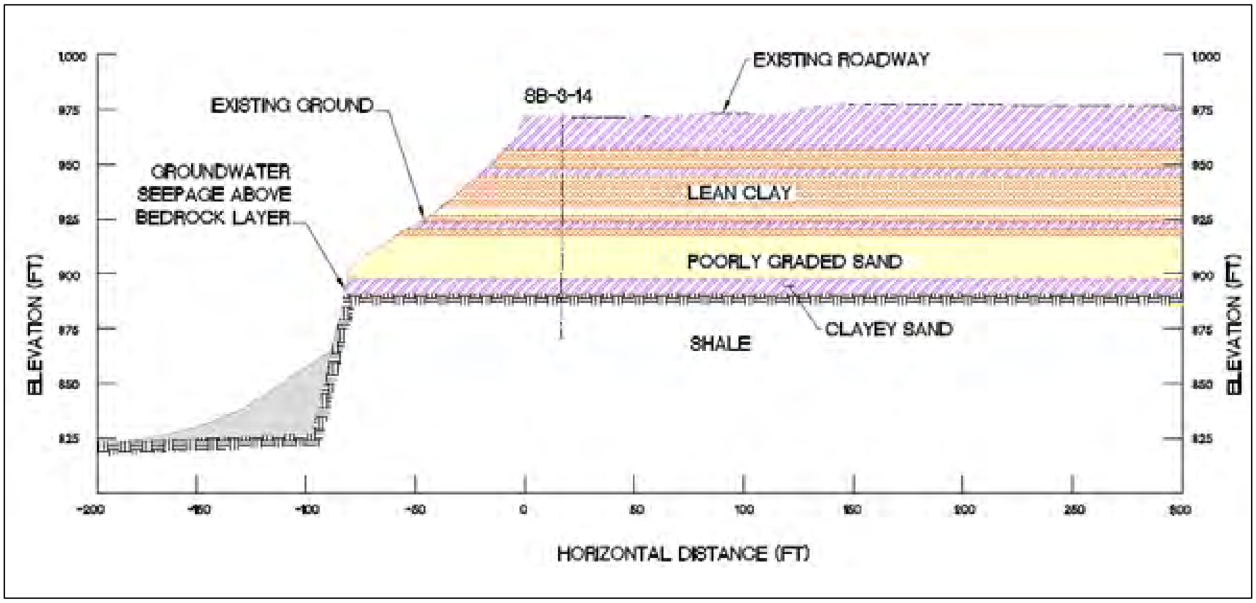



**Figure 2-2 Example modeling cross section showing stratigraphy (Middle Clay Pit cross section in Brickyard Area)**

### 2.1.3 Groundwater Conditions

Groundwater was encountered in all of the soil borings directly above the top of the bedrock. In borings performed near the East and Middle Clay Pits, there were several upper soil layers that were saturated. However, there were soils below these layers that did not exhibit elevated moisture content; thus, the upper readings recorded during drilling indicated “perched” water, likely flowing through more permeable soils, as opposed to a solid water table down to bedrock.

Seepage was observed weeping from many of the site slopes at the soil/bedrock interface, but not usually seen higher in the slopes. Therefore, the groundwater was assumed to be generally located at the soil/bedrock interface at most times of the year.

## 2.2 Slope-Stability-Simulation Modeling

SLOPE/W and SEEP/W software, part of the GeoStudio 2012 suite of programs, was used to evaluate the influence of existing topography, soil strength, and effects of seepage and saturation on the stability of the slopes within the Brickyard Area. The modeling cross-section locations, shown in Figure 2-1 focused on areas of moderate-to-large potential slope failure (not shallow, surficial sloughing).

Once the cross sections were defined, SLOPE/W (a limit equilibrium slope-stability-analysis program) was used to evaluate stability of the selected critical slope sections. Since the existing slopes have remained stable for extended periods of time, the failures are likely influenced by the presence of additional soil moisture/saturation, weakening soil and rock, and increased load at the head of the slopes. Therefore, Barr also evaluated the influence of seepage and saturation using the SEEP/W component of the

GeoStudio 2012 software suite. This component is specifically designed to perform analysis of seepage, groundwater infiltration, and effects of soil saturation on slope stability.

Detailed modeling methodology and results are provided in Appendix C of the Brickyard Area Study report.

### 2.2.1 Modeling Factor of Safety

The factor of safety of a slope is defined as the ratio of the resisting forces in the soil to the driving or mobilized forces that cause slope movement. Therefore, the point of stability is considered a factor of safety of 1.0 (driving forces equal to resisting forces). Slopes with a factor of safety less than 1.0 are considered to be unstable and would fail; slopes with a factor of safety higher than 1.0 are considered stable (or marginally stable as the safety factor approaches or hovers close to 1.0).

Natural soil slopes which are stable or marginally stable usually have minimum calculated factors of safety of 1.1 to 1.3. Factors of safety for natural slopes are representative for typical “sunny day” conditions, but may be reduced or even drop below 1.0 in the presence of excess moisture from rainfall, changes in groundwater elevations, etc. Therefore, the factor of safety for a slope should be considered for a range of anticipated conditions to determine the potential for slope failure. Analyses of several different sets of conditions to determine the potential for slope failures along the bluff line within the geotechnical study area were performed.

### 2.2.2 Soil Suction

Review of the topography throughout the site indicates that the angles of some slopes exceed the drained friction angle of the soils. If the strength of the soils was governed only by the drained friction angles, the slopes would be unstable and fail. To allow for steep slopes to remain standing, the soils must have additional strength beyond their angle of friction. The soil mechanism allowing this is called soil suction. Soil suction is formed by drying or dewatering the soils, which creates a negative pore pressure in the soil’s pore spaces and increases the strength of the soil matrix (or provides an apparent cohesion in the soil in excess of its drained friction angle).

The phenomena of soil suction can be illustrated by thinking of a common sand castle at the beach. Dry sand will only form a conical pile to a certain angle (the material’s drained friction angle). However, sand with moderate water content will allow much steeper angles to be achieved. Then, as the castle sits in the sun and dries, the sides of the castle become unstable and slough off. Or, as the tide comes in and the sand at the base of the castle becomes saturated, the sides of the castle slough and collapse. By drying or saturating the soils, the suction force is negated; the soil strengths will be governed by their friction angle and failures will occur.

Modeling of the existing slopes within the geotechnical analysis study area, including suction forces predicted by the physical index characteristics of the clay soils, suggests factors of safety ranging from about 1.1 to 1.4. However, when the soils are re-saturated the suction force is negated; the soil strengths will be reduced and slope failures will occur.

### 2.2.3 Saturation and Loss of Stability Due to Rainfall

To determine the effects of rainfall (i.e., saturation of the soils resulting in loss of suction) a unit flux line located at the ground surface in each of the cross sections was used to model the effects of groundwater infiltration. The modeling conservatively assumed full infiltration of 3.5 inches of steady precipitation over a 24-hour period. The modeling also assumed both low and high soil permeability. Lower soil permeability is associated with unsaturated conditions (low moisture content). Very little infiltration occurs in unsaturated soils, which is why flash flooding occurs in desert environments. Higher soil permeability essentially allows the full amount of precipitation to infiltrate the soil.

Modeling results using lower permeability (low infiltration) indicated that a single rainfall event of this magnitude on moderately saturated soils is not, by itself, likely to significantly reduce the stability of the slopes. However, when higher amounts of infiltration are considered, the factors of safety are reduced below stability. This indicates that if soil conditions allow for infiltration of some precipitation, the strength of the natural sand soils is reduced from loss of suction and could result in slope failures.

### 2.2.4 Saturation and Loss of Stability from High Ground Water and Ponding

Saturation and reduction of slope stability can also be caused from higher groundwater levels in the soil. Higher groundwater can result from infiltration of water from either rainfall events upstream of the area or by infiltration through the bottom of permanent stormwater ponds.

To better understand the relationship between high groundwater and a reduction in downstream slope stability, the model was adjusted to include a stormwater pond upstream of the Cherokee Heights culvert. There were two conditions assumed for this pond; a dry pond (a pond allowed to drain rapidly through a surface outlet following storm events) with groundwater just below the surface of the pond bottom, and a partially full pond (which could be a product of designing the pond to hold water for longer periods of time or may occur from slower drainage following a storm event).

The geotechnical analysis indicated that a permanent pond upstream of the Cherokee Heights culvert that retains water for a significant time may potentially allow enough stormwater to infiltrate, reducing the stability of the bluff slope to below a safety factor of 1.0 (unstable conditions). If the upstream pond functions like a temporary/dry pond, draining rapidly through the culvert following storm events and allowing only minimal infiltration (current condition), the stability of the bluff slope is reduced as compared with no infiltration from ponding, but the factor of safety remains above a factor of safety of 1.0.

As noted above, saturation of the soils resulting from significant rainfall(s) can reduce the factors of safety below stability, regardless of upstream ponding conditions.

### 2.2.5 Role of Vegetation in Stability

There is diverse vegetation on the upper soil slopes of the Brickyard Area and trees of various sizes—from saplings to mature 40-foot trees. There is also grass/weed vegetation that has formed carpet-like mats on many of the slopes within the Brickyard Area.

In certain scenarios, vegetation can help increase slope stability by reinforcing soils and absorbing water that would otherwise increase moisture content. However, trees in the study area have not stabilized the larger slides, as evidenced by the fallen trees observed during the July 2014 site visit. Furthermore, trees that are overhanging or near the edge of slopes may help trigger landslides when undermined, unstable, or blown over—dragging the surrounding soils down the slope.

Ultimately, some form of surface vegetation should be placed on the exposed slopes. Otherwise, erosion will create large amounts of downstream sediment that is both costly and time-consuming to manage. If slopes are regraded or existing vegetation is removed, re-vegetation that is suitable to park conditions and able to minimize soil erosion, such as deep-rooted understory, is recommended. Removal of larger trees overhanging or near the edge of the soil slopes may also be beneficial, reducing these as a trigger mechanism for slides.

## 2.3 Summary of Geotechnical Findings

The factor of safety of a slope is defined as the ratio of the resisting forces in the soil to the driving or mobilized forces that cause slope movement. Therefore, the point of stability is considered a factor of safety of 1.0 (driving forces equal to resisting forces). Slopes with a factor of safety less than 1.0 are considered to be unstable and would fail; slopes with a factor of safety higher than 1.0 are considered stable (or marginally stable as the safety factor approaches or hovers close to 1.0).

Seepage and soil saturation (which results in a loss of suction) can reduce stability of the slopes. Geotechnical modeling results indicate that the infiltration of approximately 3.5 inches of water in a 24-hour period is enough to impact soil stability in the study area. Loss of suction can also be realized through elevation of the groundwater table following periods of heavy precipitation or increased infiltration resulting from upstream ponding.

To better understand the relationship between upstream ponding and reduction in downstream slope stability, the geotechnical model was adjusted to include a stormwater pond upstream of the Cherokee Heights culvert. The geotechnical analysis indicated that a partially full pond upstream of the Cherokee Heights culvert that retains and infiltrates water for a significant time may potentially allow enough stormwater to infiltrate, reducing the stability of the bluff slope to below a safety factor of 1.0 (unstable conditions). If the upstream pond functions like a temporary/dry pond, draining rapidly through the culvert following storm events and allowing only minimal infiltration (current condition), the stability of the bluff slope is reduced as compared with no infiltration from ponding, but the factor of safety remains above a factor of safety of 1.0.

# 3.0 Stormwater Analysis

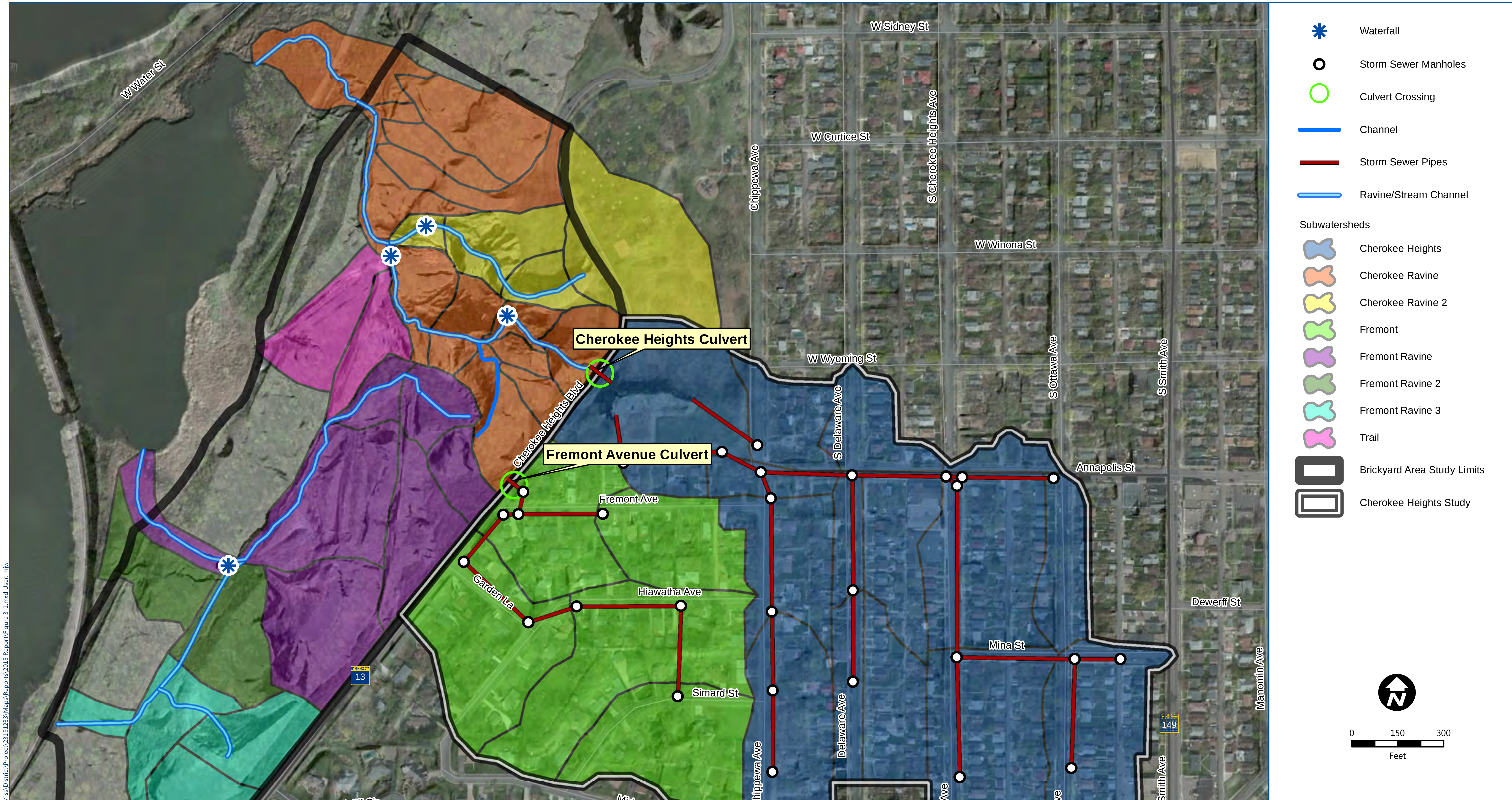
## 3.1 Stormwater Modeling

An XP-SWMM hydrologic and hydraulic model was developed to estimate stormwater runoff depths and corresponding flows and velocities in the storm sewer system, channels, and ravines throughout the study area. XP-SWMM uses rainfall and watershed characteristics to estimate local runoff, which is routed through pipe and overland-flow networks. The XP-SWMM hydrologic and hydraulic model was developed to gain a better understanding of drainage patterns throughout the study area, including runoff rates and flows and velocities through the Cherokee Heights culvert and downstream ravine. The XP-SWMM model for the Cherokee Heights study was developed in conjunction with the Brickyard Area Study.

The drainage area tributary to the Cherokee Heights culvert was delineated into subwatersheds that represent major stormwater inflow points to the storm sewer system. Subwatersheds were also delineated to the Cherokee Heights ravine and other ravines within the Brickyard Area. The subwatershed divides are shown in Figure 3-1. The model includes storm sewer information provided by the contributing cities. There are three culverts under Cherokee Heights Boulevard/TH 13 that serve as the main stormwater discharge points into the Brickyard Area of Lilydale Regional Park, including the 60-inch Cherokee Heights culvert and the Fremont Avenue culvert identified in Section 1.0. The location of the storm sewer pipes and the three culverts under Cherokee Heights Boulevard are also shown in Figure 3-1.

The ravines are modeled using representative natural channel cross sections to reflect the unique shapes of the ravines at specific locations along the bluff and throughout the Brickyard Area, based on 2011 topographic information provided by the Minnesota Department of Natural Resources.

The model was used to simulate the 1-, 2-, 5-, 10-, 50-, and 100-year frequency, 24-hour rainfall events based on National Oceanic and Atmospheric Administration (NOAA) Atlas 14 precipitation frequency estimates. Detailed modeling methodology and results can be found in Appendix A.




STORMWATER MODELING
CHEROKEE HEIGHTS CULVERT
ANALYSIS AND EROSION
CONTROL FEASIBILITY STUDY

FIGURE 3-1

## 3.1 Summary of Stormwater Findings

Site observations during the field visits identified erosion issues in the ravine downstream of the Cherokee Heights culvert. High flow rates and velocities in this channel, in combination with erodible, sandy soils appear to be (1) contributing to some localized instability of adjacent slopes; (2) removing material from the toes of the slopes; (3) destabilizing the upper slopes; and (4) causing slides into the ravine.

To adequately address the ravine erosion issues, it is important to understand the flow rates and flow velocities in the channel. Table 3-1 provides a summary of the estimated peak flow rates through the Cherokee Heights culvert for various storm event recurrence intervals.

**Table 3-1 Estimated flow rates through the Cherokee Heights culvert**

| Rainfall Event Recurrence Interval | Precipitation Depth (Atlas 14) over a 24-Hour Period (inches) | Peak Flow Rate (cfs) |
|---|---|---|
| 1 year | 2.5 | 54 |
| 2 year | 2.8 | 70 |
| 5 year | 3.5 | 109 |
| 10 year | 4.2 | 116 |
| 50 year | 6.3 | 252 |
| 100 year | 7.5 | 295 |

The peak flow velocities vary by reach, depending on contributing flow rate, channel shape, and channel slope. The Cherokee Heights culvert has a steep slope (approximately 8%), which results in high peak flow velocities estimated to range from 23 to 24 feet per second (ft/s) for the 10-year, 50-year, and 100-year frequency, 24-hour rainfall events. Estimated flow velocities in the channel downstream of the Cherokee Heights culvert are lower, as shown in Table 3-2.

**Table 3-2 Estimated flow velocities in Cherokee Heights culvert and downstream channel**

| Reach/Culvert | Peak Flow Velocity 10-year, 24-hour Rainfall (feet per second) | Peak Flow Velocity 50-year, 24-hour Rainfall (feet per second) | Peak Flow Velocity 100-year, 24-hour Rainfall (feet per second) |
|---|---|---|---|
| Cherokee Heights Culvert | 23 | 23 | 24 |
| Cherokee Heights Ravine approximately 140 feet downstream of culvert | 10 | 13 | 14 |
| Cherokee Heights Ravine- directly upstream of East Clay Pit Falls | 14 | 18 | 19 |

# 4.0 Improvement Alternatives

Several improvement options were evaluated for stabilizing the Cherokee Heights culvert and downstream slope, including: (1) downstream channel stabilization; (2) upstream modifications; and (3) downstream piped conveyance.

## 4.1 Downstream Channel Stabilization

The first improvement alternative included stabilization of the downstream ravine using a combination of engineered and bioengineered techniques. The Cherokee Heights Ravine channel is steep, with about 30 feet of drop in the approximately 300 feet between the culvert and East Clay Pit Falls. Observation of the channel revealed significant erosion along the channel bottom and side slopes, with the channel bottom scoured down to the underlying Decorah Shale bedrock at the downstream end of the ravine near the East Clay Pit Falls. While steep, the channel slope varies considerably. We recommend that the channel be built up in the low points to gain a more consistent channel slope, as shown in Figure 4-1. Raising portions of the channel would reduce flow velocities in some sections of the reach and provide increased buttressing of the channel side slopes. We have assumed that fill would be acquired from adjacent slope grading efforts.

The channel within the Cherokee Heights Ravine is fairly narrow and meanders slightly between the culvert and the East Clay Pit Falls. High flow rates and velocities through this reach, in combination with sandy, erodible soils have caused erosion that is contributing to the instability of the adjacent banks and delivery of sediment to downstream Pickerel Lake, thereby degrading its water quality. As noted above, the ravine side slopes are unstable and become more so when the ground is saturated. Several active failures of adjacent banks within the ravine were observed, most notably near the culvert outlet and approximately midway to the East Clay Pit Falls. Continued downcutting of the channel is aggravating the situation. As the channel downcuts it erodes the bottom of the adjacent side slopes (toe of the slope); the bottom of the slopes becomes much steeper than above, leading to an unstable condition.

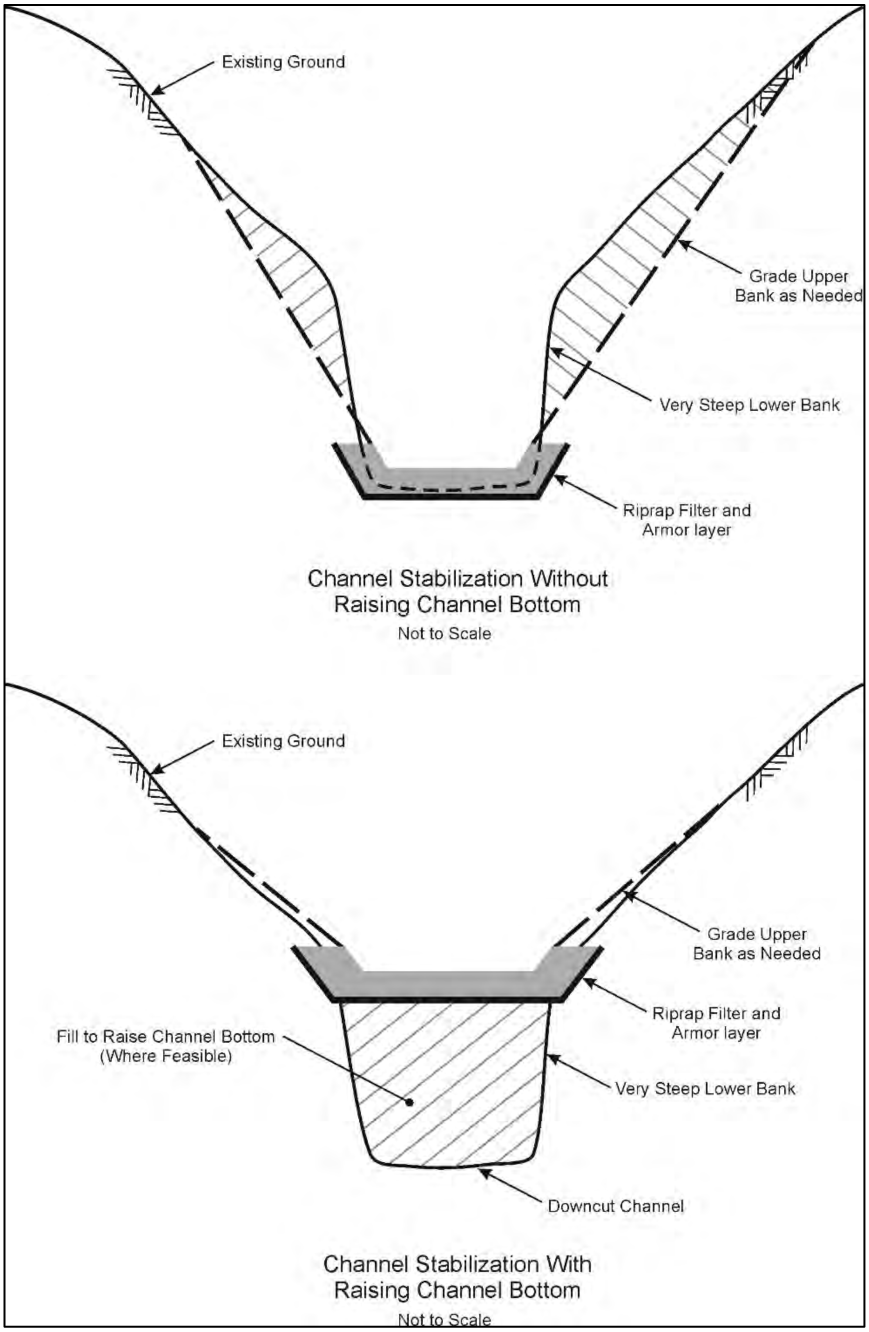


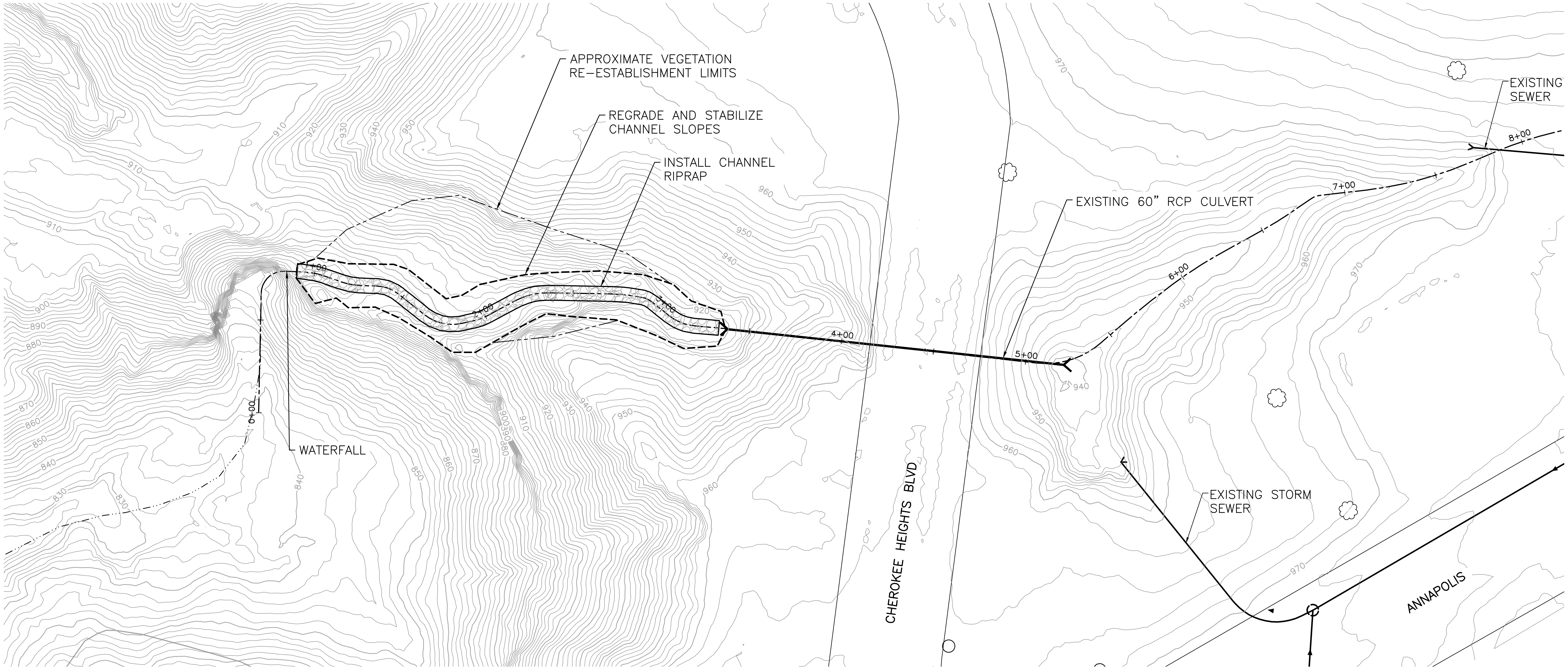

**Figure 4-1 Comparison of side slope grading with and without raising channel bottom**

Figure 4-2A describes the recommended stabilization of the ravine channel between the Cherokee Heights culvert and the East Clay Pit Falls, which includes regrading and armoring the channel with rip-rap. The proposed improvements will reduce channel erosion and minimize undercutting of the toe of the slope, which contributes to instability of the ravine side slopes. High flow velocities in the channel preclude use of many bio-engineering techniques for stabilization, as these techniques typically would not withstand the magnitude of flow velocities from large, intense storm events. In addition, the ravine is heavily shaded with little stabilizing ground vegetation. Rip-rap used for the channel stabilization would be large-diameter, with a mean diameter of 15 inches and maximum of 30 inches, based on the predicted peak velocities through this channel for a 100-year, 24-hour rainfall event (see Section 3.0). A typical channel cross section, as shown in Figure 4-2A, would include installation of properly graded filter material below the riprap to prevent migration of underlying fine-grained soils through the rip-rap.

Some of the side slopes in this ravine are steeper than the natural friction angle or angle of repose of the existing soils (see Section 2.0), and are at risk of failing under saturated ground conditions. To reduce the risk of slope failure within the ravine, the channel bottom can be raised and the side slopes can be graded to a more stable slope, as shown in the Figure 4-2A cross section and Figure 4-2B profile. Filter material and rip-rap should extend up along the face of the slope, to protect the toe of slope from undercutting erosion and to buttress the slope for increased long-term stability. This provides weight along the face of the slope to physically restrict further movement of the ravine side slopes, while also providing erosion control. Due to the steep topography within the Cherokee Heights Ravine area, it may not be feasible to regrade the ravine side slopes throughout the entire reach to more stable slopes; simply raising the channel bottom in these areas will provide some benefit. The extent of side slope regrading should be determined as part of final design based on topography constraints and desired impact footprint. However, it is important to note that side slopes steeper than the natural friction angle may continue to be unstable under saturated ground conditions and could reduce the longevity of channel improvements if slope failures occur.

The East Clay Pit Falls is located at the downstream end of the Cherokee Heights ravine. Just upstream of the East Clay Pit Falls, a berm of soil directs the flow path of the stream roughly parallel to the edge of the East Clay Pit wall. Channel stabilization design should include consideration of measures to further stabilize this area and provide energy dissipation at the falls, as necessary.

Vegetation management should be implemented with the channel armoring and slope grading. Selective removal of less-desirable mature trees and buckthorn would improve the light penetration to the forest floor, thereby promoting ground vegetation and opening up views. Removal of larger trees overhanging or near the edge of the ravine side slopes would also be beneficial, reducing these as a trigger mechanism for slides. Selective planting of desirable ground vegetation (and protection of the plantings) would promote long-term stability of the ravine.

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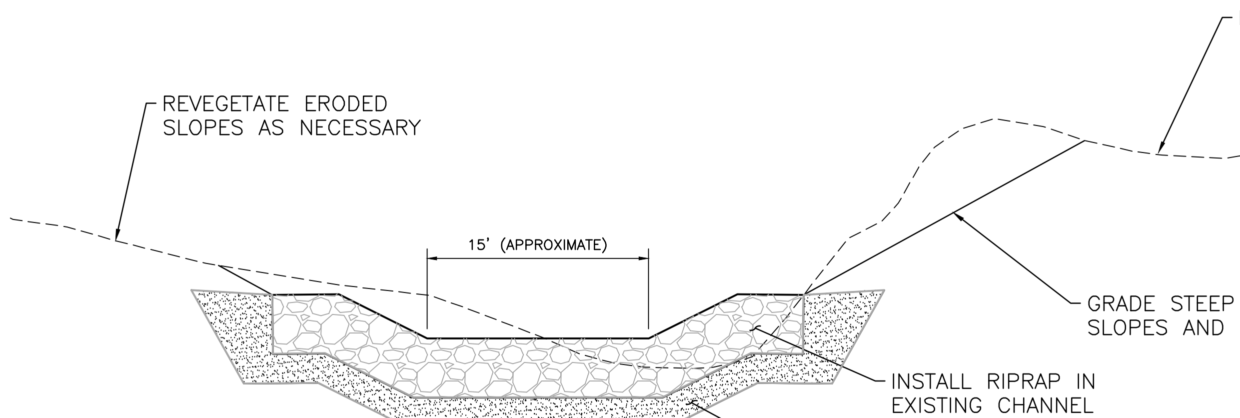


2 SECTION: TYPICAL CHANNEL SECTION – MATCH EXISTING CHANNEL ELEVATION
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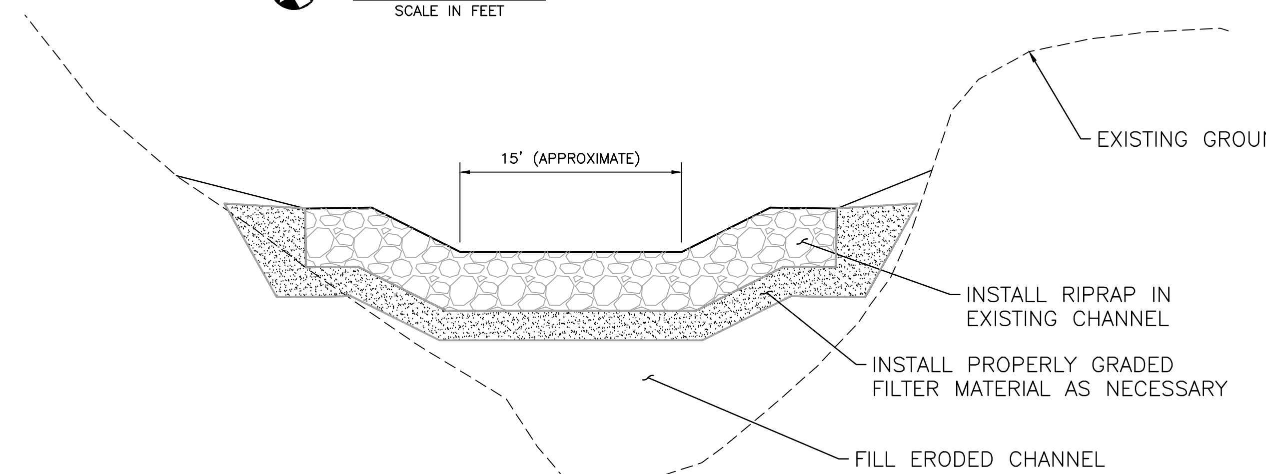


3 SECTION: TYPICAL CHANNEL SECTION – RAISE CHANNEL ELEVATION
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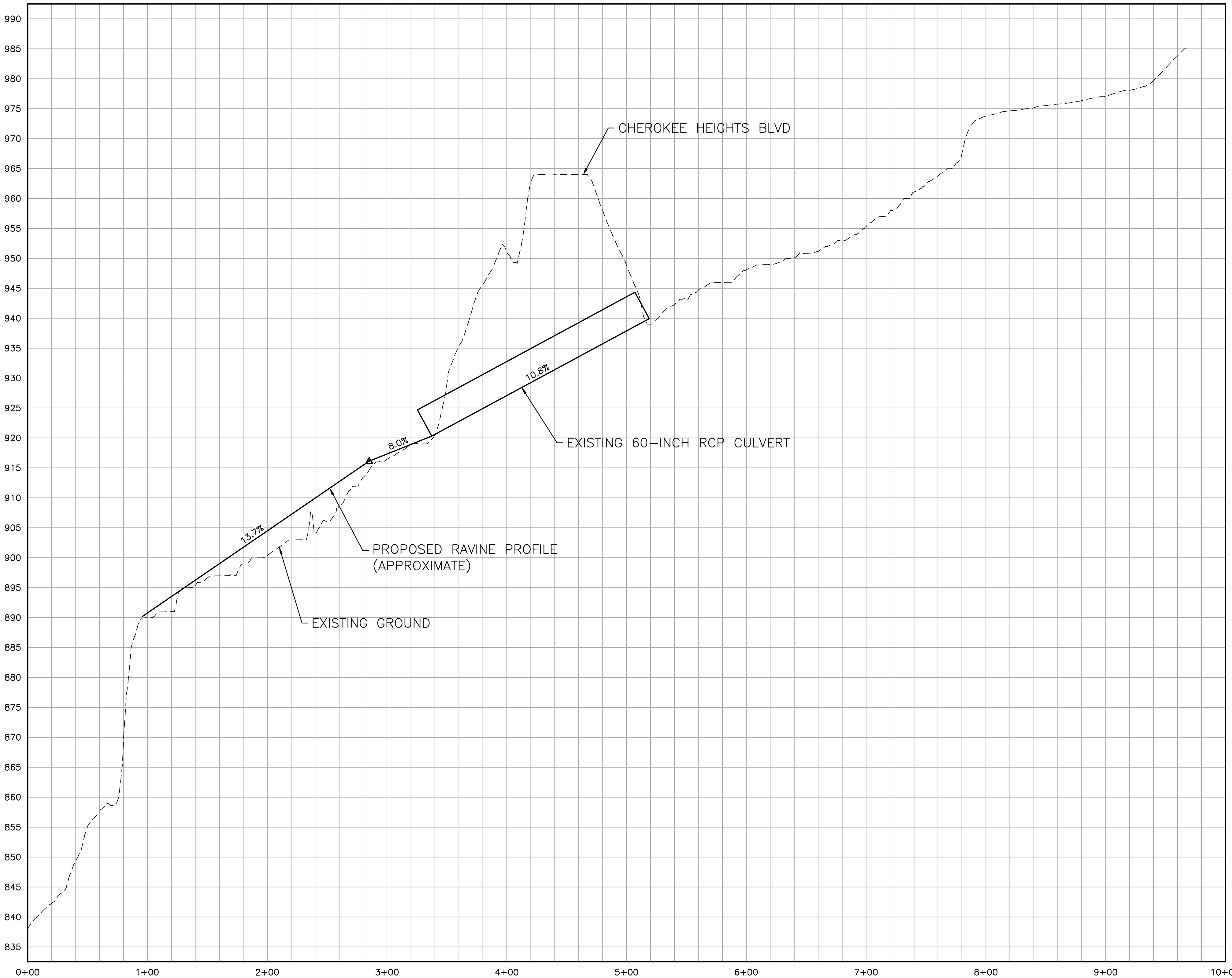
FIGURE 4–2A
DOWNSTREAM CHANNEL STABILIZATION

CHEROKEE HEIGHTS CULVERT ANALYSIS AND
EROSION CONTROL FEASIBILITY STUDY




1 PROFILE: DOWNSTREAM CHANNEL
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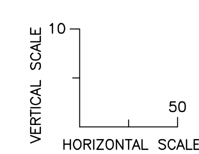



FIGURE 4–2B
DOWNSTREAM CHANNEL STABILIZATION PROFILE

CHEROKEE HEIGHTS CULVERT ANALYSIS AND
EROSION CONTROL FEASIBILITY STUDY

## 4.2 Upstream Modifications

The second improvement alternative included modifications to the upstream ravine or watershed tributary to the culvert to reduce peak flows and velocities through the culvert and minimize erosion. One of the objectives of this study was to evaluate modifications to the culvert and/or the upstream tributary drainage area to reduce the flows to the Cherokee Heights ravine. At the onset of the study, stormwater runoff volume techniques such as small scale or regional infiltration practices were considered for the upstream watershed. Construction of a permanent stormwater detention pond in the Cherokee Heights Park to reduce the peak flows to the Cherokee Heights ravine was also considered. However, results of the geotechnical analysis indicate that a pond upstream of the Cherokee Heights culvert that retains water for a significant time may potentially allow enough stormwater to infiltrate, reducing the stability of the bluff slope to below a safety factor of 1.0 (unstable conditions). Given these results, increased upstream infiltration and/or construction of a stormwater detention pond to reduce peak flows to the Cherokee Heights ravine were not considered further.

An alternate option for reducing peak flows is flow restriction and temporary, short-duration ponding/storage of runoff. The geotechnical analysis found that if the upstream pond functions like a temporary/dry pond, draining rapidly through the culvert following storm events and allowing only minimal infiltration (current condition), the stability of the bluff slope is reduced as compared with no infiltration from ponding, but the factor of safety remains above a factor of safety of 1.0. The concepts for restriction of peak flows and temporary stormwater storage described below were evaluated and discussed with stakeholders.

### 4.2.1 Culvert Modifications and Upstream Ravine Expansion

Under existing conditions, the 60-inch Cherokee Heights culvert conveys flows from most rainfalls without significant flow restriction and resulting water level bounce in the low area/ravine directly upstream of the culvert. Peak flows from the 1- and 2-year frequency, 24-hour events pass without restriction and temporary ponding. In larger, less frequent rainfall events, flows are restricted by the existing Cherokee Heights culvert and water levels in the upstream low area/ravine bounce. Table 4-1 shows the peak flows through the Cherokee Heights culvert, and approximate high water elevations and bounce in the low area/ravine directly upstream of the culvert.

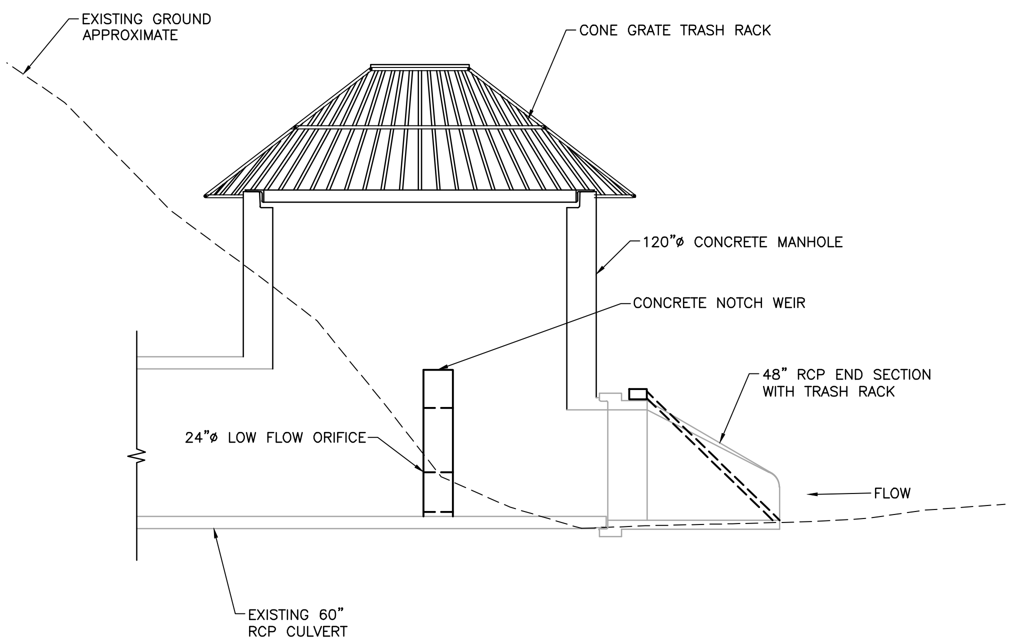
**Table 4-1 Peak flow rates through the Cherokee Heights culvert and maximum high water elevations in upstream ravine under existing conditions**

| Rainfall Recurrence Interval | Peak Flow Rates (cfs) | Approximate High Water Elevation (feet MSL) | Approximate High water Bounce[1] (feet) |
|---|---|---|---|
| 1 year | 54 | 939.3 | 1.0 |
| 2 year | 70 | 939.4 | 1.1 |
| 5 year | 109 | 939.8 | 1.5 |
| 10 year | 116 | 942.9 | 4.6 |
| 50 year | 252 | 948.8 | 10.5 |
| 100 year | 295 | 952.2 | 13.9 |

1 Based on culvert invert of 938.3 feet MSL

To reduce the peak flows from the Cherokee Heights culvert, installation of a tiered outlet structure at the upstream end of the existing culvert and expansion of the upstream storage volume was evaluated. A tiered outlet can be designed to reduce peak flows from a range of recurrence intervals. The tiered outlet configuration shown in Figure 4-3 is designed to pass low flows through an orifice at the culvert invert elevation. With the low-flow restriction, water levels in the upstream area increase until reaching the secondary overflow elevation, shown as a weir within the outlet structure (with or without a notch) in Figure 4-3. In tiered outlet designs, the secondary overflow is typically designed to be at or above the 1-year or 2-year high water elevation. The 48-inch inlet pipe shown in Figure 4-3 restricts flows during larger storm events, with the overflow grate providing an emergency overflow for storms that exceed a 100-year frequency event.

The XP-SWMM model developed for the study area was used to design and evaluate a tiered outlet for restricting flows to the Cherokee Heights ravine. Modeling results indicate that with the existing storage in the low area/ravine upstream of the culvert, outlet restrictions that achieve considerable reductions in peak discharge result in high water elevations far exceeding those of existing conditions. For example, achieving a 50% reduction in peak flow for a 1-year, 24-hour rainfall results in a bounce of approximately 13 feet. The increased frequency of significant bounce in the low area/ravine directly upstream of the Cherokee Heights culvert presents a safety concern for park users, especially considering the steep ravine topography. Therefore, we recommend that outlet modifications to reduce peak flows be accompanied by an increase in temporary storage volume.

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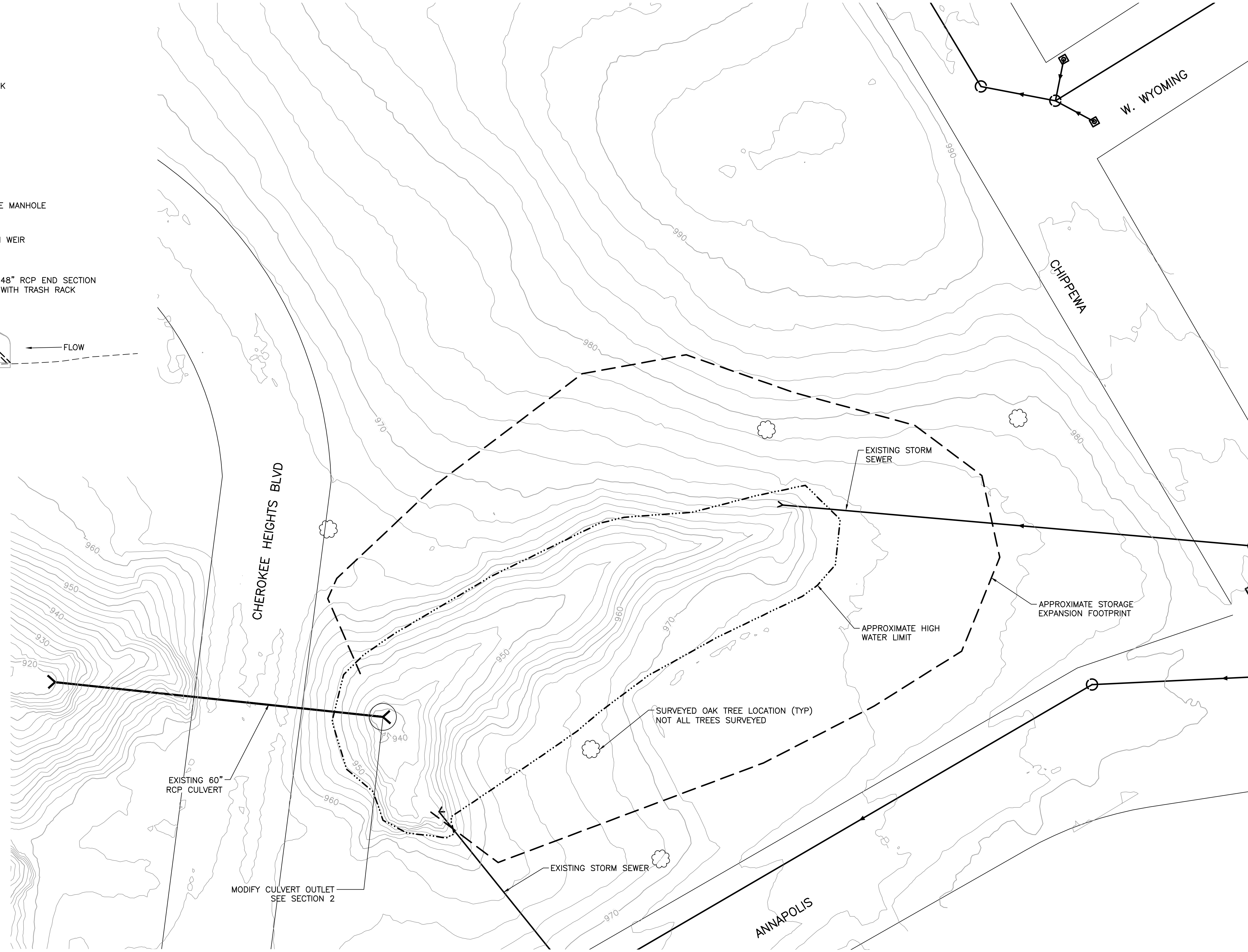


1 PLAN: CULVERT MODIFICATION
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FIGURE 4–3
UPSTREAM MODIFICATIONS

CHEROKEE HEIGHTS CULVERT ANALYSIS AND
EROSION CONTROL FEASIBILITY STUDY

CADD USER: Matt R. Podget FILE: M:\DESIGN\23191233.00\FIGURE_4-3_UPSTREAM MODIFICATIONS.DWG PLOT SCALE: 1:2 PLOT DATE: 4/7/2015 12:13 PM
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mrp M:\Design\23191233.00\Figure_4-3_Upstream Modifications.dwg Plot at 0 04/07/2015 12:12:12

For the conceptual design presented in this report, the outlet and additional storage were designed to reduce peak flows for a range of recurrence intervals, while minimizing increases in bounce as compared to existing conditions. For the 100-year, 24 hour event the high water elevation is increased by 0.6 feet under the expanded storage and modified outlet scenario, based on the assumption of 3.9 additional acre-feet of storage, with 1.3 acre-feet below the 2-year frequency high water elevation (943.3 feet MSL). Table 4-2 summarizes the peak flows and high water elevations based on the outlet shown in Figure 4-3 and the additional 3.9 acre-feet of storage. Figure 4-4 compares the 100-year, 24-hour high water elevations within the upstream low area/ravine under existing conditions and the expanded storage/modified outlet conceptual design. As shown in Figure 4-4, the duration of temporary water storage increases by 2 hours under the expanded storage/modified outlet scenario, as compared with existing conditions.

**Table 4-2 Comparison of peak flow rates under existing and expanded storage/modified outlet conceptual design**

| Rainfall Recurrence Interval | Existing Conditions Peak Flow Rates (cfs) | Expanded Storage/ Modified Outlet Peak Flow Rates (cfs) | Percent Reduction in Peak Flow (compared with existing conditions) |
|---|---|---|---|
| 1 year | 54 | 23 | 57% |
| 2 year | 70 | 30 | 57% |
| 5 year | 109 | 52 | 52% |
| 10 year | 116 | 70 | 40% |
| 50 year | 252 | 162 | 36% |
| 100 year | 295 | 212 | 28% |

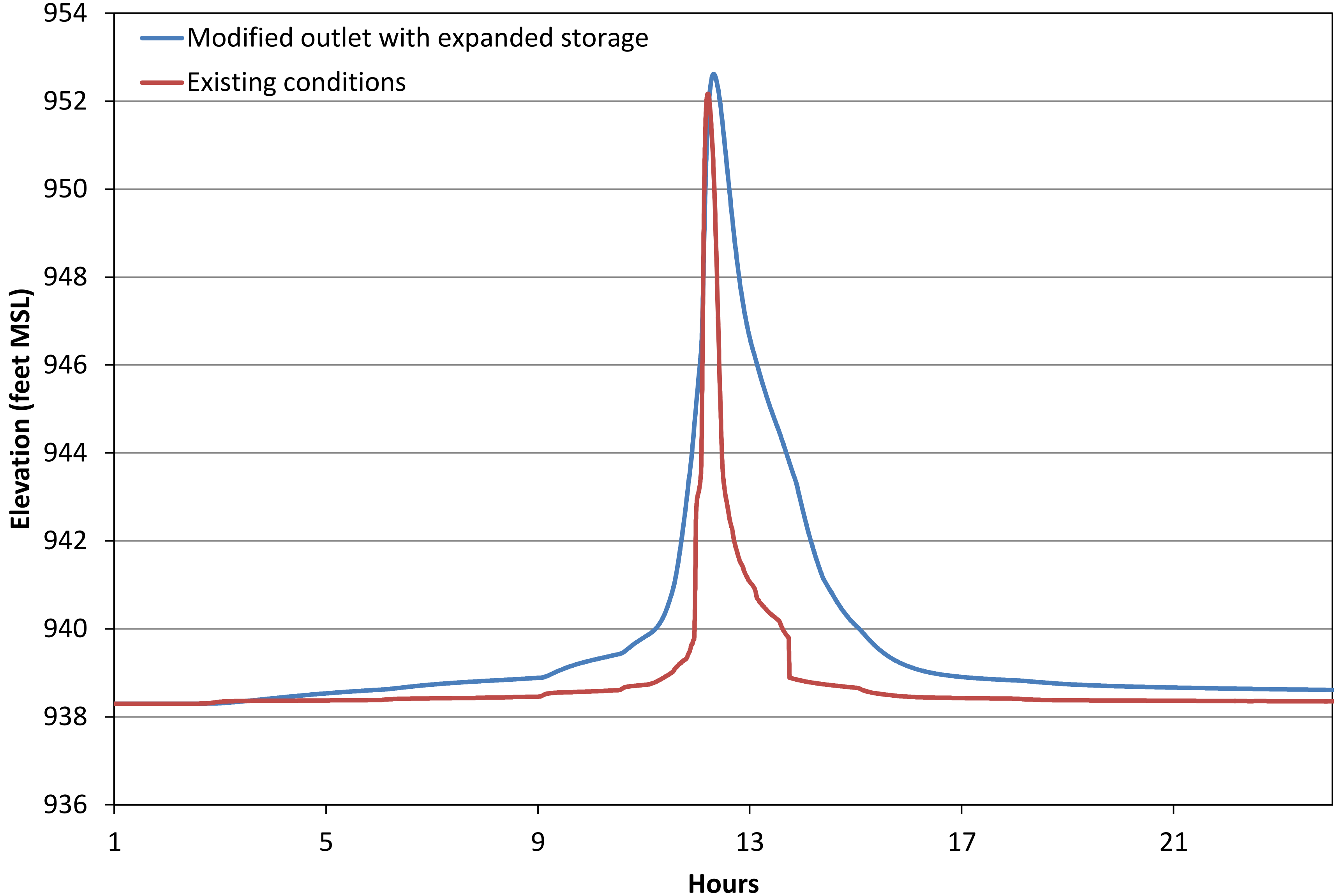


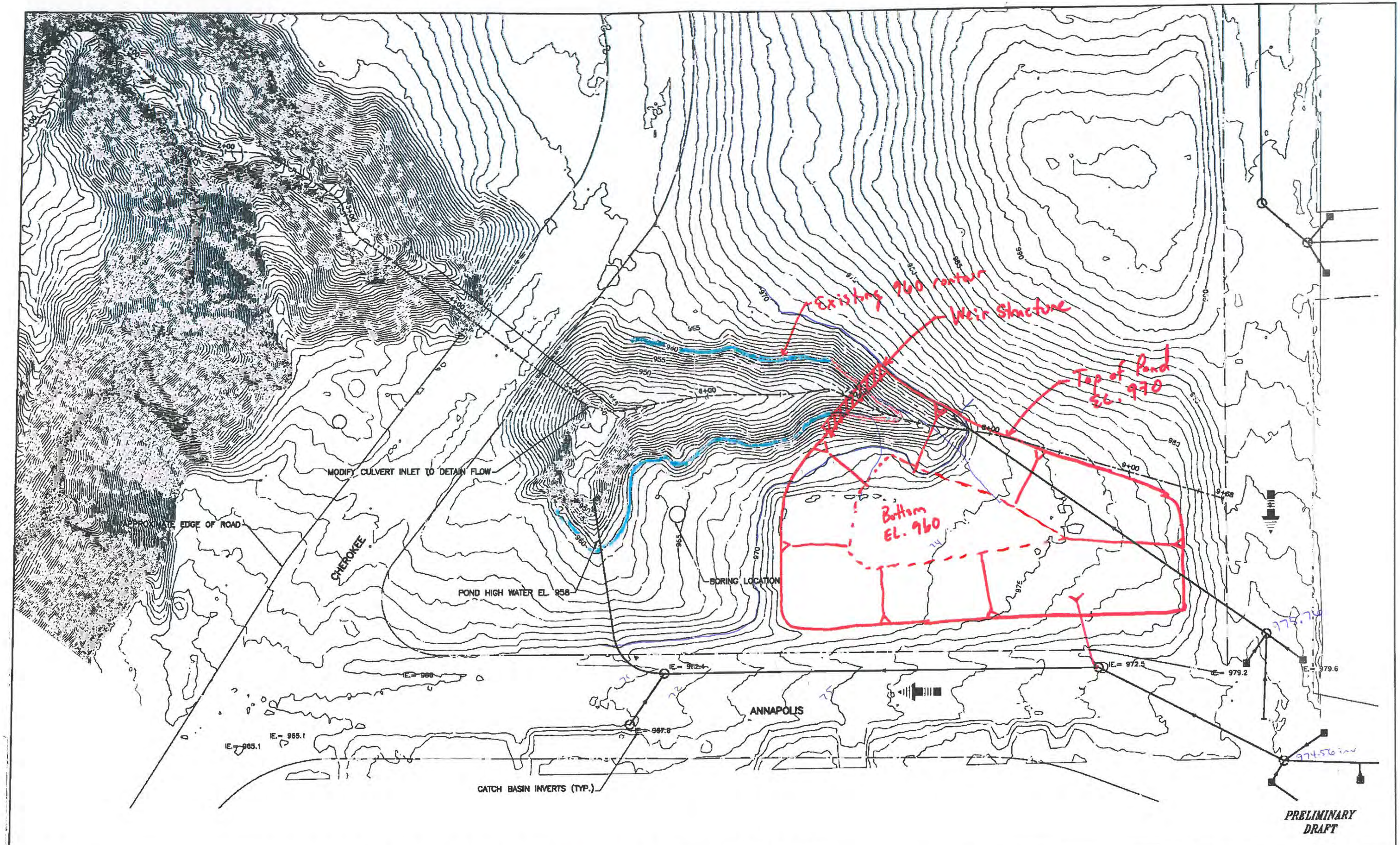

**Figure 4-4: 100-year, 24-hour flood elevations within the upstream low area/ravine under existing conditions and the expanded storage/modified outlet conceptual design**

### 4.2.2 Storage Expansion Alternatives

As mentioned above, a significant increase in storage (approximately 4 acre-feet) is necessary to reduce peak flows through the Cherokee Heights culvert while minimizing significant increases in bounce. To put the expanded storage volume into perspective, the existing low area/ravine upstream of the culvert has approximately one acre-foot of storage below the 100-year, 24-hour high water elevation (952.2 feet MSL). Due to the depth and steep side slopes of the low area/ravine, the addition of 4 acre-feet of storage below the 100-year high water elevation would require a significant amount of excavation within the ravine and regrading of the ravine side slopes. Figure 4-3 identifies an approximate storage expansion footprint, which extends well beyond the existing ravine. Although the expansion footprint shown in the figure is only approximate and could be designed to minimize impacts, the significant expansion footprint would result in the loss of many trees within the ravine and upland park area and would change the aesthetic character of the park.

To avoid significantly changing the aesthetic character of the ravine excavation, an alternative option would be to provide temporary storage further upstream. Figure 4-5 shows a red-lined sketch of the alternative option, which includes building an embankment on the upper end of the ravine to hold back stormwater and expanding a large temporary storage area into the flatter portion of the park just north of Annapolis Street and west of Chippewa Avenue. While this alternative would maintain the aesthetic characteristics of the existing ravine feature, it would significantly alter the look and usability of the flatter park area and also result in considerable tree loss.

To avoid significant changes in park aesthetics and usability, another option for expanded temporary storage is underground storage. However, due to the large desired storage volume, this alternative is likely cost prohibitive. This option would also result in considerable tree loss.

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I HEREBY CERTIFY THAT THIS PLAN, SPECIFICATION, OR REPORT WAS PREPARED BY ME OR UNDER MY DIRECT SUPERVISION AND THAT I AM A DULY LICENSED PROFESSIONAL ENGINEER UNDER THE LAWS OF THE STATE OF MINNESOTA.

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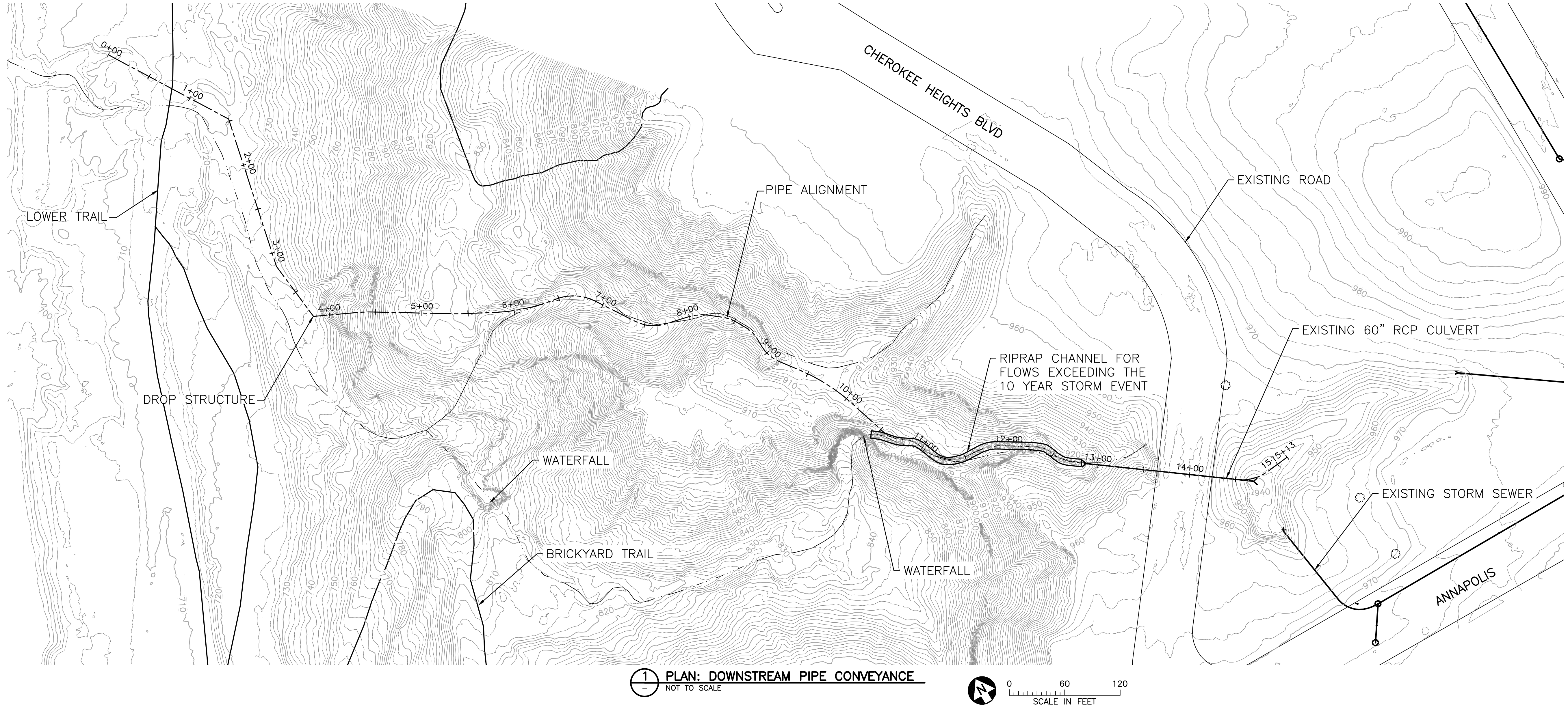
## 4.3 Downstream Piped Conveyance

The third alternative evaluated included installation of a pipe system from the Cherokee Height culvert outlet to the bottom of the bluff near Pickerel Lake. The pipe system would convey runoff from the 47-acre upstream drainage area and reduce erosion in the Cherokee Heights ravine. Figure 4-6 shows a potential pipe alignment, which follows the Cherokee Heights ravine until just upstream of the East Clay Pit Falls, then bends toward the ravine to the north. The primary advantage of a piped system is reduced flows and velocities in the Cherokee Heights ravine channel, and therefore reduced erosion. The primary disadvantages include high construction costs (see Section 5.0), costly maintenance and future system repairs/replacement, and construction-related impacts to the pipe corridor. This alternative is also the most un-natural, conveying runoff through underground infrastructure in lieu of day-lighted streams and water falls.

If the piped conveyance down the bluff was implemented, some level of channel stabilization would still be necessary in the Cherokee Heights ravine. The conceptual design and opinion of cost prepared for this alternative assume that the pipe system is sized to convey runoff from a 10-year frequency event and the downstream channel is stabilized for flows exceeding the 10-year rainfall event (up to 100-year level of protection).




1 PLAN: DOWNSTREAM PIPE CONVEYANCE
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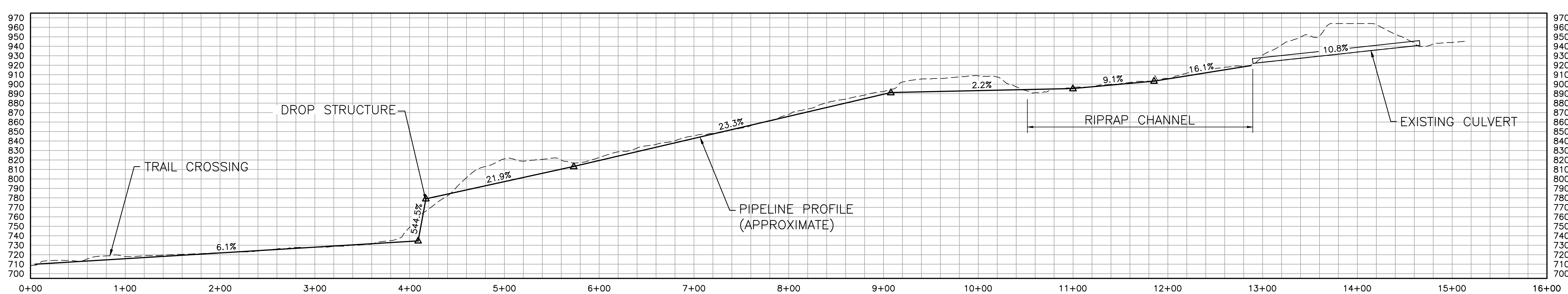


2 PROFILE: PIPE CONVEYANCE
NOT TO SCALE

FIGURE 4–6
DOWNSTREAM PIPED CONVEYANCE

CHEROKEE HEIGHTS CULVERT ANALYSIS AND
EROSION CONTROL FEASIBILITY STUDY

CADD USER: Matt R. Padget FILE: M:\DESIGN\23191233.00\FIGURE_4-6_DOWNSTREAM PIPE CONVEYANCE.DWG PLOT SCALE: 1:2 PLOT DATE: 4/8/2015 10:53 AM
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mrp M:\Design\23191233.00\Figure_4-6_Downstream Pipe Conveyance.dwg Plot at 0 04/08/2015 10:51:50

# 5.0 Planning-Level Opinion of Construction Costs

Planning-level opinions of construction cost have been developed for the improvement alternatives discussed in Section 4.0. The estimated costs are summarized in Table 5-1. More detailed cost estimates for the three improvement alternatives are provided in Appendix B.

The planning-level opinions of construction cost are intended to provide assistance in evaluating and comparing alternatives and should not be assumed as absolute values for given alternatives. These opinions of probable cost generally correspond to standards established by the Association for the Advancement of Cost Engineering (AACE). This cost estimate is characterized by limited project definition, wide-scale use of parametric models to calculate estimated costs (i.e., making extensive use of order-of-magnitude costs from similar projects or proposals), and uncertainty. The expected accuracy range for these point estimates is -30 percent to +50 percent. All estimated construction costs are presented in 2015 U.S. dollars and include costs for engineering and project administration.

**Table 5-1 Planning-level opinions of construction costs for improvement alternatives**

| Improvement Alternative | Estimated Cost | Estimated Cost Range (-30%/+50%) |
|---|---|---|
| Downstream Channel Stabilization | $400,000 | $280,000 - $600,000 |
| Upstream Culvert Modifications and Ravine Expansion | $350,000 | $250,000 - $530,000 |
| Downstream Piped Conveyance | $2,100,000 | $1,500,000 - $3,200,000 |

# 6.0 Cost Allocation

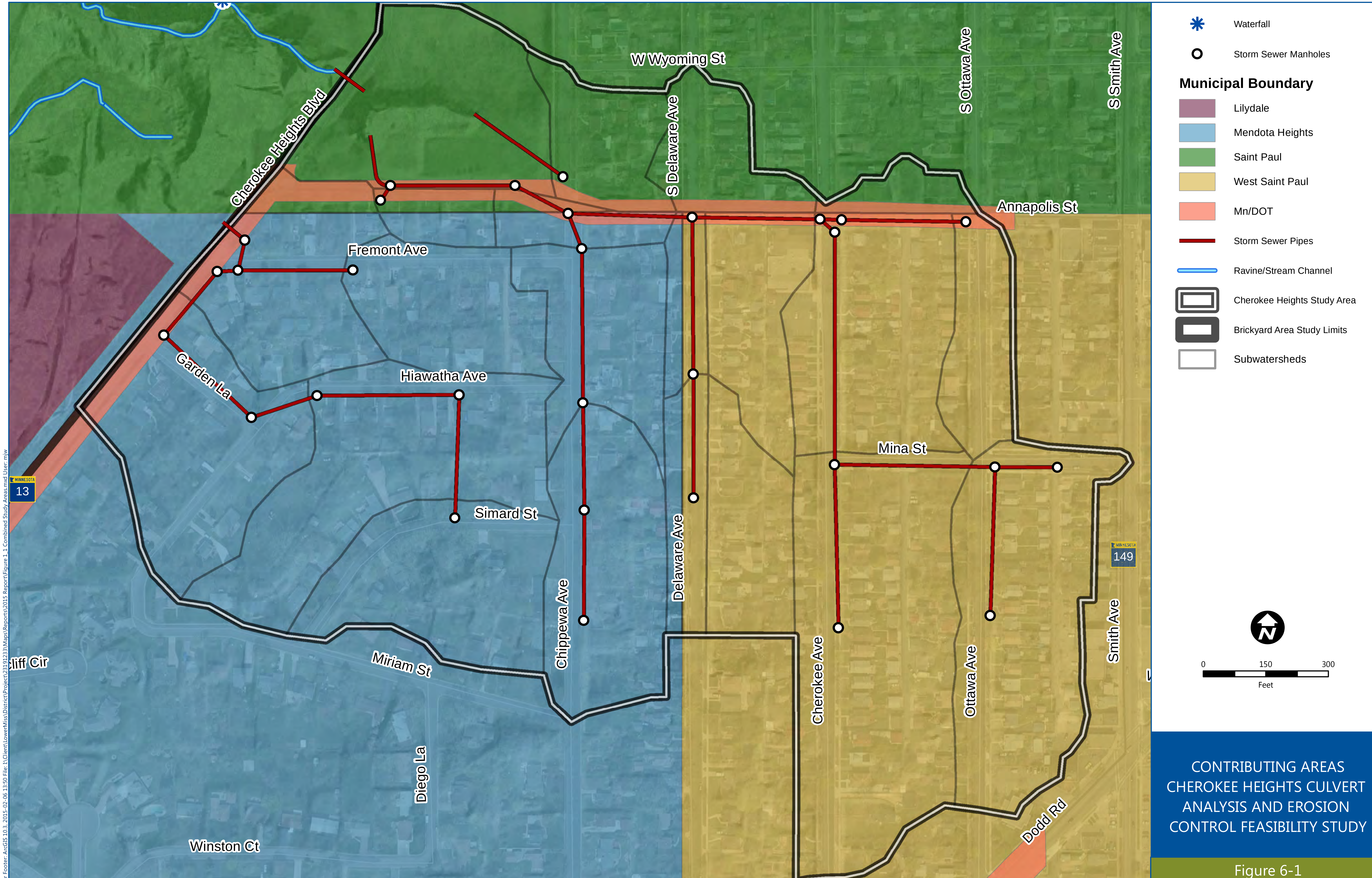
## 6.1 Contributing Drainage Areas

The drainage area directly tributary to the Cherokee Heights culvert is 47.3 acres and includes portions of Mendota Heights, West St. Paul, and St. Paul. Figure 6-1 shows the contributing areas from each municipality, as well as the portion of drainage area within Mn/DOT jurisdiction, based on Municipal Separate Storm Sewer System (MS4) boundaries provided to the LMRWMO by the Minnesota Pollution Control Agency (MPCA) in 2012. Two acres of the drainage area, Annapolis Street and State Hwy 13, falls under Mn/DOT jurisdiction. Table 6-1 summarizes the tributary drainage areas to the Cherokee Heights culvert, as well as the portion of each city that is Mn/DOT right-of-way (ROW). Of the 9.4 acres of contributing land from the City of St. Paul, 4.4 acres is parkland.

**Table 6-1 Watershed areas tributary to Cherokee Heights culvert**

| City | Non-Mn/DOT ROW (acres) | Mn/DOT ROW (acres) | Total Tributary Area (acres) | % of Total Tributary Area |
|---|---|---|---|---|
| Mendota Heights[1] | 9.4 | 0.6 | 10.0 | 21% |
| St. Paul[2] | 8.5 | 0.9 | 9.4 | 20% |
| West St. Paul | 27.3a | 0.5 | 27.8 | 59% |
| Total | 45.2 | 2.0 | 47.2 | 100% |

[1] Surface overflows from the Fremont Avenue culvert drain to the Cherokee Heights culvert. The tributary drainage area to the Fremont Avenue culvert are not included in this table.
[2] 4.4 acres attributed to St. Paul Park and Recreation Department




CONTRIBUTING AREAS
CHEROKEE HEIGHTS CULVERT
ANALYSIS AND EROSION
CONTROL FEASIBILITY STUDY

Figure 6-1

## 6.2 Allowable Flow Cost Apportionment

Allowable flow is the cost apportionment method defined in the LMRWMO Joint Powers Agreement (JPA) as a means of determining cost sharing among municipalities when stormwater management projects include drainage from more than one municipality. The allowable flow method is defined in the current LMRWMO JPA and has been further clarified in several technical memos throughout the past several decades. This method is based on the basic principles that (1) upstream communities have the right to discharge some flow (termed allowable flow) downstream without financial obligation; and (2) upstream communities should share in downstream costs of handling flow in excess of their allowable flows.

Per the LMRWMO JPA, allowable flow is a set of conditions defined for the purposes of cost apportionment for intercommunity drainage projects. The allowable flow is the amount of flow that a municipality can discharge without financial obligation and is calculated assuming "natural" land use conditions and topography that existed on the JPA enactment date (1985). The XP-SWMM model was used to compute allowable flows for each upstream municipality (Mendota Heights and West St. Paul), based on assumptions of no impervious surfaces within the watershed ("natural" land use), but existing topography and storm sewer infrastructure. Table 6-2 summarizes the peak flows through the Cherokee Heights Culvert based on the allowable flow assumption of "natural" conditions. As shown in the table, there is a greater percent difference in peak flows for the more frequent storms, but as the storm recurrence interval approaches the 100-year frequency, the difference between existing and "natural land use conditions" flows diminishes.

**Table 6-2 Comparison of peak flow rates through the Cherokee Heights culvert under existing and "natural land use" conditions**

| Atlas 14 24-Hour Storm Event | Existing Conditions Peak Flow Rates (cfs) | "Natural Land Use" Conditions Peak Flow Rates (cfs) | % Difference |
|---|---|---|---|
| 1 year | 54 | 14 | 74% |
| 2 year | 70 | 26 | 63% |
| 5 year | 109 | 52 | 52% |
| 10 year | 116 | 79 | 32% |
| 50 year | 252 | 176 | 30% |
| 100 year | 295 | 240 | 19% |

In the LMRWMO's allowable flow cost apportionment method, the excess flow (or volume) is the portion of a city's discharge that they are financially responsible. The excess flow is calculated as the difference between an upstream municipality's design flow and allowable flow. Similarly, the excess volume is calculated as the difference between an upstream municipality's design volume and allowable volume.

The following sections summarize the cost apportionment computations for each of the evaluated improvement alternatives based on the LMRWMO's allowable flow methodology.

### 6.2.1 Downstream Channel Stabilization

For an open channel conveyance system, the excess flow from an upstream city is the 100-year design flow less the 100-year allowable flow. However, for cost apportionment downstream of ponds, excess flows are adjusted to reflect the effects of ponding using the following equation:

$$Qexcess(outlet) = Qexcess(inlet) \times \frac{Qtotal(outlet)}{\Sigma Qtotal(inlet)}$$

Because the Cherokee Heights culvert restricts flows during the 100-year frequency event, acting as a temporary pond, this equation was used to calculate excess flows from the Cherokee Heights culvert. Table 6-3 summarizes the 100-year frequency allowable and design flows to the Cherokee Heights culvert, as well as the excess flows to the culvert, Qexcess(inlet), and excess flows from the culvert, Qexcess(outlet). Based on the excess flows from West St. Paul and Mendota Heights, the cost shares for the downstream channel stabilization are 19% and 12%, respectively.

**Table 6-3 Cost apportionment summary for downstream channel stabilization**

| Upstream City | Allowable Flow (cfs) | Design Flow to Cherokee Heights Culvert (cfs) | Excess Flow to Cherokee Heights Culvert (cfs) | Excess Flow from Cherokee Heights Culvert (cfs) | Cost Share (Excess Flow / Total Flow) |
|---|---|---|---|---|---|
| West St. Paul | 149 | 226 | 77 | 56 | 19% |
| Mendota Heights[1] | 68 | 117 | 49 | 36 | 12% |
| St. Paul | - | - | - | - | 69% |
| [1] Allowable and design flows include surface overflow from the Fremont Avenue culvert | | | | | |

### 6.2.2 Ravine Expansion and Upstream Culvert Modifications

This improvement alternative includes expansion of the flood storage upstream of the Cherokee Heights culvert and modifications to the low area/ravine outlet. Based on the LMRWMO allowable flow methodology, the cost apportionment for these two project components is calculated separately. The computations are summarized below.

#### 6.2.2.1 Ravine Storage Expansion

For pond (or flood storage) expansions, cost allocation is determined based on excess runoff volume, versus excess flow. For drainage areas without upstream ponding, the excess volume is calculated as the difference between design and allowable volume. The allowable volume from an upstream city is the total runoff volume from the design storm, using "natural" land use. The 100-year return frequency event was used, as the 100-year event was used as the design storm for the improvement alternatives.

Table 6-4 summarizes the 100-year frequency allowable and design (actual) runoff volumes to the Cherokee Heights culvert, as well as the excess volumes and cost shares. Based on the excess volumes from West St. Paul and Mendota Heights and a total proposed storage increase of 3.9 acre-feet, the cost shares for the ravine storage expansion are 66% and 22%, respectively.

**Table 6-4 Cost apportionment for ravine storage expansion**

| Upstream City | Allowable Volume (acre-feet) | Design Volume (acre-feet) | Excess Volume (acre-feet) | Cost Share (Excess Volume / Total Volume) |
|---|---|---|---|---|
| West St. Paul | 10.3 | 12.8 | 2.6 | 66% |
| Mendota Heights[1] | 3.9 | 4.7 | 0.9 | 22% |
| St. Paul | | | | 12% |
| 1 Allowable and design volumes include surface overflow from the Fremont Avenue culvert | | | | |

#### 6.2.2.2 Upstream Culvert Modifications

For modifications to the Cherokee Heights culvert, the excess flows from upstream cities are calculated using the same method as for the downstream channel stabilization discussed in Section 6.2.1, with excess flows adjusted to reflect the effects of ponding.

Table 6-5 summarizes the 100-year frequency allowable and design flows to the Cherokee Heights culvert, as well as the excess flows to the culvert, Qexcess(inlet), and excess flows from the culvert, Qexcess(outlet). Based on the excess flows from West St. Paul and Mendota Heights, the cost shares for the downstream channel stabilization are 19% and 12%, respectively.

**Table 6-5 Cost apportionment summary for upstream culvert modifications**

| Upstream City | Allowable Flow (cfs) | Design Flow to Cherokee Heights Culvert (cfs) | Excess Flow to Cherokee Heights Culvert (cfs) | Excess Flow from Cherokee Heights Culvert (cfs) | Cost Share (Excess Flow / Total Flow) |
|---|---|---|---|---|---|
| West St. Paul | 149 | 226 | 77 | 40 | 19% |
| Mendota Heights[1] | 68 | 117 | 49 | 26 | 12% |
| St. Paul | - | - | - | - | 69% |
| [1] Allowable and design flows include surface overflow from the Fremont Avenue culvert | | | | | |

### 6.2.3 Downstream Piped Conveyance

For the downstream piped conveyance, the excess flows from upstream cities are calculated using the same method as for the downstream channel stabilization discussed in Section 6.2.1, with excess flows adjusted to reflect the effects of ponding.

Table 6-6 summarizes the 100-year frequency allowable and design flows to the Cherokee Heights culvert, as well as the excess flows to the culvert, Qexcess(inlet), and excess flows from the culvert, Qexcess(outlet). Based on the excess flows from West St. Paul and Mendota Heights, the cost shares for the downstream channel stabilization are 19% and 12%, respectively.

**Table 6-6 Cost apportionment summary for downstream piped conveyance**

| Upstream City | Allowable Flow (cfs) | Design Flow to Cherokee Heights Culvert (cfs) | Excess Flow to Cherokee Heights Culvert (cfs) | Excess Flow from Cherokee Heights Culvert (cfs) | Cost Share (Excess Flow / Total Flow) |
|---|---|---|---|---|---|
| West St. Paul | 149 | 226 | 77 | 56 | 19% |
| Mendota Heights[1] | 68 | 117 | 49 | 36 | 12% |
| St. Paul | - | - | - | - | 69% |
| [1] Allowable and design flows include surface overflow from the Fremont Avenue culvert | | | | | |

## 6.3 Cost Allocation Summary

Table 6-7 summarizes the apportionment of costs for the evaluated improvement options based on the LMRWMO allowable flow methodology.

**Table 6-7 Summary of LMRWMO Allowable Flow cost apportionment**

| City | % Cost Share | | | |
|---|---|---|---|---|
| | Downstream Channel Stabilization | Upstream Ravine Expansion | Upstream Culvert Modifications | Downstream Piped Conveyance |
| West St. Paul | 19% | 66% | 19% | 19% |
| Mendota Heights | 12% | 22% | 12% | 12% |
| St. Paul | 69% | 12% | 69% | 69% |

# 7.0 Water Quality Considerations

High flow rates and velocities through the ravine downstream of the Cherokee Heights culvert, in combination with sandy, erodible soils, have caused erosion that is contributing to the instability of the adjacent banks and delivering sediment to downstream Pickerel Lake, thereby degrading its water quality. Pickerel Lake is a shallow, 115-acre lake located within the Lilydale Regional Park. The 1,320-acre watershed to Pickerel Lake includes portions of the municipalities of St. Paul, Lilydale, Mendota Heights, and West St. Paul and is comprised of primarily residential land use, in addition to the park and recreational space surrounding the lake.

Pickerel Lake is located in the Mississippi River floodplain. When river levels are high enough, the Mississippi River completely inundates or backs up into Pickerel Lake, which can greatly hinder the water quality of the lake. In addition to impacts from the Mississippi River, the lake also receives significant sediment and phosphorus loading from erosion of the steep bluffs and ravines that discharge to the lake. A recent study conducted by the LMRWMO and the MPCA identified ravine/bluff stabilization as a priority for protecting and improving the water quality of Pickerel Lake (MPCA, 2015).

The Revised Universal Soil Loss Equation (RUSLE) was used to estimate soil loss from the side slopes of the Cherokee Heights ravine directly downstream of the Cherokee Heights culvert. An excel version of the RUSLE created for the state of Minnesota was obtained from the Minnesota Board of Water and Soil Resources (BWSR) web site and used to estimate sediment loads under existing slope conditions. Assuming a slope grade of 50% and slope length of 80 feet, the estimated annual sediment load from side slope erosion is 13 tons/acre/year. With a direct tributary area of 1.3 acres to the Cherokee Heights ravine, the resulting estimated load from the channel is 16 tons/year. With implementation of the channel stabilization recommended as part of this study, it is assumed that sediment loads from the side slopes of the channel will be eliminated resulting in the annual sediment reduction downstream of 16 tons/year. Additional erosion benefits and sediment load reductions from reduced channel erosion (versus side slope erosion) will also be achieved.

The BWSR Water Erosion Pollution Reduction Estimator, Stream & Ditch calculator was used to estimate annual phosphorus load reduction based on the predicted sediment load reduction  Assuming silty soils throughout the Cherokee Heights ravine, the annual phosphorus load reduction is estimated to be 16 pounds per year.

# 8.0 Conclusions

The objective of this study was to evaluate options to reduce erosion in the approximately 300 feet of channel between the Cherokee Heights culvert and East Clay Pit Falls by stabilizing the channel and reducing peak flow rates and velocities, as feasible. The improvement alternatives evaluated as part of this study are described below.

## 8.1 Downstream Channel Stabilization

Observation of the Cherokee Heights ravine channel revealed significant erosion along the channel bottom and side slopes, including undercutting of the toe of the slope, which contributes to instability of the ravine side slopes. To minimize erosion of the channel and side slopes and reduce the instability of adjacent banks within the ravine, we recommend regrading and stabilizing the channel by armoring the channel with rip-rap and a properly graded filter material to prevent migration of underlying fine-grained soils through the rip-rap. Throughout portions of the ravine, we recommend that the channel be raised and the side slopes be graded to a more stable slope. Raising portions of the channel will reduce flow velocities in some sections of the reach and provide increased buttressing of the channel side slopes. Filter material and rip-rap should extend up along the face of the slope, to protect the toe of slope from undercutting erosion and to buttress the slope for increased long-term stability. This provides weight along the face of the slope to physically restrict further movement of the ravine side slopes, while also providing erosion control.

High flow velocities in the Cherokee Heights ravine channel preclude use of many bio-engineering techniques for stabilization, as these techniques typically would not withstand the magnitude of flow velocities. Vegetation management should be implemented with the channel armoring and slope grading. Selective removal of less-desirable mature trees and buckthorn would improve the light penetration to the forest floor, thereby promoting ground vegetation. Selective planting of desirable ground vegetation (and protection of the plantings) would promote long-term erosion protection of the ravine.

## 8.2 Other Improvement Alternatives Considered

### 8.2.1 Upstream Modifications

Options for reducing peak flows through the Cherokee Heights culvert and ravine were evaluated. Typical methods to reduce runoff rate and volume include increased stormwater runoff infiltration or stormwater detention ponds. However, results of the geotechnical analysis indicate that increased ponding that promotes infiltration in the area upstream of the Cherokee Heights ravine can reduce the stability of downstream slopes. Therefore, increased upstream infiltration and/or construction of a stormwater detention pond to reduce peak flows to the Cherokee Heights ravine were not considered further.

Although permanent ponding was eliminated as a potential flow reduction technique, installation of a tiered outlet upstream of the existing culvert and temporary, short duration ponding/storage of runoff was evaluated as an option. The geotechnical analysis found that if the upstream pond functions like a temporary/dry pond, draining rapidly through the culvert following storm events and allowing only minimal infiltration (similar to current condition), the stability of the bluff slope is reduced as compared with no infiltration from ponding, but the factor of safety remains above a factor of safety of 1.0.

A tiered outlet was designed to reduce peak flows from a range of storm frequencies (1-year to 100-year), along with an additional storage volume of approximately 4 acre-feet below the 100-year high water elevation to minimize increases in bounce within the ravine. Without additional storage volume, flow restrictions result in more frequent and severe bounce in the low area/ravine directly upstream of the culvert, which could present a safety concern for park users given the steep ravine topography.

Due to the depth and steep side slopes of the low area/ravine, the addition of 4 acre-feet of storage below the 100-year high water elevation would require a significant amount of excavation within the ravine and regrading of the ravine side slopes, which would extend well into the nearby parkland. Although the regrading could be designed to minimize impacts to the extent possible, the large expansion footprint would result in the loss of many trees within the ravine and upland park area and would significantly change the aesthetic character of the park. To avoid significantly changing the aesthetic character of the ravine excavation, an alternative option would be to provide temporary storage further upstream in the flatter area of the park. While this alternative would maintain the aesthetic characteristics of the existing ravine feature, it would significantly alter the look and usability of the flatter park area and also result in considerable tree loss. Another option for expanded temporary storage is underground storage. However, due to the large desired storage volume, this alternative is likely cost prohibitive. This option would also result in considerable tree loss.

It is important to note that while upstream outlet modifications and storage expansion would be effective in reducing peak flows for most storm events, the 100-year peak flow is reduced only by 30% to 212 cfs. So, the design and associated costs of the downstream channel stabilization would not be significantly reduced by implementing upstream modifications.

### 8.2.2 Downstream Piped Conveyance

The third alternative evaluated to help stabilize the channel downstream of the Cherokee Heights culvert was to install an underground pipe system down the entire bluff to convey runoff from the Cherokee Heights culvert (including the 47-acre upstream drainage area) to Pickerel Lake. While this pipe would effectively reduce flows and velocities in the Cherokee Heights ravine and the channel downstream of the East Clay Pit Falls, the project has a high construction cost, significant construction-related impacts, and some level of channel stabilization would still be necessary in the Cherokee Heights ravine for flows that exceed the capacity of the underground pipe.

## 8.3 Other Considerations

While the scope of this study is to address erosion in the approximately 300 feet of channel between the Cherokee Heights culvert and East Clay Pit Falls, additional erosion control measures may be necessary in the future in the channel downstream of the East Clay Pit Falls. Specific recommendations for that portion of the Cherokee Heights ravine would be included in the Brickyard Study. Following implementation of channel stabilization in the upper 300 feet of the ravine, visual monitoring for erosion issues throughout the entire length of channel should be continued on a regular basis.

The improvement alternatives evaluated as part of this study are intended to stabilize and protect the channel downstream of the Cherokee Heights culvert, to reduce erosion and reduce the risk of localized slope failure. It is important to distinguish between these localized slope-stability issues in the Cherokee Heights ravine and other ravines that are exacerbated by erosion from stormwater and the more wide-spread slope stability issues within the Brickyard Area. It is our opinion that even if erosion due to stormwater runoff is addressed in this stretch of the Cherokee Heights ravine, the potential for slope failure still exists. Additional information on slope failure risk throughout the Brickyard Area can be found in the Brickyard Study.

# 9.0 References


1. **The Association for the Advancement of Cost Engineering.** *AACE International Recommended Practice No. 18R-97: Cost Estimating Classification System as Applied in Engineering, Procurement, and Construction for the Process Industries.* 2005.

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5. **American Engineering Testing, Inc.** *Report of Geotechnical Exploration and Review, Cherokee Regional Trail Widening along Cherokee Heights Boulevard.* St. Paul : American Engineering Testing, 2011.

6. **American Engineering Testing.** *Report of Geotechnical Exploration and Review, Cherokee Regional Trail Widening.* St. Paul : American Engineering Testing, 2010.

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# Appendices

# Appendix A

## Stormwater Modeling Methodology

# Appendix A

## *Stormwater Modeling Methodology*

Prepared for
Lower Mississippi River Watershed Management Organization

April 8, 2015

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# Appendix A

# Stormwater Modeling Methodology

# April 8, 2015

## Contents



## List of Tables

# 1.0 Hydrologic and Hydraulic Modeling Methodology

The U.S. EPA's Storm Water Management Model (SWMM), with a computerized graphical interface provided by XP Software (XP-SWMM), was chosen as the computer modeling package for this study. XP-SWMM uses rainfall and watershed characteristics to generate local runoff, which is routed simultaneously through complicated pipe and overland flow networks. The model can account for detention in ponding areas, backflow in pipes, surcharging of manholes, as well as tailwater conditions that may exist and affect upstream storage or pipe flows. XP-SWMM Version 2014, was used to model the storm sewer, ponding, channel flow and overland flow systems for the Cherokee Heights and the Brickyard Study Areas.

## 1.1 Hydrologic Modeling

Three major types of information are required by XP-SWMM for hydrologic modeling: (1) watershed characteristics, (2) rainfall data, and (3) infiltration characteristics. This data is used by XP-SWMM to generate inflow hydrographs at various points in the drainage network. The following sections describe each of these data sets.

### 1.1.1 Watershed Data

The amount of runoff from a watershed depends on numerous factors, including the total watershed area, the soil types within the watershed, the percent of impervious area, the runoff path through the watershed, and the slope of the land within the watershed. ArcGIS (geographic information systems) software was used extensively in assessing the above mentioned characteristics of each subwatershed within the study area.

#### 1.1.1.1 Watershed Area

The watershed delineation was performed using the Minnesota Department of Natural Resources' (MNDNR's) 2011 LiDAR elevation data set covering Dakota County along with the storm sewer system (manholes, catch basins, and pipes) layout and aerial imagery. A total of 55 subwatersheds were delineated for this area - 34 subwatersheds in the Brickyard Area of Lilydale Regional Park area and 21 subwatersheds contributing stormwater flow to the park via the upstream storm sewer system (including the Cherokee Heights study area). The delineated subwatersheds are shown in Figure 3-1 of the main report.

#### 1.1.1.2 Land Use Data

Land use data was obtained to estimate both the percentage of directly and indirectly connected imperviousness within each subwatershed. The directly-connected impervious fraction consists of the impervious surfaces that are "connected" directly to stormwater conveyance systems, meaning that flows do not cross over pervious areas. The indirectly connected impervious fraction represents impervious areas with runoff that flows over pervious areas before reaching the stormwater conveyance system (rooftops, for example). These fractions were calculated by first estimating the total impervious area for each subwatershed using the National Land Cover Dataset (NLCD) 2011 impervious layer (Xian et al, 2013). Indirectly connected impervious areas were estimated using roof delineations for the Twin Cities Metropolitan Area produced by the National Geodetic Survey (NGS) in 2008 using LiDAR elevation data. Total roof area coverage located in portions of the watershed with a land use classification consistent with having indirectly connected impervious surfaces (i.e. Park/Recreational/preserve, single family attached, single family detached, and undeveloped) were calculated for each subwatershed. Other impervious area types (roads, sidewalks, driveways and parking lots) were assumed to be directly connected to the storm sewer system. Directly connected impervious areas were calculated by subtracting the indirectly connected impervious areas from the total impervious area for each subwatershed. The impervious factions were determined by dividing each impervious value by the total subwatershed area for each of the subwatersheds in the model.

#### 1.1.1.3 Watershed Width and Slope

The SWMM Runoff Non-linear Reservoir Method was used as the hydrograph generation technique for this project. This method computes outflow as the product of velocity, depth and a watershed width factor. The watershed "width" in XP-SWMM is defined as the subwatershed area divided by the flow path length. This factor is a key parameter in determining the shape of the hydrograph for each subwatershed and is often used as a calibration parameter, when calibration data is available. The main flow path length was calculated in ArcGIS and was used in conjunction with the subwatershed area to calculate the width parameter.

The average slope (ft/ft) for each subwatershed was calculated in ArcGIS (standard ArcGIS Spatial Analyst raster tools) using the MNDNR 2011 LiDAR elevation data set.

#### 1.1.1.4 Rainfall Data

The XP-SWMM model was run for the 1-year, 2-year, 5-year, 10-year, 50-year, and 100-year recurrence, 24-hour precipitation events using the Atlas 14 precipitation frequency estimates. Point-based precipitation frequency estimates for the centroid of the study area were obtained from NOAA's National Weather Service Precipitation Frequency Data Server (PFDS) located at http://dipper.nws.noaa.gov/hdsc/pfds/.

Nested 24-hour rainfall distributions were created for each modeled storm event. Each rainfall distribution was a storm hyetograph derived from the precipitation frequency estimates. A "nested" hyetograph was built, which is a hypothetical precipitation distribution where the precipitation depths for various durations within the storm have identical exceedance probabilities. This distribution maximizes the rainfall intensities by incorporating selected short duration intensities within those needed for longer durations at the same probability level. As a result, the various storm durations are "nested" within a single hypothetical distribution.

### 1.1.1.5 Infiltration Data

#### *Soils*

Soils data for the area was obtained through 2014 Gridded Soil Survey Geographic Database for the state of Minnesota (USDA, 2014) which was imported into ArcGIS. The database included the soil names and the hydrologic soil group (HSG) designation for most of the soil types. The hydrologic soil group designation classifies soils into groups (A, B, C, and D) based on the infiltration capacity of the soil (well drained, sandy soils are classified as "A" soils; poorly drained, clayey soils are classified as "D" soils). When a HSG designation was not included in the soils database, the soil description was used to estimate the HSG. If a soil description was unavailable, the most dominant soil group in the vicinity was assumed.

#### *Horton Infiltration*

Infiltration was simulated in the XP-SWMM model using the Horton Infiltration equation. This equation is used to represent the exponential decay of infiltration capacity of the soil that occurs during heavy storm events. The soil infiltration capacity is a function of the following variables: $F_c$ (minimum or ultimate value of infiltration capacity), $F_o$ (maximum or initial value of infiltration capacity), k (decay coefficient), and time.

The actual values of $F_c$, $F_o$, and k are dependent upon soil, vegetation, and initial moisture conditions prior to a rainfall event. Because it was not feasible to obtain this detailed information for each subwatershed through field samples, it was necessary to make assumptions based on the various soil types throughout the study area. Table A 1-1 summarizes the Horton infiltration values used for each HSG to calculate composite infiltration parameters for each subwatershed. The values shown in the table are based on suggested values in the *Storm Water Management Model, Version 4: User's Manual* (U.S. EPA, 1988). Composite $F_c$ and $F_o$ values were calculated for each subwatershed based on the fraction of each soil type within the subwatershed. Global databases containing the infiltration parameters for each subwatershed were developed and imported into the XP-SWMM models.

**Table A-1-1 Horton Infiltration Parameters**

| Hydrologic Soil Group | $F_o$ (in/hr) | $F_c$ (in/hr) | k (1/sec) |
|---|---|---|---|
| A | 5 | 0.38 | 0.0008 |
| B | 3 | 0.23 | 0.0008 |
| C | 2 | 0.1 | 0.0008 |
| D | 1 | 0.03 | 0.0008 |

### 1.1.1.6 Depression Storage Data

Depression storage represents the volume (in inches) that must be filled with rainfall prior to the occurrence of runoff in XP-SWMM. It characterizes the loss or "initial abstraction" caused by such phenomena as surface ponding, surface wetting, interception and evaporation. Separate depression storage input values are required in XP-SWMM for pervious and impervious areas.

The depression storage assumptions used for the models were based on the values used in the XP-SWMM model developed for the *Nine Mile Creek Watershed District Bloomington Use Attainability Analysis* (Barr Engineering, 2001). For this reference model, the depression storage was estimated by plotting total precipitation for several measured rainfall events at a Bloomington continuous-recording-precipitation gage versus runoff from several Bloomington monitoring sites. A regression analysis of the data yielded a y-intercept that was assumed to be the depression storage (in inches). Based on this analysis, the assumed impervious depression storage was 0.06 inches and the pervious depression storage was 0.17 inches. These values are in line with the range of values recommended in literature.

## 1.2 Hydraulic Modeling

### 1.2.1 Storm Sewer Network

Data detailing the storm sewer network for the area was provided by the cities of St Paul, Mendota Heights, West St. Paul and the Minnesota Department of Transportation (Mn/DOT). The storm sewer data was provided in a GIS format, with the database file containing invert elevations, pipe sizes, pipe lengths, and manhole rim elevations. Where storm sewer information was missing in the GIS data set, "as-built" drawings containing the storm sewer information were provided by the cities. A Manning's roughness value of 0.013 was applied to each storm sewer pipe.

There are three culverts under Cherokee Heights Boulevard that serve as the three main stormwater discharge points into the Brickyard Area of Lilydale Regional Park. The location of the storm sewer pipes and the three culverts under Cherokee Heights Boulevard are shown in Figure 1-1 of the main report.

### 1.2.2 Storage Areas

Three storage areas were included in the model: one located at the upstream end of the 18-inch Freemont Avenue culvert under Cherokee Heights/Highway 13, one just upstream of the 60-inch culvert under Cherokee Heights Boulevard, and a depression area located to the south of Simard Street and north of Miriam Street. Storage curves describing the elevation/area relationship were developed in GIS for each of these storage areas using the 2011 MNDNR LiDAR elevation data set.

### 1.2.3 Overland Flow Network

Since there is no known storm sewer pipe system actively conveying water within the Brickyard Area of Lilydale Regional Park, runoff from the Brickyard Area downstream of Cherokee Heights Boulevard generally flows overland following the slope of the land. Runoff from the three main stormwater discharge points (described above) flows into overland channels through the Brickyard Area. The overland channels were modeled as natural channel cross-sections. Channel lengths, upstream and downstream channel elevations, and channel shape were determined using the 2011 MNDNR LiDAR elevation data set. A Manning's roughness value of 0.05 was applied to each of the natural channel cross-sections.

A street overland flow channel network was also added to the upstream portion of the study area served by storm sewer. All street sections are represented in the XP-SWMM model using a trapezoidal channel with a 30-foot bottom width, 1:1 side slopes, and a Manning's roughness value of 0.014. Street elevations were determined using the 2011 MNDNR LiDAR elevation data set. All surface runoff that is surcharged or exceeds the capacity from the Freemont Avenue and Cherokee Heights storm sewer systems is routed through overland flow street channels into the Cherokee Heights low area/ravine, through the 60-inch culvert under Cherokee Heights, and into the Cherokee Heights ravine (modeled using a natural channel cross-section).

# 2.0 References


Barr Engineering, 2001. *Nine Mile Creek Watershed District Bloomington Use Attainability Analysis*

NOAA's National Weather Service Precipitation Frequency Data Server (PFDS) located at http://dipper.nws.noaa.gov/hdsc/pfds/

USDA, 2014. *Gridded Soil Survey Geographic (gSSURGO) Database Users Guide*. Natural Resources Conservation Service, United States Department of Agriculture. April 2014.

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# Appendix B

## Engineers Opinions of Probable Cost

**CONCEPT LEVEL ENGINEER'S OPINION OF COST**

CHEROKEE HEIGHTS CULVERT AND SLOPE STABILIZATION

4/8/2015

| **Downstream Ravine Stabilization - 100yr storm event** | | | | |
|---|---|---|---|---|
| **Item Description** | **Unit** | **Estimated Quantity** | **Unit Price** | **Extension** |
| Excavation of excess material | CY | 210 | $ 30 | $ 6,300 |
| Clearing and Grubbing | SY | 1450 | $ 10 | $ 14,500 |
| Grading and Shaping | SY | 1450 | $ 3 | $ 4,350 |
| Channel Filter Rock/Geotextile | CY | 1140 | $ 46 | $ 52,440 |
| Riprap (MnDOT Class V gradation) | CY | 1450 | $ 95 | $ 137,750 |
| Restoration | AC | 0.25 | $ 12,800 | $ 3,200 |
| Erosion Control/Temp Facilities | LS | 1 | $ 10,000 | $ 10,000 |
| MOB/DEMOB | LS | 1 | $ 50,000 | $ 50,000 |
| SUBTOTAL | | | | $ 278,540 |
| CONTINGENCY | | | (10%) | $ 28,000 |
| ESTIMATED CONSTRUCTION SUBTOTAL | | | | $ 306,540 |
| ENGINEERING & ADMINISTRATION | | | (30%) | $ 92,000 |
| **TOTAL** | | | | **$ 398,540** |
| **Estimated Accuracy Range** | | | -30% | **$ 278,978** |
| | | | 50% | **$ 597,810** |

| **Upstream Culvert Modifications with Downstream Ravine Improvements** | | | | |
|---|---|---|---|---|
| **Item Description** | **Unit** | **Estimated Quantity** | **Unit Price** | **Extension** |
| Clearing and Grubbing | SY | 8010 | $ 4.50 | $ 36,045 |
| Tree Removal | EA | 70 | $ 390 | $ 27,300 |
| Excavation | CY | 6500 | $ 9 | $ 60,450 |
| Grading and Shaping | SY | 8010 | $ 3 | $ 24,030 |
| Culvert Modification | LS | 1 | $ 27,500 | $ 27,500 |
| Vegetation | LS | 0.5 | $ 12,800 | $ 6,400 |
| Restoration | LS | 1 | $ 31,780 | $ 31,780 |
| Erosion Control | | | (5%) | $ 11,000 |
| MOB/DEMOB | | | (10%) | $ 22,000 |
| SUBTOTAL | | | | $ 246,505 |
| CONTINGENCY | | | (10%) | $ 25,000 |
| ESTIMATED CONSTRUCTION SUBTOTAL | | | | $ 271,505 |
| ENGINEERING & ADMINISTRATION | | | (30%) | $ 81,000 |
| **TOTAL** | | | | **$352,505** |
| **Estimated Accuracy Range** | | | -30% | **$ 246,753** |
| | | | 50% | **$ 528,757** |

| **Downstream Pipe Conveyance** | | | | |
|---|---|---|---|---|
| **Item Description** | **Unit** | **Estimated Quantity** | **Unit Price** | **Extension** |
| Clearing and Grubbing | SY | 2890 | $ 9.00 | $ 26,010 |
| Tree Removal | EA | 100 | $ 390 | $ 39,000 |
| Excavation and Backfill | CY | 4800 | $ 90 | $ 432,000 |
| Piping (includes bedding and pipe bends) | LF | 1300 | $ 200 | $ 260,000 |
| Manholes | EA | 5 | $ 8,000 | $ 41,600 |
| Drop Structure | EA | 1 | $ 200,000 | $ 200,000 |
| Riprap Channel (includes MnDOT Class IV riprap and filter) | LF | 260 | $ 600 | $ 156,000 |
| Restoration | AC | 1.25 | $ 12,800 | $ 16,000 |
| Erosion Control | | | (5%) | $ 59,000 |
| MOB/DEMOB | | | (10%) | $ 123,000 |
| SUBTOTAL | | | | $ 1,352,610 |
| CONTINGENCY | | | (20%) | $ 271,000 |
| ESTIMATED CONSTRUCTION SUBTOTAL | | | | $ 1,623,610 |
| ENGINEERING & ADMINISTRATION | | | (30%) | $ 487,000 |
| **TOTAL** | | | | **$ 2,110,610** |
| **Estimated Accuracy Range** | | | -30% | **$ 1,477,427** |
| | | | 50% | **$ 3,165,915** |