



TOWN OF NORWOOD

MASTER DRAINAGE PLAN & STORMWATER FEASIBILITY STUDY

AUGUST 2026



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ABBREVIATIONS

AC-FT	Acre Feet
AEP	Annual Exceedance Probability
CDOT	Colorado Department of Transportation
cfs	Cubic feet per second
CMP	Corrugated metal pipe
CNs	Curve numbers
CR	County Road
CUHP	Colorado Urban Hydrograph Procedure
EPA	United States Environmental Protection Agency
Ex	Existing
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
fps	Feet per second
GIS	Geographic Information System
HDPE	High-density polyethylene
HDS-5	Hydraulic Design Series No. 5
HEC-22	Hydraulic Engineer Circular No. 22
HEC-RAS	Hydrologic Engineering Center - River Analysis System
HOA	Homeowners Association
HSG	Hydrologic Soil Group
Hwy	Highway
LUD	Land Use Designation
MDP	Master Drainage Plan
MHFD	Mile High Flood District
No.	Number
NLCD	National Land Cover Database
NOAA	National Oceanic and Atmospheric Administration
NRCS	Natural Resources Conservation Service
O&M	Operation and Maintenance
PFDS	Precipitation Frequency Data Server
RCP	Reinforced Concrete Pipe
SCS	Soil Conservation Service, now NRCS
SSA	Storm and Sanitary Analysis
SWMM	Storm Water Management Model
UDFCD	Urban Drainage and Flood Control District
USGS	United States Geological Survey
WRCC	Western Regional Climate Center



1.0 EXECUTIVE SUMMARY

Norwood's drainage system relies on streets, inlets, storm drain pipes, culverts, ditches, irrigation features, and detention ponds, many of which predate current drainage standards. This Master Drainage Plan (MDP) gives the Town of Norwood a practical framework for understanding the system and identifying stormwater deficiencies, prioritizing improvements, and budgeting for future work.

The goal of this report is to develop a Master Drainage Plan for the Town of Norwood. The Town does not currently have a Master Drainage Plan (MDP). This MDP focuses on the following components:

- 1 Inventory the town's existing drainage infrastructure.
- 2 Assess the hydrologic methodologies considered to determine the adequacy of existing drainage infrastructure.
- 3 Determine what improvement projects and maintenance items are needed to manage minor storm events (e.g., a 5-Year or (20% AEP) return interval storm).
- 4 Identify how major storms (e.g., a 100-Year or (1% AEP) return interval storm) were predicted to move through the Town streets, storm drains, culverts, and channel systems.
- 5 Provide budgetary cost estimates of recommended improvements.

The study inventories the existing drainage network, delineates contributing basins, estimates runoff for the 5-year and 100-year design storms, and evaluates planning-level hydraulic performance. The study area includes 20 culverts, 41 storm drain pipes, 24 single inlets, 24 open-channel ditch segments (including curb and gutter and valley pan), and 6 stormwater detention ponds. Field investigations identified recurring maintenance concerns, including sedimentation, partial blockages, damaged culvert ends, vegetation overgrowth, and detention ponds that lack reinforced overflow protection.

At the planning level, approximately 50% of evaluated culverts do not meet minor-storm criteria, and approximately 60% do not meet major-storm criteria. Approximately 22% of evaluated storm drain pipes do not meet minor-storm criteria, and approximately 63% do not meet major-storm criteria. Many deficiencies reflect system interactions—downstream limitations, ditch-to-pipe transitions, shallow street conveyance, and missing collection points—rather than isolated pipe or culvert problems.

⋮ **The highest-priority system-level improvement areas are South Pine between Paradox and San Miguel Streets, Cedar Street north of Grand Avenue, and the Lucerne/Market/Summit Street area.**

Recommended improvements include selective pipe and culvert replacement, additional inlets, localized curb and gutter, valley pans, ditch regrading, and rerouted conveyance paths. Implementation should proceed from downstream to upstream so that upstream improvements do not overload unresolved downstream constrictions.

⋮ **The planning-level storm drain system improvement program cost of five major improvement projects totals approximately \$4,340,000. Identified detention pond overflow improvements total approximately \$13,300.**

- The report also recommends near-term annual stormwater operation and maintenance budgeting in the range of **\$25,000 to \$50,000**, which would reduce to **\$10,000** annually after reactive maintenance is completed. Eleven additional lower-cost pipe maintenance and replacement projects total approximately **\$68,130**. These costs are intended for planning and budgeting only and should be refined during design.

Town staff, elected officials, and project partners should use this MDP as an implementation guide, not as final design documentation. The plan supports capital improvement programming, grant applications, annual maintenance budgeting, development review, and coordination with irrigation interests and downstream landowners. Before construction, the Town should confirm ownership, maintenance responsibility, downstream acceptance, irrigation impacts, utilities, constructability, permits, easements, and design-level costs.

NORWOOD MASTER DRAINAGE PLAN: KEY FINDINGS AT A GLANCE			
Study Area	Existing Infrastructure	Modeled Deficiencies	Planning Costs
636 acres evaluated across Town and contributing offsite drainage areas. 57 existing subbasins & 69 proposed subbasins and modeled for hydrologic response.	20 culverts 41 storm drain pipes 24 inlets 24 surface drainage segments 6 detention ponds	Culverts: 50% deficient for the minor storm; 60% deficient for the major storm. Pipes: 22% deficient for the minor storm; 63% deficient for the major storm.	\$4.34M total planning-level capital program. \$13.3K detention pond overflow improvements. \$25K–\$50K recommended near-term annual O&M budget.
Highest-priority improvement areas: South Pine between Paradox and San Miguel Streets; Cedar Street north of Grand Avenue; and the Lucerne/Market/Summit Street area.		Implementation principle: complete downstream improvements before upstream collection or conveyance upgrades to avoid transferring flooding to unresolved downstream constraints.	

Note: Costs are planning-level estimates intended for budgeting, grant planning, and capital improvement programming. Values should be refined during design-level analysis. Proposed subbasins cover the same total area as the initial basins but reflect restructured drainage areas resulting from proposed inlets and street improvements.



2.0 INTRODUCTION OVERVIEW

The Town of Norwood has served as an agricultural, commercial, and cultural center for western San Miguel County, Colorado, since its incorporation in 1905. The community is at an elevation of approximately 7,000 feet. Norwood is located approximately 46 miles northwest of Telluride, along CO-145.

Norwood has experienced varied growth since 1950, with no immediate decrease in growth rate anticipated over the next decade. Like many similarly-sized municipalities across the western United States, the Town experienced rapid growth before large-scale infrastructure projects were planned. As a result, the Town is taking the opportunity to evaluate the state of this infrastructure, understand what items are undersized and jump-start forward-thinking planning processes.

Development has resulted in the loss of natural drainage features that historically connected and provided the capacity to relieve stormwater flows to perennial streams. Norwood's development inhibits its ability to rely solely on natural features to manage stormwater runoff and capture pollutants. [The Town's stormwater system relies primarily on its street and storm drain systems](#) and a few onsite detention basins to collect and direct runoff to tributaries of the Gurley Ditch drainage along its northern boundary.

After identifying the need for a Master Drainage Plan (MDP), the Town selected SGM to prepare it through a request-for-proposals process. SGM began this work in July 2025.

This report develops the Town's first modern MDP.

- [The plan establishes a planning framework for evaluating existing drainage conditions, identifying deficiencies, prioritizing improvements, and coordinating future drainage decisions with irrigation operations, downstream conveyance limitations, maintenance capacity, and anticipated infill development.](#)

Detailed objectives are summarized in Section 2.5.

2.1 AREA CLIMATE

Norwood is at an elevation of 7,025 feet atop Wright's Mesa and experiences a four-season climate. Long-term climate records are available from a climate station in Norwood, with monthly records from 1924 through 2008.

The average annual precipitation in Norwood is 15.41 inches. The summer and early fall months (July through October) have the highest average rainfall, typically in the form of afternoon and early evening thunderstorms, often with high-intensity, short-duration rain events. June typically experiences the lowest rainfall of the year. Precipitation during the winter months is generally snow. Climate data for the Norwood climate station, obtained from the Western Regional Climate Center (WRCC), are included in [Appendix C](#).

2.2 PREVIOUS MASTER DRAINAGE PLAN

The Town completed a master drainage plan in 1963. That document does not reflect current stormwater infrastructure, current development patterns, or modern hydrologic methods, so it does not provide an adequate basis for present-day planning and design decisions.

2.3 SCOPE OF 2025 STUDY

- ∴ Town staff identified the need to develop drainage solutions for short-duration,
- ∴ high-intensity storms, reduce localized flooding, and support more reliable
- ∴ infrastructure planning.

The Town asked SGM to prepare a Master Drainage Plan that presents the costs of maintaining existing drainage infrastructure and recommendations for new capital projects.

This study uses current planning practices and focuses on the storm events most likely to control new and replacement infrastructure in this region. SGM inventoried the existing drainage system, reviewed known problem areas, identified maintenance and replacement needs, and developed planning-level recommendations for larger capital projects.

The primary deliverables are a prioritized improvement program, planning-level cost estimates, and a framework for future design, maintenance, development review, and implementation.

2.4 COMMUNITY OVERVIEW

The Town is located on Wright's Mesa, which is situated 500 feet above the San Miguel River. [Figure 1-1](#) in [Appendix A](#) shows the location of the Town relative to other communities in southwest Colorado. Additionally, the Town is situated near Miramonte Reservoir, Cone Reservoir, and Gurley Reservoir, as well as several smaller impoundments that support agricultural irrigation.

- ∴ The Gurley Ditch is the principal watercourse in the study area. Although originally
- ∴ constructed for irrigation, it now also functions as a major drainage conveyance.

Potential wetlands occur throughout the study area, many of which appear to be associated with irrigation infrastructure and ditch seepage rather than natural drainage alone.

Floodplain boundaries for the Gurley Ditch have not been updated by the Federal Emergency Management Agency (FEMA) since 1988 (see [Appendix D](#)). No portion of the Town is currently in a FEMA special flood hazard area (SFHA), although nearby Zone A mapping exists downstream and north of town. **The absence of mapped Special Flood Hazard Area within Town should not be interpreted as absence of local drainage or flood risk.** Runoff from nearly all of the study area ultimately reaches the Gurley Ditch before discharging to the San Miguel River more than 15 miles downstream.

The Colorado Water Conservation Board (CWCB) has completed preliminary work to update the floodplain mapping in the area.

According to the Colorado Hazard Mapping & Risk MAP Portal, CWCB began reassessing flood risks in 2017 by collecting updated topography, survey, and hydrology information. Hydraulic models were approved by FEMA in January 2021. Data listed in the “Milestones” section for San Miguel County indicate that no additional work has been completed since 2021, except for the FEMA Base Level Engineering (BLE) review, completed on August 19, 2025. More up-to-date information can be found at the CWCB’s Hazard Mapping website:

<https://coloradohazardmapping.com/story?county=5a7553fd-3b75-47b7-a434-495eecd34d5#milestones>

Based on this information, there is no projected date for adopting of an updated flood map for the County. More detailed information from the 2020-2021 flood mapping efforts can be found in [Appendix D](#).



Figure 2-1: Vicinity Map

Population estimates are not available for the entire study area, but are available for the Town of Norwood, which comprises most of the study area's population (see [Table 2-1](#)). The State Demography Office's population estimate for Norwood in 2024 was 560. Over the 34-year period from 1980 to 2024, the Town's annual growth rate averaged approximately 0.69%. Using this growth rate for a new 20-year planning period, the current (2025) population of 564 people in the Town of Norwood is projected to reach 643 by 2045. This growth rate would result in approximately 79 additional people over the planning period.

Future growth for the Town is anticipated to be predominantly infill of currently vacant lands and/or on the periphery of the Town to the north, south, and/or east.

2.5 PLAN OBJECTIVES

The objectives of this Master Drainage Plan (MDP) establish a practical framework for understanding the Town's existing stormwater system, identifying infrastructure deficiencies, and prioritizing future drainage improvements. The plan focuses on characterizing contributing drainage areas, documenting existing infrastructure, evaluating system performance under short-duration, high-intensity rainfall conditions, and developing recommendations that support informed decision-making, capital planning, updates to local drainage standards, and advancement of select priority projects toward preliminary design.

Table 2-1: Norwood Census Data

Year	Total Population	Growth
1950	294	-
1960	443	151%
1970	408	-8%
1980	478	17%
1990	429	-10%
2000	438	2%
2010	518	18%
2020	535	3%
2025	564	0.69%
2035	602	0.69%
2045	643	0.69%

• The Master Drainage Plan (MDP) objectives included the following:

- 1 Define and characterize the areas that impact the Town's existing stormwater infrastructure.
- 2 Conduct a field survey of existing storm infrastructure and an investigation of historical documents.
- 3 Evaluate how the Town's existing stormwater infrastructure performs under a short-duration, high-intensity rainfall distribution and identify deficient infrastructure elements.
- 4 Provide general recommendations for drainage improvements necessary to meet the Town's design criteria for stormwater infrastructure.
- 5 Summarize the Town's stormwater infrastructure needs, priorities, and costs.
- 6 Develop revisions to the Town's Drainage Standards to bring them into line with a contemporary standard of care.
- 7 Develop preliminary design drawings (50% readiness) for select high-importance infrastructure.

• Collectively, these objectives provide the basis for a data-driven, prioritized approach to improving the Town's stormwater infrastructure, reducing drainage-related risks, and supporting long-term system resilience.



3.0 PLAN APPROACH

This Master Drainage Plan (MDP) uses historical observations, field information, available mapping, and firsthand accounts from Town staff to identify flooding issues and evaluate system performance. The study area was then divided into primary drainage basins, with hydrologic analysis points identified for each basin. Stormwater system elements, such as culverts, inlets, and detention ponds, define the locations of analysis points within each drainage basin. Flow paths (i.e., the paths that runoff follows from the upper elevation to the analysis point of each drainage basin were determined throughout the study area.)

The study evaluates 5-year and 100-year design storms, corresponding to 20% and 1% annual exceedance probability (AEP) events. Hydrologic modeling uses basin area, land use, soil conditions, and rainfall data to estimate peak runoff rates in cubic feet per second (cfs). Rainfall is applied uniformly and simultaneously across the study area for this planning-level analysis.

Hydraulic modeling of each basin routes estimated flows through the basin's drainage infrastructure and flags elements which exceed planning criteria. If a drainage element did not meet the Standards (i.e., culvert surcharging above design limits, street spread or curb depth exceeding design limits, pond overflowing, or similar), the element is identified as "> CAPACITY," "SURCHARGED," "FLOODED," or "OVERTOPPING" in the modeling results. Recommended upgrades are identified in the Proposed Replacement Items and Proposed Improvements Cost and Priority ([Table 8A](#) and [Table 8B](#), respectively, in [Appendix B](#)). Draft drainage standards are included in [Appendix H](#).

An additional list of Low Priority Replacement and Maintenance Recommendations is provided in [Appendix G](#). These low-priority storm drain pipes and culverts provide sufficient drainage under major storm conditions but, in most cases, do not meet the draft Standards' minimum size of 12 inches for laterals and 18 inches for mains.

Cost estimates for replacement or supplemental elements were prepared for existing elements deemed inadequate. The scope of the MDP includes preliminary design-level analyses and recommendations for high-priority projects; therefore, the solutions included in the MDP are estimates intended to provide budgetary costs to the Town. Verifying design deficiencies and determining design solutions will allow the Town to refine the budget for the storm drainage improvements.

The workflow in summarizes how the MDP moves from background information and field inventory through hydrologic and hydraulic screening, deficiency identification, improvement development, and implementation prioritization.

Master Drainage Plan Approach

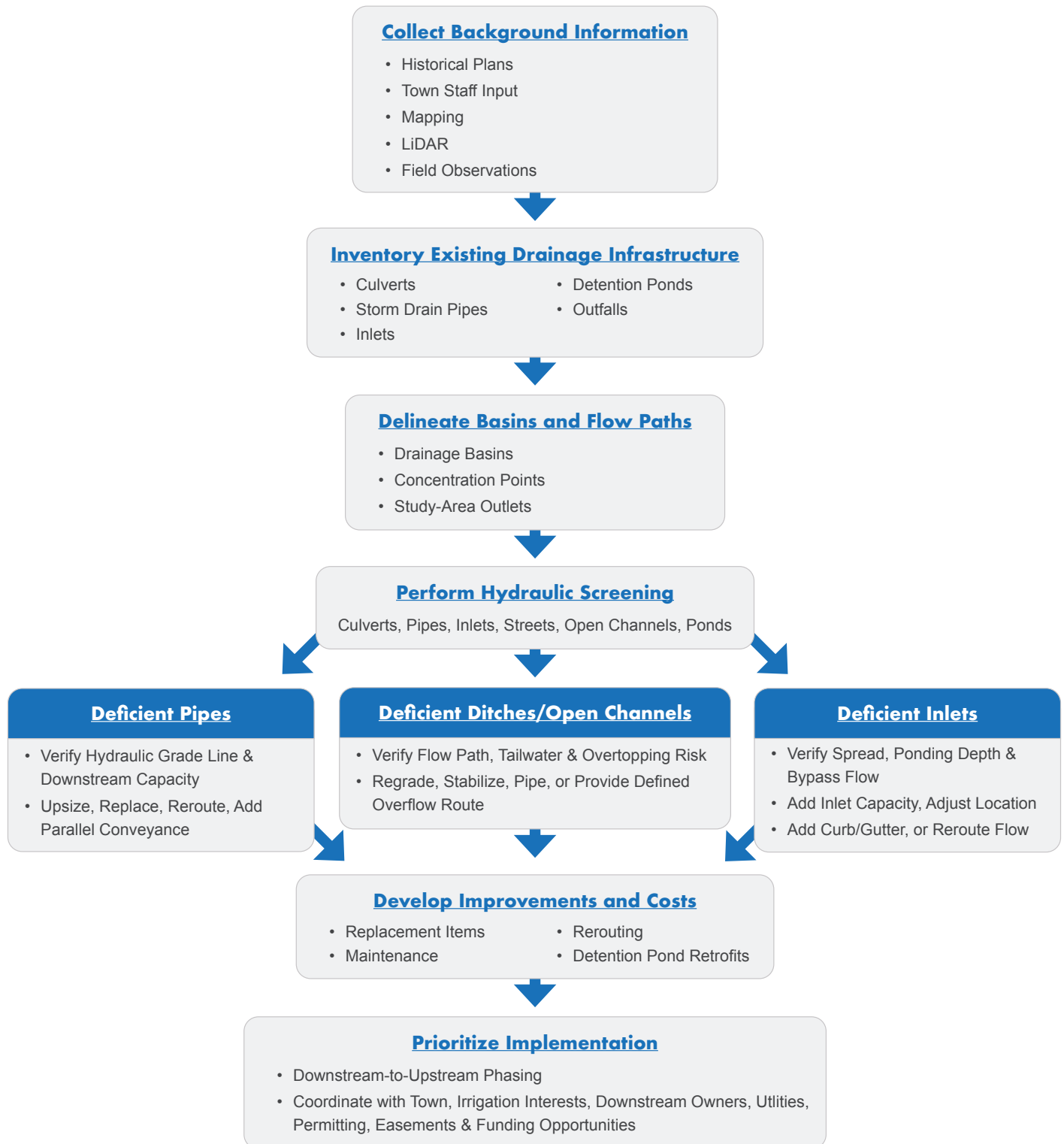


Figure 3-1: Master Drainage Plan Approach Workflow



4.0 EXISTING SYSTEM INVENTORY

4.1 BASE MAP DEVELOPMENT

4.1.1 TOWN-SUPPLIED INFORMATION

Information supplied by the town included historical drainage and engineering plans dating back to 1963. This included a drainage plan from 1963, roadway improvement plans from 1980, subdivision stormwater as-builts from 1998 for Kiwi Street and San Miguel Street, stormwater plans for the Homestead Subdivision dated 2002, as-built subdivision plans for Pinion Park Subdivision dated 2023, and a grading plan for the Lone Cone Library dated 2018. Additionally, the town provided information on historic runoff patterns and directions, the locations of problematic areas, and the co-mingled storm and irrigation network that runs through town.

Data in the plans identified the location and recorded sizes of much of the storm infrastructure; however, the accuracy of the old plans needed to be verified to reflect contemporary storm infrastructure. Data from subdivision as-builts included information related to detention ponds and stormwater conveyance. It should be noted that private culverts at driveways were not included in the analysis; only culverts that crossed streets or alleys were included. Furthermore, culverts and pipes were excluded from the analysis if their primary function was to convey irrigation water.

4.1.2 SGM SUPPLEMENTAL DATA ACQUISITION

SGM Survey and GIS teams worked with Town Public Works staff to collect field data for all identifiable infrastructure. This ensured that the invert datums for the modeled data coincided with the terrain model used in the analysis. Pertinent data captured for inlets included inverts for connecting pipes, structure invert elevations, structure sizes, rim elevations, and inlet type. Information collected by SGM for pipes and culverts included material type, pipe invert elevations, and a conditional assessment. Detention basin outlet structure information was also collected, including the orientation, size, and elevation of outlet structure components, such as orifices and weirs.

SGM built a digital surface model (DSM) using LiDAR data from the Colorado Hazard Mapping website (<https://coloradohazardmapping.com/>). The data from this source were sufficiently accurate to verify overall drainage element elevations and to develop and verify existing detention basin volumes.



Figure 4-1: Infrastructure Mapping, Inventorying, and Data Collection

Note that additional field-collected elevation points served to supplement detention basin volumes. LiDAR data for the study area were collected in 2020. The tested vertical accuracy ranged from 9.84 cm to 20.16 cm for non-vegetated and vegetated areas, respectively. Data sheets for the LiDAR data are in [Appendix I](#).

San Miguel County imagery was added to the SSA model for spatial reference.

4.2 INVENTORY OF SYSTEM ELEMENTS

The inventory of existing system elements was verified through a comprehensive field inspection of the storm drainage system. The field inspection collected information including, but not limited to, locating and identifying the type of curb and gutter in place throughout the system, size and type of inlets (i.e., curb inlets or area inlets, and types of grates on the inlets, parallel, perpendicular, curved or straight vane, locations of drainage swales and ditches, detention pond outlet structure size and geometry, identification of discharge points from detention ponds, identification of flow paths for drainage waters, and general confirmation of the drainage basins configurations).

It should be noted that some of the elements identified within the study area may be the responsibility of CDOT, San Miguel County, or private property owners, but were analyzed as part of the study to ensure a comprehensive system analysis. All stormwater ponds understood to be privately owned and maintained were analyzed in this study to identify performance deficiencies.

[Figures 3-1 through 3-11 in Appendix A](#) show the base map and the locations of the existing storm drainage elements within the study area.

Additional emergency work was completed by the Town in the summer of 2026 due to flooding impacts. Due to limited available elevation and geometric data, this updated work is not captured in the MDP. This work included piping the entirety of the system along Cedar Street, north of Grande Avenue, the installation of two junction boxes, installation of an irrigation diversion structure at the location of [Figure 4.3](#), installation of an inlet on the south west corner of Cedar and Summit, and replacement of the inlet on the south east corner of Cedar and Summit. The replaced pipes are CMP. Future work may consider keeping these pipes as-is, coating with Cured in Place Pipe (CIPP) lining to reduce the pipe roughness and increase capacity and pipe life, or replacement of the pipes.

4.2.1 CULVERT INVENTORY

The study area includes 20 existing culverts, excluding pond outlet pipes; 80% are corrugated metal pipe (CMP), and 20% are high-density polyethylene (HDPE). Culvert pipe diameters range from 6 to 24 inches. The length of culverts within the study area totals approximately 1,190 linear feet.

Field observations indicate that 9 of the 20 culverts (45%) were partially silted, crushed, closed, or otherwise obstructed at either the inlet or outlet end. These conditions increase flood risk and indicate a need for more frequent maintenance. Data on pipe size and slopes were estimated for these pipes whose ends could not be exposed.

4.2.2 STORM DRAIN PIPE INVENTORY

The study area includes 41 storm drain pipes connected to inlets, outfalls, or manholes. Approximately 60% are HDPE and the remaining 40% are CMP. Storm drain pipe diameters range from 12 to 30 inches. The length of storm drain pipes within the study area totals nearly 4,950 linear feet.

At least 2 of the 41 pipes (5%) were silted closed or otherwise obstructed on either the inlet or outlet end, suggesting that more frequent maintenance is required for these pipes. Data on pipe size and slopes were estimated for these pipes whose ends could not be exposed.

4.2.3 STORM DRAIN INLET INVENTORY

The study area includes 24 single inlets. Approximately 10 inlets, or 42% are located in sag locations, and the remaining 14 are located on grade. One inlet north of Summit Street on Market Street was not observed during field investigations, but it was assumed to exist based on discussions with Town staff and modeled as a component of the existing conditions.



Figure 4-2: On-Grade Combination Inlet on Kiwi Street

4.2.4 OPEN CHANNEL DRAINAGE INVENTORY

The Town's drainage infrastructure consists of culverts and pipes, interspersed with grassed or earthen drainage swales. Included in this study are 24 open channel ditches, 4 valley pan segments, and 4 gutter segments. The total length of modeled open channel segments is approximately 5,500 linear feet of ditches, 1,655 linear feet of gutter, and 80 linear feet of valley pan. Also included in the inventory are 8 ditch segments for culvert tailwater at outfall locations, where applicable.

Grassed and earthen ditches are often an integral part of a stormwater system, serving a multi-functional role by providing additional storage capacity, attenuating flow, and allowing sediment deposition. These systems can provide numerous benefits for stormwater conveyance and should be maintained to all practical extents possible.

4.2.5 STORMWATER POND INVENTORY

The study area includes six ponds constructed for stormwater detention purposes and one area that, while not explicitly designed for stormwater retention, serves to retain excess runoff. The designed ponds provide detention storage ranging from approximately 0.05 feet to 0.4 acre-feet. The "storage" area located south of Kiwi Street at the existing disc golf course and the proposed future school site (Stor-06) provides approximately 1.5 acre-feet of storage.

None of the ponds have a constructed, reinforced spillway, increasing the risk of uncontrolled overtopping and berm failure during larger storms. Some ponds appear to have accumulated sediment since construction, reducing the available storage volume to less than the designed capacity.

The area south of Kiwi Street serves two primary functions. First, it supports a senior water right that uses the draw as an impoundment for irrigation. Second, it delays runoff before that runoff enters the Town's stormwater system during periods of heavy rain and inundation. Because this feature is integral to both the existing and proposed drainage network, its storage and detention function should be preserved.

4.2.6 IRRIGATION DITCH AND CULVERT INVENTORY

Irrigation ditch culverts are not considered part of the Town's storm drainage infrastructure and were not analyzed for flow and capacity as part of the MDP. However, irrigation water is diverted through the Town's existing storm infrastructure.

- ∴ Two of the Town's major drainage pathways also serve
- ∴ as conduits to convey irrigation water.

These two drainages run from south to north, with one pathway from Pinion Street to Pine discharging north of Summit Street adjacent to Cedar (see [Figure 4-1](#)). The second pathway runs along Market Street, through Town.

Base flows of 2 cfs were modeled in the Pinion Street system and the Market Street system to account for storms occurring during the irrigation season. Proposed stormwater infrastructure developments shall exercise due diligence to maintain or bypass irrigation flows along historic pathways. Documents detailing the historic irrigation pathways and quantities are in [Appendix J](#) for Ditch No. 132, the *John Davis Waste & Seep Ditch*, and the *Stephens Drainage Pipeline*.

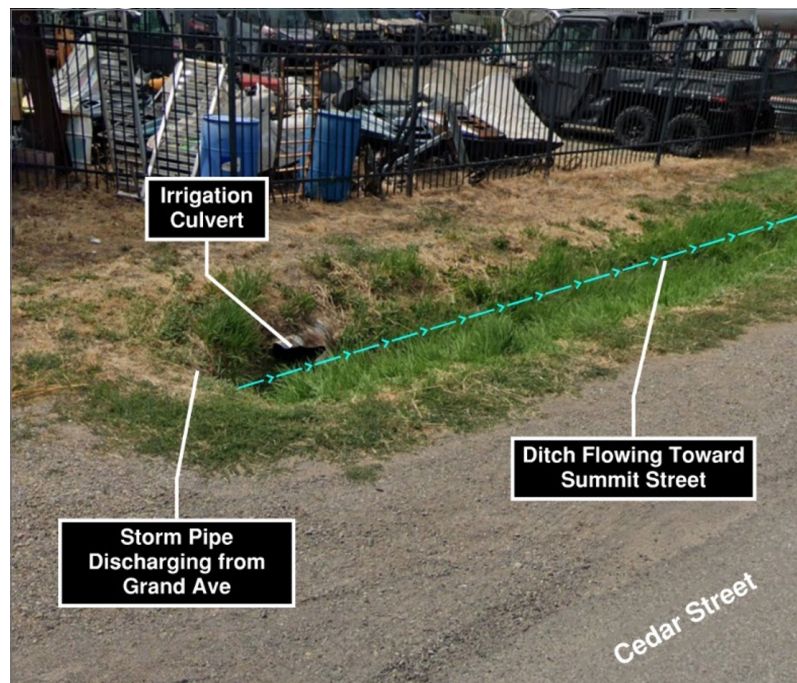
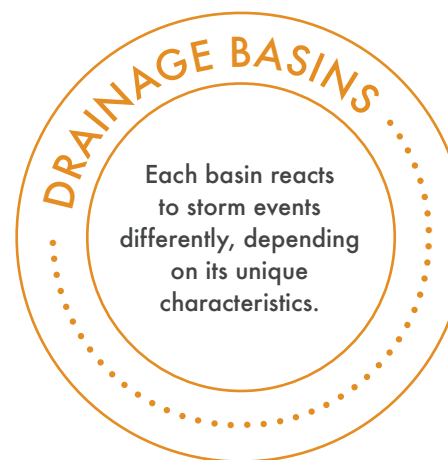


Figure 4-3: Co-Mingled Storm and Irrigation on Cedar Street



5.0 DRAINAGE BASIN DELINEATION AND CHARACTERIZATION

The characteristics of drainage basins are essential in analyzing the basins and estimating the quantity of drainage water generated and the rate at which stormwater will run off. The characteristics considered include area, soil types, infiltration parameters, surface slopes, the nature and extent of cover conditions, and other characteristics that could impact stormwater discharges from a basin.



5.1 CONCENTRATION POINTS OF HYDRAULIC ANALYSIS

Hydraulic analysis concentration points include culvert inlets, storm drain inlets, and stormwater ponds, where stormwater flows are required to assess element performance during design storm events. Many points of hydraulic analysis discharge to downstream basins or storm drain pipes; other points of hydraulic analysis discharge to study-area outlet points, where stormwater runoff leaves the study area, such as at irrigation ditches, the Gurley Ditch, or other points along the study-area boundary.

5.2 DRAINAGE BASIN AND FLOW PATHS DELINEATION

The drainage basins were configured around the points of concentration and study area outlet points using the topography of the study base map. Areas topographically upstream of each point of concentration define a drainage basin. The major drainage basins within the system drain from south to north and are composed of smaller sub-catchments or subbasins. The primary delineated basins within the study area are shown in [Figure 5-1](#) on the next page and [Figure 2-1](#) in [Appendix A](#), while the smaller, local sub-catchments are shown throughout [Figures 3 and 5](#) in [Appendix A](#).

The outermost delineated basins define the study area. The study area is primarily confined to the Town limits, with limited contributing areas outside the Town limits that drain into it. **The catchment area encompasses approximately 636 acres. The town boundary currently encompasses approximately 225 acres.** The study area boundary is shown in [Figure 2-1](#) in [Appendix A](#).

5.3 BASIN AREAS

The area of a drainage basin, along with other factors, directly correlates to the quantity of stormwater runoff from the design storm. Surface areas of basins were determined to model stormwater runoff from each basin. The study area was divided into 57 basins, ranging from 0.04 to 255.08 acres, with an average basin size of 11.99 acres. A large number of basins were required because of the considerable number of concentration points that required design flows within the study area, and predominantly within the Town limits. The contributing drainage area and estimated peak flows for each basin are shown in [Tables 1A and 1B](#) in [Appendix B](#), for the existing and proposed subbasins, respectively.

5.4 BASIN TOPOGRAPHY

The study area generally slopes from the southeast to the northwest, ranging from 7,000 to 7,610 feet in elevation, with higher elevations and steeper slopes in the southern portion and lower elevations and flatter areas in the northern portion.

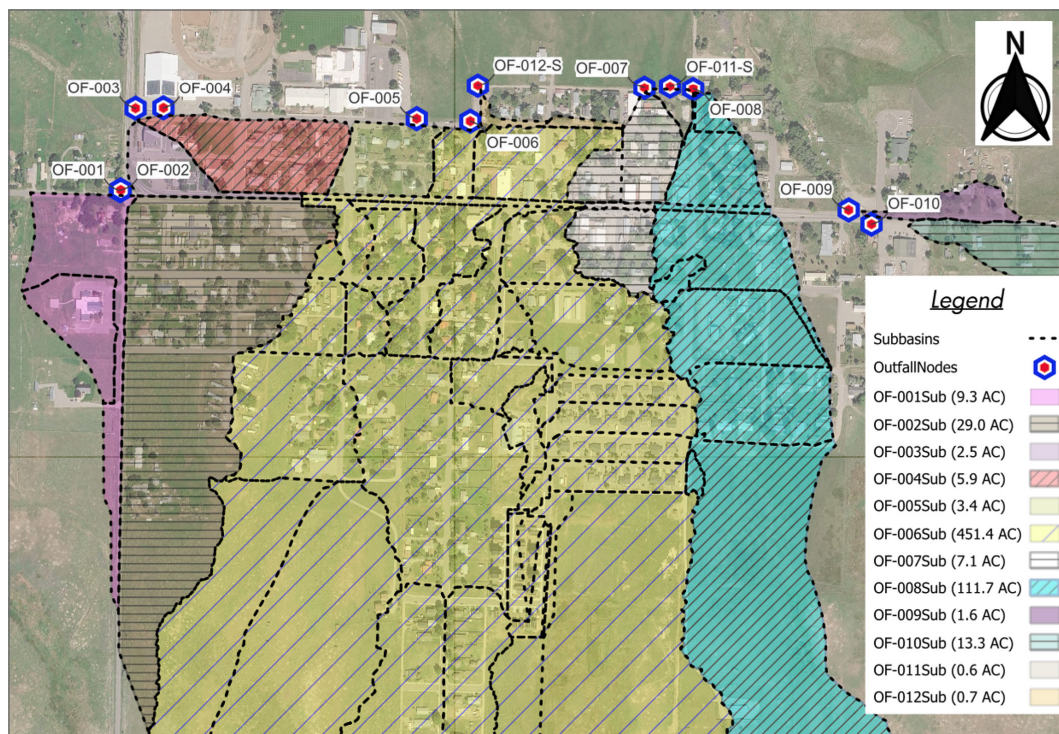


Figure 5-1: Drainage Basin and Outfall Overview

5.5 GENERAL LAND USES

Land use within the study area consists primarily of residential developments, with agricultural, grazing, and open-space uses interspersed throughout. In addition, there is a linear core area of retail, commercial, and office-professional development adjacent to CO Highway 145, ranging from Summit Street to the north and Naturita Street to the south. The periphery of the commercial core of the Town is represented as a residential area. The southern portion of the Town has seen significant development over the last ten years, a trend expected to continue in the near future.

5.6 SURFACE CONDITIONS

Identifying surface conditions in a drainage basin is essential to accurately predicting a basin's response to a storm event. The most significant surface conditions considered are cover type and slope.

Land cover type is classified as impervious or pervious. Impervious cover is a surface that allows little to no water to infiltrate the soil during rainfall. Examples of impervious cover include asphalt or concrete streets, parking lots, sidewalks, building roofs, and rock outcroppings.

Impervious cover results in higher runoff quantities and runoff rates. Cover that is not impervious is pervious. However, stormwater runoff response from pervious surfaces will vary significantly depending on the nature of the pervious cover and underlying soil type. **For example, a pervious area of compacted gravel or soil used as a parking or driving area will respond to a rain event more similarly to an impervious surface, with higher runoff quantities and runoff rates.** In contrast, an area with heavy ground cover, such as grass or other vegetation, or an area cultivated, such as a plowed field, will have much lower runoff quantities and rates, all other characteristics being equal. Vegetative cover and impervious area estimates were based on the National Land Cover Database (NLCD) land-cover and percent imperviousness raster data. Area-weighted averages were computed in QGIS for each subbasin. Impervious percentages for each basin in the study area are included in [Tables 1A and 1B](#) in [Appendix B](#) for the existing and proposed subbasins, respectively.

The second surface condition influencing drainage is surface slope. Steeper slopes result in higher runoff velocities over a shorter time period. When cover conditions and surface slopes are combined, the impact on drainage can be significantly increased. To complete the analysis of the defined drainage basins for the MDP, each basin was considered separately, and surface conditions were defined for each.

5.7 SOIL CHARACTERISTICS

Soil type impacts stormwater runoff response, mainly where impervious ground cover is absent. Soils are classified based on this study's potential for infiltration and runoff. Based on data obtained from the Natural Resources Conservation Service (NRCS) Web Soil Survey (<http://websoilsurvey.nrcs.usda.gov/>), the majority of the study area consists of soils classified as Callan Loam, with surface slopes ranging from 3% to 6% and an average of approximately 5%. An area to the south, outside the Town limits but contributing to the Town's drainage, has a soil classification of Narraguinnep Clay Loam, with surface slopes ranging from 15% to 50%, averaging approximately 32.5%. The remaining areas consist of loam soils, with a maximum surface slope of 12%.

The NRCS assigns soils to one of four hydrologic soil groups based on how quickly water infiltrates when the soils are not protected by vegetation and have a moist (normal) antecedent moisture level. According to the NRCS Web Soil Survey, the entire study area is classified as Group C. **Group C soils are characterized by slow infiltration when wet and slow water transmission.**

[Appendix E](#) contains an NRCS map showing the soil classifications and hydrologic soil groups assigned to those soils within the study area.

5.8 HYDROLOGIC FEATURES WITHIN BASINS

The hydrologic features of the Norwood drainage system vary from basin to basin and even within each basin. Much of the variation is related to the age of the improvements and developments within the basin. Older developments have unique features compared to newer developments. Portions of the older residential development in the Town lack curb and gutter. While the streets are paved in most of these areas, drainage water is conveyed generally northward through shallow, often poorly defined, earthen ditches along the street edge or, in some cases, along the rear lot line.

Roll curbs and gutters have been installed in older and newer residential areas. Hydraulic continuity of curb and gutter is not maintained throughout the Town. There appear to be several cross-sections for the roll-type curb and gutter, all of which have a reasonably shallow gutter depth of approximately 4 to 6 inches. Many of these installations are in relatively mild grades and in locations where flow directions must change to follow the curb-and-gutter alignment. **Any substantial rainfall could cause these shallow gutter sections to overtop, allowing drainage water to flow over the curb and out of the gutter channel.**

The Town Center area, located on Grand Avenue, is served by an underground piped storm sewer system that falls within the CDOT Right-Of-Way (ROW). This network also accepts drainage from the majority of the contributing drainage area to the south of Grand Avenue. The inlets along Grand Avenue and the connected drainage systems capture and convey nearly 88% of the Town's contributing drainage. This flow exits the Town drainage network at two primary outfall locations. The system comprises pipes, culverts, and curb inlets. Curb and gutter sections along Grand Avenue are all approximately 6-inch vertical curbs adjacent to crowned street sections. The bulk of the remaining drainage infrastructure consists of drainage swales, some of which are almost indiscernible, and culverts.

A significant feature of the Town's storm drainage system is the interception of runoff by, or discharge of piped stormwater runoff to, irrigation ditches and laterals. These ditches are primarily open and intercept drainage from the areas above them. These ditches and laterals intercept or accept runoff from numerous basins, primarily in the northern and western portions of the study area.

The two primary outfalls discharge into ditches on privately owned land on the northern boundary of the town. Without a recorded contractual agreement between the landowner and the town, discharged flows or volumes should not be increased.

5.9 LAND USE

The Town identified seven land-use designations on its 1994 Land Use Code, which was last amended in 2020 (<https://www.norwoodtown.com/files/cabc11b2c/revised-land-use-code-feb-20.pdf>). Additionally, the Town identifies four future land use designations on its Future Land Use Map. This can be seen in Figure 5-2 and in a larger format in Figure 4-1 in Appendix A. Three of the four land use types are within the MDP Study Area, with percentages of each designation listed in Table 5-1. The probable future percent of impervious area of each land use type was estimated using values of imperviousness recommended in the Mile High Flood District's *Urban Storm Drainage Criteria Manual Volume 1*. Note that Table 5-1

includes the Land Use Designation (LUD) *Public**.

This LUD includes community housing, existing school buildings, and open areas.

Table 5-1: Land Use and Percentage Imperviousness Values

Norwood Land Use Designation	Approximate Percentage of Study Area (%)	Percentage Imperviousness (%) (from UDFCD, Vol. 1, Ch. 6, Table 6-2)
Light Industrial	15.5%	75%
Commercial	7.3%	80%
Residential	61.3%	60%
Public*	15.9%	55%

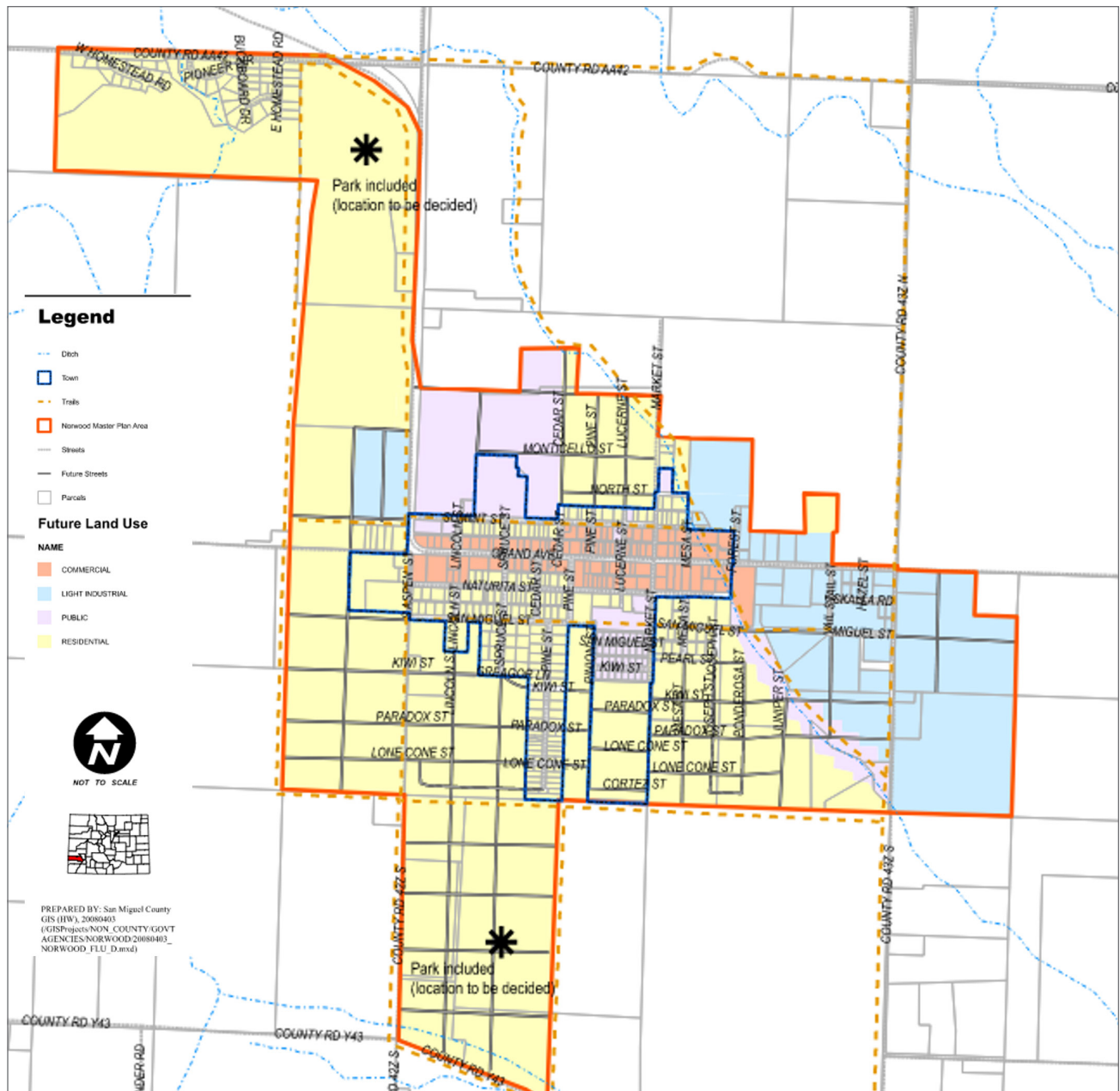


Figure 5-2: Town of Norwood Future Land Use Map

5.10 DRAINAGE SYSTEM PROBLEM AREAS AND AREAS OF CONCERN

Storm drainage problem areas were identified by the Town and taken into consideration in this MDP. Specific problem areas are listed below in no particular order and can be seen in [Figure 5-3](#) on the following page.

- 1 **Pine Street & San Miguel:** This area is problematic as Town staff has expressed concerns over the possibility of flooding residential property located immediately to the north of the intersection. Water has been observed to be within a foot of the roadway crown during heavy rainfall events.
- 2 **Southeast corner of intersection of Aspen Street and County Road Z42 (Grand):** Culvert inlet does not provide adequate interception of water. Frequent flooding has been observed at this location.
- 3 **South side of Summit Street between Cedar and Spruce Streets:** Stormwater drains into a 6-inch irrigation culvert. Ponding has been observed here during heavy precipitation events.
- 4 **Summit Street between Market and Lucerne Streets:** Discharge from the culvert crossing Summit at this location has been known to flood property adjacent to the Fire Department.
- 5 **Summit Street & Market Street:** Overflow from the culvert on Summit Street between Market and Lucerne Streets tends to pond in the intersection.
- 6 **Cedar Street north of Grand Avenue:** Reductions in pipe diameter and hydraulic capacity of ditches reduce and cause bottlenecking in the system. This bottleneck causes backup and ponding.
- 7 The Homestead Subdivision's southernmost detention basin outlet structure's outflow control is inadequately sized.
- 8 No observed hardened overflow spillways for detention basins.
- 9 **Pinion Street, adjacent to San Miguel and Kiwi Streets:** Frequent ponding and flooding noted in the eastern ditch.
- 10 **San Miguel & Spruce Streets:** Backup at acute culvert transition. Ponding noted.
- 11 **Naturita & Spruce Streets:** Backup at acute culvert transition. Ponding noted.
- 12 **West of Spruce Street & South of San Miguel Street:** Concerns over ditch capacity that runs behind Spruce Street residences.
- 13 **Town-wide:** Co-mingling of irrigation and stormwater.

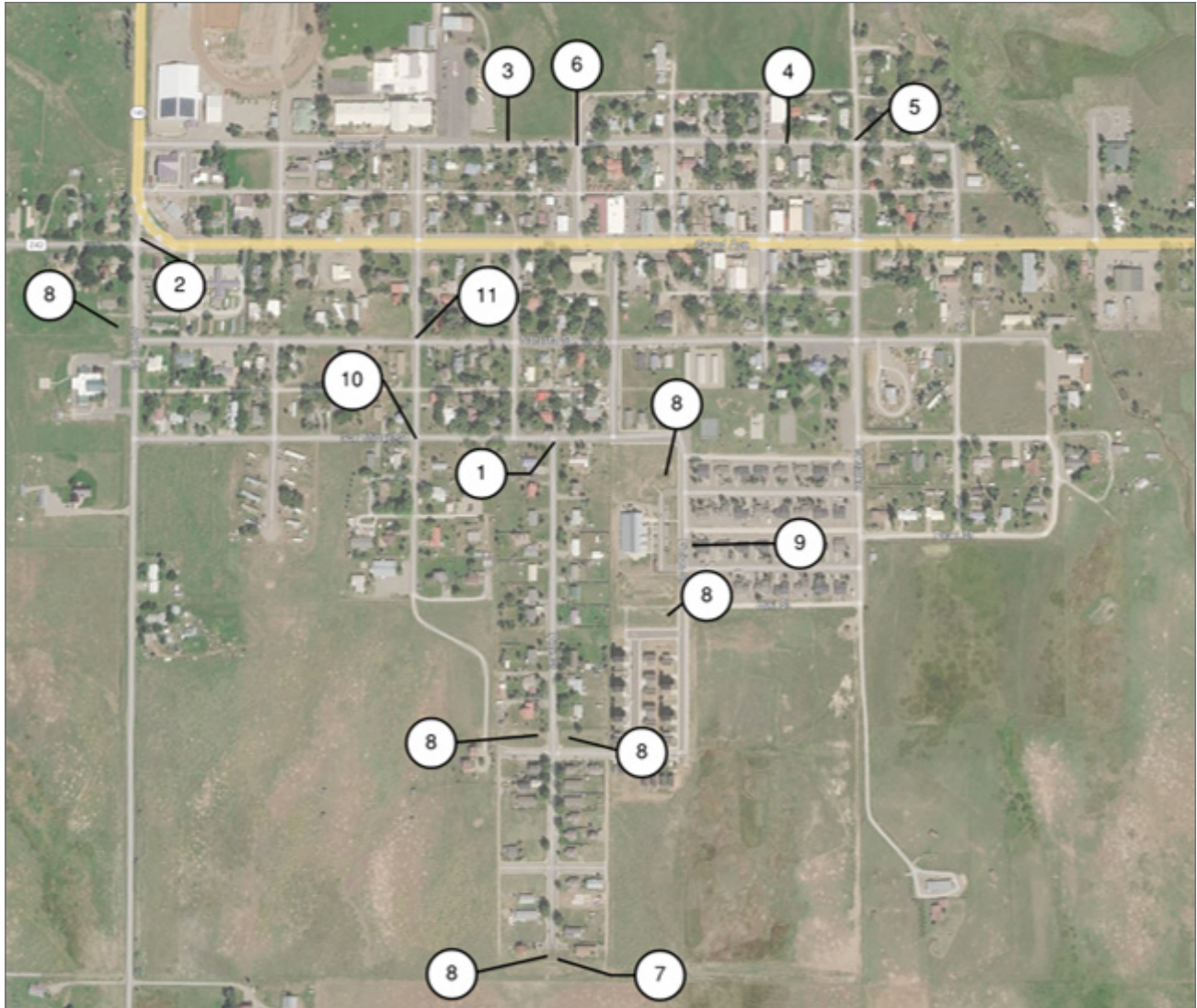


Figure 5-3: Areas of Concern

5.11 ENVIRONMENTAL CONCERNS

Environmental constraints appear limited for the existing drainage infrastructure, but new construction, replacement, and maintenance activities should be reviewed early in project development. New construction may affect wetlands, and older irrigation structures may meet State Historic Preservation Office historic requirements.

Environmental issues should be identified on a case-by-case basis as development projects enter the initial planning phase. **New development should limit post-development discharge to historic rates across the applicable design storms to avoid overloading the Town's infrastructure and downstream receiving systems.**

5.12 OPERATIONS AND MAINTENANCE CONCERNS

Operations and maintenance (O&M) are essential to the reliable performance of a storm sewer network. Routine upkeep helps preserve drainage pathways so that the system can function as intended during larger storm events.

- ⋮ Although small municipalities often must balance stormwater maintenance with other
- ⋮ competing priorities, regular inspection, cleaning, and repair of storm drainage
- ⋮ components can reduce the likelihood of clogged pipes or inlets, maintain system
- ⋮ capacity, and help limit localized flooding impacts.

Deferred or inconsistent maintenance can gradually reduce the effectiveness of the storm drainage system and may increase both operational and community impacts over time. Sediment, debris, vegetation, and damaged structures can restrict flow, reduce inlet capacity, and contribute to nuisance flooding during routine storms or more significant flooding during larger events. In addition to potential property damage and roadway disruptions, inadequate maintenance can increase long-term repair costs, shorten the service life of drainage infrastructure, and create avoidable emergency response needs. Establishing a consistent O&M program helps communities identify issues early, prioritize limited resources, and maintain a more resilient stormwater system.

SGM's field inspection identified the following **recurring O&M issues**:

Inlet grates blocked or partially blocked with trash, sediment, gravel, and/or leaves.

Detention/retention ponds overgrown with vegetation and siltation buildup.

Catch basins below grate covering, blocked or partially blocked with trash, leaves, vegetation, or sediment.

Damaged culvert ends (inlet and outlet).

Culvert openings blocked or partially blocked with trash, leaves, or sediment.

High quantities of debris/sediment in ditches and roadway gutters.



Figure 5-4: Culvert and Ditch with Debris and Sediment



6.0 HYDROLOGY STUDY

6.1 HYDROLOGIC ANALYSIS CRITERIA

The hydrologic analysis criteria used in this analysis are based on criteria adopted by neighboring municipalities and are as follows:

- Analysis of minor and major design storms of 5-year (minor) and 100-year (major) 20% and 1% Annual Exceedance Probability events, respectively.
- Use of the EPASWMM method for stormwater runoff estimates.
- Rainfall data from the National Oceanic and Atmospheric Administration's (NOAA) Atlas 14 via NOAA's Precipitation Data Frequency Server.

The Town's infrastructure design standards are incorporated into the land use code initially adopted in 1994, with the latest revision dated February 2020.

However, most of the development within the Town was completed before the current Standards were adopted; therefore, most of the existing storm drainage system was not designed to the current Standards but to unknown, likely varying criteria.

Much of the storm drainage infrastructure was likely not designed to convey or detain runoff from a major storm event. However, this event was used in the analysis to determine how storm drainage facilities perform in a major storm. Drainage facilities that do not meet the performance requirements of these studies will be recommended for upgrading or replacement.

For undeveloped areas within the study area, this analysis assumes that the developer will be required to limit post-development runoff to historic pre-development rates . SGM has prepared draft updated drainage standards to support that requirement. The proposed standards are located in [Appendix H](#).

6.2 HYDROLOGIC MODEL

The U.S. Environmental Protection Agency's (EPA) Storm Water Management Model (SWMM), operating within the Autodesk Storm and Sanitary Analysis (SSA) software (version 13.6.268.0, 3/01/2023), was used to model the hydrology of the study area. SWMM simulates runoff by breaking the study area into subbasins that receive precipitation and generate runoff. The model then tracks the runoff as it moves through pipes, channels, and other devices, such as storage and treatment units, and diversion structures. SWMM can be used to evaluate the effectiveness of stormwater control strategies, such as pipes and storm drains, and to create cost-effective hybrid solutions Typical applications of SWMM include analyzing and sizing complex drainage system components for stormwater management.

EPA SWMM utilizes the SCS Curve Number infiltration method to estimate peak discharges for urban drainage basins. EPA SWMM operates under the assumption that the calculated runoff is a function of the curve number (infiltration), impervious cover, equivalent watershed width, and a 2-hour urban rainfall distribution for Norwood. Each subbasin was linked to an outflow point or outfall corresponding to a specific culvert, inlet, or pond. Model runs were completed for both the minor and major storms.

6.3 HYDROLOGIC MODELING INPUTS

6.3.1 RAINFALL DATA

Rainfall data were obtained from the National Oceanic and Atmospheric Administration's (NOAA) Precipitation Frequency Data Server (PFDS) (<http://hdsc.nws.noaa.gov/hdsc/pfds/>), which interpolates NOAA Atlas 14 precipitation frequency estimates and associated information for a specified location. The PFDS then outputs local precipitation information as a table with rainfall depths as a function of storm duration and average recurrence interval. [Appendix F](#) includes the rainfall data for the Norwood area generated from the NOAA PFDS.

• The 2025 SSA model uses a 2-hour distribution to assess short-duration, high-intensity rainfall events that are more common in southwest Colorado due to monsoonal rainfall in mid-summer to early fall and stress drainage infrastructure with high peak runoff values. These short-duration, high-intensity rainstorms account for most flooding.

The storm distribution used in this analysis was generated using the Colorado Urban Hydrograph Procedure (CUHP) developed by Mile High Flood District (MHFD, <https://mhfd.org/>). The 1-hour point precipitation values from NOAA Atlas 14 are distributed into 5-minute increments to develop temporal distribution patterns for use with CUHP. The 1-hour depth-point rainfall from NOAA Atlas 14 is converted to a 2-hour design storm. This method is outlined as follows:

- 1 The 1-hour, 5-year rainfall depth of 0.774 inches was obtained from NOAA Atlas 14 for watershed coordinates. Rainfall data from NOAA-14 are included in [Appendix F](#).
- 2 The 2-hour rainfall distribution was generated by multiplying the 5-year rainfall depth by a percentage as a function of time throughout the temporal distribution as outlined in Table 5-2 of the MHFD Criteria Manual, Volume 1, Chapter 5.
- 3 The 1-hour, 100-year rainfall depth of 1.65 inches was obtained from NOAA Atlas 14 for watershed coordinates. Rainfall data from NOAA-14 are included in [Appendix F](#).
- 4 The 2-hour rainfall distribution was generated by multiplying the 100-year rainfall depth by a percentage as a function of time throughout the temporal distribution as outlined in Table 5-2 of the MHFD Criteria Manual, Volume 1, Chapter 5.

The rainfall distributions generated from the above steps are listed in [Table 6-1](#) and [Table 6-2](#).

Table 6-1: 2-hour, 5-year Rainfall Distribution

(from MHFD, Vol. 1, Ch. 5, Table 5-2)

Minutes	% of 1-Hour	Depth (in.)
0	0	0.000
5	2	0.015
10	3.7	0.029
15	8.7	0.067
20	15.3	0.118
25	25	0.194
30	13	0.101
35	5.8	0.045
40	4.4	0.034
45	3.6	0.028
50	3.6	0.028
55	3	0.023
60	3	0.023
65	3	0.023
70	3	0.023
75	2.5	0.019
80	2.2	0.017
85	2.2	0.017
90	2.2	0.017
95	2.2	0.017
100	1.5	0.012
105	1.5	0.012
110	1.5	0.012
115	1.5	0.012
120	1.3	0.010
TOTAL	115.7	0.896

Table 6-2: 2-hour, 100-year Rainfall Distribution

(from MHFD, Vol. 1, Ch. 5, Table 5-2)

Minutes	% of 1-Hour	Depth (in.)
0	0	0.000
5	1	0.017
10	3	0.050
15	4.6	0.076
20	8	0.132
25	14	0.231
30	25	0.413
35	14	0.231
40	8	0.132
45	6.2	0.102
50	5	0.083
55	4	0.066
60	4	0.066
65	4	0.066
70	2	0.033
75	2	0.033
80	1.2	0.020
85	1.2	0.020
90	1.2	0.020
95	1.2	0.020
100	1.2	0.020
105	1.2	0.020
110	1.2	0.020
115	1.2	0.020
120	1.2	0.020
TOTAL	115.6	1.907

6.3.2 BASIN SURFACE

LiDAR data from the Colorado Hazard Mapper LiDAR portal was used to determine the relative slope of each drainage basin. The LiDAR data were used to generate a digital surface model, a composite of 2-foot and 30-foot grids. This was completed by processing point cloud data and filtering for ground shots.

The finer 2-foot-resolution data were used to model areas in town, while the 30-foot-resolution data were used to model the contributing drainage basins south of town. Additionally, LiDAR data were used as supplemental elevation data to estimate relative surface elevations in areas where survey data were not available. Areas with dense vegetation or tree cover tend to produce greater uncertainty in surface elevations. Considering this, the LiDAR surface provides adequate detail for a high-level overview. The tested vertical accuracy ranged from 9.84 cm to 20.16 cm for non-vegetated and vegetated areas, respectively. LiDAR data sheets are available in [Appendix I](#).

6.3.3 BASIN AREAS

Basin areas determined in Section 4.3 were input for each modeled basin or sub-basin. Areas can be viewed in [Tables 1A](#) and [1B](#) in [Appendix B](#) for existing and proposed basins.

6.3.4 TIME OF CONCENTRATION

The time of concentration is generally considered the minimum time that must elapse for all portions of a drainage basin to contribute flow to the basin outlet due to rainfall. EPA SWMM computes the time of concentration using the kinematic wave formulation, which depends on the subbasin length and equivalent width. The computed time of concentration for each subbasin is shown in [Tables 1A](#) and [1B](#) in [Appendix B](#), for the existing and proposed subbasins, respectively.

6.3.5 LAND USE/COVER

In conjunction with soil type, the land cover of a drainage basin is one of the primary factors contributing to storm runoff. The EPA SWMM methodology uses a single, composite Soil Conservation Service (SCS) Curve Number for the basin's pervious regions. Land use and cover affect the amount of runoff generated over an area, considering the hydrologic activity of the vegetation covering the land. The bulk of the pervious areas, without any improvements or structures, can be considered to have grass cover (50%-75%) or fair pastureland. **Generated runoff is a function of both land use and hydrologic soil group.** These combined values determine the corresponding SCS Curve Number. More in-depth discussion on the Curve Number derivations and their source is discussed in Section 6.3.6.

6.3.6 CURVE NUMBERS

Infiltration of rainfall from the pervious area of each subbasin was estimated using the Soil Conservation Service (SCS) Curve Number infiltration method.

∴ **Curve numbers (CNs) are factors indicating the potential for excess runoff from a given rainfall event. Higher CNs indicate higher runoff potential, and lower CNs indicate lower runoff potential.**

In the EPA SWMM method, subbasins are divided into pervious and impervious subareas. Surface runoff can infiltrate into the upper soil zone of the pervious subarea, but not through the impervious subarea. The impervious portion of the subbasin area is assumed to translate entirely into runoff, less any specified depression depth for the impervious area.

Therefore, infiltration parameters, such as a composite curve number value, should be specified for the pervious portion of the subbasin. Impervious areas are specified directly using the percent impervious parameter and are not included in the composite curve number.

Estimation of CNs for the pervious portion of subbasins was based on the drainage area's hydrologic soil group, land use, and ground cover. Curve numbers within the study area were assumed to be a constant value of 79 and were classified as pasture, grassland, or range. [Tables 1A](#) and [1B](#) in [Appendix B](#) illustrate the CN assigned to each subbasin.

6.3.7 IMPERVIOUS AREA

The percentage of subbasin area that is impervious (i.e., roofs, asphalt roadways, and sidewalks) was estimated from an NLCD imperviousness raster dataset obtained in QGIS and area-averaged across subbasins. Percent impervious data for each subbasin are shown in [Tables 1A](#) and [1B](#) in [Appendix B](#).

6.3.8 MANNING'S ROUGHNESS

The Manning's roughness coefficient used for overland flow over the impervious portion of the subbasin was 0.015. The Manning's roughness coefficient used for overland flow over the pervious portion of the subbasin was 0.10.

6.3.9 DEPRESSION DEPTH AND COVER

- ∴ The depth of depression storage value is intended to account for “localized ponding”
- ∴ after a rainstorm, when small puddles are scattered.

The assigned depth of depression storage on the impervious portion of the subbasin was 0.1 inches over 75% of the subbasin area. The assigned depth of depression storage on the pervious portion of the subbasin was 0.2 inches.

6.4 EXISTING CONDITIONS HYDROLOGIC MODELING RESULTS

Modeling outputs for each subbasin include the calculated time of concentration, the estimated peak runoff rate for both the minor and major design storms. The modeling inputs and results are in [Tables 1A](#) and [1B](#) of [Appendix B](#).

The calculated time of concentration ranged from approximately 5 minutes to 3.5 hours for the minor storm and from approximately 4 minutes to 2.5 hours for the major storm.

Estimated peak flows for each subbasin ranged from 0.03 cubic feet per second (cfs) to approximately 9.23 cfs for the minor storm and 0.10 cfs to approximately 25.90 cfs for the major storm. **The major storm peak discharge rate is approximately 6.5 times the minor storm peak flow across all subbasins.** This substantial increase can be attributed to a greater proportion of minor storms being stored in depressions and infiltrating into the ground. The greater intensity of the major storm quickly exceeds the land cover's natural ability to infiltrate and store excess rainfall.



7.0 LIMITED HYDRAULIC STUDY

This hydraulic analysis is intentionally limited to planning-level evaluation. Available topographic data, including LiDAR-based surface modeling, are sufficient for system-wide screening and conceptual improvement planning but lack the fine detail needed for street cross-slopes, curb profiles, precise headwater elevations, or exact hydraulic controls at every concentration point. The analysis therefore uses reasonable planning assumptions to identify likely deficiencies and guide future design work. Assumptions regarding elevations and slopes used in this limited hydraulic study are detailed in the sections discussing the model input parameters for the specific drainage elements.

7.1 HYDRAULIC DESIGN CRITERIA

The Town of Norwood does not currently maintain a complete set of formal hydraulic design criteria for storm drainage infrastructure. For this study, SGM applies criteria commonly used by comparable communities in southwest Colorado to establish reasonable planning-level performance standards.

Table 7-1 summarizes the planning criteria used for the limited hydraulic screening. **In other words, the minor storm criteria focus on frequent nuisance flooding and road usability, while the major storm criteria focus on public safety, property protection, and emergency access.**

Table 7-1: Hydraulic Performance Criteria for Minor and Major Storms

System Element	Minor Storm Performance Criteria	Major Storm Performance Criteria	Design Criteria
Culverts	Headwater depth generally limited to the culvert diameter.	Headwater remains 1 foot below the roadway low point or property-damage threshold.	Pipe grades should achieve a minimum velocity of 3 fps when flowing full.
Storm Drain Pipes	Convey flow without surcharge.	Surcharging is permitted provided the hydraulic grade line remains 1 foot below the rim or inlet-control limits.	Pipe grades should achieve a minimum velocity of 3 fps when flowing full.
Streets	Curb or easement containment within right-of-way or drainage easement. Allowable flow spread: - Local streets: to the street crown. - Collector streets: at least one 10-foot lane free of water. - Arterial streets: at least one 10-foot lane free of water in each direction.	Curb or easement containment within right-of-way or drainage easement. Allowable flow spread: - Local streets: to the street crown. - Collector streets: at least one 10-foot lane free of water. - Arterial streets: at least one 10-foot lane free of water in each direction.	
Detention Ponds	Limit post-development discharge to historic pre-development flow.	Limit post-development discharge to historic pre-development flow.	Provide an overflow spillway capable of conveying the undetained flow from a major storm.

The hydraulic methodology for this limited study includes the following:

- Analysis of street flow depth, spread, and inlet capture and bypass using methods described in the Federal Highway Administration's (FHWA) Urban Drainage Design Manual, Hydraulic Engineering Circular No. 22 (HEC-22). An inlet clogging factor of 50% (see commentary on this value in Section 7.3) was used in the analysis.
- Flow depths in storm drain pipes and culverts are determined using the Manning equation for open channel flow when pressure conditions are not present. When pressure flow conditions are present, flows are computed using the Hazen-Williams equation. These equations are used to determine the friction slope within the pipe or culvert, which is then incorporated into Saint-Venant's equations, which apply the continuity and momentum equations.
- Analysis of culvert flow using methods presented in FHWA's Hydraulic Design of Highway Culverts, Hydraulic Design Series No. 5 (HDS-5).

7.2 HYDRAULIC MODEL INPUTS

7.2.1 DESIGN FLOWS

Design flows for minor and major storms were used to model the hydraulics of the storm drainage infrastructure. Peak flows were used to analyze culverts, streets, inlets, and storm drain pipes, while hydrographs (flow over time) were used to analyze detention pond performance. [Tables 1A](#) and [1B](#) in [Appendix B](#) detail the flows from the hydrologic model, at each subbasin.

7.2.2 CULVERT PARAMETERS

Culvert modeling parameters include physical parameters identified during the field survey, such as pipe invert elevations, length, shape, diameter, and material. Additional parameters include the design flow determined as part of the hydrologic modeling, calculated pipe slope, Manning's roughness coefficient, allowable headwater depth, and the calculated tailwater depth.

Manning's roughness coefficient measures resistance to flow and varies from 0.012 for smooth interior high-density polyethylene (HDPE) and reinforced concrete pipe (RCP) to 0.024 for corrugated metal pipe (CMP).

Allowable headwater depths are a headwater depth equal to the culvert pipe diameter for a minor storm, and a minimum of 1 foot below the low point on the road or the elevation required to prevent damage to adjacent properties. As discussed above, the topographic data lacked sufficient detail to estimate the locations of low points on roads or other topographic features. For modeling purposes, the major storm was analyzed assuming a headwater elevation of 1 foot above the top of the pipe. The 1-foot submergence assumes that culverts will typically have at least 1 foot of cover over the pipe and that the typical installation is in a crowned roadway section. This is a conservative approach, as culverts may have more than 1 foot of cover, and culverts showing insufficient capacity during a major storm should be reviewed in more detail to determine whether additional headwater depth is available.

Tailwater depth was determined by calculating the design-flow depth in the channel downstream of the culvert using channel parameters estimated from the topographic data. As discussed above, limitations in topographic detail reduce the accuracy of these estimates and may yield overly conservative tailwater depth estimates, which could affect the estimated capacity of some culverts.

All culverts were modeled with a generic overflow weir to simulate roadway overtopping and to ensure the continuity of flows downstream of the culvert capacity to downstream basins, given the assumed conditions discussed above. The overflow weir was modeled as either a 20-foot or 40-foot-wide level section of paved road.

7.2.3 STORM DRAIN PIPE PARAMETERS

Storm drain pipe modeling parameters include physical parameters identified during the field survey, such as pipe invert elevations, lengths, diameters, and materials. Additional modeling parameters include the design flow, calculated pipe slope, and Manning's roughness coefficient. Design flows were the sum of the upstream pipe flows, and the flow intercepted by the inlet(s) at the upstream end of the pipe. Tailwater depths were based on the headwater depth of the downstream pipe. Pipes are modeled using the same principles as culverts, where applicable, as discussed in the previous section.

7.2.4 OPEN CHANNEL PARAMETERS

Open-channel segments of the system were modeled based on survey data. While slopes within the channel were not always constant, the channel slopes were modeled according to the next connecting link, which, in most cases, was either a pipe or a culvert. Grassed and earthen channels were modeled with a Manning's coefficient of 0.032 to account for the relative roughness of weeds or other vegetative debris that may accumulate in the channel. Channels were modeled as triangular, trapezoidal, or rectangular shapes. Channels with driveways were modeled as a single continuous reach, and driveway culverts, or the lack thereof, were not considered.



Figure 7-1: Typical Culvert and Ditch System Along South Spruce Street

7.2.5 STREET AND INLET PARAMETERS

Street modeling parameters include street width, longitudinal slope, cross-slope or crown slope, and the presence or absence of a curb and gutter. Streets were assumed to have crowned slopes of 2%; longitudinal slopes were determined point-to-point between inlets. Design flows were determined at the design points, typically inlet locations.

Inlet modeling parameters include the physical parameters identified during the field survey, including the size of the inlet and inlet type (standard inlets are combination inlets with inlet grates located in the gutter pan with curb openings in the face of the curb compared to inlets designated as surface inlets or gutter pan inlets, which are inlet grates, only), grate type (orientation of grate bars), and inlet location (on continuous grade or in a sag/sump condition). Inlet cross-slopes were assumed to be 2%, with longitudinal slopes equal to the street longitudinal slope at the inlet location.

7.2.6 DETENTION POND PARAMETERS

Storm drain detention pond modeling parameters include the physical parameters identified in the field survey, including the elevation of the pond bottom and top of pond berm and overflow spillway (maximum water surface elevation); outlet pipe invert elevations, pipe diameter, pipe material; and outlet structure details, including orifice diameter, number, configuration, and grate size, type, and elevation. Pond areas were generated from SGM field survey data (August 2025) and supplemental 2020 LiDAR data from the Colorado Hazard Mapper LiDAR portal and used to develop stage-storage curves for each pond, assuming a linear increase in area with depth. For this study, the ponds were assumed to start empty of water before the design storm event.

7.3 HYDRAULIC MODELING AND SETTINGS

Hydraulic modeling of storm drainage structures within the study area was completed using Autodesk Storm and Sanitary Analysis (SSA) software. The SSA model for Norwood employed the hydrodynamic method. This method is well-suited to existing complex stormwater management systems. It can deal with backwaters from detention basins, pipes with negative slopes, and junctions with multiple inflows. It requires smaller time steps for computational stability; one second worked well in this case. The model was run with an inertial term “Dampen” (recommended), with a 100-second lengthening time step to account for longer conduit, gutter, and swale runs. Junction surface area was left at the default value, as most manholes in the storm drain section of the model are 4 feet in diameter.

Except for flow data, the data in the preceding sections were entered into input files; flow data was transferred within the software from the hydrologic modeling to the design points for hydraulic modeling.

Model runs were completed for both the minor and major storms.

Design flows were transmitted from upstream design points and basins to downstream design points and basins. Culverts were analyzed as stand-alone structures using FHWA equations, including the pipe length, slope, and diameter, the maximum headwater depth, the assumed tailwater depth, and the design flow.

Streets, inlets, and storm drain pipes are interconnected and analyzed as a system. Street geometries (longitudinal and cross-slope, and width) and design flows at the design points (inlet locations) determine the flow spread (width within the roadway) and the depth at the design points. The capture of the street flow by an inlet depends on several factors, including whether the inlet is located on grade or in a sump/sag condition. Other factors include the size and number of inlets, the percentage of inlet clogging, and curb openings.

Inlet clogging is 50%, resulting in a minor reduction in performance according to HEC-22 calculations. In contrast, SWMM assumes a linear reduction in performance related to inlet clogging. Therefore, SWMM reduces inlet performance by 50% when 50% clogging is assumed.

An inlet on grade may capture only a portion of the street flow if the spread of the flow exceeds the width of the inlet grate. Analysis of an inlet on grade includes estimating the flow intercepted by the inlet and the flow bypassing it. The bypassed flow is transmitted in the model to the next downstream design point, and any additional surface runoff is produced between the design points. Intercepted flow is transferred to the downstream storm drain pipe, where it is combined with upstream pipe flows, and the performance of the downstream pipe segment is analyzed.



Figure 7-2: Typical Street Section with On-Grade Inlets at Pinion Street

In contrast, an inlet in sag will intercept 100% of the flow if it is physically possible to achieve the required depth to move the flow into the inlet (i.e., the depth is not so great that it overflows the surrounding ground surface to adjacent land at a lower elevation).

When an inlet is identified as being in a sag condition, the model will calculate the required depth and whether it is physically possible. The flow is then transferred to the downstream storm drain pipe, where it is added to any upstream flows, and the performance of the downstream pipe segment is analyzed. A depth significantly below curb height indicates that the results should be reviewed more closely to determine whether the depth is physically possible or whether the flow would overtop the curb and discharge to a different downstream location.

Pipe performance was analyzed using the hydrodynamic routing method to account for both open-channel flow conditions (when pipes are flowing at less than full depth) and pressure-flow conditions (when pipes are flowing at full depth and under pressure head). Using Manning's roughness coefficients, pipe diameter, and pipe slope, Manning's equation determined the water depth and velocity for both open channel and pressurized flow conditions.

Pond performance was analyzed using specific stage-storage-discharge curves for each pond and the pond's inflow hydrograph. As applicable, pipe flow, orifice flow, and/or weir flow calculations were used to develop a discharge curve for each outlet structure. Flows were routed through the pond, and ponds with insufficient volume for the inflow routed excess flows downstream by overtopping an emergency spillway modeled as a weir.

7.4 HYDRAULIC MODELING RESULTS

Results of the hydraulic modeling of storm drainage elements within the study area are presented in this section. All results are at the Master Drainage Plan level of analysis. As discussed in previous sections, select parameter values were assumed where detailed topographic data were unavailable. The assumed values may be conservative, leading to the identification of elements that do not meet the design criteria. An engineering analysis at the design level will be required to make this determination.

7.4.1 CULVERT ANALYSIS RESULTS

Tables 2A and 2B present the results of the culvert analyses for the minor and major storms, respectively. The tables summarize the physical parameters of the pipes and the modeling results.

The estimated headwater depth is the key evaluation parameter for determining whether the modeling standards adequately sized the culvert. If the calculated headwater depth (the water depth at the culvert's inlet end required to push the design flow through the pipe) is greater than the allowable headwater depth for the design storm (based on modeling criteria), then the pipe has insufficient capacity for the estimated basin flow and tailwater conditions. Culverts identified as ">Capacity" or "Surcharged" in Tables 2A and 2B are identified as potentially not meeting the modeling standards and require additional study to verify this result; culverts identified as "Sufficient" meet the modeling standards even under the conservative parameters assumed in this study and need no additional study.

- Table 2A shows that 10 culverts (50% of the total evaluated) are estimated not to meet the design standards for the minor storm. Table 2B indicates that 12 culverts (60% of the total evaluated) are estimated not to meet the design standards for the major storm.

The culverts recommended for additional investigation and replacement are listed in Tables 8A and 8B in Appendix B and shown in Figures 5-1 to 5-6 in Appendix B.

7.4.2 STORM DRAIN PIPE ANALYSIS RESULTS

Tables 3A and 3B in Appendix B present the results of the storm drain pipe analyses for the 5-year and 100-year storms, respectively. The tables summarize the physical parameters of the pipes and the modeling results. The estimated headwater depth is the key evaluation parameter for determining whether the storm drain pipe is sufficiently sized. If the calculated headwater depth is greater than the allowable headwater depth for the design storm the pipe cannot convey the estimated flow captured by the upstream inlets, nor can it convey flow to the downstream or tailwater conditions. Storm drain pipes identified as ">Capacity" or "Surcharged" in Tables 3A and 3B are identified as not meeting the design standards and require additional study to verify this result; pipes identified as "Sufficient" meet the design standards under the estimated analysis conditions.

- Table 3A reveals that 9 storm drain pipes (22% of the total evaluated) are estimated not to meet the design standards for the minor storm, and Table 3B reveals that 26 storm drain pipes (63% of the total evaluated) are estimated not to meet the design standards for the major storm.

The storm drain pipes recommended for additional investigation and replacement are listed in [Tables 8A and 8B](#) in [Appendix B](#) and shown in [Figures 5-1 to 5-6](#) in [Appendix A](#).

7.4.3 OPEN CHANNEL ANALYSIS RESULTS

[Tables 4A and 4B](#) in [Appendix B](#) illustrate the results of the analyses for the minor and major storms, respectively. Data in these tables list the approximate geometric parameters used to model each open channel segment and indicate whether the channel was modeled as having adequate capacity (“Calculated”) or as “Flooded,” in which case the flow capacity exceeded the channel.

Due to the limitations of a one-dimensional analysis, the extent and impacts of flooding are dynamic and depend on the adjacent topography; therefore, their spread is difficult to quantify. The better metric to evaluate performance, aside from the qualitative “Flooded” designation, is the “Max Flow/ Design Flow Ratio” shown in [Tables 4A and 4B](#). One ditch exceeded the “Max Flow/ Design Flow Ratio” for the minor storm (4%) and two exceeded the ratio for the major storm (8%).

7.4.4 STORM DRAIN INLET ANALYSIS RESULTS

[Tables 5A and 5B](#) present the results of the inlet analyses for the minor and major storms, respectively. The tables summarize the physical parameters of the inlets and their peak flows. The key evaluation parameters are the maximum gutter spread and depth during peak flow. The analysis standards for minor storms require at least one 10-foot lane to be water-free on collector and arterial streets, and no overtopping of curbs on all streets, where curbs are present. In contrast, the standards for a major storm require that the water depth be less than 12 inches at the gutter flow line and that the flow be contained within street rights-of-way or drainage easements.

As previously discussed, this study's topographic and right-of-way information is insufficient to allow a detailed analysis of these design parameters. *For example, curb height, street cross-slope, gutter depression, street width, right-of-way limits, and other detailed parameters can affect whether the storm drain system meets the design criteria.* However, information at this level of detail was unavailable for each inlet location, and the assumptions outlined in [Section 6.2.4](#) were used in the inlet analyses. In addition, to evaluate the sufficiency of each inlet, flow characteristics as outlined in [Table 7-2](#) for the minor storm and [Table 7-3](#) for the major storm, were employed. These values depend on the inlet location and the characteristics of the roadway where it is used.

Table 7-2: Allowable Pavement Encroachment and Depth of Flow for Minor Storm Runoff

Street Classification	Maximum Encroachment*
Local	No curb overtopping. Flow may spread to the street crown.
Collector	No curb overtopping. Flow spread must leave the equivalent of one 10-foot driving lane clear of water.
Arterials	No curb overtopping. Flow spread must leave the equivalent of two 10-foot driving lanes clear of water, one in each direction.

Note: * Indicates that where no curbing exists, encroachment shall not extend past the rights-of-way lines.

Table 7-3: Allowable Pavement Encroachment and Depth of Flow for Major Storm Runoff

Street Classification	Maximum Encroachment*
Local & Collector	Residential dwellings and public, commercial, and industrial buildings should be no less than 12 inches above the 100-year flood at the ground line or lowest water entry of the building. The water depth over the gutter flow line should not exceed 12 inches.
Arterials	Residential dwellings and public, commercial, and industrial buildings should be no less than 12 inches above the 100-year flood at the ground line or lowest water entry of the building. The water depth should not exceed the street crown to allow emergency vehicles to operate. The depth of water over the gutter flow line should not exceed 12 inches.

All of the inlets had sufficient capacity for the minor storm; however, 7 out of 24 inlets (29%) did not meet the criteria listed below for the major storm. This is again assuming that all inlets are clean prior to receiving storm runoff and are functioning as intended. Lastly, due to limited topographical information, area inlets adjacent to roadways should be assessed on a case-by-case basis for their effectiveness in receiving runoff. One example of this is INLET-004 on the western side of South Pine Street at its junction with Grand Avenue. The recommended inlets for additional investigation, replacement, removal, or additional new inlets are listed in [Tables 8A and 8B in Appendix B](#) and shown in [Figures 5-1 to 5-6 in Appendix A](#). The inlets recommended for replacement are recommended due to elevation and grade changes in the connecting conduits. **New inlets are recommended where problem areas exist, and open-ditch systems are converted to piped connections.**

7.4.5 STORMWATER POND ANALYSIS RESULTS

[Tables 6A and 6B](#) present the results of the detention-pond analyses for minor and major storms, respectively. The tables summarize the physical parameters of the ponds and the peak outflow relative to the peak inflow. Evaluation of ponds against current design standards would require estimating the pre-development peak flow rate, which is difficult to determine retrospectively and beyond the scope of this study. Instead, ponds were evaluated primarily based on whether they are predicted to overtop during minor and major storm events, the magnitude of the estimated overtopping flow, and their estimated ability to reduce the peak flow from the drainage basin.

[Table 6A](#) reveals that none of the ponds within the study area are estimated to overtop during the minor storm, and [Table 6B](#) reveals that none of the ponds within the study area are estimated to overtop during the major storm. Overtopping indicates that the existing pond volume, combined with the existing outlet structure, is insufficient to contain the design storm.

∴ **In ponds without reinforced (typically riprap) spillways to protect the berms in the event of an overflow, overtopping could result in rapid erosion and failure of the berms, as well as the rapid release of stored water.**



8.0 PROPOSED IMPROVEMENTS AND PRELIMINARY BUDGETARY COSTS

One of the primary goals of the Master Drainage Plan was to introduce solutions to localized problem areas experiencing flooding. Each problem area exhibited unique issues, which include but are not limited to:

Piped areas discharging to open ditches

Ditches discharging to constricted pipes

Undersized pipes or culverts

Ditches modeled as over capacity during the minor storm

Lack of infrastructure to collect and/or transport water from ponded areas

Detention Pond deficiencies or deferred maintenance

This section identifies specific improvement recommendations for the system and the budget estimated to implement each recommendation.

Undersized pipes and culverts were previously discussed in Section 7.4, and a comprehensive list of proposed improvements relative to existing infrastructure is provided in [Table 8A](#) in [Appendix B](#). Proposed improvements include component replacements, the installation of previously non-existent components, or general excavation (ditches). These improvements include:

- 11 culverts
- 24 pipes
- 8 ditches
- 3 valley pan segments
- 4 inlets
- 3 manholes

Specific project details and budgetary estimates and quantities are presented below and provided in [Table 8B](#) in [Appendix B](#).

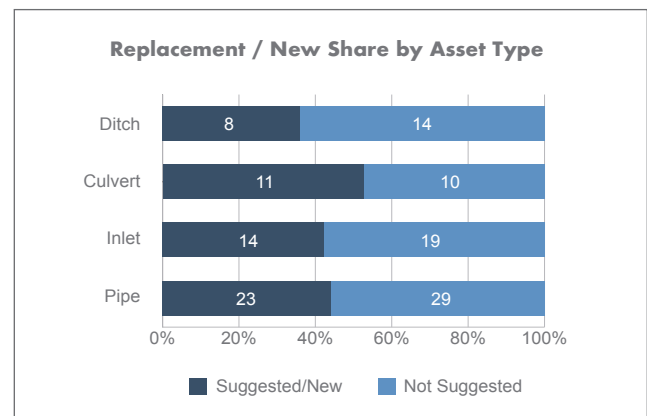


Figure 8-1: Improvement Summary Compared to Existing Inventory

8.1 PROPOSED DRAINAGE IMPROVEMENTS

8.1.1 FOCUSED PROJECT AREAS

- Of the five outfall systems identified for infrastructure improvements, three areas present the most significant drainage constraints: South Pine Street between Paradox Avenue and San Miguel Street, Cedar Street north of Grand Avenue, and the Lucerne Street and Market Street areas near Summit Street.

In these areas, upsizing existing pipes or culverts would provide limited benefit, and the existing topography does not support reliable swale drainage without the risk of overflow. Potential improvements include piping selected open ditch segments, installing additional inlets, adding curb and gutter to redirect runoff, and constructing valley pans at intersections. **These three areas are priority locations; however, the required downstream conveyance improvements must be completed before the focus-area projects to establish the hydraulic capacity needed to safely convey additional runoff and avoid transferring or worsening flood impacts to downstream properties.**

8.1.1.1 South Pine Improvements

In the South Pine area, the recommended improvements are intended to limit the risk of floodwaters overtopping the crown of San Miguel Street and to reduce localized ponding on the south side of San Miguel, where existing curb-and-gutter sections begin. The suggested improvements in this area resulted from discussions with Town staff. These alterations include the addition of curb and gutter on South Pine from San Miguel to Paradox Street, along the south side of San Miguel from South Pine to the east to match the existing curb and gutter, and finally along the north side of San Miguel from South Cedar to South Pine. Additionally, it is suggested that an inlet be installed at the southeast corner of the South Pine and San Miguel intersection, piping stormwater to the east before being intercepted by another inlet that should be installed to serve as a junction to transition the flow north into the existing storm network that runs along South Pine, south of Naturita Street. A graphic showing these suggested improvements is shown in [Figure 5-5 in Appendix A](#).

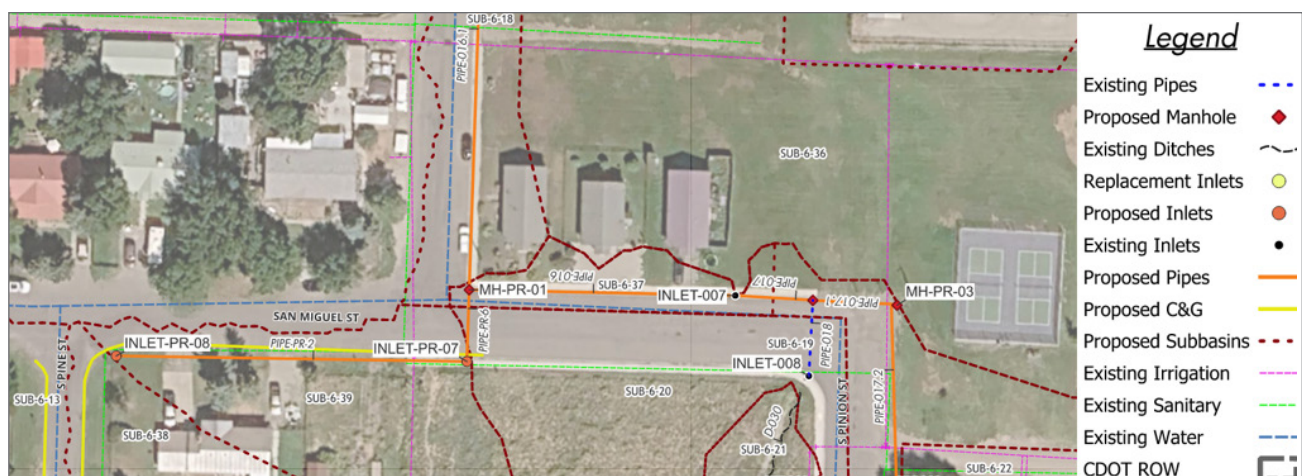


Figure 8-2: Proposed Improvements at Pine and Pinion

8.1.1.2 Cedar Street north of Grand Ave

Cedar Street North of Grand Avenue serves as a major collection point for stormwater drainage. This area is subject to localized flooding as it feeds into one of the primary discharge points north of town. Suggested improvements include piping the open ditch along the west side of Cedar Street, adding curb and gutter to better direct the flow, installation of an irrigation diversion structure, upsizing existing pipes to expand stormwater conveyance through this system, and the installation of new storm inlets on the south side of the intersection at Summit Street.

Major improvements were completed by Town staff in the summer of 2026. Improvements were previously described in Section 4.2. Final surveying and engineering evaluation of those improvements should be completed prior to expanding the downstream system to ensure adequate conveyance. If capacity is found to be inadequate, then options such as installing CIPP lining to the CMP may reduce the n-value from 0.024 to a value as low as 0.012 which may drastically increase pipe capacity. Any recommendations presented in this document are based on initial survey data captured in August 2025.

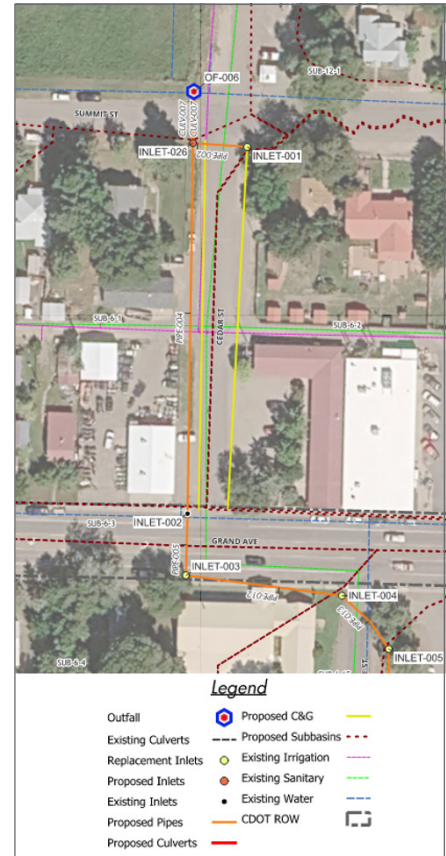


Figure 8-3: Cedar Street Proposed Improvements

8.1.1.3 Lucerne and Market Street areas at Summit Street

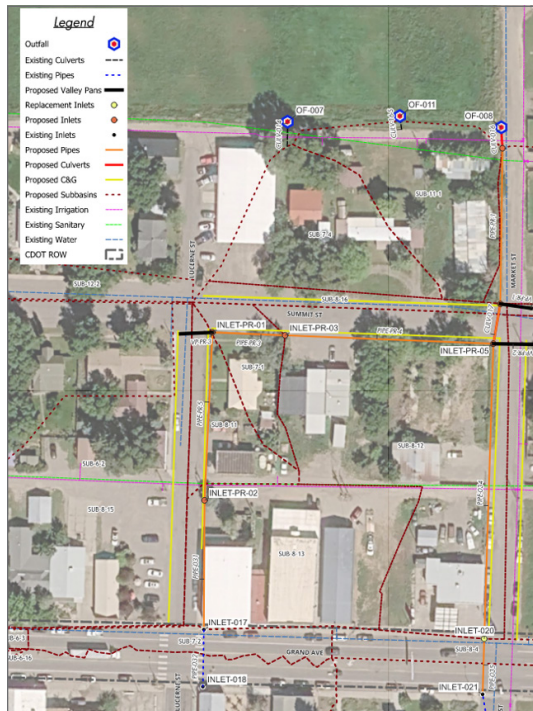


Figure 8-4: Proposed Improvements on Lucerne and Market Streets

The intersection of Market Street and Summit Street experiences localized flooding. This is most likely due to low areas adjacent to the intersection and diminishing downstream conduit sizes. This system also serves as a possible bypass to reduce the Town's impact on flooding along the east side of the existing fire protection district building, where current drainage pathways are routed through a small earthen ditch.

A summary of improvements includes the addition of five inlets on Lucerne, Summit, and Market streets, the addition of curb and gutter, the addition of a piped system to move water from Lucerne, bypassing the historic drainage pathway adjacent to the fire protection district building, discharging to the east along Summit, and ultimately tying into the Market Street system. Moreover, improvements include adding valley pans at the intersection of Summit and Market. Suggested improvements in this area are detailed in [Figure 5-6](#) in [Appendix A](#).

These proposed improvements are contingent on the Town acquiring a written agreement with the landowner to the north of Town, who currently receives existing drainage, to alter the current discharge patterns and to receive the increased flows the proposed system would produce. A summary of existing and proposed drainage peak discharges at system outfalls, where stormwater to a limited level leaves the Town's infrastructure, can be found in [Table 7](#) in [Appendix A](#).

8.1.2 DETENTION POND IMPROVEMENTS

The Town has historically relied on individual property owners and/or subdivision owners to develop detention ponds to serve individual properties or subdivisions, as the Town lacks a core drainage collection and regional detention system. This has resulted in many small ponds, some of which are of limited functionality. Future development, particularly in the southern portions of Town, may consider a larger regional detention pond to provide ample storage volume while minimizing the loss of real estate.

Four of the existing ponds are on town-owned parcels, and two are on third-party-owned land, according to San Miguel County GIS data. One pond is on library-owned property, and the other is on property at the existing medical complex.

Recommendations for detention pond improvements are limited because the ponds were designed to various design standards and were previously approved by the Town. It is unlikely that the Town could retroactively require the ponds to be modified to current design standards, nor would it be physically possible in most cases. Therefore, the principal recommendations for ponds are to:

- 1 Regularly clean sediment deposits and excess vegetation,
- 2 Construct reinforced overflow conveyances for ponds that do not currently include this feature, and
- 3 Consider regional detention ponds in developments north and south of town.

∴ **Cleaning ponds is necessary to maintain the designed volume and provide the designed detention function. Many ponds experience sedimentation during construction and before paved surfaces or site vegetation are established.**

Still, this soil has generally not been cleaned out after the site was stabilized. This results in a reduction in pond volume, providing less detention volume than designed, and may cause the design storm to overtop rather than be detained as intended. In general, small ponds are more sensitive to sedimentation-induced volume reduction.

A reinforced overflow conveyance, typically a riprapped spillway, provides a semi-controlled, erosion-resistant flow path if a storm exceeds a pond's design capacity or if the outlet structure becomes obstructed. Ponds without such a feature are susceptible to erosion of the pond berm during an overflow event, and overtopping could cause berm failure, release of a large volume of water over a brief period, and flooding of downstream properties. None of the stormwater ponds within the study area appear to have a reinforced overflow conveyance, as shown in [Table 8B](#) in [Appendix B](#).

Areas developed to the north of Town must accommodate existing flows and volumes while still providing adequate detention for new developments. Lastly, detention ponds may be considered for multi-purpose use and are not limited to simply holding water during rainfall. Examples include pocket parks, open spaces, combination detention/retention ponds, and outdoor recreational areas.

8.1.3 STORM DRAIN SYSTEM IMPROVEMENTS SUMMARY

A directory of tables detailing hydrologic and hydraulic changes which could result from implementing the proposed improvements is provided in [Table 8-1](#). Note that each table listed indicates whether the component was replaced. The inlets currently proposed for replacement meet the standards listed in [Table 7-2](#) and [Table 7-3](#); however, they will need to be replaced if the suggested improvements are adopted.

Proposed inlets are intended to alleviate localized ponding or serve as junctions at bends. Proposed manholes, listed in [Tables 8A and 8B](#) in [Appendix B](#), are intended to replace inaccessible miters or tees that could hinder maintenance. Upsized pipes and culverts, and suggested ditch improvements, provide additional hydraulic capacity and efficiency. It is worth noting that due to cover and elevation limitations, 7 pipes and 4 culverts fail to meet the minimum scour velocity (3 ft/s) criterion listed in Section 7.1. Due to limitations with this study's topographical detail, particular attention to these velocities should be considered if these improvements are adopted. Adherence to this criterion ensures that the pipes are effectively "self-cleaning," reduces maintenance, and therefore decreases the likelihood of a pipe or culvert becoming clogged with sediment.

NOTE: Implementation of the proposed improvements should be completed in phases, with downstream components prioritized. The improvements proposed herein are contingent on the functional and adequate capacity of the downstream, receiving network components. Improvement recommendations listed in [Appendix B - Table 8B](#) generally follow implementation priority for each network.

Prioritizing downstream components is integral to the proper functioning of all proposed improvements. Failure to prioritize downstream components may result in undesirable impacts including flooding and property damage.

[Table 8-2](#) provides a total for each component of quantities detailed in [Appendix B Table 8B](#). These values are provided only for conceptual-level planning. The locations of these improvements are shown in [Figures 5-1 through 5-6](#) in [Appendix A](#).

Table 8-1: Appendix B Proposed Improvements Directory

Component	Appendix B Table
Proposed Subbasins	1 B
Culverts (5-year)	2C
Culverts (100-year)	2D
Pipes (5-year)	3C
Pipes (100-year)	3D
Ditches (5-year)	4C
Ditches (100-year)	4D
Inlets (5-year)	5C
Inlets (100-year)	5D

Table 8-2: Proposed Improvement Quantity Summary

Component	Quantity
New Pipes	900 LF
Replacement Pipes	2073 LF
Replaced Culverts	456 LF
New Inlets	9
Replaced Inlets	5
Ditch Regrading	1342 LF
New Valley Pan	102 LF

8.1.4 MAINTENANCE OF EXISTING STORM DRAINAGE SYSTEM

Several culverts were clogged with sediment, and digging was required before survey data could be collected for the pipes. Due to the nature of development in the Town, the maintenance demands on Town staff for storm drainage infrastructure are significant. Elements such as limited curb and gutter, numerous earthen ditches, numerous culverts and drainage pipes, small private detention ponds, area drains in highly vegetated settings, and many more similar features create a maintenance burden.

Proper maintenance is essential to keeping the system functioning properly and at its highest capacity. This includes regular and periodic cleaning and repair of pipes, cleaning of detention ponds and associated piping, clearing and cleaning of grated inlets, cleaning and maintaining earthen ditches, and repair of curbs and gutters.

- The Town should implement a regular maintenance program for Town-owned infrastructure and inspect privately owned detention ponds.

The detention pond inspection program should allow the Town to perform or hire pond maintenance and to be reimbursed if the pond owner does not perform the required maintenance within six months of notification. Table 8-3 provides an example maintenance program. Consistent documentation of maintenance activities and asset conditions is a critical component of the program because it supports long-term budgeting, planned asset replacement, and eligibility for grant funding opportunities.



Figure 8-5: Clogged/Buried Inlets Along Pinion Street

Table 8-3: Example O&M Actions and Frequency

Stormwater Asset	Key O&M Actions	Recommended Frequency	Notes / Follow-Up
<u>Inlets and Catch Basins</u>	Remove sediment, trash, vegetation, and debris; check grates and inlet openings	Before/after major storm seasons and after large storm events	Record recurring problem locations and needed repairs
<u>Pipes and Culverts</u>	Inspect for blockage, sediment buildup, damage, or joint separation; clean as needed	Annually and after major runoff events	Prioritize locations with known backup or flooding issues
<u>Ditches and Channels</u>	Remove excessive vegetation, debris, and sediment; repair erosion or flow restrictions	Seasonally and after high-flow events	Coordinate with adjacent landowners where access or ownership is unclear
<u>Detention Ponds</u>	Inspect outlets, embankments, sediment buildup, vegetation, and erosion	Semiannually and after major storms	Track sediment accumulation and outlet condition

Stormwater Asset	Key O&M Actions	Recommended Frequency	Notes / Follow-Up
<u>Roadside Drainage</u>	Clear curb flowlines, swales, driveway culverts, and roadside conveyance paths	As needed	Track recurring ponding or nuisance flooding areas
<u>Program Management</u>	Maintain inspection logs, repair lists, photos, and annual maintenance priorities	Ongoing; review annually during budgeting	Supports budgeting, grant applications, and defensible maintenance records
<u>Street Sweeping</u>	Semi-annual street sweeping to reduce sediment buildup at inlets and culverts	Before/after major storm seasons and after large storm events	Preventative maintenance limits debris.

8.2 BUDGETARY COSTS FOR DRAINAGE IMPROVEMENTS

High-level budgetary costs were developed for proposed improvements to assist with capital planning, long-term goal identification, project feasibility, and project ranking.

8.2.1 FOCUSED PROJECT AREAS

Planning-level budgetary costs were developed for the three focused project areas identified in Section 8.1.1: South Pine Street between Paradox Avenue and San Miguel Street, Cedar Street north of Grand Avenue, and the Lucerne Street and Market Street areas near Summit Street. These areas were selected because they represent the most significant drainage constraints among the five outfall systems evaluated and because localized pipe or culvert upsizing alone would not fully resolve the observed system limitations.

Each focused project area includes multiple phases or work elements, such as curb and gutter installation, storm drain inlets, enclosed pipe segments, valley pans, ditch modifications, irrigation-related improvements, and downstream conveyance improvements.

- ∴ The preliminary costs are intended to support capital improvement planning, grant applications, project prioritization, and coordination with roadway, utility, irrigation, and private-property improvements.

The values should be considered planning-level budget estimates and refined during preliminary engineering, surveying, utility coordination, easement review, permitting, and final design.

- ∴ Because the focused project areas rely on downstream conveyance capacity, required downstream improvements must be completed before upstream or localized focus-area improvements are constructed. This sequencing is necessary to establish adequate hydraulic capacity, prevent newly collected or redirected runoff from exceeding downstream system capacity, and avoid transferring or worsening flooding on downstream properties.

- ∴ The recommended implementation approach is downstream-to-upstream, with each
- ∴ phase confirmed to have adequate receiving capacity before upstream collection,
- ∴ inlet, pipe, curb and gutter, or valley pan improvements are advanced.

Table 8B in Appendix B provides the detailed budget breakdown for the focused project areas. A summary table is provided below to allow project costs, phases, and implementation priorities to be inserted as the Town refines the capital improvement program.

Table 8-4 summarizes the focused project areas identified for planning-level implementation, including the primary work scope, required sequencing considerations, estimated budgets, and key dependencies for each project area. The table is intended to support comparison of the proposed improvements and preliminary budgeting while emphasizing that downstream conveyance improvements must be completed before upstream or contributing focus-area improvements are constructed to provide adequate hydraulic capacity and avoid increasing downstream flood impacts. The estimated budget for the South Pine Street & San Miguel Street to Paradox Street improvements includes all associated work required to complete the improvements south of Grand Avenue.

Table 8-4: Section 8.1.1 Focus Project Descriptions and Budgets

Project Area	Primary Work Scope	Sequencing Requirement	Budget	Key Dependencies
Cedar Street North of Grand Avenue Section of Outfall OF-006	Pipe the existing open ditch along the west side of Cedar Street; add curb and gutter to improve runoff collection and routing; install an irrigation diversion structure; upsize existing pipes; and install new storm drain inlets on the south side of the Summit Street intersection.	Complete required downstream conveyance improvements first to provide adequate downstream hydraulic capacity for increased collection and conveyance.	\$525K	Open ditch conversion, irrigation diversion requirements, utility conflicts, roadway drainage grades, and discharge capacity north of Town.
South Pine Street Improvements & San Miguel Street to Paradox Street Section of Outfall OF-006	Add curb and gutter along South Pine Street and connecting portions of San Miguel Street; install inlet improvements at South Pine/San Miguel; pipe stormwater east from the proposed inlet; and install a second inlet or junction structure to route flow north into the existing South Pine storm network. Includes riprap ditch improvements along Pinion adjacent to the library. Note: This budget includes all work to the south and east of INLET-003 on the south side of Grand Avenue at Cedar Street south to Paradox and Includes Improvements along South Pine (Paving, Curb and Gutter, and Inlets).	Complete required downstream conveyance improvements first to establish adequate hydraulic capacity and avoid transferring or worsening flooding on downstream properties.	\$1.2M	Roadway grades, curb and gutter tie-ins, inlet placement, and connection to the existing South Pine storm network. Contingent on completing Cedar Street North of Grand Avenue work first

Project Area	Primary Work Scope	Sequencing Requirement	Budget	Key Dependencies
Lucerne/Market/Summit Street Area Section of Outfall OF-008	Install five storm drain inlets on Lucerne, Summit, and Market streets; add curb and gutter; install a piped system from Lucerne Street; bypass the historic drainage pathway adjacent to the fire protection district building; convey flow east along Summit Street into the Market Street system; and add valley pans at Summit Street and Market Street.	Complete required downstream conveyance improvements first to establish adequate hydraulic capacity and avoid increasing downstream flood impacts.	\$1.8M	SWritten agreement from the downstream landowner north of Town to accept altered discharge patterns and increased flows.

8.2.2 DETENTION POND IMPROVEMENTS

Budgetary costs for the construction of overflow weirs are provided in [Table 8B](#) in [Appendix B](#) if the Town decides to retrofit the existing detention ponds as part of the maintenance of the Town's storm drainage infrastructure. The total cost of emergency overflow weir construction identified in this study is approximately \$13,300. The cost of annual maintenance is covered in Section 8.4.

Four of the existing ponds are on town-owned parcels, and two are on third-party-owned land, according to San Miguel County GIS data. One pond is on library-owned property, and the other is on property at the existing medical complex.

It may be that a portion of these costs, particularly the removal of sediment and vegetation from the ponds, could be completed or at least partially reimbursed by the pond owner(s). Ultimately, it is in the pond owner's interest to properly maintain the ponds, as they could be held liable if stormwater discharge causes damage to downstream properties, and it can be shown that the pond owner did not properly maintain the pond.

8.2.3 STORM DRAIN SYSTEM IMPROVEMENTS

Budgetary costs were developed for upgrading existing culverts, pipes, and inlets, and for new capital, or "special," projects that did not meet the major storm event design criteria. Storm drain system elements may or may not also meet the minor storm design criteria, but the most critical criteria were assumed to be the major storm design criteria due to the greater safety hazard and potential property damage posed by the higher flows associated with the major storm event. Storm drain system deficiencies may consist of not meeting the depth or spread of flow at the gutter, a function of inlet number and spacing, or surcharging of culverts and pipes to depths exceeding the design criteria, a function of the pipe size, slope, material, the flow intercepted by inlets, and the flow from upstream storm drain pipes.

The resolution of deficiencies in the storm drain system is not a straightforward design solution.

For example, adding an inlet to a storm drain system that captures additional runoff can cause a pipe section that is currently determined to be sufficient to become deficient because of the additional flow it must now convey; this impact can also be transferred to downstream pipes and receiving private property.

For example, adding an upstream inlet to a storm drain system may reduce bypass flow enough that a downstream inlet currently identified as deficient will meet the design criteria due to reduced gutter flow approaching the inlet; this impact can also be transferred to additional downstream inlets. Therefore, developing specific recommendations for the number and location of additional inlets, as well as for increased pipe sizes, requires detailed design analysis before construction of the improvement.

To develop reasonable budgetary costs for the storm drain system improvements, the following assumed costs were included for elements identified to be investigated:

- 1 The full value of one additional inlet for each proposed item or replacement inlet.
- 2 Pipes recommended for replacement were assumed to be replaced with a minimum pipe size of 18 inches for mains and 12 inches for laterals.
- 3 Culverts recommended for replacement were assumed to be replaced with a minimum pipe size of 12 inches.

Table 8B in Appendix B lists storm drain systems recommended for investigation and the associated budgetary costs for improvements. Total budgetary costs are approximately \$4,223,150 for high-priority storm drain system improvements, \$62,900 for medium-priority storm drain system improvements, and \$53,800 for low-priority storm drain system improvements. Total budgetary costs for the construction of storm drain system improvements are estimated at approximately \$4,340,000.

Additionally, Appendix G, Low Priority Replacement and Maintenance Recommendations, lists low-priority pipes that are performing adequately, provided they are clean and free of debris, and may be undersized relative to the analysis standards. We recommend upsizing these pipes as indicated if an improvement project is planned in the area. An overview of replacement and maintenance items is shown in Figure 6-1 of Appendix A.

Unit prices for developing preliminary budgetary costs are based on recent bid projects in the region and on actual block-to-block construction contract prices from area communities. Actual costs will depend on the project's scale, with smaller projects being relatively more expensive and larger projects relatively more cost-effective. Costs are in Present Value (2026) and do not include inflation factors for future construction. Budgetary construction costs were increased by 33% for professional design services, contractor fees, and contingencies. Detailed quantities and current prices should be used when developing design solutions.

8.2.4 MAINTENANCE OF EXISTING SYSTEM

A major necessity that would have substantial returns for little investment is to implement a regular maintenance program for Town-owned infrastructure and inspect privately owned detention ponds. The detention pond inspection program should allow the Town to perform or hire pond maintenance and to be reimbursed if the pond owner does not perform the required maintenance within six months of notification. Table 8-3 provides an example maintenance program. Consistent documentation of maintenance activities and asset conditions is a critical component of the program because it supports long-term budgeting, planned asset replacement, and eligibility for grant funding opportunities.

We recommend that the Town budget \$25,000 - \$50,000 annually for stormwater system labor, equipment, and materials. Due to delayed maintenance of the storm drain system, initial operations and maintenance costs will be high but will taper down once the key elements are restored. We believe annual operating costs may taper off to around \$10,000 as equipment needs decrease over time.

[Appendix G](#) lists low-budget maintenance items aimed at increasing the current systems' efficiency and functionality. Maintenance items include cleaning pipes, culverts, inlets, and the installation of angular, ungraded riprap (D50 = 6-8") along the eastern side of S. Pinion Street from San Miguel Street (alley) on the north to Kiwi Street (alley) on the south. The total estimated cost of Low-Priority Replacements and Maintenance is approximately \$62,900. [Figure 6-1](#) in [Appendix A](#) details the locations of suggested maintenance items and [Appendix G](#) details estimated costs.

8.3 TOTAL BUDGETARY COSTS FOR IMPROVEMENTS AND OPERATIONS

The budgetary cost summary is intended to establish an order-of-magnitude funding baseline for long-term stormwater planning, rather than a final construction estimate. The values presented should be interpreted as planning-level costs that can support budgeting, grant applications, capital improvement programming, and future rate or fee discussions. As individual projects advance, costs should be refined through survey, preliminary engineering, geotechnical evaluation, utility coordination, permitting review, and contractor input.

Because Norwood does not currently fund a dedicated stormwater management program, the preliminary program cost represents a new municipal service commitment rather than an incremental adjustment to an existing budget. Implementation will likely need to be phased based on available funding, project urgency, maintenance needs, and opportunities to coordinate stormwater improvements with roadway, utility, or development projects. **Establishing a dedicated funding mechanism would allow the Town to move from reactive drainage response toward a more predictable program for maintenance, pond improvements, capital projects, and regulatory compliance.** [Table 8B](#) in [Appendix B](#) provides detailed breakdowns of capital costs and pond improvements.

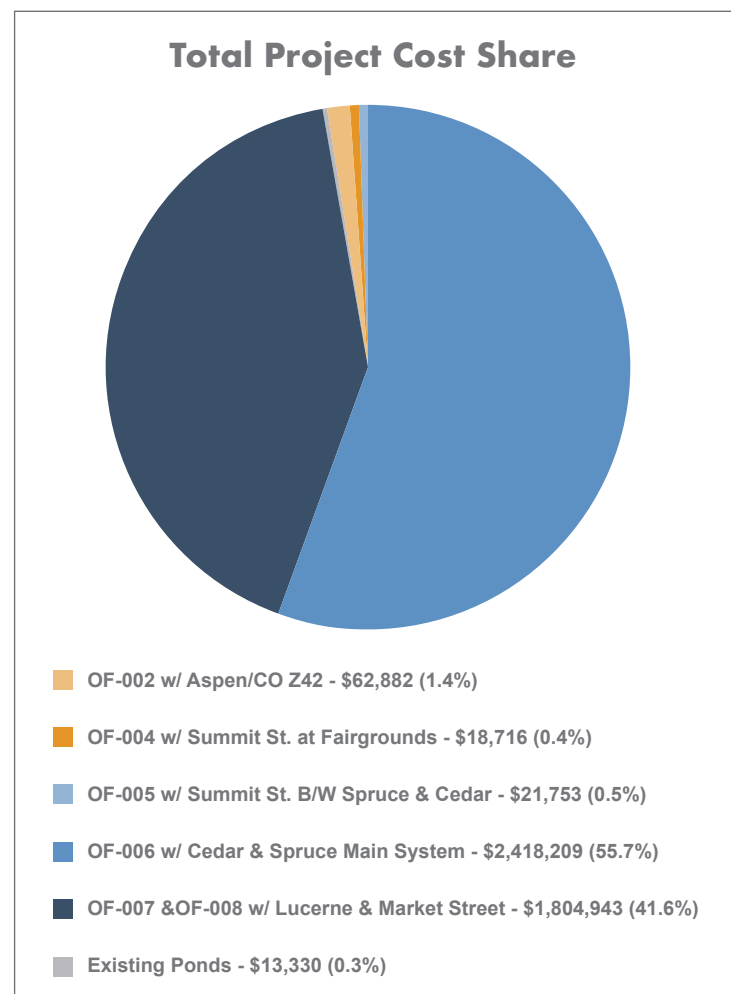


Table 8-5: Summary of Projected Stormwater Expenditures

Specific Capital Improvement Projects	Cost	Priority
Aspen / CO Z42 - OF-002	\$62,882.40	MEDIUM
Summit St. @ Fairgrounds - OF-004	\$18,715.76	LOW
Summit St. BW Spruce & Cedar - OF-005	\$21,753.48	LOW
Cedar & Spruce Main System - OF-006	\$2,418,208.66	HIGH
Lucerne & Market St - OF007 & 008	\$1,804,943.00	HIGH
Detention Ponds	\$13,300	LOW
TOTAL	\$4,339,803.30	
Annual Storm Drainage Maintenance		
Initial Reactive	\$50,000.00	
Annual Preventative	\$10,000.00	
Low-Cost Replacements and Maintenance		
Pipe and Culvert Maintenance	\$6,630.00	
Inlet Maintenance	\$500.00	
Swale Maintenance	\$15,000.00	
Pipe and Culvert Replacement	\$46,000.00	
TOTAL	\$68,130.00	



9.0 CONCLUSIONS AND RECOMMENDATIONS

This Master Drainage Plan provides the Town of Norwood with a practical framework for improving stormwater system performance, reducing localized flooding, and supporting long-term drainage infrastructure planning. The study identifies vulnerable portions of the existing drainage system, evaluates planning-level hydrologic and hydraulic performance, outlines recommended improvements, and establishes preliminary costs and priorities to guide future maintenance, capital improvements, development review, and funding decisions. Because the analysis was completed at a master-planning level, the recommendations should be used to guide decision-making and project development but should not be considered a substitute for detailed engineering, survey, utility coordination, permitting, easement review, or final design.

The recommendations in this section are intended to help the Town move from reactive stormwater response toward a more deliberate, phased, and sustainable drainage program.

Future implementation should consider updated rainfall information, emergency overland flow pathways, infrastructure ownership, downstream conveyance limitations, irrigation interactions, and the long-term maintenance needs of both public and private stormwater facilities. The Town should also maintain the stormwater inventory as a living GIS-based asset management tool, use a transparent prioritization framework to distinguish between project urgency and implementation readiness, and phase improvements from downstream to upstream so that new collection or conveyance improvements do not transfer flooding to unresolved downstream constraints.

SGM recommends that the Town consider **the following overall strategies** identified in this Master Drainage Plan:

Establish a proactive stormwater maintenance program for Town-owned drainage infrastructure, including routine inspection, cleaning, condition documentation, and annual maintenance budgeting.

Implement capital improvements in a phased downstream-to-upstream sequence so that downstream conveyance capacity, easements, and landowner agreements are addressed before upstream runoff collection is increased.

Adopt formal storm drainage design standards that require new development to manage both minor and major storm runoff and limit post-development peak discharge rates to historic predevelopment rates.

Use a transparent project prioritization framework that considers flood risk, public safety, infrastructure condition, maintenance burden, downstream dependency, constructability, funding opportunity, and implementation readiness.

Maintain and update a GIS-based stormwater asset inventory to support maintenance tracking, capital planning, emergency response, grant applications, and long-term infrastructure management.

Evaluate future regional detention opportunities as development occurs north, south, and east of Town, particularly where regional facilities may reduce reliance on small, privately maintained detention ponds.

Develop a long-term funding strategy that combines local budgeting, grants, development contributions, interagency coordination, project bundling, and, if appropriate, a dedicated stormwater funding mechanism.

Collectively, these actions would provide Norwood with a more resilient, maintainable, and financially sustainable stormwater program while preserving flexibility to refine individual projects as better survey data, funding opportunities, downstream agreements, and design-level analyses become available.

9.1 FUTURE DESIGN CONSIDERATIONS

Stormwater planning should account for uncertainty in future rainfall patterns, infrastructure performance, and system maintenance conditions. This Master Drainage Plan relies on the best available rainfall information at the time of analysis, including NOAA Atlas 14 precipitation data and locally applicable short-duration storm distributions. However, future updates to precipitation-frequency estimates, including NOAA Atlas 15 when available, may revise design rainfall depths and should be reviewed before advancing major capital projects to final design. If future rainfall estimates are higher than those used in this study, the Town should evaluate whether critical conveyances, roadway crossings, detention facilities, and emergency access routes require additional capacity, freeboard, or overflow protection.

Future design should also consider how stormwater moves through the Town during events that exceed the design capacity of pipes, culverts, ditches, and inlets. **Emergency overland flow pathways should be identified and preserved so that excess runoff can move through streets, drainage easements, ditches, or other designated routes without increasing risk to structures, emergency access, or downstream properties.** Particular attention should be given to areas where roadway crowns, shallow curb sections, low intersections, irrigation ditches, or private downstream receiving areas influence the direction and depth of flood flows.

∴ **Capital projects should verify that both the underground conveyance system and the surface overflow pathway function together under major storm conditions.**

Sediment and debris loading should also be incorporated into future design and maintenance decisions. The Town's system includes earthen ditches, roadside swales, irrigation features, detention ponds, and older culverts that are susceptible to sediment deposition, vegetation growth, and debris blockage. These conditions can reduce hydraulic capacity and may cause otherwise adequate infrastructure to perform poorly during storm events. Future designs should consider inlet protection, debris access, sediment forebays where appropriate, maintainable ditch sections, reinforced overflow paths, and pipe slopes or velocities that reduce sediment accumulation. Sediment and debris considerations are especially important where stormwater and irrigation flows are comingled or where runoff is conveyed through vegetated or unlined channels.

Because the Master Drainage Plan identifies drainage issues at a planning level, the modeling results are subject to limitations in detail and precision. The **following considerations** should be incorporated into future capital projects:

Capital projects should be implemented from downstream to upstream to avoid unanticipated damage and impacts.

Costs should be refined as actual design solutions are developed.

Basin boundaries should be verified to confirm the design flows to the suspect elements. Delineation of basin boundaries at this Master Drainage Plan level may have missed subtle boundary divisions that could affect the results.

A detailed hydraulic analysis and engineering design process should determine the actual sizes and locations of replacement culverts, storm drain pipes, and additional inlets. Culvert and storm drain pipe sizes and additional inlets are estimated in this study for preliminary budgeting purposes only. The feasibility of the recommended design size (e.g., adequate cover over the larger pipe, lack of conflict with other underground utilities, and similar) should be verified during the design process.

Ownership of storm drainage infrastructure should be confirmed to assess responsibility for maintenance and recommended upgrades. This may entail participation by other stakeholders to achieve the intended goal.

Irrigation and Drainage are comingled in areas and that must be included in future design. Two of the Town's major drainage pathways also serve as conduits to convey irrigation water. Proposed stormwater infrastructure developments shall provide due diligence in maintaining or bypassing irrigation flows along historic pathways.

9.2 PRIORITIZATION FRAMEWORK

The Town should consider using a transparent prioritization framework to guide project sequencing, annual budgeting, and grant applications. The priority assigned to each project should reflect more than hydraulic deficiency alone. Projects should be evaluated based on the severity and frequency of flooding, risk to structures or public safety, impact to emergency access and roadway function, known maintenance burden, condition of existing infrastructure, economic impacts, downstream dependency, constructability, permitting or easement complexity, opportunity to coordinate with other capital projects, and overall cost effectiveness.

A recommended prioritization approach is to score each project using categories such as hydraulic need, consequence of failure, maintenance benefit, implementation readiness, downstream dependency, and funding opportunity. Projects with high flood-risk reduction, clear downstream capacity, manageable ownership constraints, and strong grant or coordination opportunities should generally receive the highest implementation priority. Projects that are hydraulically deficient but dependent on unresolved downstream agreements, complex easements, or major utility conflicts may remain high need but should be classified as lower readiness until those constraints are resolved.

This framework would allow the Town to distinguish between urgency and readiness. For example, a project may be technically important because it addresses a significant flooding problem, but it may not be immediately implementable if downstream conveyance rights or easements are unresolved.

Conversely, a smaller maintenance or replacement project may be appropriate for near-term implementation if it is low cost, shovel-ready, and reduces recurring operational problems. Using a consistent prioritization framework will help the Town explain project decisions to elected officials, residents, funding agencies, and downstream stakeholders while maintaining flexibility as conditions and funding opportunities change.

9.3 ASSET MANAGEMENT AND GIS INTEGRATION

The Town should maintain the storm drainage inventory developed for this Master Drainage Plan as a living GIS-based asset management database. At a minimum, the database should include asset identification numbers, location, ownership, material, size, condition, inspection history, maintenance history, known deficiencies, replacement priority, photographs, and links to design or record drawings where available. Maintaining this information in GIS will allow the Town to track recurring maintenance issues, document completed work, plan future capital improvements, and support grant applications with defensible asset-condition information. **Over time, the inventory should be updated after maintenance activities, private development, capital projects, and field inspections so that the Town's stormwater records remain current and useful for budgeting, design, emergency response, and long-term infrastructure planning.**

9.4 TOWN STORM DRAINAGE SYSTEM INFRASTRUCTURE DESIGN STANDARDS

The Town's current storm drainage system design standards are limited and do not provide a complete framework for reducing downstream flooding risk as development continues.

The current code requires that the major storm runoff from the developed site be handled by the proposed infrastructure. Aside from this provision, storm drainage is currently evaluated on a case-by-case basis for proposed subdivisions. This standard alone will not sufficiently reduce peak flows and contributes to potential downstream flooding by failing to adequately maintain expected, existing flows.

A central recommendation of this MDP is that the Town require post-development peak discharge rates for both the minor and major design storms to be limited to historic predevelopment rates for all new development, not only subdivisions. Below are additional suggestions that we recommend for consideration in the next iteration of the drainage design standards:

- Detention pond outlet structure construction standards should identify materials, minimum elements, and details.
- Detention and retention basin vegetation standards and maintenance responsibilities are needed.
- Reward developers for land dedications of natural drainage areas. Provide density bonuses for the natural stream course area set-asides.
- Encourage disconnected impervious cover and emphasize using Low Impact Development practices to minimize runoff concentrations.

- Use the CUHP\MHFD basin sizing spreadsheet; minimum elements we recommend include Extend Detention Basin and WQCV volumes.
- The rational method's maximum basin size should be limited to 90 acres.
- If used, retention basins need to have a back-to-back 1% storm storage volume.

Appendix H provides recommended standards for new drainage infrastructure that are comparable to those used by other municipalities in the region and are intended to support a functional, maintainable, lower-maintenance stormwater collection and conveyance system.

9.5 DOWNSTREAM USERS' ACCEPTANCE OF INCREASED FLOWS

The proposed updates presented in this document are contingent on the Town brokering an agreement with the landowner to the north. Currently, this property is undeveloped and used for agricultural production, yet the landowner accepts a large amount of stormwater on their land. Changes in runoff rates and volumes will affect this landowner.

As detailed in Section 8.1, SGM recommends that a written agreement be brokered between the Town and the affected landowner prior to making any improvements to contributing infrastructure.

This agreement could acknowledge that the landowner would assume all associated indemnifications for receiving the increased runoff. Short of a contractual agreement between the Town and the landowner slated to receive the excess stormwater, an adequately sized detention basin would be required to emulate historic, existing maximum flow rates.

Potential areas for regional detention are limited and would incur significant expense to implement if the aforementioned agreement is not realized.

9.6 FUTURE DEVELOPMENT REGIONAL DETENTION PONDS

The Town has historically relied on individual property owners and/or subdivision owners to develop detention ponds to serve individual properties or subdivisions, as the Town lacks a core drainage collection and regional detention system. This has resulted in many small ponds, some of which are of limited functionality. Future development, particularly in the southern portions of Town, may consider a larger regional detention pond to provide ample storage volume while minimizing the loss of real estate.

Areas developed to the north of Town must consider passing existing flows and volumes while still providing adequate detention for new developments. **Detention ponds may be considered for multi-purpose use and are not limited to simply holding water during rainfall.** Examples include pocket parks, open spaces, combination detention/retention ponds, and outdoor recreational areas.

9.7 FUNDING MECHANISMS AND FINANCIAL IMPLEMENTATION

Implementation of the recommended stormwater improvements will require a combination of local funding, outside grants, cost sharing, and coordination with other infrastructure projects. Because Norwood does not currently operate a dedicated stormwater utility or enterprise fund, the capital and maintenance costs identified in this Master Drainage Plan represent a new or expanded municipal funding need.

- ∴ Establishing a predictable funding mechanism would allow the Town to move
- ∴ from reactive drainage response toward a planned program for maintenance,
- ∴ capital replacement, detention pond improvements, development review, and
- ∴ regulatory compliance.

Potential funding resources and implementation mechanisms are included in the following table.

Table 9-1: Funding Resources and Mechanisms

Funding Resource / Mechanism	Potential Use	Notes
<u>Annual General Fund allocation</u>	Routine maintenance, small repairs, inspection program, local match	Provides flexible local funding but may compete with other Town priorities.
<u>Dedicated stormwater fee or drainage enterprise fund</u>	Ongoing O&M, capital replacement, debt service, grant match	Provides the most predictable long-term funding source for a stormwater program.
<u>State and federal grants</u>	Major capital improvements, hazard mitigation, resilience projects, water quality improvements	May require local match, benefit documentation, and project readiness.
<u>DOLA or other local government assistance programs</u>	Infrastructure planning, design, and construction	May be well suited for small-community infrastructure needs.
<u>FEMA hazard mitigation programs, where eligible</u>	Flood-risk reduction, critical drainage improvements, resilience projects	Eligibility depends on documented risk, benefit-cost considerations, and program requirements.
<u>CWCB or watershed/flood mitigation funding opportunities</u>	Drainage, flood mitigation, watershed resilience, planning, or design support	May be applicable where projects reduce flood risk or improve drainage resilience.
<u>CDOT coordination along CO-145 / Grand Avenue</u>	Drainage improvements connected to highway right-of-way or roadway reconstruction	Requires early coordination where Town drainage interacts with CDOT facilities.
<u>San Miguel County or regional cost sharing</u>	Shared drainage corridors, downstream impacts, roadway-related drainage, regional facilities	Appropriate where benefits extend beyond Town-owned infrastructure.
<u>Developer contributions or drainage impact fees</u>	Growth-related conveyance, detention, or regional drainage improvements	Helps ensure new development contributes proportionally to increased drainage demands.

Funding Resource / Mechanism	Potential Use	Notes
<u>Private-property reimbursement or maintenance agreements</u>	Privately owned detention ponds, shared outfalls, easements, downstream conveyance	Useful where facilities are privately owned but affect public drainage performance.
<u>Project bundling with roadway or utility work</u>	Curb and gutter, storm drain replacement, valley pans, ditch improvements	Can reduce mobilization costs and limit repeated disturbance.
<u>Local financing or debt instruments</u>	Large capital projects or phased improvement packages	May be appropriate if paired with a dedicated repayment source.

The Town should review funding options annually as part of budgeting and capital improvement planning.

- ∴ **Projects with clear public safety benefits, downstream capacity, completed preliminary engineering, secured easements, and refined cost estimates will generally be more competitive for outside funding.**

Where projects alter discharge locations or increase runoff conveyed to downstream properties, funding plans should also account for legal agreements, easement acquisition, landowner coordination, and long-term maintenance responsibilities.



10.0 LIMITATIONS

This document was prepared solely for the Town of Norwood, Colorado, in accordance with professional standards at the time the services were performed and in accordance with the contract/Work Order between the Town of Norwood and SGM. This document is governed by the specific scope of work authorized by the Town of Norwood; it is not intended to be relied upon by any other party except for regulatory authorities contemplated by the scope of work. SGM has relied on information or instructions provided by the Town of Norwood, its own field investigations, and other parties, and, unless otherwise expressly indicated, has made no independent investigations as to the validity, completeness, or accuracy of such information except as noted herein.

This document was prepared as a Master Planning Document and is not intended to be and should not be considered or inferred to be design-level documentation. Any recommendations made herein are presented as planning-level recommendations. Advancing any recommendations for project implementation will require additional engineering analysis and design effort.



11.0 ACKNOWLEDGEMENTS

We would like to acknowledge the support and cooperation received from the Town of Norwood, its Town Manager, Sara Owens, and its Public Works Director, Randy Harris. Special thanks to all those who have contributed to the successful completion of this report. SGM Durango survey team; SGM GIS team, Steve Kirk, and Nick Treankler; and SGM civil engineering team, Catherine Carella, Eric Krch, David Rothwell, Sandra Rico, Cassidy Knab, Jevon Poston, and Diana Rooney.



12.0 ENGINEER'S CERTIFICATION

I, Eric Krch, a duly registered professional engineer in the State of Colorado (PE # 28583), have supervised the preparation of this Master Drainage Plan. To the best of my knowledge, the information included is accurate and consistent with professional practices in the State of Colorado.