



Development Review Committee Agenda

Wednesday, August 12, 2026

80 S Main Street in the Council Chambers at Library Hall

Development Review Committee Agenda

PUBLIC NOTICE is hereby given that the Development Review Committee of Spanish Fork, Utah, will hold a meeting in the City Council Chambers at Library Hall, on the second floor, 80 South Main Street, Spanish Fork, Utah, commencing at 10 a.m. on August 12, 2026.

SPANISH FORK CITY does not discriminate on the basis of race, color, national origin, sex, religion, age or disability in the employment or the provision of services. The public is invited to participate in all Spanish Fork City Development Review Committee. Meetings located at the City Council Chambers at Library Hall, 80 South Main Street, Spanish Fork. If you need special accommodation to participate in the meeting, please contact the Community Development Office at 801-804-4580. This meeting is not available to attend virtually.

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5. Master Plan Update

5.1 Storm Drain Master Plan Update

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This proposal is an update to the Storm Drain Master Plan to match the newly adopted Land Use Element of the General Plan. [Final Spanish Fork SDMP 8-3-2026.pdf](#) 

6. Discussion Item

6.1 Northeast Bench Sewer Line

7. ADJOURNMENT

Draft Minutes
Spanish Fork City Development Review Committee
80 South Main Street
Spanish Fork, Utah
August 5, 2026

Staff Members Present: Cory Pierce, Public Works Director; Seth Perrins, City Manager; Dave Anderson, Community Development Director; Brandon Snyder, Senior Planner; David Mann, Senior Planner; Vaughn Pickell, City Attorney; Joshua Nielsen, Assistant City Attorney; John Little, Chief Building Official; Byron Haslam, Assistant City Engineer; Marcie Clark, Engineering Department Secretary; Jered Johnson, Engineering Division Manager; Jared Bartel, Information Systems Director Kevin Taylor, Senior Power Utility Planner; Jake Theurer, Power and Light Superintendent; Bart Morrill, Parks Maintenance Supervisor; Bryton Shepherd, Landscape Architect; Jason Turner, Fire Marshal; Bronco Hunter, Parks and Recreation Director, Dillon Muirbrook, Traffic Engineer.

Citizens Present: Rich Smith, Cindy Smith, Andrade Christensen, Gavin West, Ashley Christiansen.

Cory Pierce called the meeting to order at 10:00 a.m.

MINUTES

July 29, 2026

Jake Theurer moved to approve the minutes of July 29, 2026.

Jered Johnson seconded and the motion **passed** all in favor.

FINAL PLAT

ANDY RIDGE SUBDIVISION

Brandon Snyder presented the Final Plat for the proposed six-lot subdivision, noting that the application is consistent with the previously approved Preliminary Plat. He recommended approval based on the findings and conditions outlined in the staff report,

explaining that Conditions 3, 5, and 6 are carryover requirements from the Preliminary Plat approval, while Condition 7 was added by the City Council as part of the associated Zone Change approval. He concluded his presentation by inviting questions from the Planning Commission for staff or the applicant's engineer.

The discussion concluded with Cory Pierce asking whether the Engineering Department had any additional questions or comments. He noted that staff had been working with the applicant on the project for an extended period and acknowledged that, while several concerns had been identified throughout the review process, those issues have been addressed through the recommended conditions of approval.

Jered Johnson **moved** to approve the proposed Andy Ridge Subdivision Final Plat based on the following finding and subject to the following conditions:

Finding:

1. That the proposal is consistent with the City's General Plan Designation of Low Density Residential.
2. That the proposal is consistent with the Preliminary Plat.

Conditions:

1. That the Applicant meets the City's Development & Construction standards, zoning requirements, and other applicable City ordinances.
2. That the Applicant addresses any red-lines and pays any remaining fees prior to recording the plat.
3. That the Applicant complies with all environmental requirements as outlined in the Subsurface Soil Gas Sampling Report prepared by Wasatch Environmental.
4. That prior to building permits, the Applicant or Builder complete additional testing to make sure there are no vapors present. If no vapors are present, no other action will be required. If vapors are present, the Applicant or Builder will be required to install a vapor mitigation system. Or the Applicant or Builder can forgo testing and install a vapor mitigation system in each lot in the Andy Ridge Subdivision.
5. That the frontage of lot 5 on the cul-de-sac be improved, maintained, and treated like a front setback for purposes of landscaping, structures, fencing, and any other setback related requirements.
6. That lots 1, 2, 3, 4 and 6 be addressed and accessed only from the cul-de-sac and that Lot 5 be addressed off of its current address on Quail Run Road.

7. That a 6-foot-tall privacy fence is required along the rear lot line of Lot 4 prior to a Certificate of Occupancy being issued for a home on that lot.

Seth Perrins **seconded** and the motion **passed** all in favor.

CONCEPT REVIEW

RIVER BOTTOMS RECEPTION AND EVENT CENTER

Rich and Cindy Smith presented their proposal to establish a unique greenhouse-style event and reception center within the community. They explained that they had been searching for a suitable property in southern Utah County and believe the site is well-suited to their vision, despite requiring improvements and modifications. The applicants noted that their goal is to create a venue unlike any currently available in the area and expressed their enthusiasm for the project. Acknowledging their limited development experience, they requested feedback from staff on the proposal and guidance on the steps necessary to move the project forward.

Seth Perrins welcomed the applicants as they are former neighbors to him. He inquired if they plan to retain the existing home on the property.

Mr. Smith explained that retaining the existing home on the property would significantly reduce the amount of new construction required for the project. He stated that the primary construction would focus on the main event gathering space, while the existing residence would be remodeled to support event operations. He noted that the home's existing kitchen and interior layout could be adapted to provide bridal suites, family gathering areas, and other amenities. Mr. Smith also shared his vision for creating comfortable, themed spaces where wedding parties and guests could relax before or during events, enhancing the overall experience for those using the venue.

Mr. Perrins clarified that this would be to repurpose the home in conjunction with the event space and not to be utilized as a caretaker residence.

In response to Mr. Perrins' concerns, Mr. Smith explained that the existing home would be repurposed to support the event center rather than serve as the primary event space. He stated that the residence would provide functional amenities such as a kitchen, storage, restrooms, and support areas, minimizing the need for demolition and new construction. Mr. Smith emphasized the applicants' desire to preserve the property's mature landscaping and natural character while incorporating only the parking and site improvements necessary to meet operational and regulatory requirements.

Byron Haslam stated that his engineering comments would be uploaded to Citizenserve for the applicants' review. He explained that both adjoining roadways are classified as collector roads, resulting in more extensive frontage improvements than would typically be required. Mr. Haslam also noted that two irrigation ditches traverse the property and advised that coordination with the irrigation company would be necessary as part of the development process.

In response, Mr. Smith explained that the existing residence would be repurposed to support the event center by providing amenities such as a kitchen, storage, restrooms, and other operational spaces, reducing the need for extensive new construction. He emphasized the applicants' commitment to preserve the property's mature landscaping and natural character while limiting site improvements to those necessary to meet operational needs and applicable development requirements.

Dave Anderson explained that the feasibility of the proposal would largely depend on the site's ability to accommodate the required parking and anticipated improvements to River Bottoms Road. He noted that the property's unique location and characteristics could support consideration of a non-residential zoning designation, despite its proximity to residential neighborhoods. Mr. Anderson recommended that the applicants first develop a more refined site plan addressing parking, roadway and sidewalk improvements, and other engineering requirements before submitting a Zone Change application. He also advised that a masonry wall would be required along the western property boundary adjacent to neighboring residential properties.

Cindy Smith acknowledged that the applicants had received and reviewed the staff comments identifying the primary site constraints.

Staff discussed the City's hillside development standards, emphasizing the importance of preserving the natural hillside and existing vegetation. Seth Perrins stated that protecting the integrity of the hillside is a significant priority for the City. Rich Smith acknowledged the property's development challenges, noting that he had walked the rear portion of the site and observed the dense vegetation and mature trees. Dave Anderson added that the City generally discourages development that would compromise hillsides or require the removal of existing vegetation. Byron Haslam further noted that the City would seek to preserve the public right-of-way and identify any necessary roadway improvements as early as possible in the development process.

Brandon Snyder asked whether staff was recommending that the applicants prepare a more detailed concept plan illustrating parking, roadway improvements, and other site requirements before pursuing a Zone Change application, allowing them to better evaluate

the project's feasibility before making a significant financial investment. Mr. Anderson agreed with this approach, explaining that a refined concept plan would provide the City Council with a realistic understanding of what could be developed on the property and assist in evaluating any future General Plan or zoning amendment requests.

Mr. Snyder also expressed concern that rezoning the property to a standard commercial designation could permit a broader range of commercial uses than intended. While acknowledging that an event center is a permitted use within the C-2 zone, he suggested that the C-O (Commercial Office) zone could also be considered because it allows less intensive commercial uses.

Mr. Smith asked whether the proposed use could instead be approved through a Conditional Use Permit, explaining that once the greenhouse event center is constructed, he believed the property would have little practical value for other commercial uses.

Mr. Anderson responded that a C-2 zoning designation combined with a Development Enhancement Overlay could provide an appropriate solution by tying the zoning approval to the specific proposed development, thereby limiting future uses to those approved through the overlay.

Mr. Perrins explained that while the concept review was intended to be a high-level discussion, future presentations to decision-making bodies would require a more realistic and refined representation of the proposed development. He advised the applicants to ensure that any conceptual renderings closely reflect what can ultimately be constructed, as decision-makers and the public will develop expectations based on the images presented.

Mr. Smith noted that this imagery is very close to what they envision for their development.

Mr. Perrins encouraged the applicants to engage an architect early in the design process to ensure the proposed concept is both realistic and financially feasible. Drawing from previous experience, he cautioned that significant differences between conceptual renderings and the final project can create challenges during the approval process. He emphasized the importance of developing accurate, achievable plans before seeking formal approvals to establish realistic expectations for both decision-makers and the public. Mr. Perrins noted that while the Development Review Committee can provide valuable guidance during the concept stage, there comes a point in the process where applicants must invest in professional design and engineering services.

Mr. Smith explained that they are not large-scale developers and that the project will primarily be undertaken by their family. While they recognize the need to budget for

professional services and development costs, they expressed hesitation to invest approximately \$10,000 before having greater confidence that the proposal is feasible.

Mr. Anderson advised that the applicants should expect to invest approximately \$5,000 to \$10,000 for a professional survey and preliminary engineering plans illustrating future curb lines, roadway improvements, and other required infrastructure. He explained that until this information is available, staff cannot adequately evaluate the proposal or determine its overall feasibility.

The discussion then turned to parking requirements. Mr. Perrins questioned whether the City's current parking standards for event centers always reflect actual demand, noting that parking needs often vary depending on the type and popularity of individual events. He observed that a successful venue may attract attendance beyond the minimum parking requirements.

Mr. Smith responded that the proposed site layout exceeds the City's minimum parking requirements and that approximately 60 parking stalls should adequately serve the facility. He added that additional parking could be incorporated into the design if necessary, although the current layout was intentionally designed to preserve the property's mature trees and landscaping.

Mr. Perrins noted that the City may consider the existing residence to be part of the overall event center, which could increase the parking requirement because the home would function as an integral component of the commercial use. Mr. Smith acknowledged that the existing residence contains approximately 3,500 square feet but maintained that, even when included in the parking calculations, the proposed 60 parking stalls should satisfy the City's requirements while still preserving the site's natural character.

Mr. Anderson advised the applicants that investing in professional engineering and design services early in the process would be beneficial. He explained that site improvements and utility infrastructure often cost significantly more than anticipated and emphasized the importance of preparing a detailed site plan and cost estimate to fully understand the project's feasibility before pursuing a formal Zone Change and City Council approval.

Mr. Smith acknowledged the value of staff's recommendation and stated that the applicants had already begun consulting with professionals to develop preliminary cost estimates for site improvements, including parking, utilities, and other infrastructure. He added that they were generally comfortable with those estimates but requested additional clarification regarding the site's electrical service requirements and the anticipated power connection.

Mr. Theurer explained that new developments are typically required to provide electrical infrastructure that serves both the project and allows for future system extensions. He noted that the final electrical design would depend on the project's service demand and load requirements, and that required street improvements, including street lighting, would also need to be incorporated into the overall utility design.

Mr. Smith stated that the current property owner had provided very limited information regarding the existing electrical service. Staff noted that if the project's electrical demand requires an increase in service capacity, the power infrastructure would need to be upgraded to accommodate the future development.

Mr. Snyder asked whether the existing residence includes a basement, and the applicants confirmed that it does. Mr. Snyder then inquired about access spacing requirements due to both adjoining roadways being classified as collector roads. Byron Haslam confirmed that Engineering would require the proposed access spacing to be clearly shown on the site plans during the review process.

Ms. Smith asked what the minimum distance would be from the intersection to the first driveway entrance. Mr. Haslam explained that the first access point must be located at least 125 feet from the intersection. Mr. Smith responded that the proposed concept already exceeds that minimum requirement and noted that the design could be modified to provide additional spacing if requested. He added that the proposed looped circulation pattern provides efficient vehicle access while maintaining the site's functionality.

Cory Pierce asked whether the same spacing requirement would apply between the two proposed access points. Mr. Smith confirmed that the submitted design satisfies those requirements and reiterated that the spacing could be increased if Engineering identified any concerns during the review process.

Mr. Snyder asked Mr. Haslam whether the slope restrictions previously identified for the site would also apply to the proposed parking areas. He requested clarification on whether a slope analysis would be required to determine which portions of the property could be developed for parking improvements.

Ms. Smith explained that the property remains relatively level until the tree line near the roadway, noting that the proposed site layout was intentionally designed to avoid encroaching into the hillside slope area.

John Little advised that converting the existing residence from a residential to a commercial use would trigger additional building code requirements, including updated restroom facilities and compliance with ADA accessibility standards. Ms. Smith explained that the proposed greenhouse addition would include new public restrooms adjacent to

the garage and greenhouse, providing both standard and ADA-compliant facilities for men and women. Mr. Little recommended that the applicants work closely with an architect to determine the applicable ADA requirements, required restroom fixture counts, and fire sprinkler standards. Mr. Smith acknowledged that staff had already identified sprinkler requirements and understood that both the existing residence and the proposed structures would require fire sprinkler systems.

David Mann asked whether utilizing the basement of the existing residence for commercial purposes would result in additional costs. Mr. Little confirmed that any additional requirements would depend on the square footage and intended use of the space. Mr. Smith responded that the basement is proposed to be used for storage.

Mr. Anderson advised the applicants that, in addition to the previously discussed development costs, converting the property from a residential to a commercial use would also generate applicable impact fees. He suggested that staff prepare a preliminary estimate of those fees once the project's design and building square footage are more clearly defined. Staff briefly discussed the timeline for developing the estimate and noted that transportation impact fees would likely represent the most significant portion of the total cost.

Mr. Perrins asked about the current ownership of the property. The applicants explained that the property is held in a trust established by the late Delores Wagner and that her daughter is serving as the executor of the estate.

Mr. Perrins expressed support for the proposed use, noting that he believed it was a better fit for the property than previous development concepts. He explained that the site's unique characteristics also present significant development challenges and associated costs due to current engineering and development standards. He encouraged the applicants to retain an experienced engineer to help navigate the process, understand regulatory requirements, and accurately evaluate project feasibility. Mr. Perrins also advised that the property's development constraints should be considered during purchase negotiations, as the cost of required improvements may affect its overall value.

Mr. Smith thanked staff for their feedback and explained that the purpose of the concept review was to determine whether the proposal was viable before making a significant financial investment. He asked whether, from staff's perspective, the project appeared feasible if the applicants were willing to meet all applicable development requirements, noting that they wanted confidence in the concept before moving forward with the property.

Mr. Perrins responded that he believed the proposal had a reasonable opportunity for success, provided the applicants carefully addressed the feedback received during the concept review. He acknowledged that the proposal differs from the property's planned residential use, making it important for the applicants to clearly demonstrate how the project would enhance the community, preserve the surrounding neighborhood, and provide a compelling vision for the Planning Commission and City Council to consider.

Mr. Anderson commented that Mr. Perrins had provided a thoughtful response to a difficult question. He acknowledged that staff could not guarantee the proposal's ultimate success but agreed that the best path forward would be for the applicants to prepare a detailed plan demonstrating compliance with City standards. He explained that providing the City Council with a well-developed proposal would help them better evaluate the request and become more comfortable with the concept. Mr. Anderson noted that while the Council considers the property owner's vision and objectives, it also carefully weighs the concerns and input of neighboring property owners when making land use decisions.

Mr. Perrins encouraged the applicants to engage with neighboring property owners, particularly those most directly affected by the proposed development, early in the process. He recommended listening to their concerns and identifying ways to address potential impacts before the proposal advances through the public review process. He noted that traffic would likely be one of the primary concerns, especially given the property's proximity to the City's annual Festival of Lights. While the community is accustomed to the temporary traffic associated with the festival, he explained that an event center would generate traffic throughout the year, making it important to carefully evaluate and mitigate those impacts.

Vaughn Pickell commented on the proposed parking layout and expressed concern that larger events, such as weddings, could result in overflow parking along adjacent streets, potentially affecting neighboring properties and creating public safety concerns. Staff discussed the importance of providing sufficient on-site parking to minimize the potential for street parking.

Mr. Perrins concluded the discussion by stating that the City would not modify the established traffic routing for the Festival of Lights, as doing so could create unsafe conditions by allowing traffic to back up onto the highway. He emphasized that the existing route through the surrounding surface streets would remain in place for public safety reasons, and staff briefly discussed how traffic along River Bottoms Road would relate to the proposed development.

DISCUSSION ITEM 45:50

TRAILS MASTER PLAN

Dave Anderson opened the discussion by apologizing for concluding the previous meeting without asking if staff had any additional items to discuss. He then invited Bryton Shepherd to present an update regarding the proposed regional trail alignment and frontage improvements adjacent to the future Maverik development.

Mr. Shepherd noted that the current Trails Master Plan exhibit contains minor inaccuracies and is slightly outdated. Dillon Muirbrook explained that the exhibit still depicts bike lanes along Main Street, which are no longer included in the current Main Street reconstruction plans. He stated that the City's long-term vision is for the regional trail to extend north to approximately 900 North, where it will connect to the existing regional trail network near 1000 North. Mr. Muirbrook also explained that the red alignment shown on the exhibit represents a regionally significant trail and that the long-term plan is to extend the trail beneath Interstate 15 along Main Street, creating a continuous regional connection.

Staff discussed the preferred trail alignment through the interchange area. Mr. Muirbrook explained that while the trail could transition to either side of Main Street, the current design favors the east side because the airport trail segment has already been constructed and the interchange improvements were designed to accommodate the trail in that location. He stated that the existing pedestrian crossing is prioritized on the east side of the interchange and that continuing the trail along that alignment before transitioning west north of Interstate 15 would provide the most practical solution. Seth Perrins agreed with this recommendation.

Brandon Snyder asked whether the future Maverik development would ultimately require a signalized intersection. Mr. Muirbrook responded that, while a traffic signal is not currently planned, the intersection would be difficult to coordinate due to its proximity to the railroad crossing. Jered Johnson added that maintaining traffic flow would require careful coordination with the railroad. Mr. Muirbrook explained that the Transportation Master Plan has not identified the intersection as warranting a traffic signal based on existing traffic volumes; however, he acknowledged that Maverik is expected to be a significant traffic generator, and a future Traffic Impact Study could determine that a signal is warranted.

Staff also discussed potential signal placement, noting that the recent addition of a second northbound lane on Main Street has reduced the available right-of-way. The discussion further highlighted the high volume of pedestrian activity near Deer Creek, particularly

during lunchtime, and emphasized the importance of providing a safer and more connected shared-use trail system for pedestrians in the area.

Dillon Muirbrook shared preliminary public engagement results from the Trails Master Plan update. He explained that the City's consultant hosted a booth during Fiesta Days to gather feedback from approximately 250 participants in an informal setting. Mr. Muirbrook reported that trails were identified as the community's highest-priority infrastructure improvement, receiving more support than roadway expansions, roundabouts, or traffic signals. He noted that the feedback demonstrates strong public interest in the continued expansion of the City's shared-use trail network.

Cory Pierce agreed and he feels that this would be a very important trail connection.

Staff concluded the discussion by recognizing that the ongoing Transportation Master Plan and Active Transportation Plan provide an appropriate opportunity to evaluate regional trail connections, identify gaps in the existing trail network, and establish priorities for future improvements. Staff discussed the importance of coordinating trail planning with future roadway and railroad crossing projects, acknowledging that some improvements may require significant funding and regional partnerships. The Committee also discussed pursuing future Transportation Improvement Program (TIP) funding opportunities to advance priority trail projects as additional planning and design work is completed.

Dave Anderson moved to adjourn the meeting at 11:09 a.m.

Adopted:

Kasey Woodard
Community Development Division
Secretary



Nicolson Industrial Plat A
Final Plat
999 West 3800 North
7.46 acres
I-1 Zone
Industrial General Plan Designation



PROPOSAL

The Applicant applied for Final Plat approval to create a 3-lot subdivision at 999 West and 3800 North. The Preliminary Plat application was approved by the Planning Commission on July 1. A corresponding Site Plan will also be presented to the DRC for approval that shows the proposed layout of buildings on each lot. A cross-access easement is noted on the plat for parking areas and drive aisles.

STAFF RECOMMENDATION

That the proposed Nicolson Industrial Plat A Final Plat be approved based on the following findings and subject to the following conditions:

Findings

1. That the proposal conforms to the City's General Plan Designation and Zoning Map.
2. That a Preliminary Plat was approved by the Planning Commission on July 1, 2026.

Conditions

1. That the Applicant meets the City's development and construction standards and other applicable City ordinances.
2. That any remaining redlines are addressed prior to the plat being recorded and building permits being issued.

EXHIBITS

1. Area Maps
2. Final Plat

EXHIBIT 1

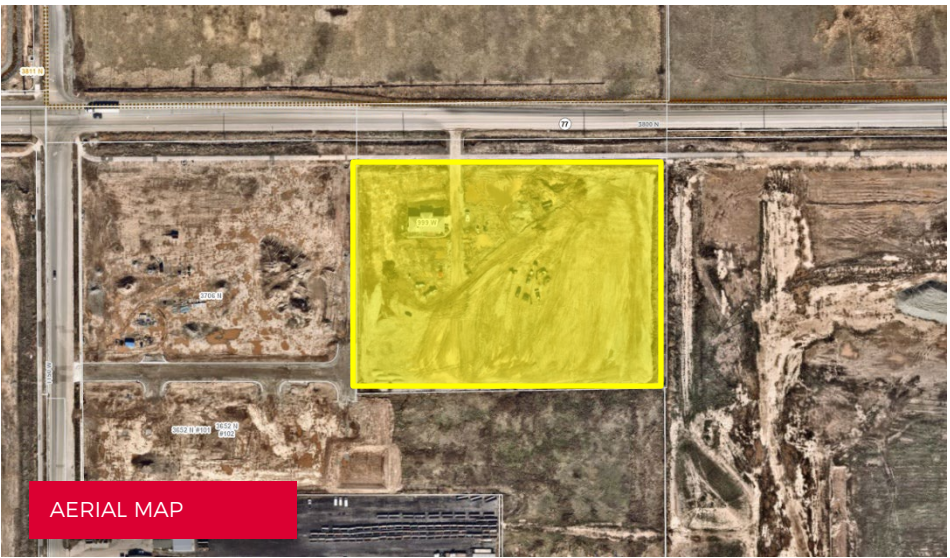
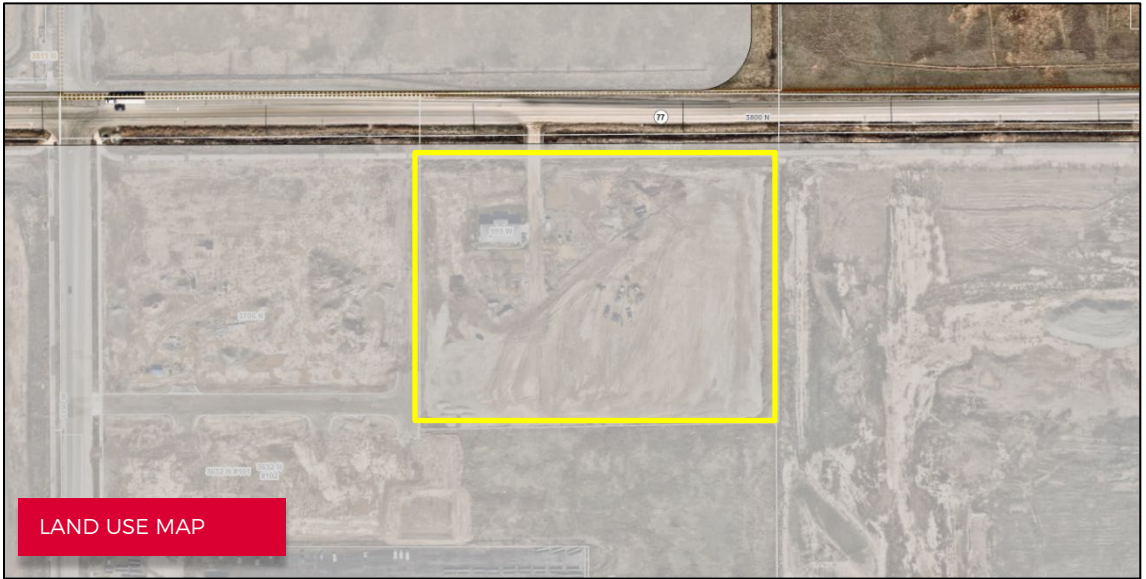
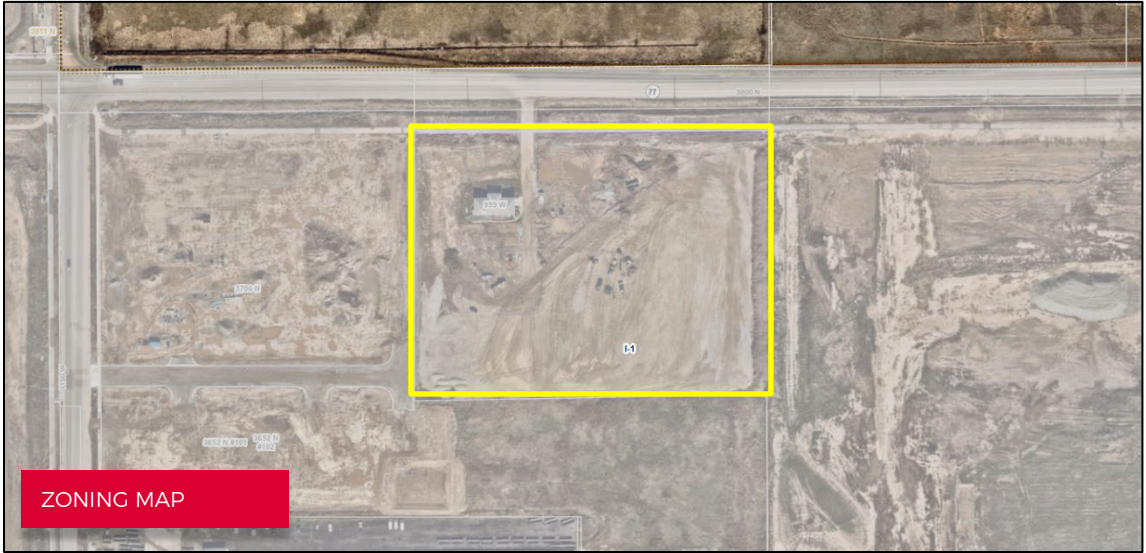
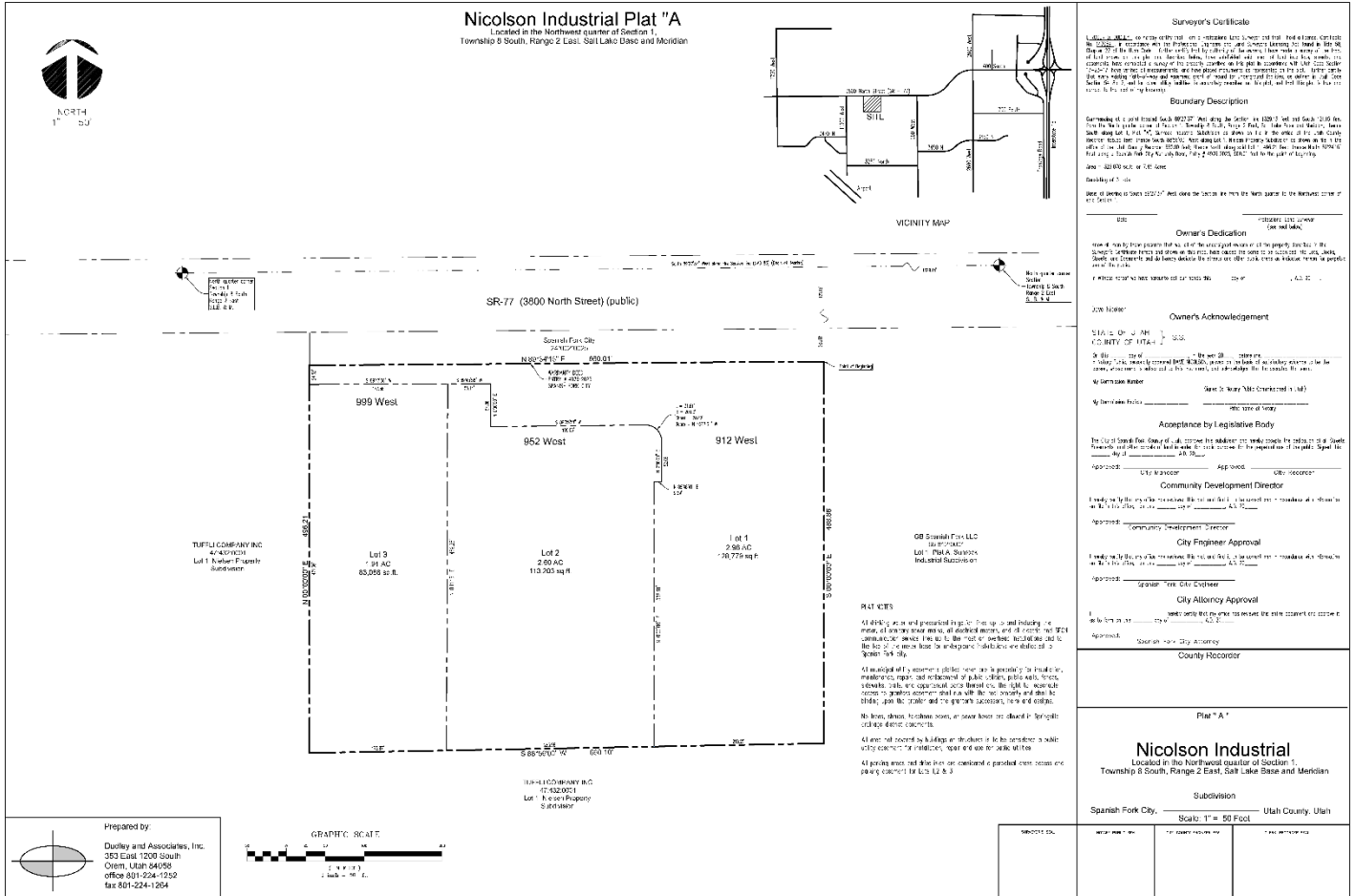


EXHIBIT 2





Nicolson Industrial Park
Site Plan
999 West 3800 North
7.46 acres
I-1 Zone
Industrial General Plan Designation



PROPOSAL

The Applicant applied for Site Plan approval to create an industrial park at 999 West and 3800 North. The project is shown in 3 phases that will create a total of 12 units with approximately 10,000 square feet of floorspace in each unit. UDOT granted approval for the access onto 3800 North, which will be the only access to this property. A cross-access easement is noted on the plat for parking areas and drive aisles.

STAFF RECOMMENDATION

That the proposed Nicolson Industrial Plat A Final Plat be approved based on the following findings and subject to the following conditions:

Findings

1. That the proposal conforms to the City's General Plan Designation and Zoning Map.
2. That a Preliminary Plat was approved by the Planning Commission on July 1, 2026.
3. That access to 3800 North was approved by UDOT on April 15, 2026.

Conditions

1. That the Applicant meets the City's development and construction standards and other applicable City ordinances.
2. That any remaining redlines are addressed prior to building permits being issued.

EXHIBITS

1. Area Maps
2. Final Plat
3. Site Plan
4. Landscape Plan
5. Elevations

EXHIBIT 1

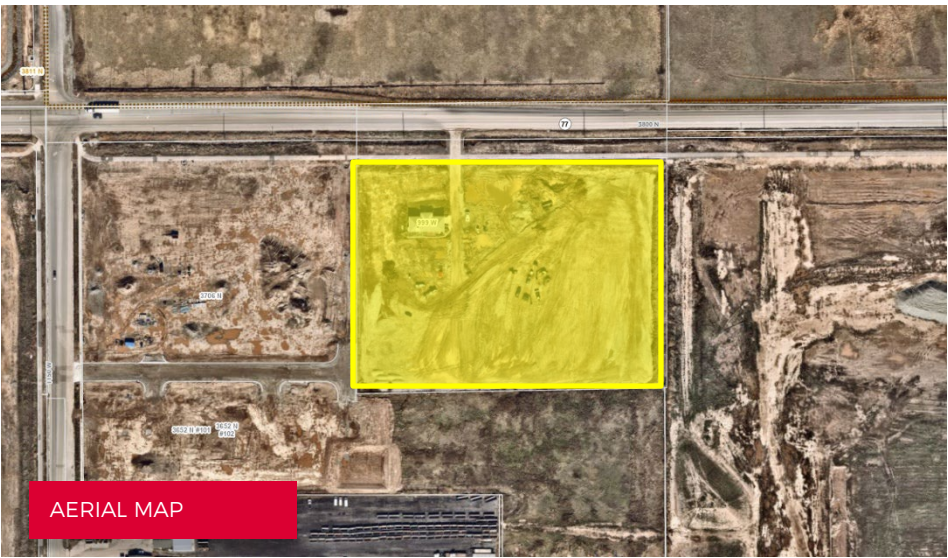
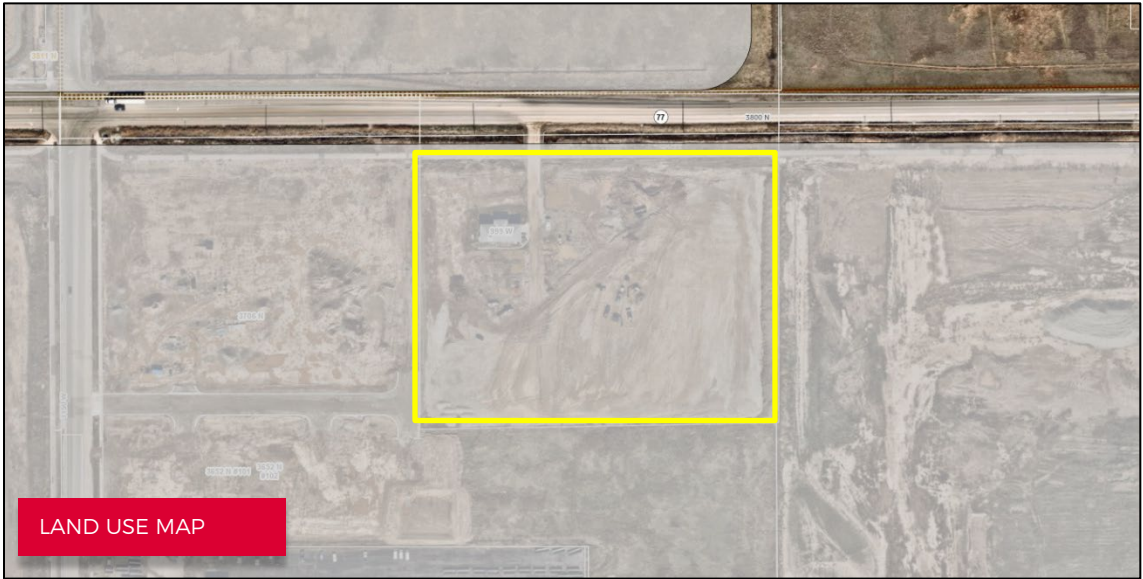


EXHIBIT 2

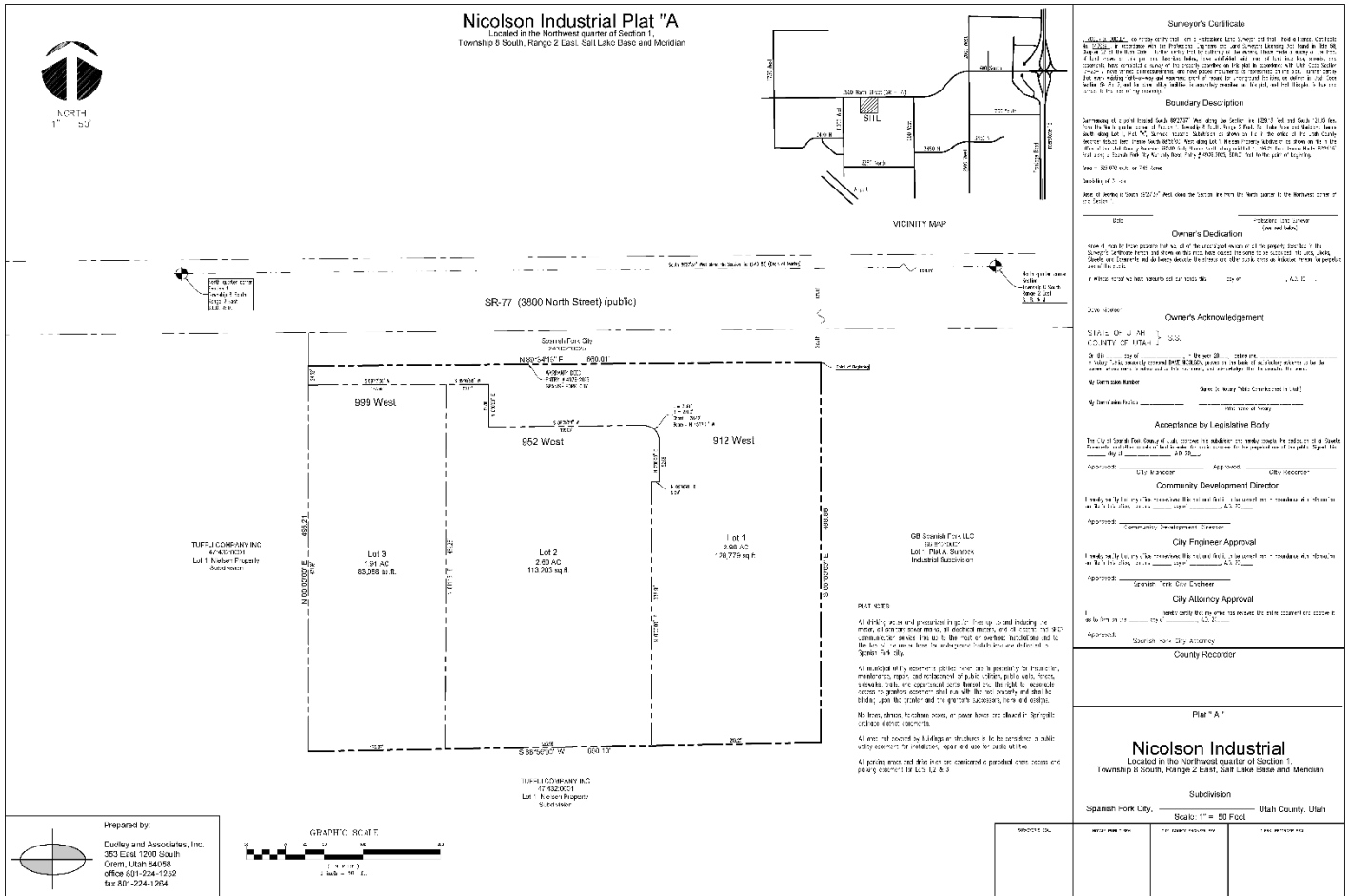


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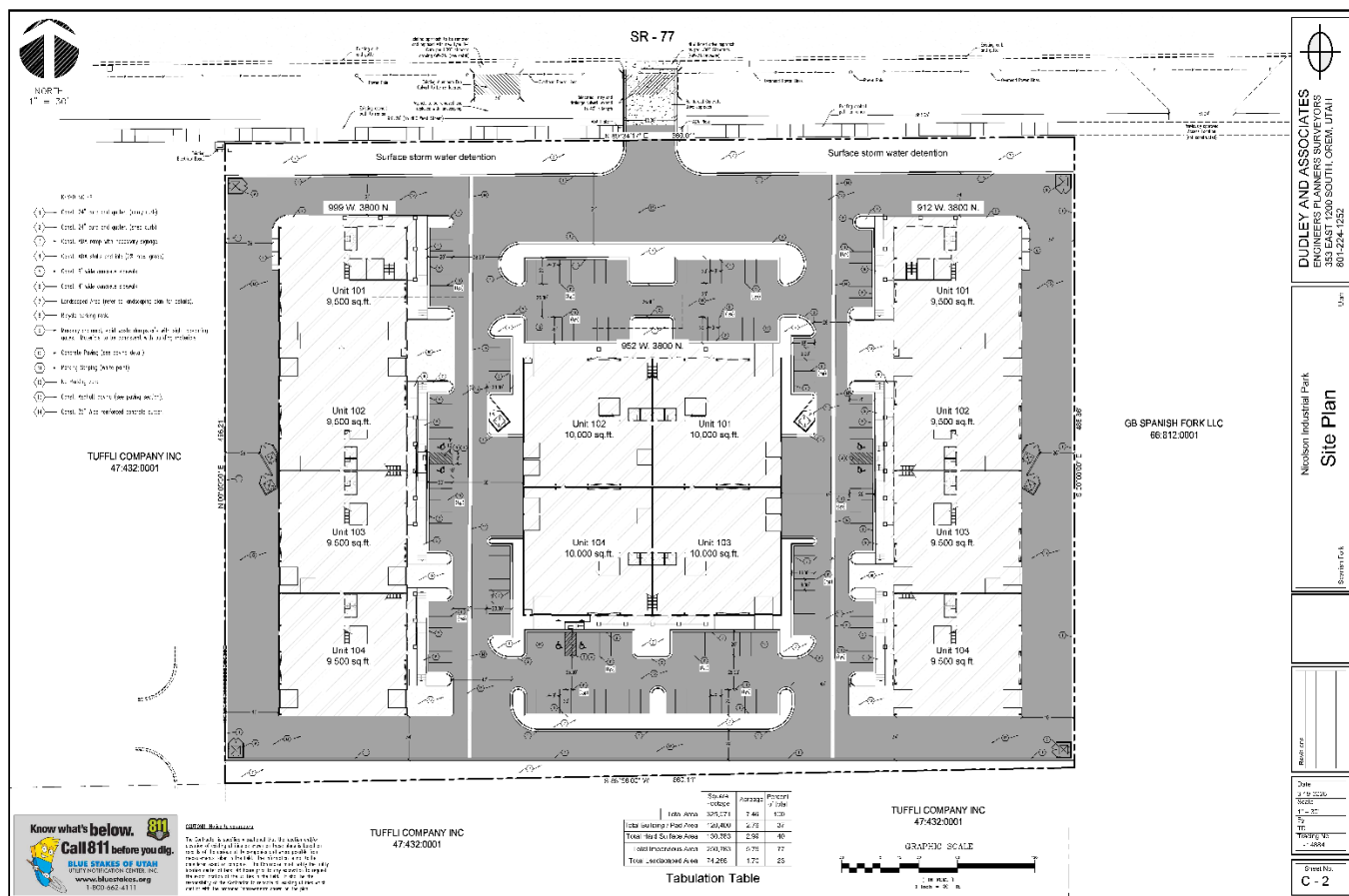


EXHIBIT 4

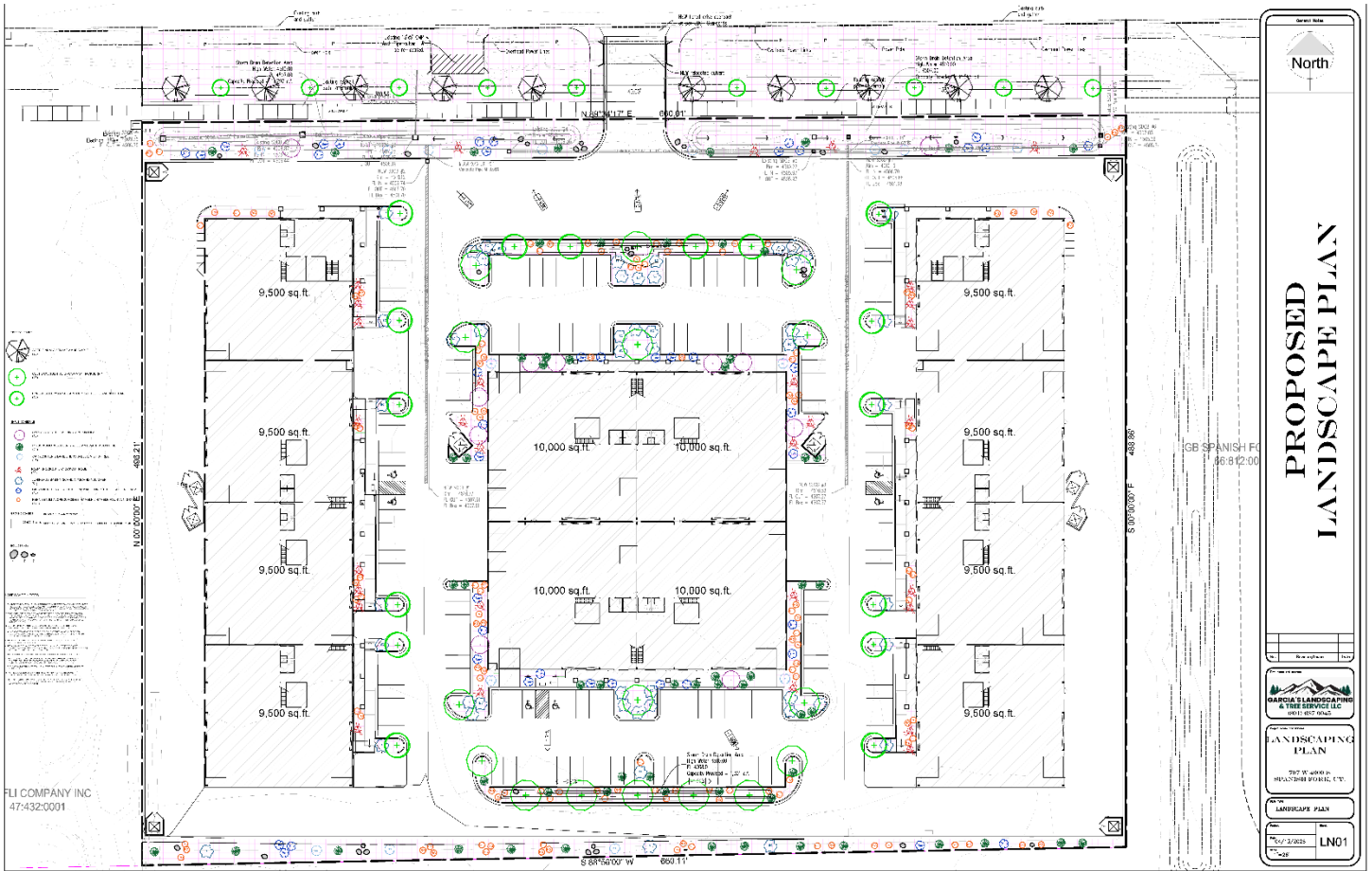
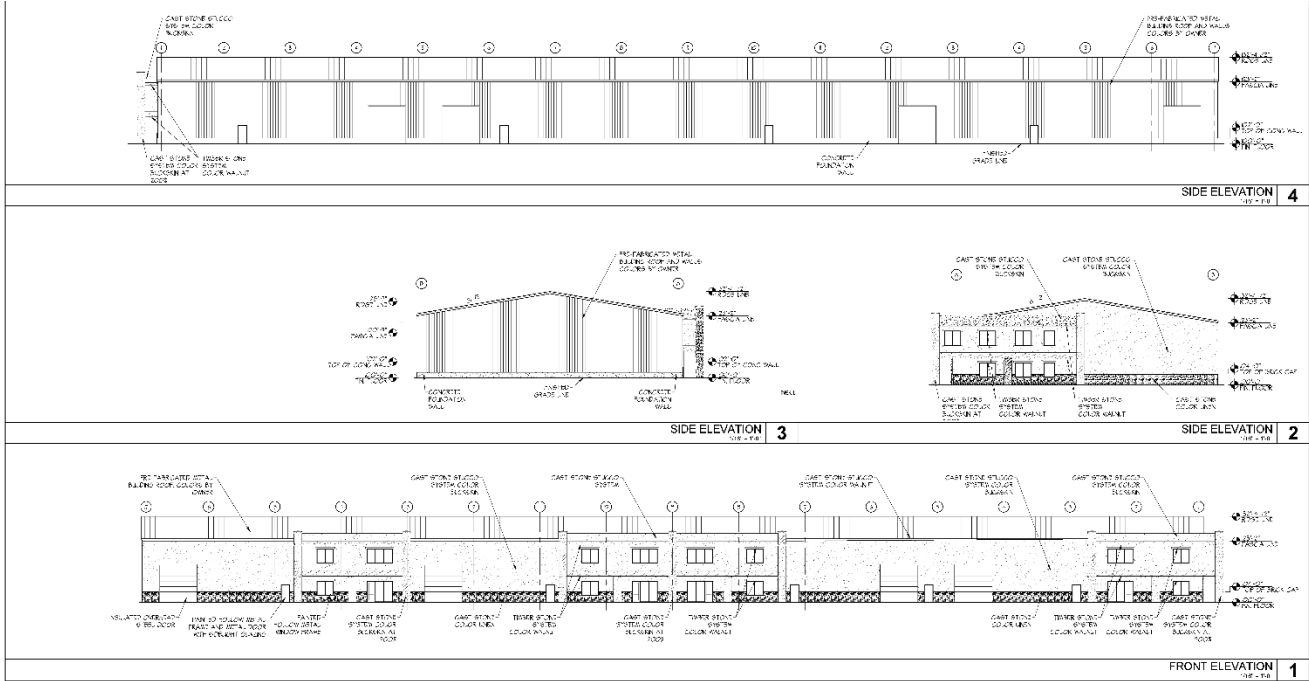
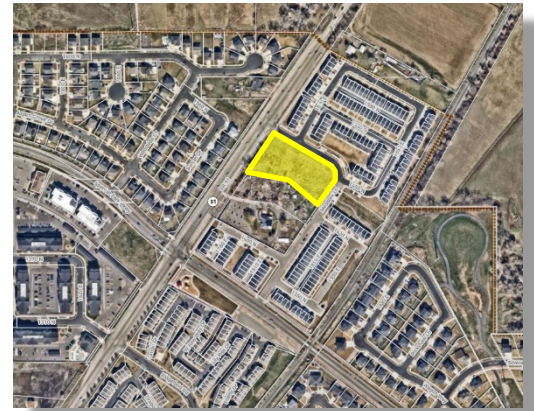


EXHIBIT 5





Meadow Subdivision Plat A
Preliminary Plat
Approx. 1425 North 1870 East
1.85 acres
R-1-6 Zone
High Density Residential General Plan
Designation



PROPOSAL

The Applicant applied for Preliminary Plat approval to create a 6-lot subdivision on property located at the intersection of Meadow Lane and SR-51. The subject property is heavily encumbered by an existing power line easement and right-of-way totaling 150 feet in width. An existing home is located on the adjacent property to the south and the remainder of the surrounding property is made up of the Meadow Creek Ridge townhome development. The current zoning of R-1-6 was established in 2014 as part of the larger Meadow Creek Ridge Preliminary Plat.

The Preliminary Plat shows 6 single family lots ranging in size from 10,509 square feet to 18,433 square feet in size. The majority of each lot will be front yard, due to the power corridor, with front setbacks ranging from 95-135 feet. The smallest building envelope is shown on lot 4 at just over 1100 square feet. Staff has made the Applicant aware of the landscaping requirements in the municipal code, which limit the front yard area to 60% impervious surface. Notes have been added to the plat stating that no access to SR-51 will be allowed on lots 1 and 2.

STAFF RECOMMENDATION

That the proposed Meadow Subdivision Plat A Preliminary Plat be approved based on the following finding and subject to the following conditions:

Finding

1. That the proposal conforms to the City's General Plan Designation and Zoning Map.

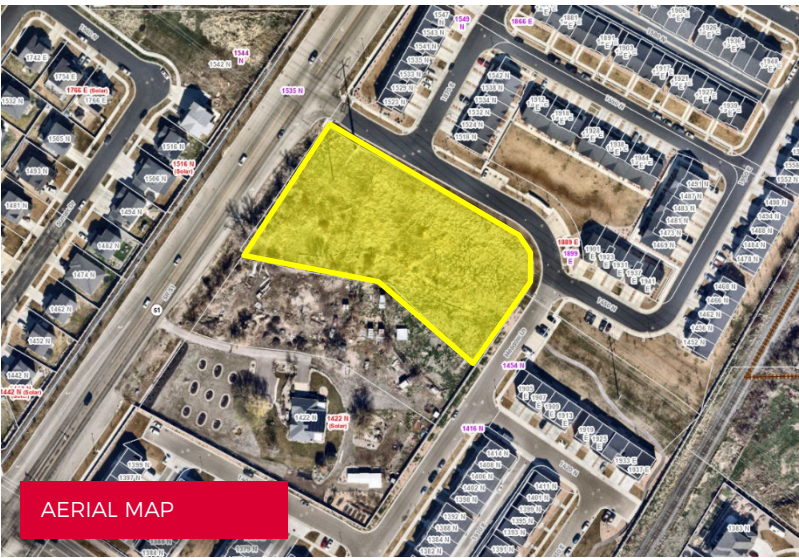
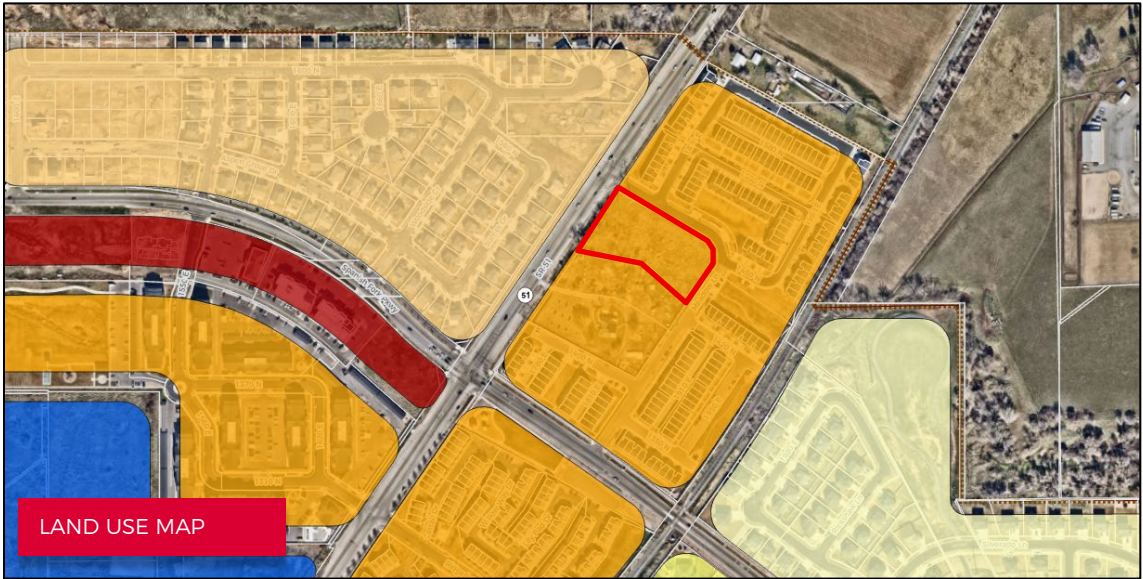
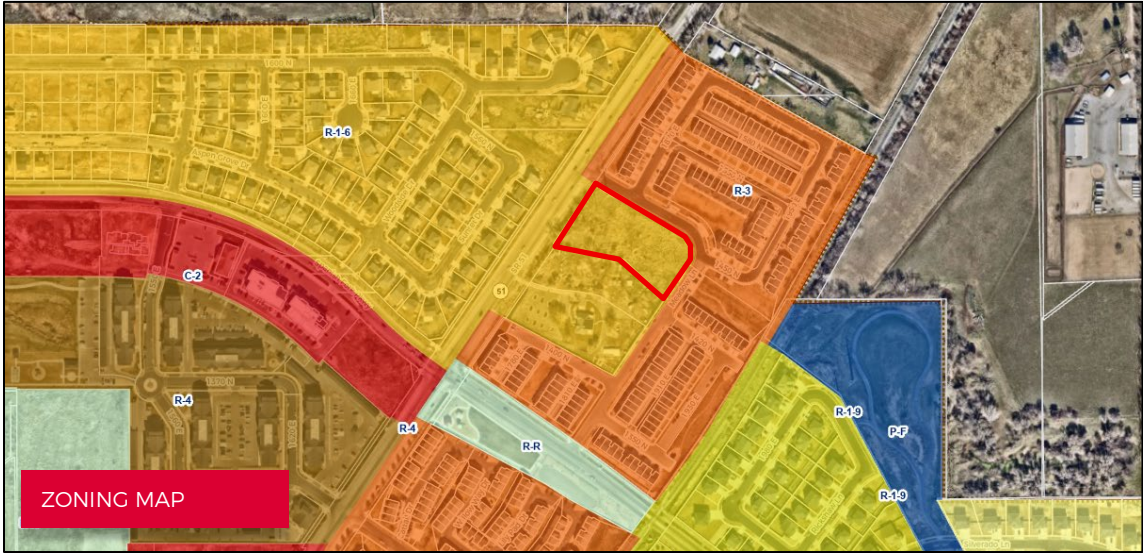
Conditions

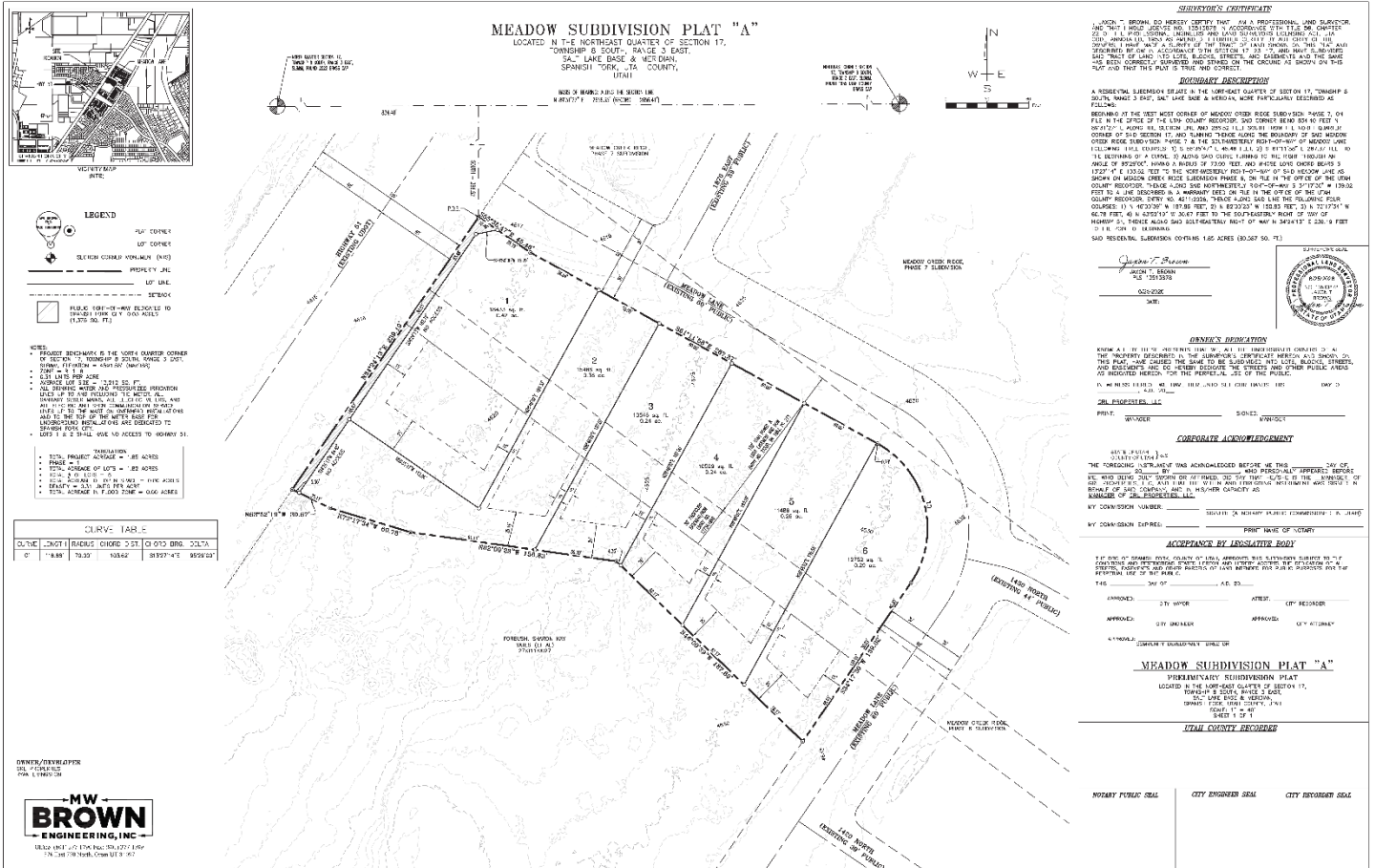
1. That the Applicant meets the City's development and construction standards and other applicable City ordinances.
2. That any remaining redlines are addressed prior to the plat being recorded and building permits being issued.

EXHIBITS

1. Area Maps
2. Preliminary Plat

EXHIBIT 1







STORM DRAIN MASTER PLAN

(HAL Project No.: 348.30.100)

July 2026

THE CITY OF SPANISH FORK

STORM DRAIN MASTER PLAN

(HAL Project No.: 348.30.100)



Kayson M Shurtz, P.E.
Project Manager



July 2026

ACKNOWLEDGMENTS

Several individuals contributed to the successful completion of this study. We sincerely appreciate the cooperation, assistance, and expertise provided by members of the project team:

The City of Spanish Fork
Cory Pierce, Public Works Director
Jered Johnson, Assistant Public Works Director
Byron Halsem, P.E., City Engineer
Josh Wagstaff, P.E., Assistant City Engineer

Hansen, Allen & Luce, Inc.
Gordon Jones, P.E., Principal in Charge
Kayson Shurtz, P.E., Project Manager
Ry Weber, M.S., Project Engineer

ABBREVIATIONS AND UNITS

AF	Acre-Foot
cfs	Cubic Foot Per Second (ft ³ /s)
CIP	Capital Improvement Plan
CMP	Corrugated Metal Pipe
Cont.	Contingency
CY	Cubic Yard
E	East
Eng	Engineering
ERU	Equivalent Residential Unit
FF	Farmer–Fletcher (1971) Storm Distribution
ft.	Foot
GIS	Geographic Information System
HAL	Hansen, Allen & Luce, Inc.
HEC	Hydrologic Engineering Center (U.S. Army Corps of Engineers)
HMS	Hydrologic Modeling System
ID	Identification
In	Inches
LID	Low Impact Development
N	North
NOAA	National Oceanic and Atmospheric Administration
NRCS	National Resources Conservation Service (formerly SCS)
RCP	Reinforced Concrete Pipe
s	Second
S	South
SF	Square Foot
SCS	Soil Conservation Service (now NRCS)
SSA	Storm and Sanitary Analysis
SWMM	Stormwater Management Model
TR-55	Technical Release 55 (NRCS 1986)
USGS	U.S. Geological Survey
W	West
yr	Year

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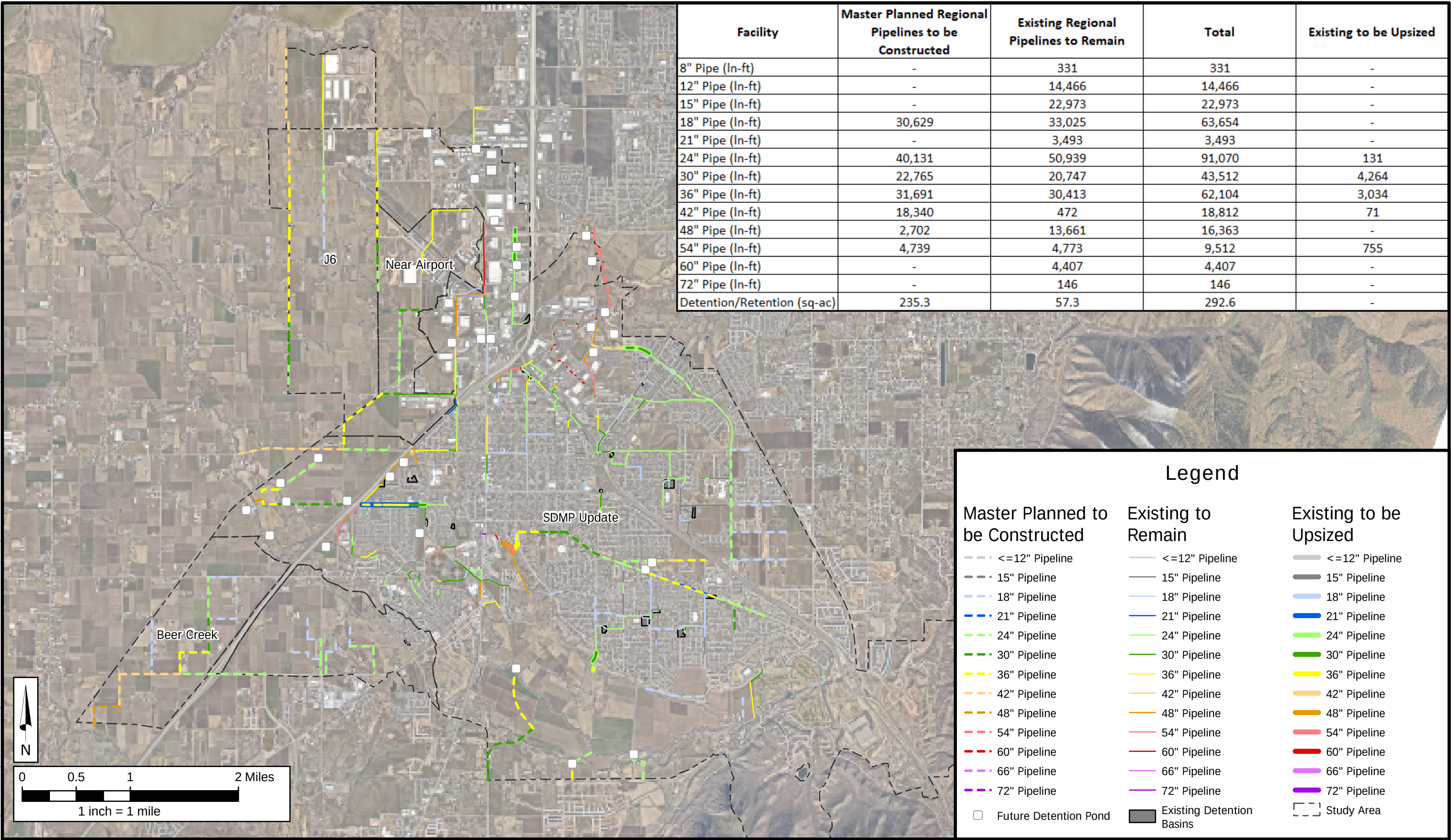
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EXECUTIVE SUMMARY

The City of Spanish Fork Storm Drain Master Plan evaluates the capacity and performance of the existing storm drainage system, identifies deficiencies, and establishes projects to support both existing development and future growth. Hydrologic and hydraulic modeling of approximately 17,130 acres identified key deficiencies including failing sump systems, surcharged trunklines, inadequate overflow capacity, and missing conveyance infrastructure. Recommended improvements focus on increasing system reliability through trunkline upsizing, new conveyance facilities, detention storage, and rehabilitation of aging infrastructure. The recommended Capital Improvement Plan (CIP) is illustrated on **Figure ES-1** (adapted from Figure 5-2A), which compares existing and buildout master-planned conveyance systems and highlights the major projects needed to achieve the City's desired level of service.

The CIP includes both near-term projects that address existing system deficiencies and long-term improvements required to accommodate buildout conditions. Existing system improvements are summarized in **Table E-1** (adapted from Table 5-1) and include projects totaling approximately **\$31 million**. Future growth-related improvements are summarized in **Table E-2** (adapted from Table 5-2) and total approximately **\$116.5 million**. As summarized in **Table E-3** (adapted from Table 5-3), the overall recommended storm drainage capital program has an estimated planning-level cost of **\$147.5 million**, including engineering and contingency. Implementation of these projects, together with ongoing maintenance, system inventory management, and periodic updates to the Master Plan, will provide a framework for managing stormwater, reducing flooding risk, and supporting continued growth throughout Spanish Fork.

Date: 7/22/2026
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SPANISH FORK CITY

STORMWATER MASTER PLAN
Updated With Recent Studies

FIGURE
ES-1

Table ES -1 Capital Improvement Plan for Existing Projects

Project ID	Deficiency ID	Location	Problem Description	Preferred Solution	Total with Eng & Cont.	Impact Fee
Spanish Fork City Master Plan Projects						
A01-N	A01	South of Canyon Road between 2300 E and 2900 E.	Failing sumps throughout neighborhoods.	Route flows north on 2600 E and west along Canyon Road (A01-A, A07, A03). Retrofitting and upsizing existing Canyon Road conveyance. Installation of 3 acre-foot detention pond.	\$8,695,000	-
A01-S	A01	South of Canyon Road between 2300 E and 2900 E.	Failing sumps throughout neighborhoods.	Connect Abbie Court detention to Canyon Elementary School detention and further west to 1100 E trunkline (A01-S, A02).	\$1,204,000	-
A02	A02	1100 E from Patriots Park south.	Missing trunkline from Patriots Park south to existing trunkline.	Add 24" dia. trunkline from Patriots Park south to existing trunkline and increase trunkline from 1750 S to outfall at Mill Race Canal to 30" and 36" dia (A02).	\$911,000	-
A03	A03	300 E and Scenic Drive to Mill Race Canal.	Tie in drainage from 300 E into Mill Race Canal.	Provide 36" dia. trunkline bypass from failing sumps on 400 S to 300 E south to continued 36" trunkline down to Mill Race Canal (A07, A03). Upsize Mill Race Canal in location of 300 E trunkline outfall to 54" dia. (A03).	\$1,090,000	-
A04	A04	Spanish Fork Parkway from 970 N to Hertiage.	Improve overflow capacity from failing sumps in neighborhoods east of railroad tracks.	Provide connection and increase capacity for overflow by installing 24", 30", and 42" trunklines along Spanish Fork Parkway (A04).	\$3,599,000	-
A05	A05	200 E from railroad tracks to outfall in Dry Creek.	Significant surcharging along 200 E.	Upside trunkline to 30" dia and provide detention for tributary areas south of railroad tracks, detaining to 0.15 cfs/acre (A05).	\$2,312,000	-

Project ID	Deficiency ID	Location	Problem Description	Preferred Solution	Total with Eng & Cont.	Impact Fee
Spanish Fork City Master Plan Projects						
A06	A06	100 S from Spanish Fields Drive east to Spanish Trails Blvd.	Surcharging along 100 S.	Improve conveyance capacity along 100 S by upsizing trunkline east of 630 W to 30" and 36" dia. pipe (A06).	\$2,440,000	-
A07	A07	400 S from 300 E to 820 E Canyon Road.	Failing sumps along 400 S.	Provide 36" dia. trunkline bypass from failing sumps on 400 S to 300 E south to continued 36" trunkline down to Mill Race Canal (A07, A03). Upsize Mill Race Canal in location of 300 E trunkline outfall to 54" dia. (A03).	\$2,385,000	-
A08	A08	Outfall of Mill Race Canal into Spanish Fork River	370 LF of 54" dia CMP.	CIPP	\$496,000	-
A09	A09	Trunkline along Scenic Drive.	160 LF of 18" dia CMP, 1,165 LF of 24" dia CMP, and 2,200 LF of 36" dia CMP.	CIPP	\$2,515,000	-
A10	A10	Trunkline along Canyon Road west to 2300 E.	605 LF of 18" dia CMP and 265 LF of 24" dia CMP.	CIPP	\$347,000	-
A11	A11	Trunkline along River Bottom Road outfalling into Spanish Fork River	1,340 LF of 24" dia CMP.	CIPP	\$690,000	-
A12	A12	Previous enclosed irrigation ditch along Canyon Road between 1700 E and 1975 E Canyon Road.	1,180 LF of 21" dia CMP.	CIPP	\$511,000	-
A13	A13	Trunkline on Main Street from 700 N to 1000 N and under trail.	420 LF of 18" dia CMP, 50 LF of 24" dia CMP, 245 LF of 30" dia CMP, and 305 LF of 48" dia CMP.	CIPP	\$1,840,000	-

Project ID	Deficiency ID	Location	Problem Description	Preferred Solution	Total with Eng & Cont.	Impact Fee
Spanish Fork City Master Plan Projects						
A14	A14	Trunkline along Kirby Lane from Canyon Creek Parkway to Commerce Way.	1,115 LF of 48" dia CMP.	CIPP	\$1,313,000	-
A15	A15	Trunkline along US-6 west from Expressway Lane and under US-6 from 800 N.	205 LF of 18" dia CMP and 1,200 LF of 24" dia CMP.	CIPP	\$690,000	-
Spanish Fork City Projects Subtotal					\$31,038,000	-

Total with Engineering and Contingency assumes a 30% on project cost.

Table ES -2 Capital Improvement Plan for Buildout Projects

Project ID	Diameter (in.); Length (ft.)	Total with Eng. & Cont.
C01	(18; 362)	\$257,000
C02	(48; 344),(36; 2,168),(30; 3,280)	\$4,897,000
C03	Detention	\$384,000
C04	Detention	\$859,000
C05	Detention	\$2,527,000
C06	Detention	\$343,000
C07	Detention	\$143,000
C08	Detention	\$93,000
C09-A	Detention	\$65,000
C09-B	Detention	\$65,000
C10	(36; 3,219),(30; 3,969)	\$6,455,000
C11	(24; 929),(36; 689)	\$1,172,000
C12	(24; 1,702)	\$931,000
C13	(18; 1,167)	\$401,000
C14	(42; 1,213),(24; 1,488),(36; 1,420)	\$3,919,000
C15	(18; 8,101),(24; 5,412)	\$6,208,000
C16	(18; 2,906)	\$1,253,000
C17	Detention	\$1,801,000
C18	Detention	\$2,693,000
C19	Detention	\$6,486,000
C20	Detention	\$1,670,000
C21	Detention	\$1,941,000
C22	Detention	\$950,000
C23	Detention	\$896,000
C24	Detention	\$2,182,000
C25	Detention	\$2,951,000
C26	Detention	\$1,560,000
C27	Detention	\$2,279,000
C28	Detention	\$1,150,000
C29	(36; 955),(24;2,731)	\$4,848,000
C30	Detention	\$730,000
C31	(30; 2,457)	\$709,000
C32	(24; 2,212)	\$1,590,000
C33	(48; 2,357),(42; 4,541)	\$7,949,000
C34	(24; 4,159)	\$2,505,000
C35	(30; 1,638),(36; 2,453),(24; 2,036)	\$4,142,000
C36	(18; 1,372)	\$592,000
C37	(18; 2,581)	\$1,113,000

Project ID	Diameter (in.); Length (ft.)	Total with Eng. & Cont.
C38	(18; 1,372)	\$471,000
C39	(24; 977), (30; 2,080), (36; 7,537), (42; 5,871)	\$12,843,000
C40	(18; 3,261), (24; 1,890)	\$2,354,000
C41	(36; 2,622), (30; 1,334), (24; 1,314)	\$3,734,000
C42	(24; 3,546), (27; 1,076)	\$2,408,000
C42A	(42; 5,132), (36; 3,644), (24; 2,085)	\$9,227,000
C43	(18; 1,640), (24; 1,638)	\$1,385,000
C44	(18; 1,636)	\$562,000
C45	(18; 881), (24; 1,037)	\$729,000
C46	(24; 2,488), (18; 1,038)	\$1,378,000
Spanish Fork City Buildout Projects Subtotal		\$116,509,000

Total with Engineering and Contingency assumes a 30% on project cost.

Table ES-3 Capital Improvement Plan Summary

Projects	Cost	Impact Fee Eligible
City of Spanish Fork Existing Projects	\$31,038,000	-
City of Spanish Fork Buildout Projects	\$116,509,000	-
Total Cost	\$147,547,000	-

CHAPTER 1 – INTRODUCTION

PURPOSE

This storm drainage master plan (Master Plan) for the City of Spanish Fork (City) presents an update to technical activities for managing and regulating storm drainage runoff caused by development and to help mitigate flooding and environmental impacts. The Master Plan will educate developers, private property owners, City staff, and elected officials regarding the capability and needs of the City's storm drainage system. The Master Plan examines the existing storm drainage system identifying existing deficiencies and the preferred solution. Alternatives are presented with conceptual cost estimates. A Capital Improvement Plan (CIP) is developed with Master Plan projects.

The Master Plan will include the combination of recent storm water analysis Hansen, Allen, and Luce (HAL) has developed for various regions of Spanish Fork as well as updates to the current Master Plan through the production and amendment of hydrologic and hydraulic models. Computer models for the Master Plan are used to simulate runoff and conveyance during storm events in the City. Not only are the model's vital tools in analyzing storm drainage and nuisance flooding for Master Planning efforts, but they will allow the City to continuously update and analyze potential storm drainage deficiencies as well as facilitate conceptual design of future projects.

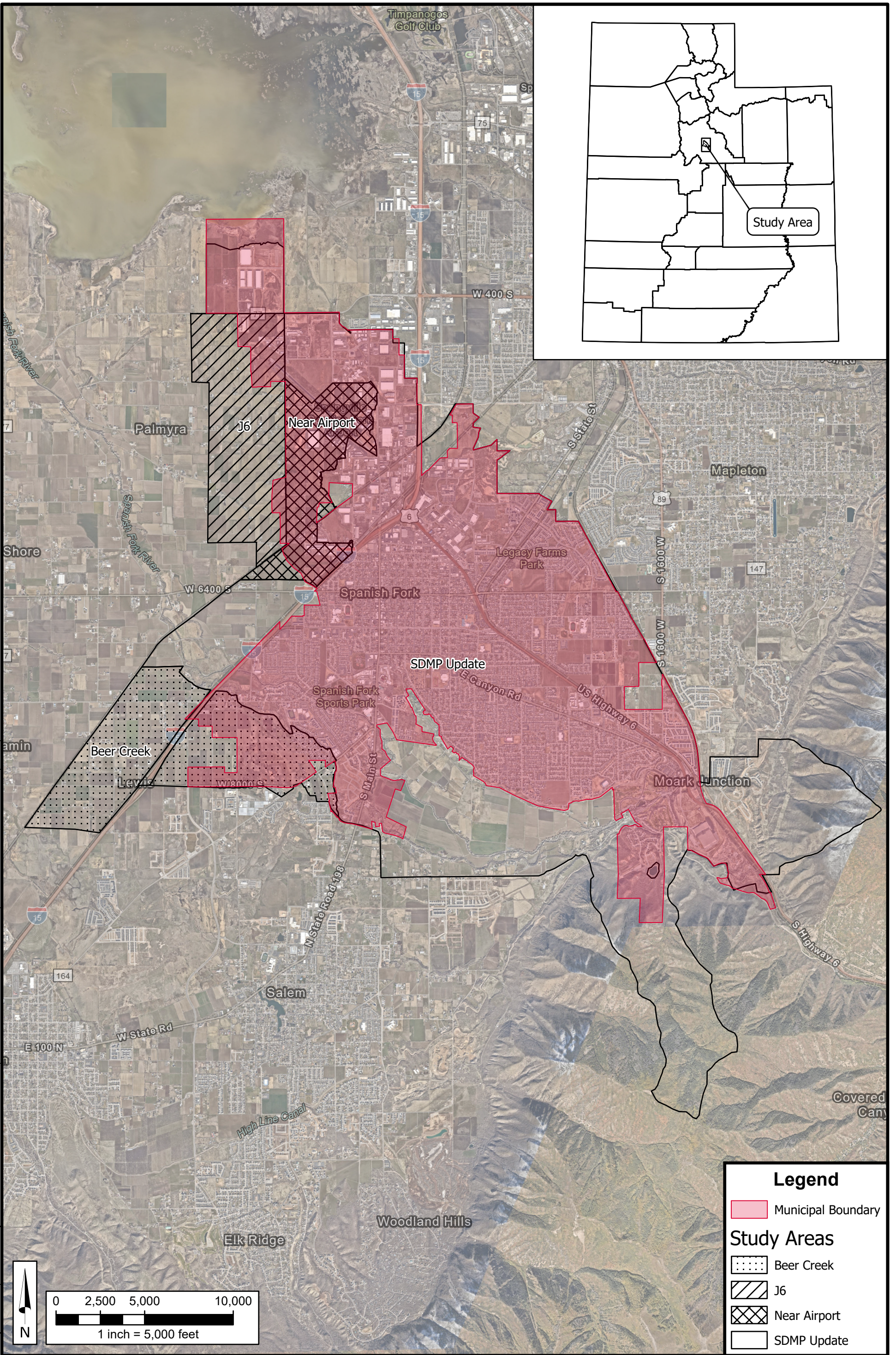
BACKGROUND

Located along the Wasatch Front in central Utah County, Utah, the City of Spanish Fork extends to just south of Provo Bay on Utah Lake to the Wasatch Mountains at the mouth of Spanish Fork Canyon and just North of Southfield Road. The terrain has a vertical relief of approximately 400 feet from the foothills of the Wasatch Mountains to Utah Lake on the northwest. Soil types range from cobbly fine sandy loam to very fine sandy loam. Land use varies from dense urban developments including residential, commercial and industrial uses to undeveloped farmland and foothills. The City of Spanish Fork was incorporated in 1855 and continues to experience growth and development, particularly to the northwest and southwest. As of 2023, the population of Spanish Fork is 45,557. The City has grown by approximately 5,571 people over the past five years. In an ongoing effort to regulate growth and redevelopment, the City desires to plan an effective storm drainage system to manage nuisance water, prevent flooding, and protect downstream waters from adverse quality and quantity impacts.

STUDY AREAS

The study area for the Master Plan includes the incorporated area of Spanish Fork, flows from the foothills of the Wasatch Mountains to the east of the City boundary, and flows from basins which border the South and Southwest City boundary. Portions of the study area that fall outside of the City boundary were included for a more conservative estimate of potential flows which may burden the City's storm drainage infrastructure assuming future city growth. Approximately 17,130 acres directly tributary to the City's storm drainage system are hydrologically modeled to determine the runoff. The 17,130 acres also includes mountainous watersheds and basins outside of Spanish Fork City limits. The study area and City boundaries are shown on **Figure 1-1**. From **Figure 1-1** it should be noted that although mountainous watersheds were delineated, very few of the runoff results were included in the hydraulic modeling calculations. Also shown in **Figure 1-1** are the boundaries showing the various study regions encompassing the update. The small area studies brought into the update include the 2024 Beer Creek Master Plan (HAL), the 2024 Beer Creek Spanish Fork Allowable Release Rate Memo (HAL), the 2022 J6 Area Drainage Study

(HAL), and the 2024 Near Airport Drainage Study (HAL). See **Appendix D** for individual small area studies.



CHAPTER 2 – EXISTING STORM DRAINAGE SYSTEM

This section discusses the features that make up the storm drainage facilities in Spanish Fork. **Figure 2-1** shows the non-exhaustive map of existing storm drainage system including manholes, pipes (stormwater), open channel (stormwater), culverts, and detention/retention basins. Elements not included in this figure include but are not limited to pretreatment structures, basin control inlets/outlets, combination boxes, pumps, LID, and diversion structures.

NATURAL DRAINAGES

The City of Spanish Fork incorporates several natural waterbodies in its storm drainage system. Natural channels in the City generally flow from east to west and from south to north. The natural waterbodies serve as collectors for outfalls for the City's storm drainage system. These natural channels eventually terminate in Utah Lake. It is valuable to note that parts of the City consist of an interwoven network of irrigation ditches and storm drainage open channels.

Dry Creek

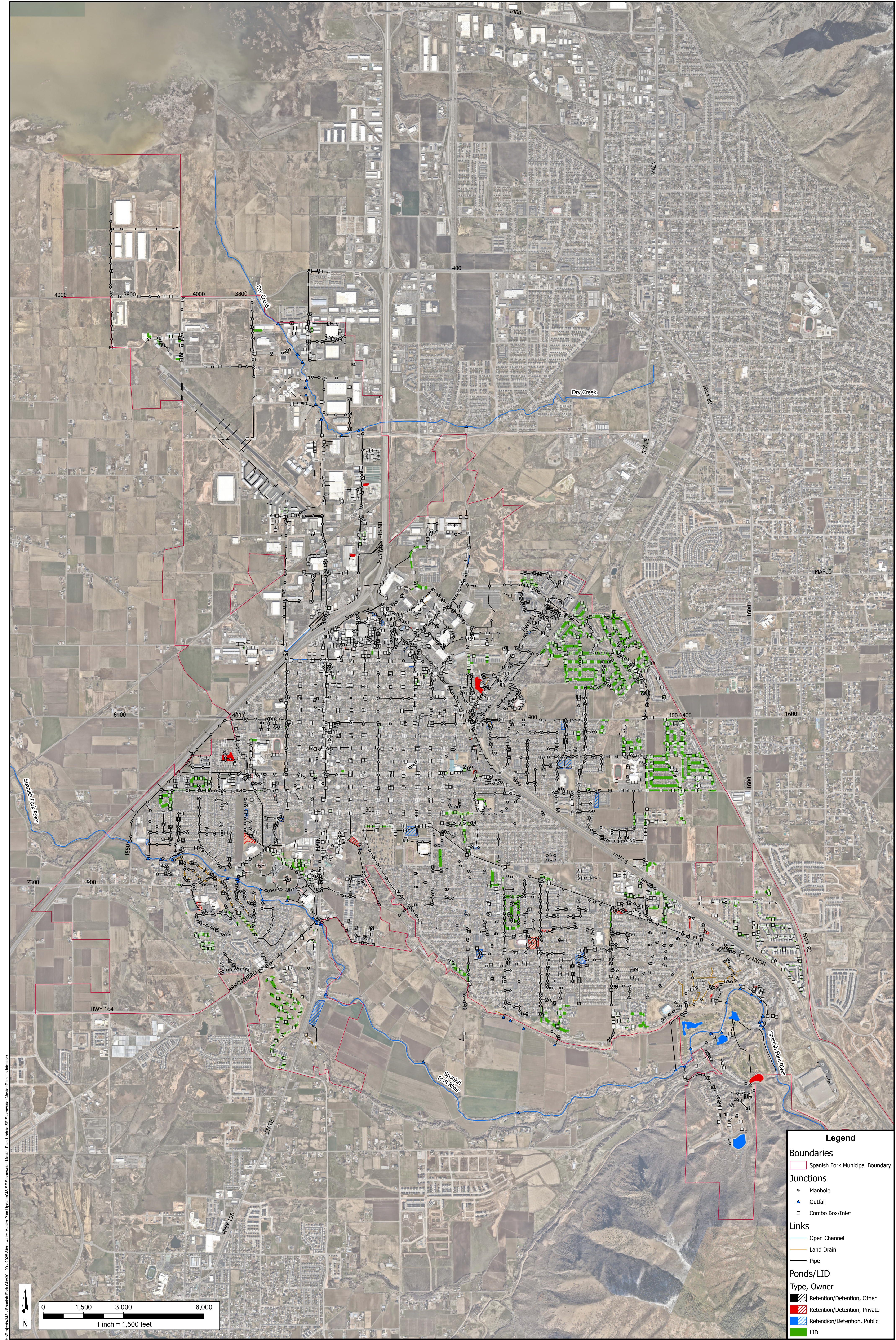
Dry Creeks' headwater begins in Springville between State Street and Main Street, south of 950 S. Dry Creek is a perennial stream driven by ground water. Dry Creek flows south-west towards 4800 South. When Dry Creek reaches 1375 S in Springville, it acts as a multi-use conveyance facility for both irrigation and stormwater. At 1200 W, Dry Creek runs west parallel to 4800 S under I-15 towards Main Street where it heads north-west terminating in Utah Lake. Much of Dry Creek acts as a terminating facility for both Springville and Spanish Fork stormwater trunkline conveyances. Due to the nature of this facility having multiple stakeholders and serving multiple uses, HAL was asked to complete a master plan for Dry Creek in 2024. Pertinent results of the Dry Creek CIP will be incorporated into this master plan update.

Spanish Fork River

Spanish Fork River is a perennial stream with a predominantly snowmelt driven hydrograph. Spanish Fork Rivers' headwaters are in the Wasatch Front where it runs south-east to north-west out of Spanish Fork Canyon and into the City of Spanish Fork. Spanish Fork River continues to flow north-west through the City paralleling its southern boundary. Spanish Fork River is a terminating facility for the City's southern portion of the storm water network, collecting runoff prior to its termination at Utah Lake. It was treated as a terminating feature during the modeling efforts.

IRRIGATION CONVEYANCES

Irrigation canals, ditches, and diversion structures are common in many parts of Spanish Fork. HAL is aware of some existing shared facilities between irrigation and storm drainage conveyances purely through spatial proximity. Identification of all shared facilities is difficult as there are no shared ID's between the features in the stormwater GIS database and features in the irrigation GIS database. Irrigation facilities which are used for storm drainage are with approval from the irrigation company through a written agreement. The Mill Race Canal is the only irrigation facility included in the modeled, the canal was modeled with a base flow of 60-cfs. The city does not plan on using irrigation facilities for storm drain purposes and is working to separate their storm drain from irrigation facilities when feasible.



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EXISTING STORM DRAIN SYSTEM

FIGURE
2-1

FLOWS OUTSIDE OF SPANISH FORK

There is one primary conveyance in which outside storm water flows may influence storm drain capacity. Dry Creek is shared between Spanish Fork and Springville providing storm drainage conveyance. The Dry Creek Master Plan details the projects necessary to convey a 3-hour 25-year event considering tributary flow from both Spanish Fork and Springville. See Dry Creek Master plan for further details.

STORM DRAINAGE FACILITIES

The City provided HAL with their existing storm drainage related GIS data. The GIS data provides an accurate depiction of where the storm drainage facilities are located, but in some cases lacked sufficient data for detailed storm drainage modeling to be performed. The City provided HAL with a Storm Water Management Model (SWMM) based model that included conveyance and detention facility details from the previous master plan completed by BCA in 2016. Data from previous local drainage related master plan studies completed by HAL were also incorporated into the city-wide master plan update.

As infrastructure is added to the system it is recommended that more detailed information be collected and included in the GIS dataset moving forward. Data that should be compiled in the system inventory moving forward include ID, location, descriptions, invert and rim elevations for structures, up/down invert elevations for pipes, up/down structure ID's for pipes, as well as, pipe and structure dimension, and pipe material.

City staff provided GIS files that included all updates to their storm drainage facility inventory since the last master plan was completed. The GIS dataset was used to identify missing information and data gaps. HAL had a data gaps meeting with city personnel summarizing and discussing the missing data needed for model development. As needed, assumptions and corrections were made to the data for modeling purposes. In the case of detention, if the GIS information or other data was unavailable, assumptions were made such that detention facilities met the current City detention requirements.

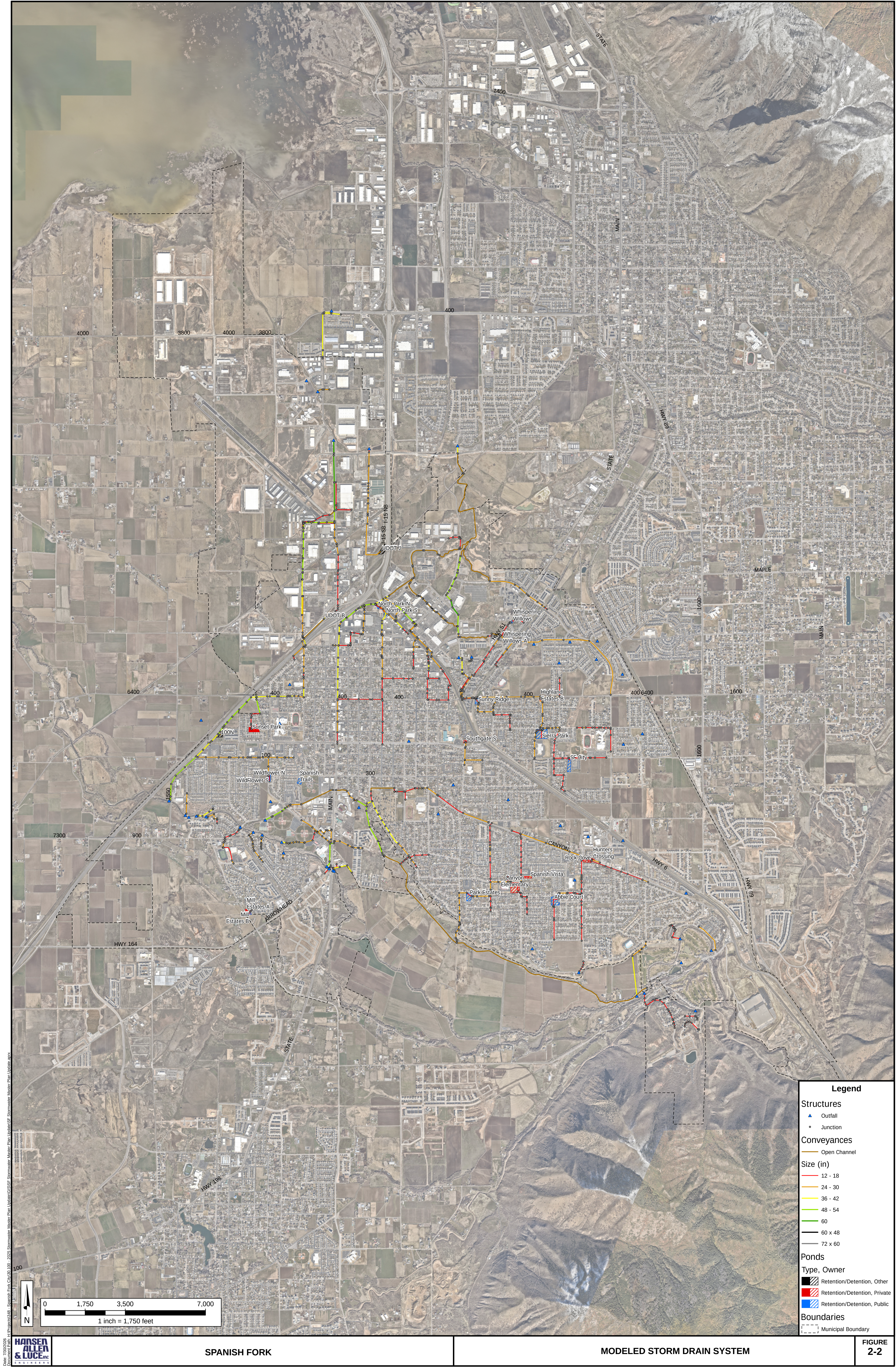
Collection and Conveyance

The City of Spanish Fork has roughly 133.9 miles of storm drainage pipes varying in size from 12-inches in diameter to 60-inches in diameter as well as open channel conveyances. The City has an estimated 4,843 inlets that introduce runoff into the storm drainage network. The storm drainage system also relies on natural drainages and roadside swales convey runoff to the Utah Lake.

As part of our scope, HAL was given an existing SWMM model by the City of Spanish Fork which included the storm drainage collection and conveyance system to be modeled in this Master Plan. Through discussion with City engineering staff, modifications to the given model were made which included appending collection and conveyance structures deemed necessary to have within the model. The modeled portion of the Spanish Fork storm drainage system can be seen in **Figure 2-2**.

Detention

Spanish Fork maintains numerous detention facilities designed to detain runoff from up to the 25-year storm event. HAL developed stage-storage curves for these basins using GIS data, model information, and available design drawings. Spanish Fork has 67 georeferenced detention basins; 57 are maintained by the City, of the 57, 23 are represented in the Storm and Sanitary Analysis



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Map: Spanish Fork, Utah - Stormwater Master Plan Update
Map: Spanish Fork, Utah - Stormwater Master Plan Update

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MODELED STORM DRAIN SYSTEM

FIGURE
2-2

(SSA) model. The modeled basins include those carried forward from the previous Master Plan and others added as needed in the updated model alignment. Current detention basins provided by the City can be seen in **Figure 2-1**. Modeled detention basins can be seen in **Figure 2-2** and are listed in **Table 2-1**. Detention basins are colored based on the entity that owns or manages the basin. In **Figures 2-1 and 2-2**, detention basins are color-coded according to ownership: City-owned (owned and maintained by Spanish Fork), Private (installed and maintained by private developers), and Other (ownership undetermined based on available GIS data).

Table 2-1 Inventory of Modeled Detention Ponds

Basin/Model Name	Approximate Volume (Ac.-Feet)
100N	2.5
Abbie Court	0.5
Black Horse	2.3
Canyon Elementary	10.7
Highland Estates B	2.0
Hunters Crossing	0.4
Mill Estates A	0.4
Mill Estates B	0.1
North Park N	1.5
North Park S	1.3
Park Estates	4.8
Spanish Fork City	4.5
Sierra Park	13.4
South Gate South	0.5
Spanish Trails	0.6
Spanish Vista	1.1
Sunny Ridge	2.5
Sunset Park	4.3
Whispering Willows	0.4
Whispering Willows A	0.4
Wildflower N	0.8
Wildflower S	0.4

CHAPTER 3 – METHODOLOGY

The project team adopted a workshop-based approach with City engineering staff to determine the design criteria, study areas, analysis processes, deficiencies, alternatives, and solutions. This section describes the methodology followed in developing the Master Plan.

HYDROLOGY

Hydrology is the science that examines the occurrence, movement, and distribution of water through the hydrologic cycle. In the context of storm drainage management, it focuses on how precipitation is transformed into runoff, how that runoff is conveyed and stored, and how these processes influence flooding, drainage infrastructure, and water resources. For this Master Plan, hydrologic analysis included: selecting rainfall design frequencies and storm distributions, delineating and calculating subbasin areas, estimating runoff potential using soil data, land cover, and impervious surface characteristics, determining peak runoff timing. The following sections describe these methods in greater detail.

Design Frequencies

The City selected design storm frequencies of 10-year (10% chance of being equaled or exceeded in any given year), 25-year (4% chance of being equaled or exceeded in any give, year), and 100-year (1% chance of being equaled or exceeded in any given year) for this study. Criteria included:

- 10-year design capacity for the initial storm drainage system. The initial storm drainage system includes pipes, gutters, and roadside ditches.
- 25-year design capacity includes the initial storm drainage system as well as detention ponds.
- 100-year design capacity includes the initial storm drainage system as well as detention and retention ponds.

Section 3 from Spanish Fork's design criteria manual seen below, describes the cities design guidelines for 10, 25, and 100-year events.

3.1 Design Storm

3.1.1 Frequency

Storm Drain facilities shall be designed to include major and minor conveyance facilities and storage facilities as described below:

Minor System

Minor system facilities shall be designed to collect and convey storm water runoff from a storm with a return frequency of 10 years. Minor system facilities include local catch basins, storm water pipes and manholes.

...

Storage Facilities

Detention basins shall be designed to detain runoff from a storm with a return frequency of 25 years. Retention basins shall be designed to retain runoff from a storm with a return frequency of 100 years.

Design Storms

The pattern of rainfall intensity over the duration of a storm is known as its temporal distribution. Selecting an appropriate storm distribution is critical because it affects peak flows in pipes and channels, as well as peak storage volumes in detention ponds. These results, determined in part by storm distribution, ultimately dictate what is flagged as a deficiency and what projects are planned for as capital improvements.

For this Master Plan, the Modified Farmer–Fletcher storm distribution was selected, as it best represents the thunderstorm events typically experienced along the Wasatch Front (**Figure 3-1**). This design storm includes the intensity of a one-hour Farmer Fletcher (E. Farmer and J. Fletcher, 1971) storm with a 3-hour storm volume. **Figure 3-1** shows the hyetograph used in developing the design storm for updated storm drain masterplan.

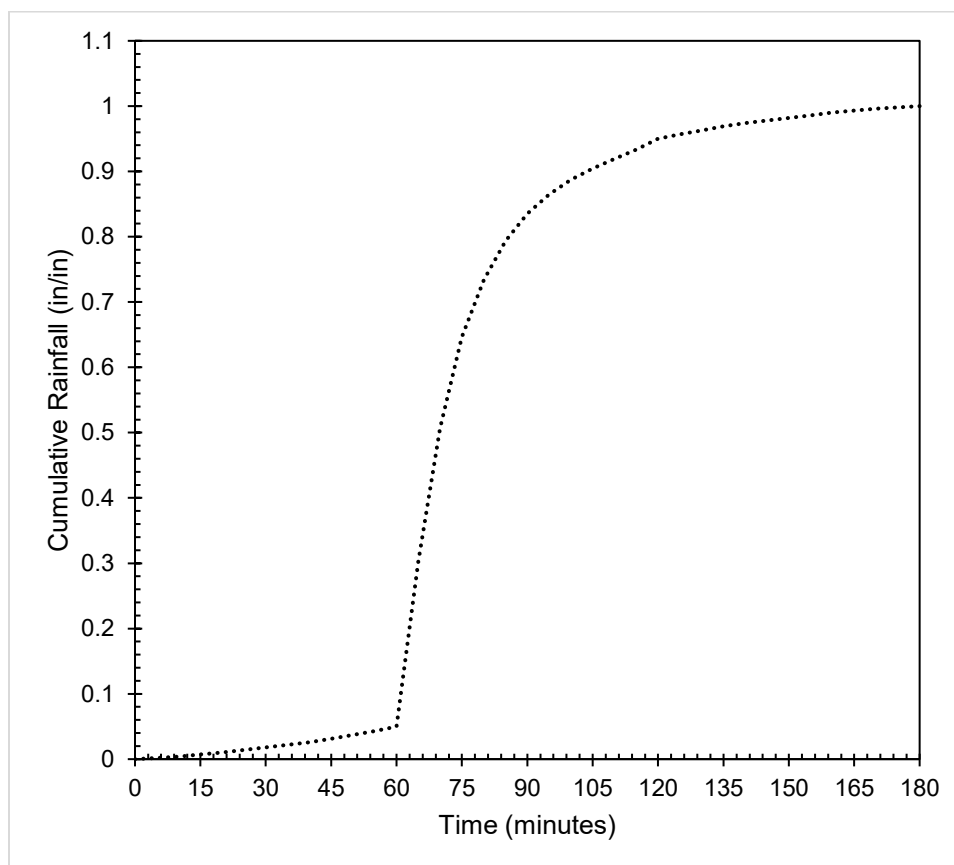


Figure 3-1 Design Rainfall Hyetograph

Precipitation depths were obtained from *NOAA Atlas 14: Precipitation-Frequency Atlas of the United States* (Bonnin et al. 2004; NOAA 2013) at location 40.1102, -111.6470. The design storm rainfall depths modeled for this Master Plan are seen in **Table 3-1**.

Table 3-1 Modeled Rainfall Depths

Storm Frequency	3 hr
10-yr Rainfall Depth (in)	1.04
25-yr Rainfall Depth (in)	1.31

DEVELOPMENT OF THE HYDROLOGIC MODELS

Subbasins

A storm drainage basin, also called a subbasin, watershed or catchment, is an area in which all rainfall or snowmelt runoff will collect to a common point (the lowest point in the basin). Storm drainage basin boundaries depend upon both the topography and the location of storm drainage facilities. Subbasin characteristics developed for this plan were based on aerial imagery, soil data, GIS mapping, land use information from the City, and engineering literature. Key characteristics include area, hydrologic soil group, percentage of impervious surface, SCS Curve Number (CN), and overland flow characteristics. The methods used to develop these parameters are based on guidance from *Technical Release 55: Urban Hydrology for Small Watersheds* (NRCS, 1986), hereafter referred to as TR-55.

Subbasin Area

The amount of runoff is typically proportional to the area of the subbasin, assuming similar hydrologic characteristics. The study area includes 256 delineated subbasins, with a median area of approximately 45 acres. **Figure 3-2** is a map providing the delineated subbasins updated as part of this plan and delineated subbasins from previous studies completed by HAL as an amendment to this plan.

Hydrologic Soil Group

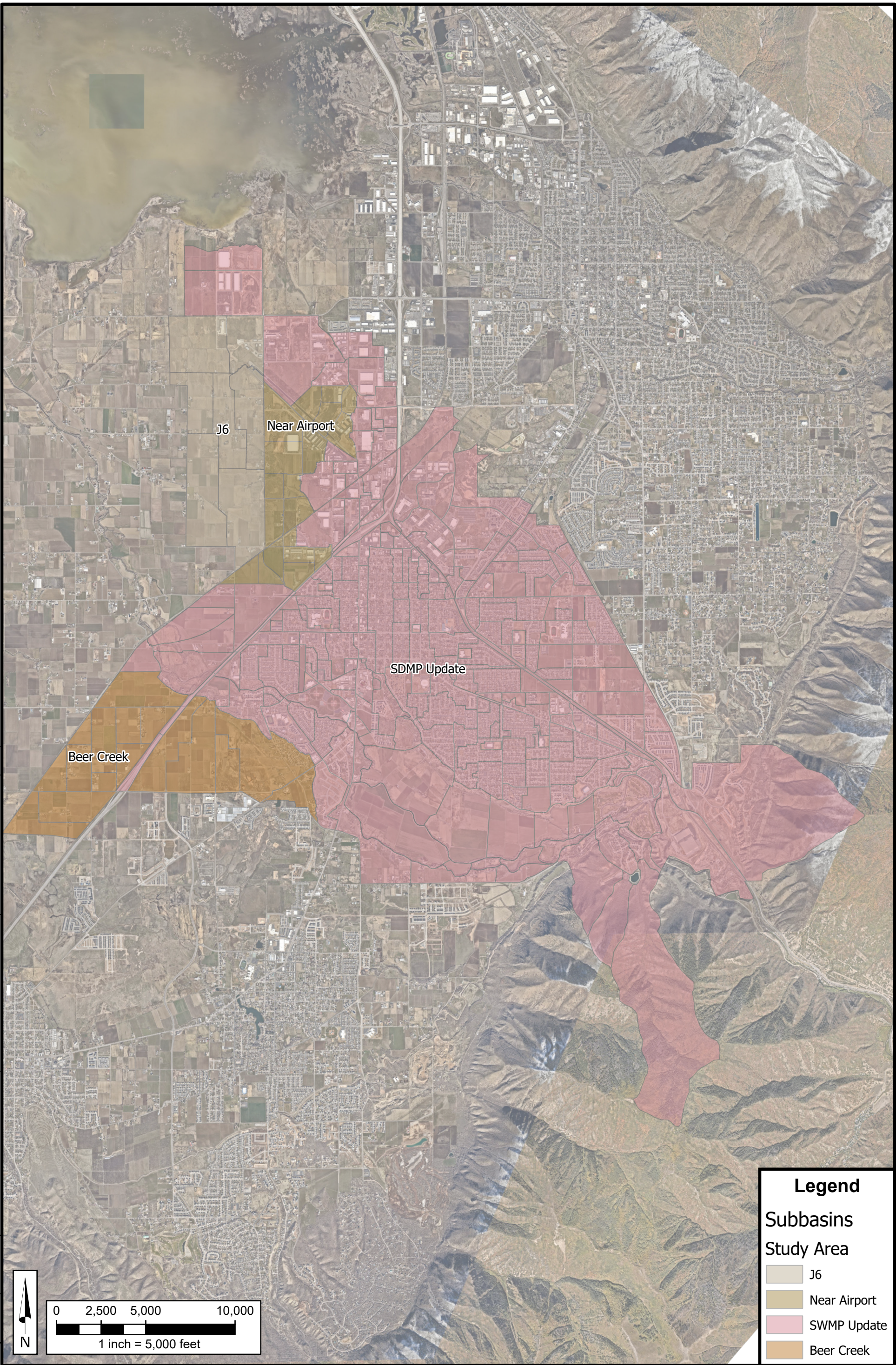
The hydrologic soil group is a factor used to determine the CN for each subbasin. Hydrologic soil group is a general indication of a soil's infiltration capacity and is a key determinant of runoff behavior. The Natural Resources Conservation Service (NRCS) has classified soils into four hydrologic groups—A, B, C, and D—based on infiltration rate: Group A: High infiltration; minimal runoff (e.g., permeable gravels and well-drained sands). Group B: Moderate infiltration; moderately coarse to moderately fine textures.

Group C: Low infiltration; finer textures or layers that impede percolation. Group D: Very low infiltration; highest runoff potential (e.g., silts, clays, and poorly drained soils). Soil groups are described in TR-55 (NRCS, 1986).

All four soil groups are found in the City, with groups C and D soils being the most prevalent. The C and D soil groups make up approximately 42% and 36% of the soil within the study area, respectively. Dual soil groups were identified within the study area; it was assumed that the areas with dual soil groups are classified as group D. Soil data for this study originated from the NRCS Web Soil Survey (NRCS 2020). A soil map of the study area is shown in **Figure 3-3**.

Land Use

Existing land use for the storm drain master plan modeling efforts was developed from the City's existing zoning data. Quality control was performed by comparing zoning information with 2024 Nearmap imagery to verify consistency. **Figure 3-4** shows a simplified land use map and **Table 3-2** shows the general mapping of zones and general plan designation types to their assumed hydrologic land use. **Table 3-2** provides typical impervious cover percentages for each zoning type, as well as the proportion of impervious cover which is directly connected (DCIA), based on TR-55 and various case studies within Spanish Fork.

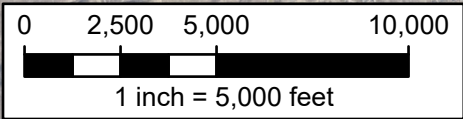


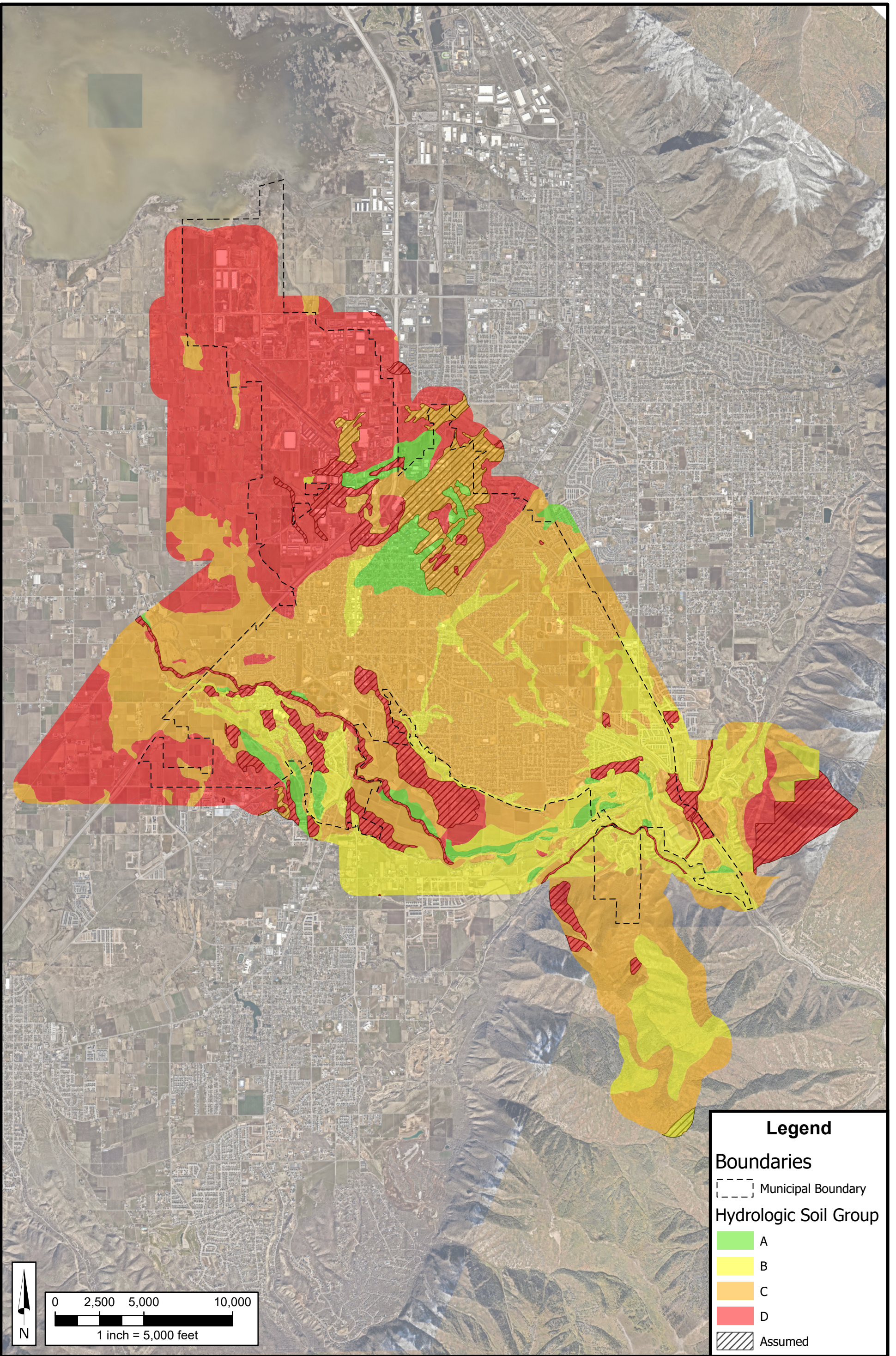
Legend

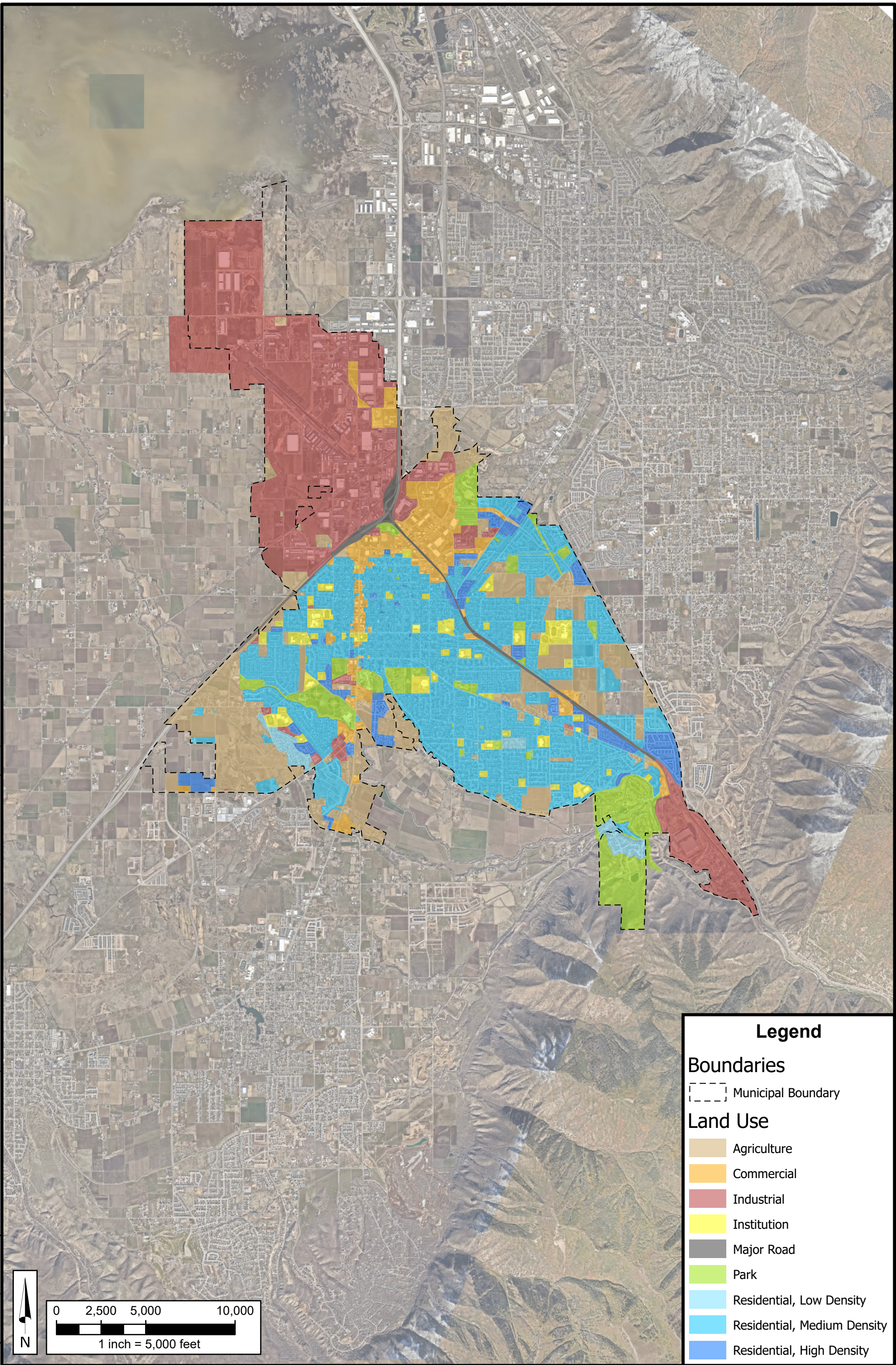
Subbasins

Study Area

	J6
	Near Airport
	SWMP Update
	Beer Creek







Impervious Area

Impervious areas within each subbasin were estimated using multispectral image analysis and the type of land uses within each subbasin are specified below in **Table 3-2**. There are two recognized types of impervious area; directly connected impervious areas (DCIA) and unconnected impervious areas (UCIA). Directly connected impervious areas provide a direct path for runoff to a conveyance such as a pipe, gutter, or channel and include roadways, parking lots, driveways, and roofs. Runoff from unconnected impervious areas crosses a pervious area before reaching the storm drainage node for the subbasin. Examples of unconnected impervious areas include sidewalks that are not adjacent to the curb, patios, sheds, and portions of house roofs.

Table 3-2 Land Uses in Spanish Fork

Zone	Assumed Hydrologic Land Use Type	% Impervious	% DCIA of Impervious
A-E, R-R, R-1-80, R-1-60, R-1-40, R-1-30	Agriculture [†]	0.08%	51%
C-O, C-D, C-1, C-2, S-C, C-UV, B-P, Mixed Use, Commercial, Business Park	Commercial [†]	84%	94%
I-1, I-2, I-3, Industrial	Industrial [†]	91%	97%
P-F, R-O, Public Facilities	Institution [†]	52%	64%
	Major Road	100%	95%
Public Facilities, Floodplain, Sensitive Lands	Park [†]	11%	67%
R-1-20, R-1-15, R-1-12, Estate Density Residential, Low Density Residential	Residential, Low Density [†]	24%	48%
R-1-9, R-18, R-1-6, R-5, Medium Density Residential	Residential, Medium Density [†]	53%	49%
R-3, R-4, R-5, Urban Density Residential, High Density Residential	Residential, High Density [†]	68%	44%

[†]Case studies were completed for these land use types.

It is important to distinguish between directly connected and unconnected impervious areas. Runoff from the directly connected impervious areas reach the storm drainage conveyance system and usually determines the magnitude of the peak flow rate. Impervious areas such as backyard patios which drain grass or landscaped areas have much less impact on peak runoff.

Unconnected impervious area for residential land use is included in the pervious area with a composite curve number based on an area-weighted average. The directly connected impervious area is included explicitly in the subbasin characteristics as a percentage of impervious area. See **Figure 3-5** for example case studies done for low density, medium density, high density,

institution, commercial, industrial, agriculture, and park land uses within the study area for the analysis on DCIA and UCIA.

SCS Curve Number

Each subbasin was assigned an SCS curve number based on hydrologic soil group, land use, and ground cover type as outlined in Chapter 2 of TR-55 (NRCS, 1986). The curve number describes the relationship between precipitation and runoff for the pervious and unconnected impervious portions of the subbasin. Practical curve numbers range from 30 to 98. Areas that are more pervious have lower curve numbers. For example, a well vegetated subbasin with sandy soils and little unconnected impervious area would have a lower curve number than a poorly vegetated subbasin with clay soils and a significant amount of unconnected impervious area. The vegetation condition and unconnected impervious area were determined based on the land use. See **Table 3-3** for the criteria for curve number assignment.

Table 3-3 Curve Number Assignment Table

Land Use	TR-55 Cover	Curve Number by Soil Type			
		A	B	C	D
Industrial [†]	Open space, fair	49	69	79	84
Commercial [†]	Open space, good	39	61	74	80
Major Road	Open space, good	39	61	74	80
Residential – High Density [†]	Open space, good	39	61	74	80
Residential – Medium Density [†]	Open space, good	39	61	74	80
Residential – Low Density [†]	Open space, good	39	61	74	80
School/Hospital/Church/ Public Facility [†]	Open space, good	39	61	74	80
Agricultural [†]	Open space, good	39	61	74	80
Park [†]	Open space, good	39	61	74	80
Impervious	Impervious	98	98	98	98

[†]The zoning case study was used to determine how much area is impervious and how much of the impervious area is directly connected. Unconnected impervious and pervious CN is based on TR-55, cover description of Open space, good condition.

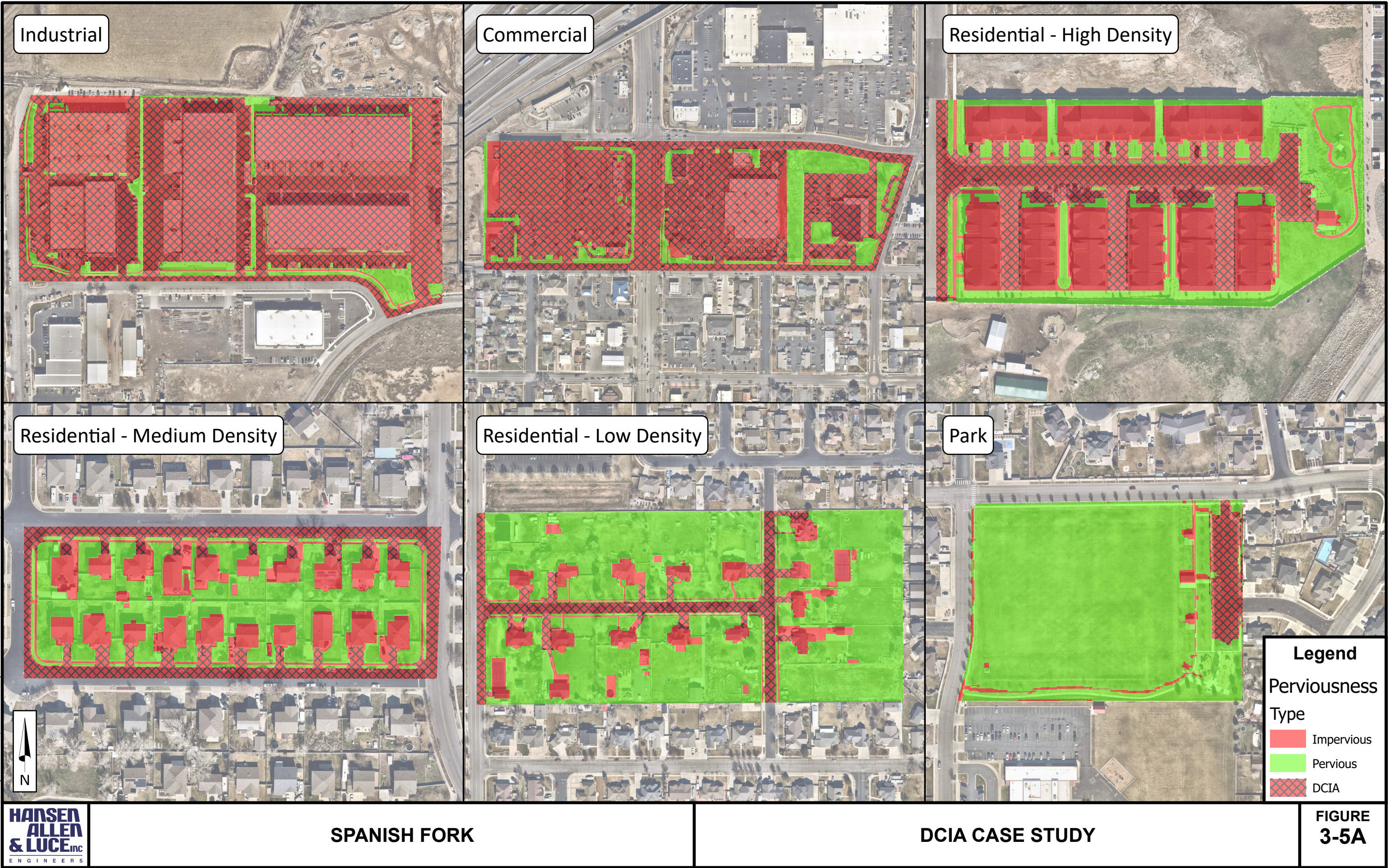
DEVELOPMENT OF THE HYDRAULIC MODELS

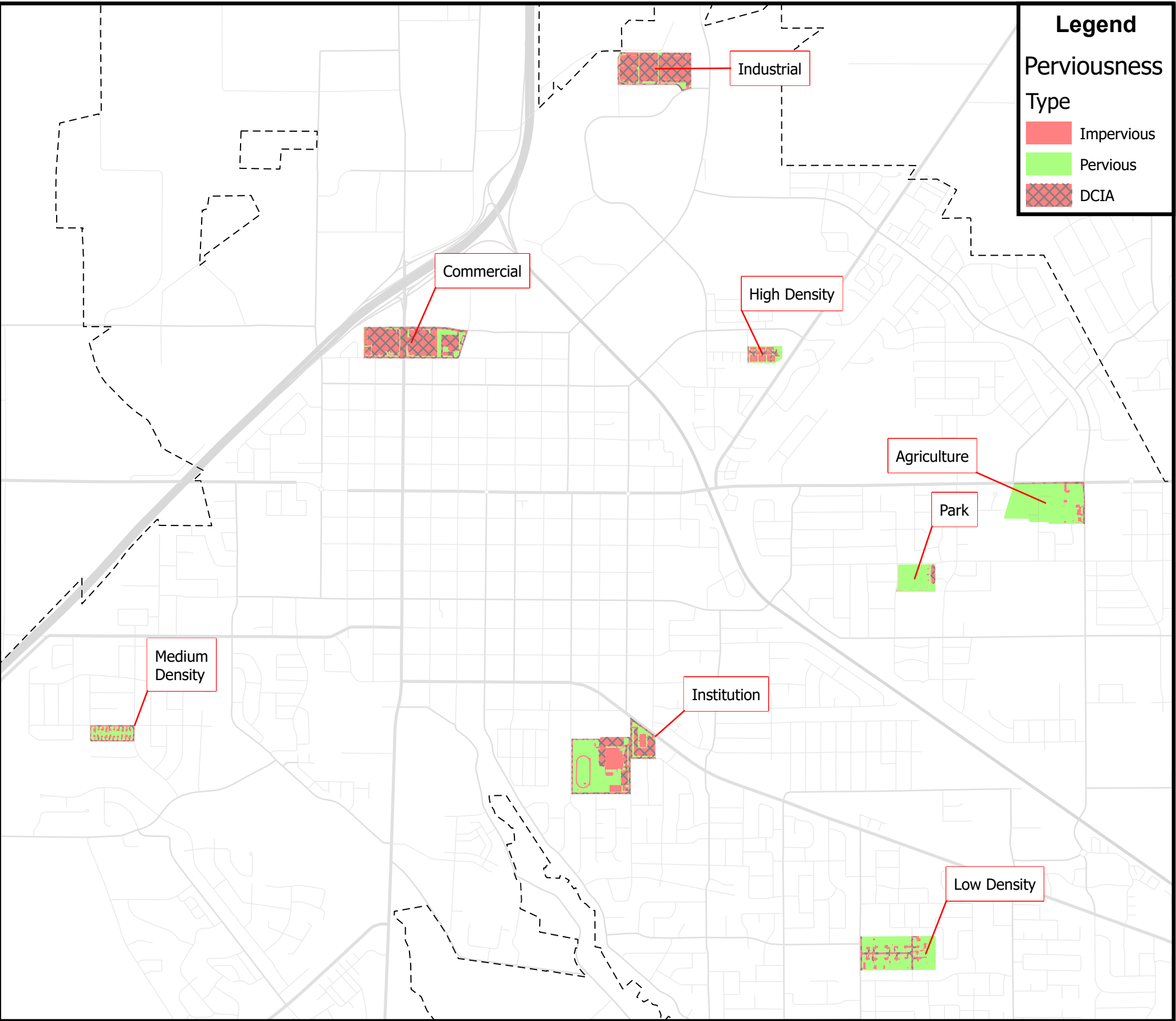
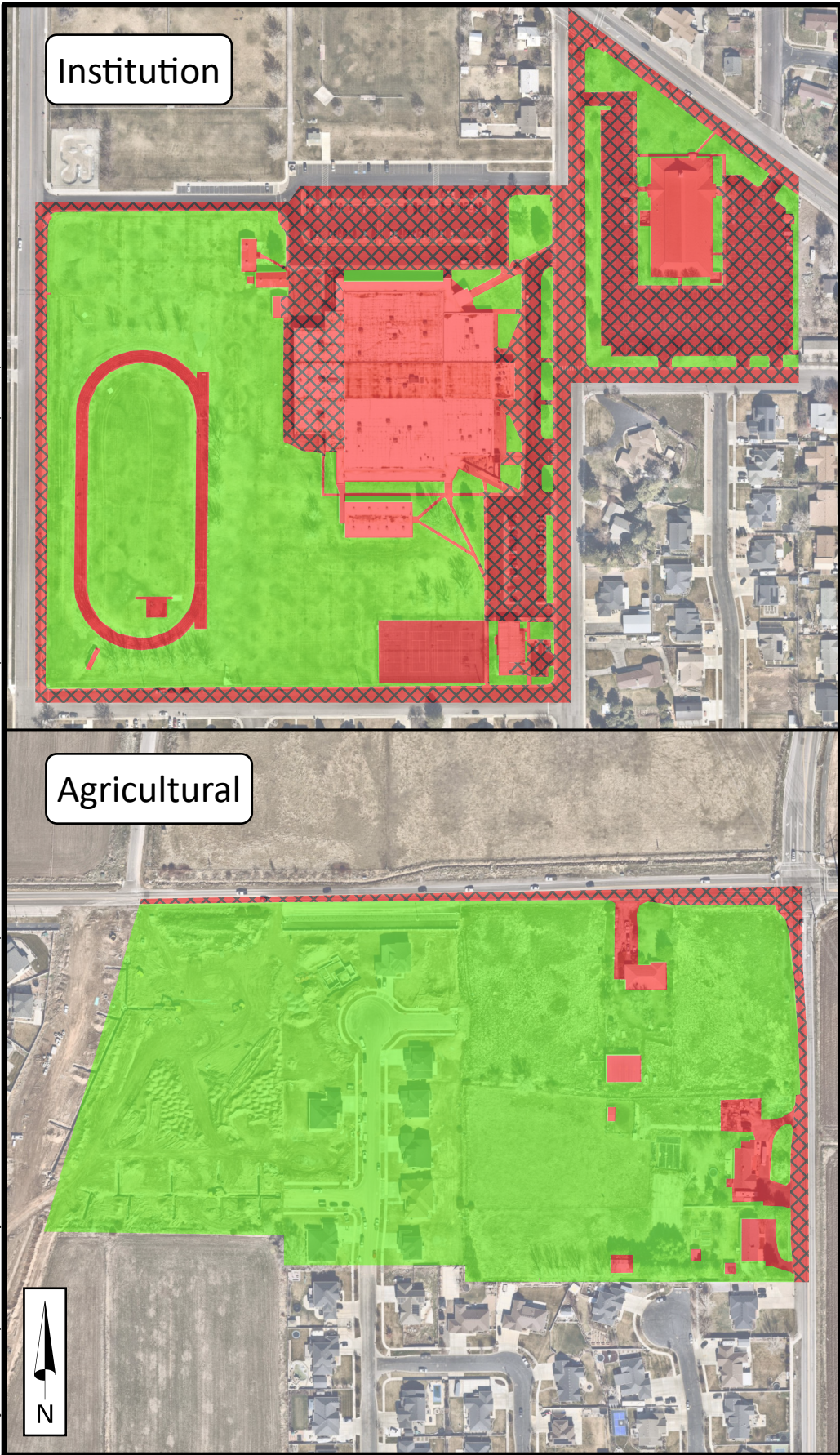
Conveyances

Conveyances convey runoff between the junctions in the model. Typical conveyances include pipes, box culverts, ditches, canals, natural channels, outlet pipes, orifice plates, weirs, pumps, and in some cases gutters. Conveyance characteristics for each model were defined from the SWMM model provided by the City. Amended data regarding links was also provided by the City. This information is used in the model to perform the hydraulic analysis for timing and routing of storm runoff hydrographs from the subbasin outlet.

Some uncertainty is associated with modeled storm drainage paths. In much of the City storm drainage conveyances frequently coincide with irrigation pipes and ditches. Flow paths in irrigation conveyances can be altered by gates and diversions, complicating the modeling of flow paths. Irrigation base flows were provided by the City.

SWMM based models will limit flow through conduits based on the conduit's capacity. For example, if a subbasin has a peak flow of 10-cfs while the conduit only has a capacity of 6-cfs, flooding would occur at the upstream node and the conduit would be limited to its peak capacity





SPANISH FORK

DCIA CASE STUDY

FIGURE
3-5B

creating surcharged conditions. The upstream node would experience flooding near 4-cfs. This is helpful when analyzing how the system functions but can hinder efforts to determine the potential peak flows at a given point. During the design process conveyances can be modified to increase the size of the conduits so that the peak flows are not restricted and specific points in the system experience the maximum possible flows during a precipitation event. Changing pipe sizes was done to evaluate potential capital improvements.

Nodes

In the model, nodes are points where conveyances or subbasins connect. Nodes generally represent manholes, inlets, outfalls, pond outlet structures, detention basins, or points where multiple conveyances combine. Nodes contain elevation data which determines the invert or rim elevation of the node. Nodes like manholes also have a diameter or shape to determine volume within the structure. Accurate rim elevation data is particularly important to determine if the point is experiencing flooding due to surcharging conditions.

Flooding at nodes can be handled in several ways. If the surface ponding option is used, the volume of water at a node above the maximum depth is stored at the node and released back into the system once surcharging conditions recede. If the surface ponding option is not used, any flooding is lost from the system. To conservatively estimate the volume of water entering storage nodes, the surface ponding option was used in this Master Plan.

IRRIGATION INPUTS

The SWMM model can add external inflows at nodes. This function was used to add irrigation flows to the model (which were modeled as a constant baseflow) only where irrigation facilities convey stormwater runoff under existing agreements. Irrigation flows were determined based on discussions with City personnel. Irrigation flows were added to nodes based on the current model alignment and cumulative magnitudes of irrigation flows moving upstream in the network. Within the model, an inflow of 60-cfs was added to the Mill Race Canal as an external inflow.

DESIGN FLOW RATES

The model computes hydrographs for each subbasin, conveyance, and junction. The City storm drainage system was analyzed with the 10-year 3-hour and 25-year 3-hour precipitation events. The 25-year 3-hour precipitation event was used to determine regional detention pond sizing for the future CIP projects.

CAPACITY ANALYSIS

The SWMM model can determine the water depth, flow rate, and capacity of conveyances during a given precipitation event. This detailed analysis includes calculating surcharged conditions and backwater effects if the Dynamic Wave routing method is selected. This capability makes the SWMM engine a very useful tool in analyzing the hydraulic capability of a storm drainage system.

CHAPTER 4 – STORM DRAINAGE ANALYSIS

The Spanish Fork storm drainage system was analyzed using the model, observations from City staff and best management practices for the industry.

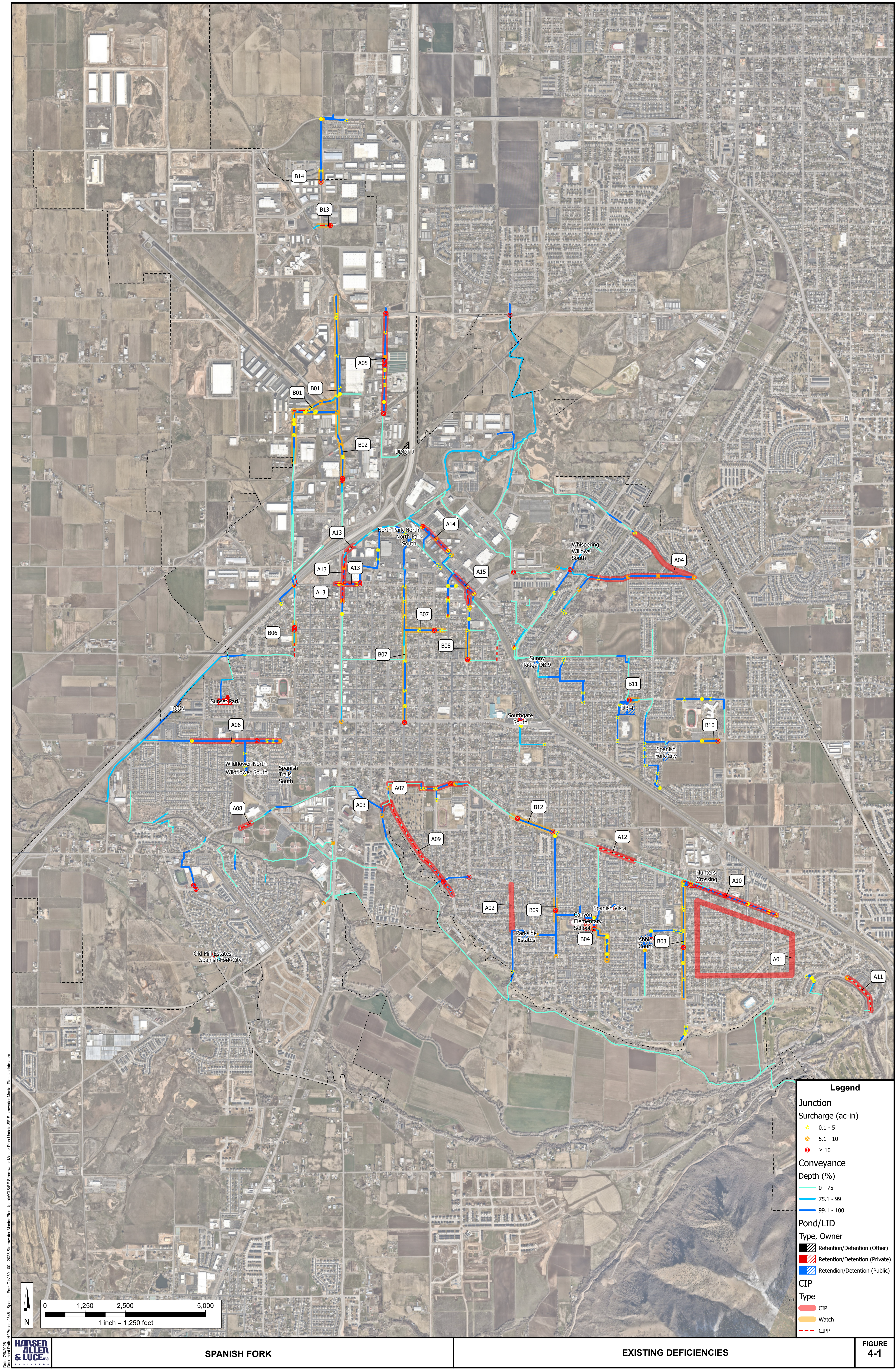
DEFICIENCIES

Deficiencies were identified based on input from City staff and on results from the model. Locations where the City had experienced flooding were analyzed in the model to determine the cause of the flooding. For example, a cul-de-sac that frequently floods could have issues with inlet capacity, pipe capacity, backwater effects, runoff from areas not directly tributary to the cul-de-sac, or the curb and gutter configuration.

Generally, storm drainage systems are designed to carry the peak 10-year runoff event, with the 100-year runoff event being conveyed in the roadway, pipes, regional detention, natural channels, and via land drains which ultimately discharge into Utah Lake. It was determined that for the existing Spanish Fork storm drainage system, full pipe flow is acceptable with minor surcharging being contained in the curb and gutter system.

Table 4-1 and **Figure 4-1** summarize the storm drainage deficiencies identified in this study. Each deficiency has a Deficiency ID (used in this study), a location description, and problem definition.

Not all surcharging is considered a deficiency, and not all deficiencies necessitate capital improvements. Because storm drainage systems are designed to convey the 10-year event, a 10-year event will produce flows at or near the pipe capacity of the system. Some nodes in the model, where areas of surcharging were identified, were determined not to be deficiencies because the surcharging is contained in the curb and gutter system, or because it was deemed a non-issue through anecdotal evidence from City staff. A pipe at capacity, or a surcharged node in the model, was only added to the deficiency list if it fell within a highlighted project area identified during discussions with the City. For this study, a threshold of 5 acre-in was identified as appropriate for defining significant flooding in these designated areas. Deficiencies identified both in the model and or by the city are highlighted with red in **Figure 4-1**; labeled with ID prefix, A##, and described in **Table 4-1**. Deficiencies identified as not warranting action will be monitored by City staff for flooding. This includes locations identified by the model as being deficient, but for which City staff were unaware of flooding problems. In **Figure 4-1**, locations that are highlighted orange and labeled with an ID prefix, B##, fall within a 'watch' list. This signifies locations where City personnel should monitor for flooding and collect additional data to determine if model updates are needed, which may either remove these locations from the 'watch' list or confirm that they should be added to a future capital improvement plan.



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Table 4-1 Storm Drainage Deficiencies†

Deficiency ID	Location	Problem Description
A01	South of Canyon Road between 2300 E and 2900 E.	Failing sumps throughout neighborhoods.
A02	1100 E from Patriots Park south.	No trunkline draining Patriots Park south to existing trunkline.
A03	300 E and Scenic Drive to Mill Race Canal.	Tie in drainage from 300 E into Mill Race Canal.
A04	Spanish Fork Parkway from 970 N to Heritage.	No overflow capacity from failing sumps in neighborhoods east of railroad tracks.
A05	200 E from railroad tracks to outfall in Dry Creek.	Significant surcharging along 200 E.
A06	100 S from Spanish Fields Drive east to Spanish Trails Blvd.	Surcharging along 100 S.
A07	400 S from 300 E to 820 E Canyon Road.	Failing sumps along 400 S.
A08	Outfall of Mill Race Canal into Spanish Fork River	370 LF of 54" dia CMP.
A09	Trunkline along Scenic Drive.	160 LF of 18" dia CMP, 1,165 LF of 24" dia CMP, and 2,200 LF of 36" dia CMP.
A10	Trunkline along Canyon Road west to 2300 E.	605 LF of 18" dia CMP and 265 LF of 24" dia CMP.
A11	Trunkline along River Bottom Road outfalling into Spanish Fork River	1,340 LF of 24" dia CMP.
A12	Previous enclosed irrigation ditch along Canyon Road between 1700 E and 1975 E Canyon Road.	1,180 LF of 21" dia CMP.
A13	Trunkline on Main Street from 700 N to 1000 N and under trail.	420 LF of 18" dia CMP, 50 LF of 24" dia CMP, 245 LF of 30" dia CMP, and 305 LF of 48" dia CMP.
A14	Trunkline along Kirby Lane from Canyon Creek Parkway to Commerce Way.	1,115 LF of 48" dia CMP.
A15	Trunkline along US-6 west from Expressway Lane and under US-6 from 800 N.	205 LF of 18" dia CMP and 1,200 LF of 24" dia CMP.

Projects are not ordered by significance.

CHAPTER 5 – CAPITAL IMPROVEMENT PLAN

This Capital Improvement Plan (CIP) presents the problems, alternatives, and recommendations identified in the study to improve storm drainage in the City of Spanish Fork. The CIP was developed from the hydrologic models, deficiency analysis, and workshops with City personnel.

PREFERRED STORM DRAINAGE PLAN DEVELOPMENT

The solution to correct a deficiency is based on the deficiency type. Some deficiencies, such as maintenance or replacement, have a straightforward linear resolution. Resolving a deficiency related to flooding, however, often has multiple solution options. Where these deficiencies existed, HAL evaluated possible alternatives and whether they could be resolved with upstream detention and/or required a pipe or inlet improvement. Selection of the preferred alternative for each problem was a process of evaluation and refinement rather than a simple choice between alternatives.

The process of selecting a preferred alternative included:

- reviewing the list of storm drainage inadequacies
- brainstorming possible solutions
- screening alternatives based on feasibility and public acceptance
- developing alternatives
- comparing cost, function, human safety, and damage prevention
- selecting the preferred alternative

Design criteria included:

- 10-year minimum capacity
- 25-year detention capacity
- 100-year detention/retention capacity

HAL has completed several small area studies assisting Spanish Fork with determining and updating master planned release rates, conveyance sizes, and levels of service for parts of the City where future growth is expected to occur. The master planned infrastructure for storm drainage may not fall within the typical level of service as outlined in 3.1.1 but were determined by the City to provide cost effective solutions for storm drainage management in the small area study locations.

PRECISION OF COST ESTIMATES

When considering cost estimates, there are several levels or degrees of precision depending on the purpose of the estimate and the percentage of detailed design that has been completed. The following levels of precision are typical:

<u>Type of Estimate</u>	<u>Precision</u>
Master Planning	±50%
Preliminary Design	±30%
Final Design or Bid	±10%

For example, at the Master Planning level (or conceptual or feasibility design level), if a project is estimated to cost \$1,000,000, then the precision or reliability of the cost estimate would typically be expected to range between approximately \$500,000 and \$1,500,000. While this may seem very imprecise, the purpose of Master Planning is to develop general sizing, location, cost, and scheduling information on several individual projects that may be designed and constructed over a period of many years. Master Planning also typically includes the selection of common design criteria to help ensure consistency and compatibility among future individual projects. Details such as the exact capacity of individual projects, the level of redundancy, the location of facilities, the alignment and depth of pipelines, the extent of utility conflicts, the cost of land and easements, the construction methodology, the types of equipment and materials to be used, the time of construction, interest, inflation rates, permitting requirements, etc., are typically developed during the more detailed levels of design.

At the preliminary or 10% design level, some of the aforementioned information will have been developed. Major design decisions such as the size of facilities, selection of facility sites, pipeline alignments and depths, and the selection of the types of equipment and materials to be used during construction will typically have been made. At this level of design, the precision of the cost estimate for a \$1,000,000 project would typically be expected to range between approximately \$700,000 and \$1,300,000.

After the project has been completely designed and is ready to bid, all design plans and technical specifications will have been completed and nearly all the significant details about the project should be known. At this level of design, the precision of the cost estimate for the same \$1,000,000 project would typically be expected to range between approximately \$900,000 and \$1,100,000.

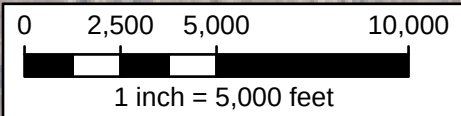
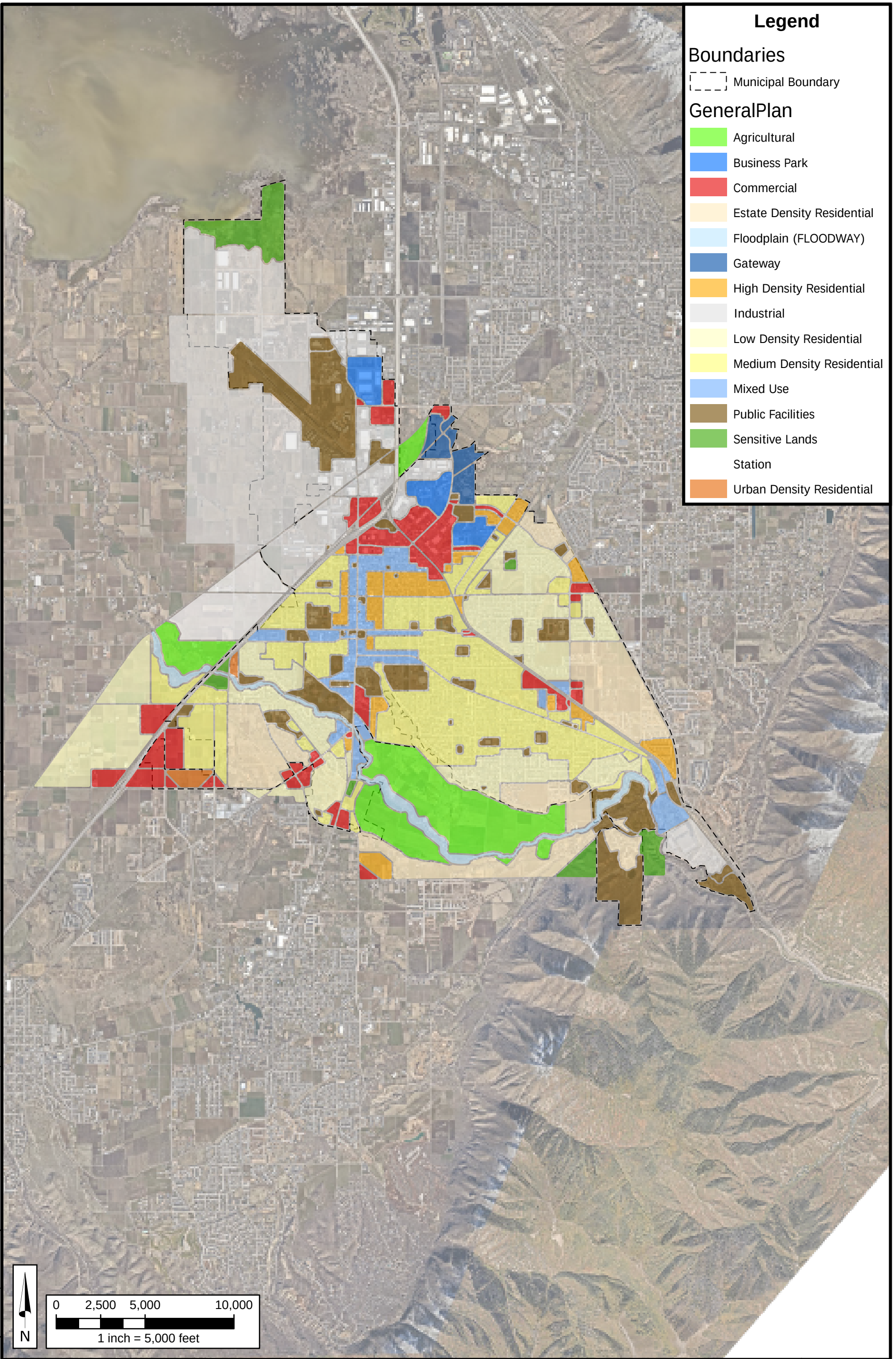
The flows and pipe diameters provided in the following Capital Improvement Plan (CIP) descriptions are approximate and are for planning purposes only. A detailed hydrologic and hydraulic analysis shall be performed during the design process of the projects to identify final design and sizing.

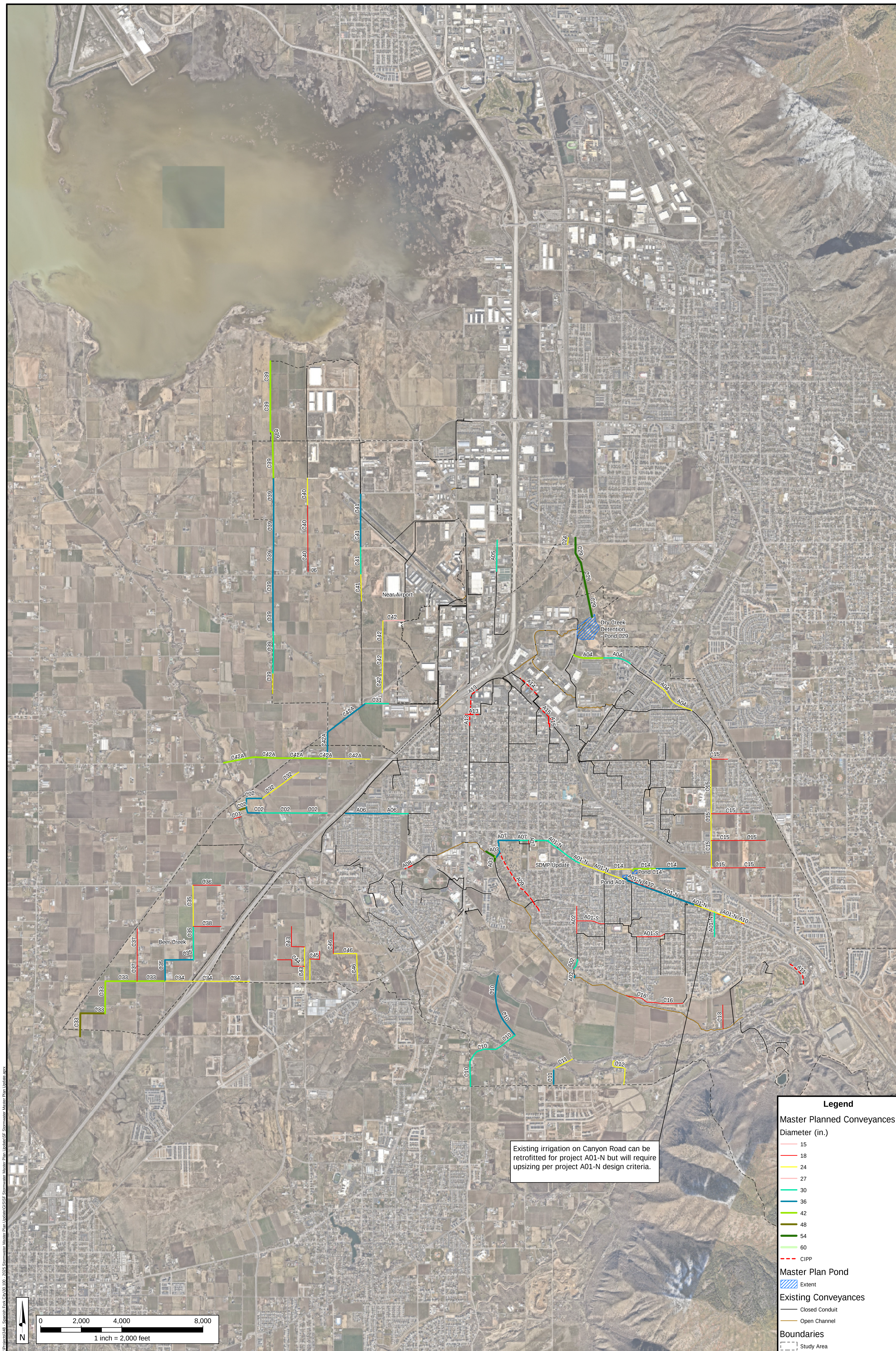
ESTIMATED CONSTRUCTION COSTS

Cost estimates are based on conceptual-level engineering. Unit construction costs were estimated based on construction cost indices (ENR 2025), communication with material suppliers, heavy construction data references (RSMeans 2025), and HAL's experience with similar construction. Engineering cost estimates given in this study should be regarded as conceptual and appropriate for use as a planning guide. Only during final design can a definitive and more accurate estimate be provided. Cost estimate assumptions of each alternative is provided in **Appendix C**.

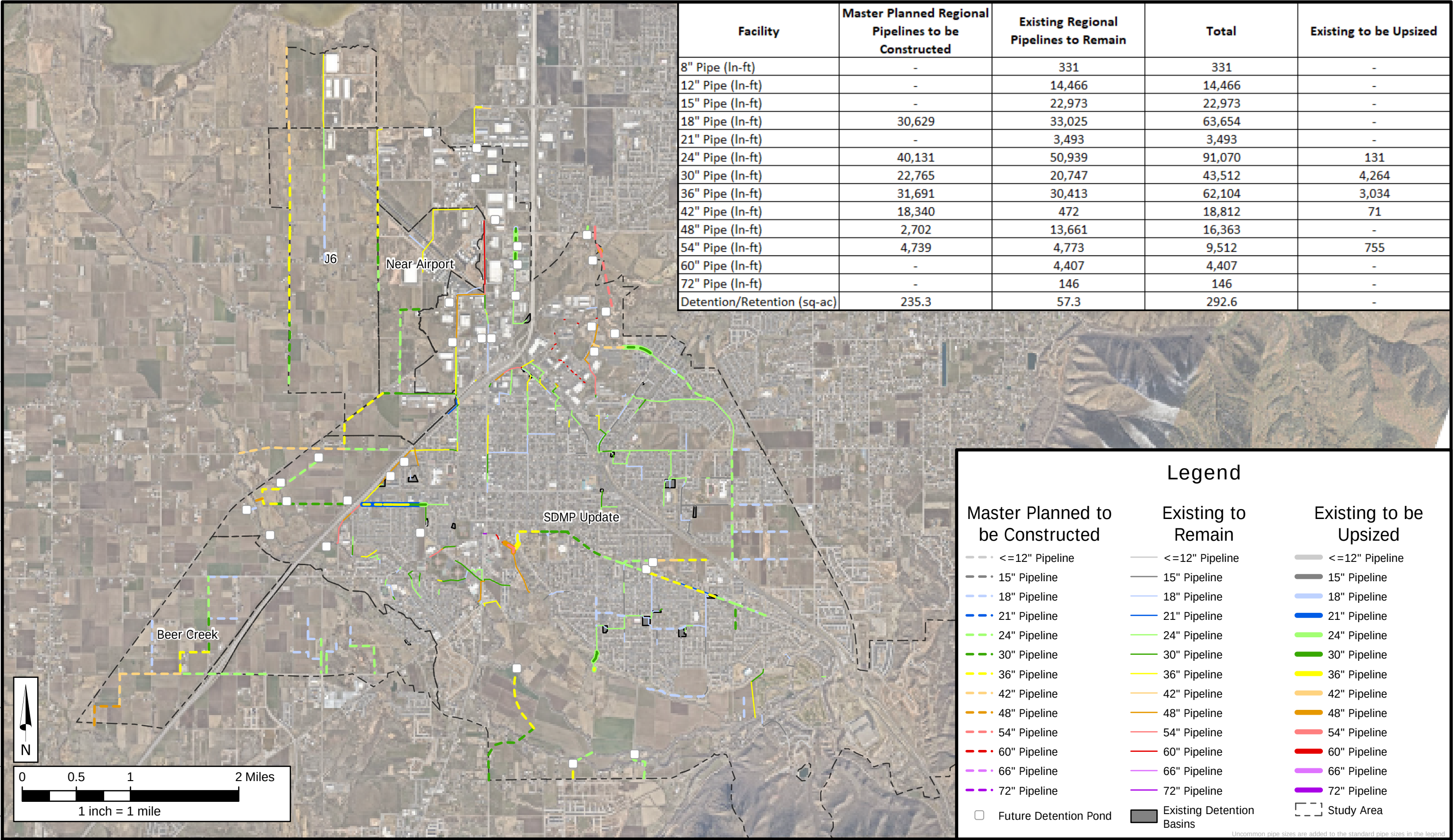
CAPITAL IMPROVEMENT PLAN

The CIP project list was based on a master planned zoning and future land use map provided by the City. Future land use used the general plan (**Figure 5-1**) and zoning with the same hydrologic land use assumptions and curve number associations found in **Tables 3-2** and **3-3** respectively. A figure of the future hydrologic land use assumptions can be seen on **Figure D-1**. **Table 5-1** presents the recommended capital improvements which are shown on **Figures 5-2** and **5-2A**. **Table 5-2** includes the CIP for future storm drainage alignment projects including the previously completed small area studies (J6, Near Airport, and Beer Creek); see **Figure 5-2** for general





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Facility	Master Planned Regional Pipelines to be Constructed	Existing Regional Pipelines to Remain	Total	Existing to be Upsized
8" Pipe (ln-ft)	-	331	331	-
12" Pipe (ln-ft)	-	14,466	14,466	-
15" Pipe (ln-ft)	-	22,973	22,973	-
18" Pipe (ln-ft)	30,629	33,025	63,654	-
21" Pipe (ln-ft)	-	3,493	3,493	-
24" Pipe (ln-ft)	40,131	50,939	91,070	131
30" Pipe (ln-ft)	22,765	20,747	43,512	4,264
36" Pipe (ln-ft)	31,691	30,413	62,104	3,034
42" Pipe (ln-ft)	18,340	472	18,812	71
48" Pipe (ln-ft)	2,702	13,661	16,363	-
54" Pipe (ln-ft)	4,739	4,773	9,512	755
60" Pipe (ln-ft)	-	4,407	4,407	-
72" Pipe (ln-ft)	-	146	146	-
Detention/Retention (sq-ac)	235.3	57.3	292.6	-



SPANISH FORK CITY

STORMDRAIN MASTER PLAN
Updated With Recent Studies

FIGURE
5-2A

project locations with project ID and **Figure 5-2A** for comparison between master planned conveyances for existing versus buildout conveyances. There are a multitude of release rates and designs for different levels of service, the levels of service and release rates are defined both in **Figure 5-3** which is a release rate map of Spanish Fork. Regional pond tributary area with required total volume and cumulative peak outflow are in **Figure 5-4**. Detention sizing, pipe sizing, costs, and level of service are in **Appendices B and C**.

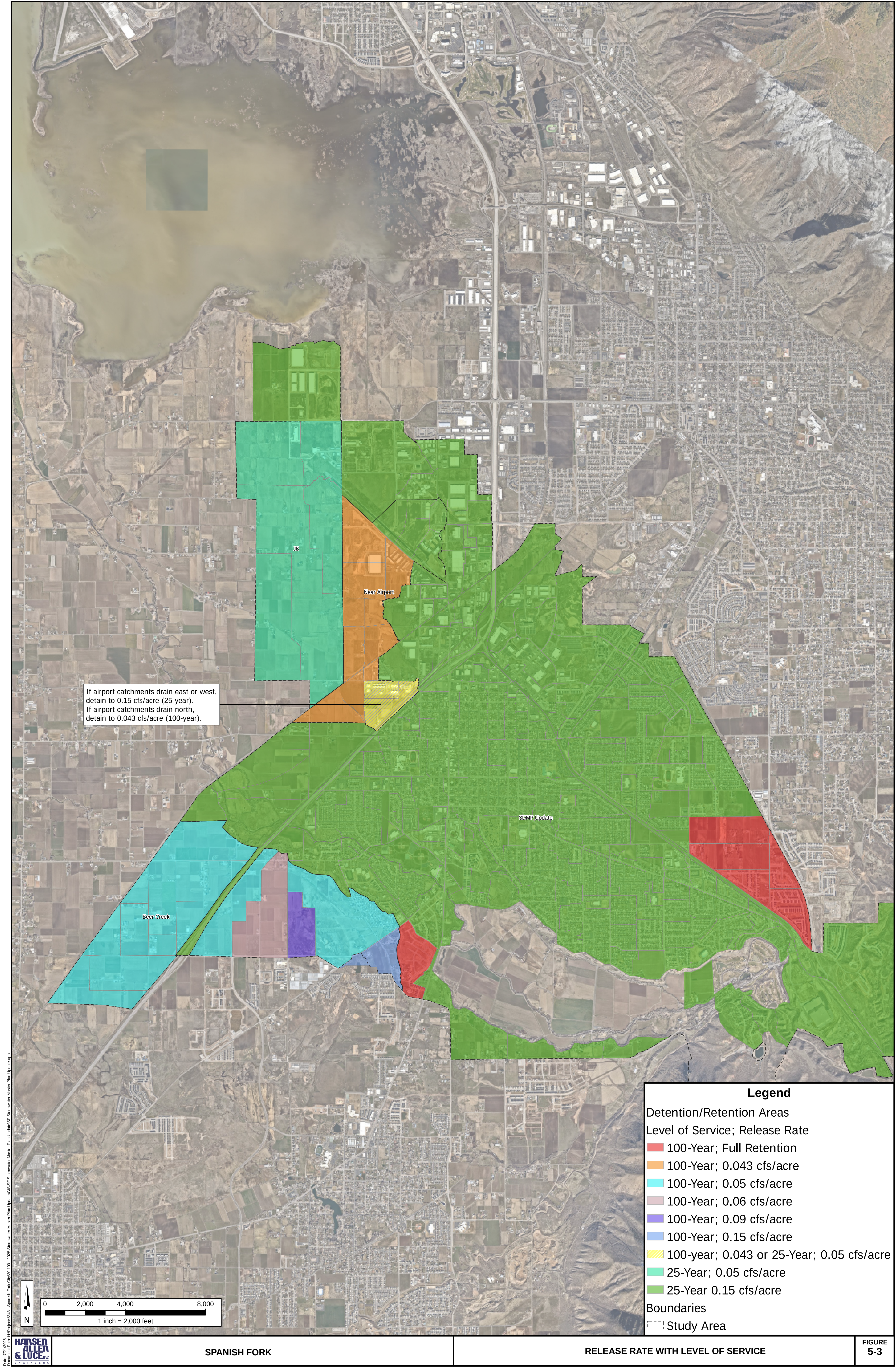


Table 5-1 Capital Improvement Plan for Existing Projects

Project ID	Deficiency ID	Location	Problem Description	Preferred Solution	Total with Eng & Cont.	Impact Fee
Spanish Fork City Master Plan Projects						
A01-N	A01	South of Canyon Road between 2300 E and 2900 E.	Failing sumps throughout neighborhoods.	Route flows north on 2600 E and west along Canyon Road (A01-N, A07, A03). Retrofitting and upsizing existing Canyon Road conveyance. Installation of 3 acre-foot detention pond.	\$8,695,000	-
A01-S	A01	South of Canyon Road between 2300 E and 2900 E.	Failing sumps throughout neighborhoods.	Connect Abbie Court detention to Canyon Elementary School detention and further west to 1100 E trunkline (A01-S, A02).	\$1,204,000	-
A02	A02	1100 E from Patriots Park south.	Missing trunkline from Patriots Park south to existing trunkline.	Add 24" dia. trunkline from Patriots Park south to existing trunkline and increase trunkline from 1750 S to outfall at Mill Race Canal to 30" and 36" dia (A02).	\$911,000	-
A03	A03	300 E and Scenic Drive to Mill Race Canal.	Tie in drainage from 300 E into Mill Race Canal.	Provide 36" dia. trunkline bypass from failing sumps on 400 S to 300 E south to continued 36" trunkline down to Mill Race Canal (A07, A03). Upsize Mill Race Canal in location of 300 E trunkline outfall to 54" dia. (A03).	\$1,090,000	-
A04	A04	Spanish Fork Parkway from 970 N to Heritage.	Improve overflow capacity from failing sumps in neighborhoods east of railroad tracks.	Provide connection and increase capacity for overflow by installing 24", 30", and 42" trunklines along Spanish Fork Parkway (A04).	\$3,599,000	-
A05	A05	200 E from railroad tracks to outfall in Dry Creek.	Significant surcharging along 200 E.	Upsize trunkline to 30" dia and provide detention for tributary areas south of railroad tracks, detaining to 0.15 cfs/acre (A05).	\$2,312,000	-

Project ID	Deficiency ID	Location	Problem Description	Preferred Solution	Total with Eng & Cont.	Impact Fee
Spanish Fork City Master Plan Projects						
A06	A06	100 S from Spanish Fields Drive east to Spanish Trails Blvd.	Surcharging along 100 S.	Improve conveyance capacity along 100 S by upsizing trunkline east of 630 W to 30" and 36" dia. pipe (A06).	\$2,440,000	-
A07	A07	400 S from 300 E to 820 E Canyon Road.	Failing sumps along 400 S.	Provide 36" dia. trunkline bypass from failing sumps on 400 S to 300 E south to continued 36" trunkline down to Mill Race Canal (A07, A03). Upsize Mill Race Canal in location of 300 E trunkline outfall to 54" dia. (A03).	\$2,385,000	-
A08	A08	Outfall of Mill Race Canal into Spanish Fork River	370 LF of 54" dia CMP.	CIPP	\$496,000	-
A09	A09	Trunkline along Scenic Drive.	160 LF of 18" dia CMP, 1,165 LF of 24" dia CMP, and 2,200 LF of 36" dia CMP.	CIPP	\$2,515,000	-
A10	A10	Trunkline along Canyon Road west to 2300 E.	605 LF of 18" dia CMP and 265 LF of 24" dia CMP.	CIPP	\$347,000	-
A11	A11	Trunkline along River Bottom Road outfalling into Spanish Fork River	1,340 LF of 24" dia CMP.	CIPP	\$690,000	-
A12	A12	Previous enclosed irrigation ditch along Canyon Road between 1700 E and 1975 E Canyon Road.	1,180 LF of 21" dia CMP.	CIPP	\$511,000	-
A13	A13	Trunkline on Main Street from 700 N to 1000 N and under trail.	420 LF of 18" dia CMP, 50 LF of 24" dia CMP, 245 LF of 30" dia CMP, and 305 LF of 48" dia CMP.	CIPP	\$1,840,000	-

Project ID	Deficiency ID	Location	Problem Description	Preferred Solution	Total with Eng & Cont.	Impact Fee
Spanish Fork City Master Plan Projects						
A14	A14	Trunkline along Kirby Lane from Canyon Creek Parkway to Commerce Way.	1,115 LF of 48" dia CMP.	CIPP	\$1,313,000	-
A15	A15	Trunkline along US-6 west from Expressway Lane and under US-6 from 800 N.	205 LF of 18" dia CMP and 1,200 LF of 24" dia CMP.	CIPP	\$690,000	-
Spanish Fork City Projects Subtotal					\$31,038,000	-

Total with Engineering and Contingency assumes a 30% on project cost.

Table 5-2 Capital Improvement Plan for Buildout Projects

Project ID	Diameter (in.); Length (ft.)	Total with Eng. & Cont.
C01*	(18; 362)	\$257,000
C02*	(48; 344),(36; 2,168),(30; 3,280)	\$4,897,000
C03	Detention	\$384,000
C04	Detention	\$859,000
C05	Detention	\$2,527,000
C06	Detention	\$343,000
C07	Detention	\$143,000
C08	Detention	\$93,000
C09-A	Detention	\$65,000
C09-B	Detention	\$65,000
C10*	(36; 3,219),(30; 3,969)	\$6,455,000
C11*	(24; 929),(36; 689)	\$1,172,000
C12*	(24; 1,702)	\$931,000
C13	(18; 1,167)	\$401,000
C14*	(42; 1,213),(24; 1,488),(36; 1,420)	\$3,919,000
C15	(18; 8,101),(24; 5,412)	\$6,208,000
C16	(18; 2,906)	\$1,253,000
C17	Detention	\$1,801,000
C18	Detention	\$2,693,000
C19	Detention	\$6,486,000
C20	Detention	\$1,670,000
C21	Detention	\$1,941,000
C22	Detention	\$950,000
C23	Detention	\$896,000
C24	Detention	\$2,182,000
C25	Detention	\$2,951,000
C26	Detention	\$1,560,000
C27	Detention	\$2,279,000
C28	Detention	\$1,150,000
C29*	(36; 955),(24; 2,731)	\$4,848,000
C30	Detention	\$730,000
C31	(30; 2,457)	\$709,000
C32*	(24; 2,212)	\$1,590,000
C33*	(48; 2,357),(42; 4,541)	\$7,949,000
C34*	(24; 4,159)	\$2,505,000
C35	(30; 1,638),(36; 2,453),(24; 2,036)	\$4,142,000

Project ID	Diameter (in.); Length (ft.)	Total with Eng. & Cont.
C36	(18; 1,372)	\$592,000
C37	(18; 2,581)	\$1,113,000
C38	(18; 1,372)	\$471,000
C39	(24; 977),(30; 2,080),(36; 7,537),(42; 5,871)	\$12,843,000
C40	(18; 3,261),(24; 1,890)	\$2,354,000
C41	(36; 2,622),(30; 1,334),(24; 1,314)	\$3,734,000
C42	(24; 3,546),(27; 1,076)	\$2,408,000
C42A	(42; 5,132),(36; 3,644),(24; 2,085)	\$9,227,000
C43	(18; 1,640),(24; 1,638)	\$1,385,000
C44	(18; 1,636)	\$562,000
C45	(18; 881),(24; 1,037)	\$729,000
C46	(24; 2,488),(18; 1,038)	\$1,378,000
Spanish Fork City Buildout Projects Subtotal		\$116,509,000

Total with Engineering and Contingency assumes a 30% on project cost. Costs with * also include detention.

SUMMARY OF CAPITAL IMPROVEMENTS

Costs of the Capital Improvements are summarized in **Table 5-3**.

Table 5-3 Capital Improvement Plan Summary

Projects	Cost	Impact Fee Eligible
City of Spanish Fork Existing Projects	\$31,038,000	-
City of Spanish Fork Buildout Projects	\$116,509,000	-
Total Cost	\$147,547,000	-

OPERATIONAL RECOMMENDATIONS

Pipe Type

Some storm drainage pipes in Spanish Fork are composed of materials with high roughness, such as CMP, which has an assumed roughness of 0.024. Manning's equation and modeling of the system suggests that generally many of the rougher pipes, such as corrugated metal, significantly reduce the capacity to convey 10-year, 3-hour flows. By decreasing the roughness by half, the capacity of the pipe is roughly doubled. If any of these pipes are due for replacement the City should consider utilizing a smoother pipe as a replacement to provide the additional capacity needed. A particularly useful time for replacing CMP pipe is during road reconstruction. If replacement is not a reasonable solution, lining the pipe is another option to improve surface roughness. When evaluating the lining option, consideration should be given to the decrease in cross-sectional area and how it impacts the overall pipe capacity.

Inventory

The current Master Plan consolidated different inventories of the Spanish Fork storm drainage network. These inventories include an inventory contained as a SWMM model and a GIS inventory of the system. Multiple inventories increase the likelihood of incorrect or outdated information being used and require more time to maintain. For example, as part of this Master Plan, the City sent HAL their GIS data and the models from the previous Master Plan. Both datasets were evaluated, and it was evident that discrepancies and missing data existed between the two. This required additional time to integrate all updated GIS data into the current model.

Therefore, it is recommended that the City maintain and update a master GIS inventory of the storm drainage system, ensuring that basic model inputs are available. These basic model inputs include ID, inverts, sizes, shape, material, upstream/downstream ID, and rim elevations. The storm drainage model should be maintained and updated based upon the master GIS inventory. The GIS inventory should be maintained separate from the SWMM model to increase accessibility by City personnel. Updates should occur as information about additional land use, conveyance, capacity, and detention data become available.

Irrigation and Storm Drainage Conveyances

Several older areas of the Spanish Fork Storm drainage system have conveyances which serve as both storm drainage facilities and irrigation facilities. The dual stormwater/ irrigation conveyances generally lack the capacity to convey the 10-year flow through the system.

Watch and Maintenance Recommendations

Areas identified as not warranting immediate action include those shown in **Figure 4-1** and classified as “areas to watch.” These areas should be monitored for future flooding. If conditions become unacceptable, a project should be added to the CIP to remedy the deficiency. Further analysis should be conducted to understand the impact of storm drainage on the Spanish Fork River and Mill Race Canal given the 10-year, 25-year, and 100-year events. Complementary to addressing capacity concerns for the City’s open channel conveyances via analysis, HAL recommends annual or bi-annual maintenance on the City’s open channels. Maintenance can and should be done on the City’s open channel storm drainage conveyances.

Storm Drainage Master Plan Updates

The storm drainage master plan should be periodically reviewed and updated dependent upon change and new development, at least every 5 years.

CHAPTER 6 – REFERENCES

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APPENDIX A

System Data

APPENDIX B

Capital Projects Detention

Table B-1 Detention Design Parameters

Pond ID	Contributing Area	Contributing Area	Release Rate	Allowable Discharge	Volume	Depth
	Subbasins	Acres	cfs/Acre	cfs	AF	Feet
Pond A01	SFE93, SFE91, SFE1, SFE7-A,	179	0.10	18	3	3
Pond A05	DCW4, DCW42	92	0.15	14	5	3
Pond C01	SFW9	54	0.15	8	0.3	3
Pond C02	SFW5	127	0.15	19	3.2	3
Pond C03	SFW3	156	0.15	23	1.4	3
Pond C04	SFW4	101	0.15	15	3.2	3
Pond C05	SWF6	159	0.15	24	9.7	3
Pond C06	SFE86	48	0.15	7	1.2	3
Pond C07	SFE62	12	0.15	2	0.4	3
Pond C08	SFE83	7	0.15	1	0.2	3
Pond C09-A	SFE25	10	0.15	2	0.1	3
Pond C09-B	SFE25	10	0.15	2	0.1	3
Pond C10	SFE40	187	0.15	28	4.6	3
Pond C11	SFE41	87	0.15	13	0.7	3
Pond C12	SFE44	47	0.15	7	0.2	4
Pond C14	SFE75, SFE32, SFE81	70	0.15	10	2.8	3
Pond C17	DCW16	101	0.15	15	6.9	3
Pond C18	DCW17	129	0.15	19	10.3	3
Pond C19	DCW15	353	0.15	53	25.1	3
Pond C20	DCW41	78	0.15	12	6.4	3
Pond C21	DCW5	90	0.15	13	7.4	3
Pond C22	DCW40	51	0.15	8	3.6	3
Pond C23	DCW1, DCW2	30	0.15	5	3.4	3
Pond C24	DCN36	124	0.15	19	8.3	3
Pond C25	DCN34	159	0.15	24	11.3	3
Pond C26	DCN21	83	0.15	12	5.9	3
Pond C27	DCN20	162	0.15	24	8.7	3
Pond C28	DCN35	166	0.15	25	4.3	3
Pond C29	Dry Creek	0	0.15	10	96	9
Pond C30	DCW11	34	0.15	5	2.7	3
Pond C32	SFW7	54	0.15	8	2.5	3
Pond C33	DCW3	54	0.15	8	3.7	3
Pond C34	DCW12	21	0.15	3	1.5	3

APPENDIX C

Cost Data

Table C-1 Itemized Table of Pipe Segments in CIP Projects

Project ID	Pipe ID	Level of Service (yr)	In Street	Length (feet)	Design Flow (cfs)	Size (inches)	\$/LF	Cost	Total Cost	Engineering Cost and Cont.
A01-N	SDP-A01-A-6	25	Y	1122	6	24	\$386	\$433,237	\$6,060,128	\$7,878,166
	SDP-A01-0	10	Y	1209	22	30	\$480	\$579,989		
	SDP-A01-A-0	10	Y	1573	9	24	\$386	\$607,224		
	SDP-A01-A-1	10	Y	1047	16	24	\$386	\$404,315		
	SDP-A01-A-2	10	Y	3289	37	36	\$658	\$2,163,384		
	SDP-A01-A-3	10	Y	222	56	36	\$658	\$145,995		
	SDP-A01-A-5	10	Y	170	56	36	\$658	\$111,672		
	SDP-A01-A-7	25	Y	1173	14	24	\$386	\$452,642		
	SDP-A01-A-8	25	Y	1932	34	30	\$480	\$926,486		
	SDP-A01-A-9	25	Y	490	62	30	\$480	\$235,183		
A01-S	SDP-A01-1	10	Y	1407	6	18	\$332	\$466,524	\$926,097	\$1,203,926
	SDP-A01-2	10	Y	1386	7	18	\$332	\$459,573		
A02	SDP02635	10	N	83	111	36	\$542	\$44,837	\$700,436	\$910,567
	SDP02636	10	N	198	25	30	\$407	\$80,427		
	SDP02681	10	N	55	37	30	\$407	\$22,549		
	SDP02683	10	N	33	69	30	\$407	\$13,386		
	SDP02684	10	N	181	66	30	\$407	\$73,684		
	SDP02746	10	N	50	109	30	\$407	\$20,337		
	SDP-A02-0	10	Y	725	2	18	\$332	\$240,270		
	SDP-A02-1	10	Y	618	5	18	\$332	\$204,948		
A03	SDP00091	10	N	71	273	42	\$642	\$45,395	\$838,344	\$1,089,848
	SDP02872	10	N	284	277	36	\$542	\$154,100		
	SDP03512	10	Y	197	81	54	\$1,044	\$205,374		
	SDP03513	10	N	509	75	54	\$852	\$433,475		
A04	SDP05070	10	Y	1127	72	30	\$480	\$540,629	\$2,767,763	\$3,598,091
	SDP05525	10	Y	246	62	30	\$480	\$117,969		
	SDP11458	10	Y	131	41	24	\$386	\$50,715		
	SDP-A04-0	10	Y	2359	32	24	\$386	\$910,695		
	SDP-A04-1	10	Y	1512	63	42	\$759	\$1,147,755		
A05	SDP01926	10	Y	164	17	30	\$480	\$78,783	\$731,465	\$950,905

Project ID	Pipe ID	Level of Service (yr)	In Street	Length (feet)	Design Flow (cfs)	Size (inches)	\$/LF	Cost	Total Cost	Engineering Cost and Cont.
	SDP01929	10	Y	602	24	30	\$480	\$288,606		
	SDP02078	10	Y	759	18	30	\$480	\$364,077		
A06	SDP00714	10	Y	351	49	36	\$658	\$230,829	\$1,876,362	\$2,439,271
	SDP00968	10	Y	234	14	30	\$480	\$111,984		
	SDP01884	10	Y	136	14	30	\$480	\$65,425		
	SDP01885	10	Y	88	29	30	\$480	\$42,376		
	SDP03219	10	Y	362	34	36	\$658	\$237,941		
	SDP03220	10	Y	189	44	36	\$658	\$124,394		
	SDP03221	10	Y	82	35	30	\$480	\$39,346		
	SDP03222	10	Y	308	31	30	\$480	\$147,903		
	SDP03223	10	Y	435	19	36	\$658	\$286,232		
	SDP03224	10	Y	566	35	36	\$658	\$372,061		
	SDP03229	10	Y	65	21	36	\$658	\$42,529		
	SDP05762	10	Y	54	33	36	\$658	\$35,541		
	SDP05763	10	Y	187	65	36	\$658	\$123,143		
	SDP05788	10	Y	25	5	36	\$658	\$16,659		
A07	SDP02864	10	Y	251	46	36	\$658	\$164,952	\$1,834,485	\$2,384,830
	SDP02871	10	Y	182	46	36	\$658	\$119,949		
	SDP-A07-0	10	Y	513	62	30	\$480	\$245,868		
	SDP-A07-1	10	Y	471	90	30	\$480	\$225,936		
	SDP-A07-2	10	Y	449	73	30	\$480	\$215,230		
	SDP-A07-3	10	Y	1223	72	36	\$658	\$804,734		
	SDP-A07-4	10	Y	174	16	18	\$332	\$57,815		
C01	SDP-C01-0	25	Y	362	15	18	\$332	\$120,092	\$120,092	\$156,119
C02	SDP-C02-0	25	N	344	63	48	\$749	\$257,795	\$3,113,063	\$4,046,982
	SDP-C02-1	25	N	1251	31	36	\$542	\$678,343		
	SDP-C02-2	25	Y	918	43	36	\$658	\$603,746		
	SDP-C02-3	25	Y	3280	29	30	\$480	\$1,573,179		
C10	SDP-C10-0	10	Y	3219	77	36	\$658	\$2,117,047	\$4,020,518	\$5,226,673
	SDP-C10-1	10	Y	3969	17	30	\$480	\$1,903,471		
C11	SDP-C11-0	25	N	929	26	24	\$316	\$293,615	\$747,015	\$971,112

Project ID	Pipe ID	Level of Service (yr)	In Street	Length (feet)	Design Flow (cfs)	Size (inches)	\$/LF	Cost	Total Cost	Engineering Cost and Cont.
	SDP-C11-1	10	Y	689	31	36	\$658	\$453,400		
C12	SDP-C12-0	10	Y	1702	14	24	\$386	\$657,038	\$657,038	\$854,150
C13	SDP-C13-0	25	N	1167	9	18	\$264	\$307,903	\$307,903	\$400,274
C14	SDP-C14-0	10	Y	1213	58	42	\$759	\$920,966	\$2,429,585	\$3,158,461
	SDP-C14-1	25	Y	1488	16	24	\$386	\$574,587		
	SDP-C14-2	10	Y	1420	13	36	\$658	\$934,033		
C15	SDP-C15-0	Overflow	Y	2662	7	18	\$332	\$882,422	\$4,774,752	\$6,207,178
	SDP-C15-2	Overflow	Y	2674	3	18	\$332	\$886,344		
	SDP-C15-3	Overflow	Y	1320	10	24	\$386	\$509,416		
	SDP-C15-4	Overflow	Y	1918	2	18	\$332	\$635,775		
	SDP-C15-5	Overflow	Y	2741	21	24	\$386	\$1,057,955		
	SDP-C15-6	Overflow	Y	848	1	18	\$332	\$281,187		
	SDP-C15-1	Overflow	Y	1351	14	24	\$386	\$521,652		
C16	SDP-C16-0	10	Y	1584	25	18	\$332	\$525,187	\$963,326	\$1,252,325
	SDP-C16-1	10	Y	1322	14	18	\$332	\$438,140		
C29	SDOC17	25	N	905	428	36	\$542	\$490,995	\$1,380,574	\$1,794,747
	SDP01156	25	N	50	128	36	\$542	\$26,890		
	SDP-C29-0	25	N	2731	320	24	\$316	\$862,689		
C31	SDP-C31-2	25	Y	1136	16	30	\$480	\$544,721	\$544,721	\$708,138
C32	SDP-C32-0	25	N	2212	12	24	\$316	\$698,667	\$698,667	\$908,267
C33	SDP-C33-1	100	Y	1217	51	48	\$868	\$1,056,980	\$5,357,965	\$6,965,355
	SDP-C33-4	100	Y	1409	35	42	\$759	\$1,069,552		
	SDP-C33-3	100	Y	1542	41	42	\$759	\$1,170,575		
	SDP-C33-2	100	Y	1590	44	42	\$759	\$1,207,190		
	SDP-C33-0	100	N	1140	56	48	\$749	\$853,668		
C34	SDP-C34-0	100	Y	4159	8	24	\$386	\$1,605,422	\$1,605,422	\$2,087,049
C35	SDP-C35-2	100	Y	1638	16	30	\$480	\$785,790	\$3,185,699	\$4,141,408
	SDP-C35-0	100	Y	1044	25	36	\$658	\$686,605		
	SDP-C35-1	100	Y	1410	19	36	\$658	\$927,182		
	SDP-C35-3	100	Y	2036	10	24	\$386	\$786,122		
C36	SDP-C36-0	100	Y	1372	0	18	\$332	\$454,860	\$454,860	\$591,318

Project ID	Pipe ID	Level of Service (yr)	In Street	Length (feet)	Design Flow (cfs)	Size (inches)	\$/LF	Cost	Total Cost	Engineering Cost and Cont.
C37	SDP-C37-0	100	Y	2581	1	18	\$332	\$855,641	\$855,641	\$1,112,333
C38	SDP-C38-0	100	N	1372	4	18	\$264	\$361,920	\$361,920	\$470,496
C39	SDP-C39-0	25	Y	977	25	24	\$386	\$376,967	\$9,878,869	\$12,842,529
	SDP-C39-1	25	Y	2080	41	30	\$480	\$997,706		
	SDP-C39-2	25	Y	2769	55	36	\$658	\$1,821,373		
	SDP-C39-3	25	Y	1490	62	36	\$658	\$979,973		
	SDP-C39-8	25	N	4017	52	42	\$642	\$2,578,009		
	SDP-C39-5	25	Y	1362	84	36	\$658	\$895,778		
	SDP-C39-4	25	N	1917	71	36	\$542	\$1,039,696		
	SDP-C39-6	25	N	1853	101	42	\$642	\$1,189,365		
C40	SDP-C40-0	25	Y	850	0	18	\$332	\$281,826	\$1,810,662	\$2,353,860
	SDP-C40-1	25	Y	972	0	18	\$332	\$322,288		
	SDP-C40-2	25	Y	1439	0	18	\$332	\$476,932		
	SDP-C40-3	25	Y	1890	0	24	\$386	\$729,615		
C41	SDP-C41-2	25	Y	2622	45	36	\$658	\$1,724,776	\$2,871,568	\$3,733,038
	SDP-C41-1	25	Y	1334	30	30	\$480	\$639,538		
	SDP-C41-0	25	Y	1314	19	24	\$386	\$507,254		
C42	SDP-C42-4	100	Y	2160	0	24	\$386	\$833,635	\$1,851,746	\$2,407,270
	SDP-C42-5	100	Y	1387	0	24	\$386	\$535,337		
	SDP-C42-6	100	Y	1076	0	27	\$449	\$482,774		
C42A	SDP-C42A-3	10	Y	5132	0	42	\$759	\$3,895,683	\$7,097,413	\$9,226,637
	SDP-C42A-1	10	Y	3644	0	36	\$658	\$2,397,061		
	SDP-C42A-2	10	Y	2085	0	24	\$386	\$804,669		
C43	SDP-C43-3	100	N	994	7	18	\$264	\$262,219	\$1,064,945	\$1,384,428
	SDP-C43-0	100	Y	674	14	24	\$386	\$260,278		
	SDP-C43-2	100	N	646	7	18	\$264	\$170,411		
	SDP-C43-1	100	Y	964	14	24	\$386	\$372,038		
C44	SDP-C44-0	100	N	1636	7	18	\$264	\$431,729	\$431,729	\$561,247
C45	SDP-C45-1	100	N	881	6	18	\$264	\$232,411	\$560,111	\$728,144
	SDP-C45-0	100	N	1037	12	24	\$316	\$327,700		
C46	SDP-C46-0	100	N	2488	12	24	\$316	\$785,839	\$1,059,701	\$1,377,611

Project ID	Pipe ID	Level of Service (yr)	In Street	Length (feet)	Design Flow (cfs)	Size (inches)	\$/LF	Cost	Total Cost	Engineering Cost and Cont.
	SDP-C46-1	100	N	1038	6	18	\$264	\$273,862		

Assumptions include an average distance of 200 feet per manhole, one set of inlets every 100 feet of pipe, and three feet of cover and engineering with contingency is an additional 30%.

Table C-2 Itemized Table of CIP Detention

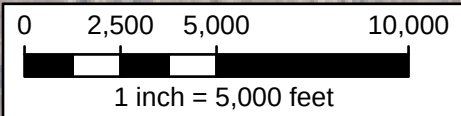
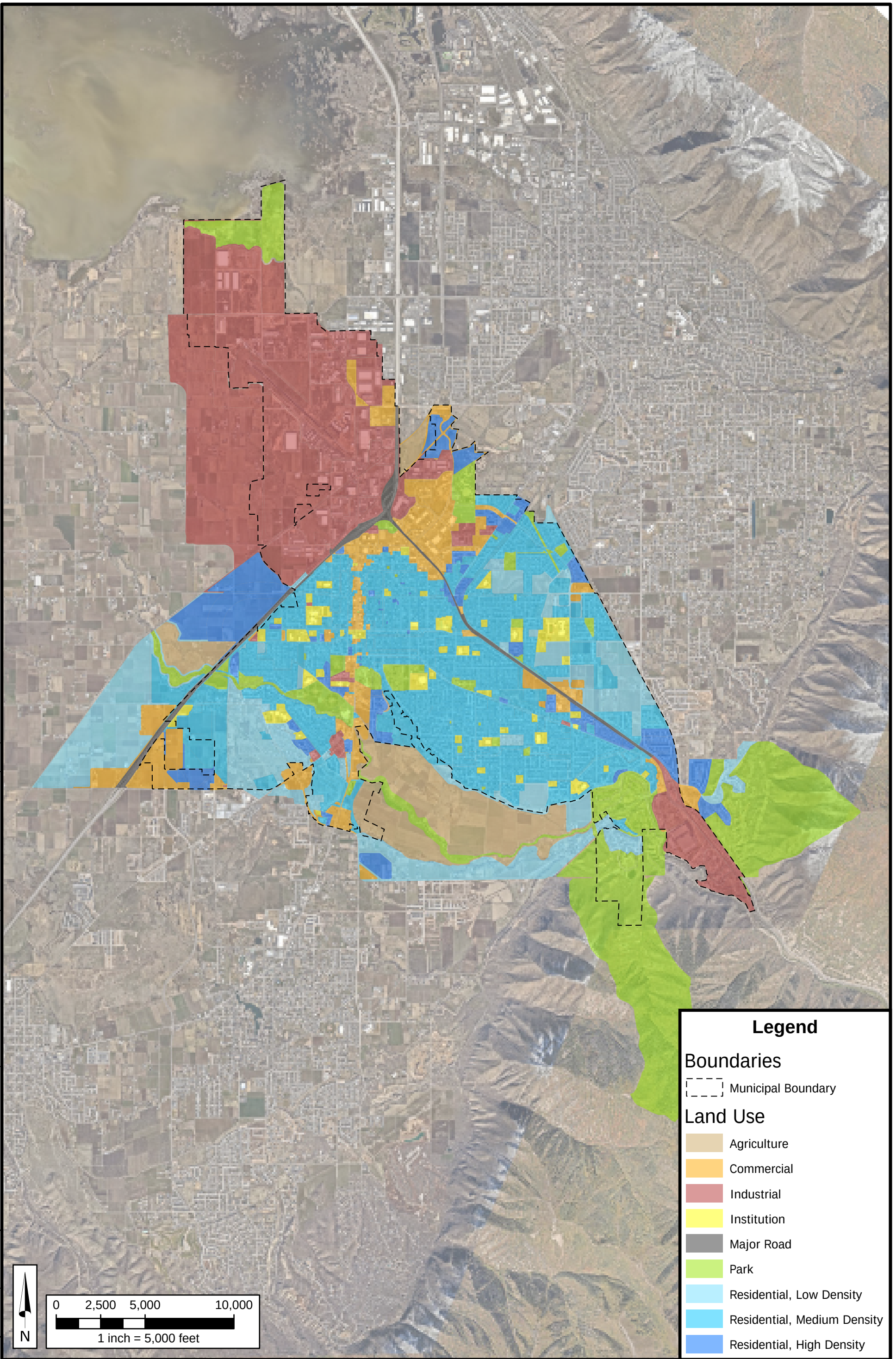
Pond ID	Cost of Excavation			Cost of Land			Cost of outlet structure		Cost of vegetation		Project Cost	With Eng & Cont.
	per AF	AF	Cost	per acre	acres	Cost	Lump Sum	25000	per sf	Cost		1.3
Pond A01	24,200	3.04	\$73,660	\$500,000	1.0	\$507,299	1	\$25,000	\$0.50	\$22,098	\$628,057	\$816,474
Pond A05	24,200	5.16	\$124,751	\$500,000	1.7	\$859,169	1	\$25,000	\$0.50	\$37,425	\$1,046,344	\$1,360,247
Pond C01	24,200	0.27	\$6,419	\$500,000	0.1	\$44,209	1	\$25,000	\$0.50	\$1,926	\$77,554	\$100,820
Pond C02	24,200	3.17	\$76,809	\$500,000	1.1	\$528,986	1	\$25,000	\$0.50	\$23,043	\$653,837	\$849,988
Pond C03	24,200	1.36	\$32,937	\$500,000	0.5	\$226,843	1	\$25,000	\$0.50	\$9,881	\$294,659	\$383,057
Pond C04	24,200	3.21	\$77,590	\$500,000	1.1	\$534,374	1	\$25,000	\$0.50	\$23,277	\$660,237	\$858,308
Pond C05	24,200	9.68	\$234,320	\$500,000	3.2	\$1,613,791	1	\$25,000	\$0.50	\$70,296	\$1,943,388	\$2,526,404
Pond C06	24,200	1.20	\$29,117	\$500,000	0.4	\$200,531	1	\$25,000	\$0.50	\$8,735	\$263,380	\$342,394
Pond C07	24,200	0.43	\$10,372	\$500,000	0.1	\$71,430	1	\$25,000	\$0.50	\$3,111	\$109,912	\$142,886
Pond C08	24,200	0.23	\$5,646	\$500,000	0.1	\$38,884	1	\$25,000	\$0.50	\$1,694	\$71,223	\$92,590
Pond C09-A	24,200	0.13	\$3,043	\$500,000	0.0	\$20,956	1	\$25,000	\$0.50	\$913	\$49,911	\$64,884
Pond C09-B	24,200	0.13	\$3,043	\$500,000	0.0	\$20,956	1	\$25,000	\$0.50	\$913	\$49,911	\$64,884
Pond C10	24,200	4.64	\$112,282	\$500,000	1.5	\$773,292	1	\$25,000	\$0.50	\$33,685	\$944,258	\$1,227,536
Pond C11	24200	0.65	\$15,791	\$500,000	0.2	\$108,757	1	\$25,000	\$0.50	\$4,737	\$154,282	\$200,567
Pond C12	24200	0.17	\$4,143	\$500,000	0.1	\$28,531	1	\$25,000	\$0.50	\$1,243	\$58,915	\$76,590
Pond C14	24200	2.83	\$68,404	\$500,000	0.9	\$471,115	1	\$25,000	\$0.50	\$20,521	\$585,025	\$760,533
Pond C17	24200	6.86	\$166,086	\$500,000	2.3	\$1,143,882	1	\$25,000	\$0.50	\$49,826	\$1,384,757	\$1,800,184
Pond C18	24200	10.33	\$249,909	\$500,000	3.4	\$1,721,192	1	\$25,000	\$0.50	\$74,973	\$2,071,015	\$2,692,319
Pond C19	24200	25.06	\$606,350	\$500,000	8.4	\$4,176,114	1	\$25,000	\$0.50	\$181,905	\$4,989,219	\$6,485,984
Pond C20	24200	6.36	\$153,803	\$500,000	2.1	\$1,059,287	1	\$25,000	\$0.50	\$46,141	\$1,284,191	\$1,669,448
Pond C21	24200	7.41	\$179,275	\$500,000	2.5	\$1,234,727	1	\$25,000	\$0.50	\$53,783	\$1,492,735	\$1,940,556
Pond C22	24200	3.56	\$86,139	\$500,000	1.2	\$593,267	1	\$25,000	\$0.50	\$25,842	\$730,222	\$949,289
Pond C23	24200	3.35	\$81,095	\$500,000	1.1	\$558,527	1	\$25,000	\$0.50	\$24,328	\$688,925	\$895,603
Pond C24	24200	8.34	\$201,931	\$500,000	2.8	\$1,390,770	1	\$25,000	\$0.50	\$60,579	\$1,678,216	\$2,181,681

Pond ID	Cost of Excavation			Cost of Land			Cost of outlet structure		Cost of vegetation		Project Cost	With Eng & Cont.
	per AF	AF	Cost	per acre	acres	Cost	Lump Sum	25000	per sf	Cost		1.3
Pond C25	24200	11.33	\$274,200	\$500,000	3.8	\$1,888,517	1	\$25,000	\$0.50	\$82,260	\$2,269,886	\$2,950,852
Pond C26	24200	5.93	\$143,442	\$500,000	2.0	\$987,939	1	\$25,000	\$0.50	\$43,032	\$1,199,364	\$1,559,173
Pond C27	24200	8.72	\$211,058	\$500,000	2.9	\$1,453,645	1	\$25,000	\$0.50	\$63,317	\$1,752,945	\$2,278,829
Pond C28	24200	4.34	\$104,936	\$500,000	1.4	\$722,736	1	\$25,000	\$0.50	\$31,481	\$884,113	\$1,149,347
Pond C29	24200	96.00	\$2,323,200	\$500,000	0.0	\$0	1	\$25,000	\$0.50	\$0	\$2,348,200	\$3,052,660
Pond C30	24200	2.71	\$65,525	\$500,000	0.9	\$451,303	1	\$25,000	\$0.50	\$19,658	\$561,460	\$729,898
Pond C32	24200	2.52	\$60,981	\$500,000	0.8	\$420,006	1	\$25,000	\$0.50	\$18,294	\$524,255	\$681,532
Pond C33	24200	3.69	\$89,286	\$500,000	1.2	\$614,960	1	\$25,000	\$0.50	\$26,786	\$755,993	\$982,791
Pond C34	24200	1.49	\$36,152	\$500,000	0.5	\$249,000	1	\$25,000	\$0.50	\$10,846	\$320,982	\$417,277

Assumptions include engineering with contingency is an additional 30%.

APPENDIX D

Supplementary Data



MEMORANDUM

DATE: June 8, 2022

TO: Michael Clark
Assistant City Engineer
40 South Main St, Suite 240
Spanish Fork, UT 84660

FROM: Kayson Shurtz, P.E.
Daniel Jones, P.E., CFM
Hansen, Allen & Luce, Inc. (HAL)
859 West So. Jordan Pkwy – Suite 200
South Jordan, Utah 84095

SUBJECT: 2022 Storm Drain Master Planning Assistance

PROJECT NO.: 348.20.100



BACKGROUND

The City of Spanish Fork (City) requested assistance from Hansen, Allen, & Luce, Inc. (HAL) related to the J6 development and surrounding areas. Previous master plans had assumed this area would remain agricultural; however, in recent years that has changed; updated master planning solutions are needed in this area. This project's scope of work includes modeling development of a hydrologic model, evaluation of design flows and project alternatives to handle them, and workshops with the City to review the findings and solutions and select the preferred solution.

The study area is generally bounded on the north by 4000 South, on the east by 800 West, on the West by 1450 West, and on the south by 5950 South. It is expected that this area will completely develop into an industrial region, rather than an agricultural one. The City has asked that HAL provide them with the master plan sizing recommendations for conveyance and detention, with conveyance across 4000 South being designed to the 100-year flows.

HYDROLOGY

To estimate the magnitude of peak future flows and total runoff volume, the area which contributes runoff must be delineated. The US Army Corps of Engineers hydrology software HEC-HMS version 4.9 was used to do this. 0.5-meter LiDAR elevation flown in 2013-2014 was downloaded from UGRC Raster Data Discovery and was loaded into HMS. Sinks were filled, flow accumulation was run, and outlets were established. The percent of existing directly connected impervious area (DCIA) was assumed to be 0%

whereas the DCIA percent for the future case was assumed to be 72% (a typical DCIA percentage for industrial developments). The existing percent DCIA was assumed to be zero. Curve number was assumed to remain the same between existing and future with the only difference being the percent DCIA increased from 0 to 72. Curve numbers for this area range from the low 70s to 80, with the typical CN being about 78-79. This is based on about 90% of the soils being classified as soil type D (the remaining portion classified as soil type C) and an assumed pervious cover as defined by TR-55 of open space, good cover.

The longest flowpath and channel slope were used to estimate the time of concentration for each basin. In the existing case, a typical velocity is expected to be around 1.5 fps; for the future case, the typical velocities are expected to be around 5 fps.

The storm distribution selected by the City and used in this study is the modified 3-hour Farmer Fletcher, commonly known as the 3-hr Salt Lake County (SLCo) distribution. The design events evaluated as part of this study included the 10-, 25-, and 100-year events. The SLCo distribution for the 10- and 100-year events can be seen below in **Figure 1**.

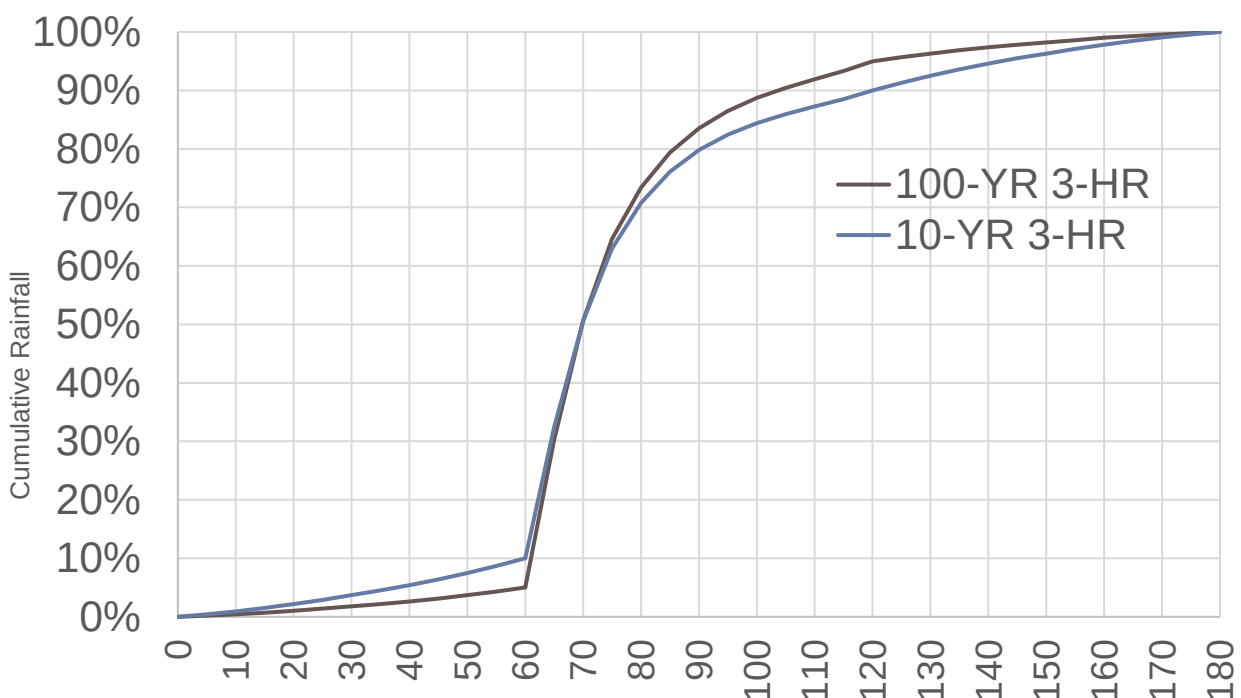


FIGURE 1. SALT LAKE COUNTY 3-HOUR UNITIZED DISTRIBUTION

The total rainfall depths as obtained from NOAA Atlas 14: Precipitation-Frequency Atlas of the United States (Bonnin et al. 2004; NOAA 2013) and can be seen below in **Table 1**. Using the study area of 2.17 square miles (5.62 km²), a depth-area reduction factor (DARF) of 0.925 (study completed for Walnut Gulch, AZ (Hydro-40)) was applied to the point precipitation estimates to obtain a more reasonable average precipitation depth for the area. When developments come in, they will be much smaller than the study area and

should be designed to the point precipitation estimate and not use the area-adjusted rainfall depths.

TABLE 1. DESIGN RAINFALL DEPTHS

Design Storm	10-year	25-year	100-year
3-hour duration	1.02	1.29	1.80
With DARF	0.94	1.19	1.67

For reference, existing peak flows approach 25 cfs for the 10-year storm (0.03 cfs/ac) and 210 cfs for the 100-year storm (0.15 cfs/ac). As mentioned above, future development will increase the impervious area, and therefore the runoff potential. This study evaluated various detention and release rate scenarios to determine the preferred alternative for this area.

COST AND PROTECTION EVALUATION

The potential detention release rate for this area which HAL evaluated were the 0.05 cfs/acre, 0.10 cfs/acre, and 0.15 cfs/acre. The greater the release rate, the greater the flows the pipes need to handle and the greater the pipe size. However, the more flow allowed to pass in the pipes, the required detention volume is reduced. An analysis was conducted to determine the most economic option for the desired level of protection.

The model was established so that runoff was stored in offline detention ponds with an orifice sized appropriately to contribute the design flowrate at a stage of three feet above the bottom of the pond. The pond volume was then expanded or reduced to get the design stage (and hence design flows) in the pipe. Once the design outflows were reached, the volume was recorded and a new basin model saved to evaluate the different design scenarios. The results of the analysis can be seen in **Table 2**.

TABLE 2. TOTAL STORAGE REQUIRED TO DETAIN TO DIFFERENT DESIGN RATES

Design Release Rate	Outlet Flows	Approximate Pipe Costs	Total Storage (acre-ft)		
			10-year	25-year	100-year
0.05 cfs/ac	70 cfs	\$10.3M	78.7	106.5	159.9
0.10 cfs/ac	140 cfs	\$13.9M	68.1	95.7	150.0
0.15 cfs/ac	210 cfs	\$15.9M	61.6	88.3	142.3

Note: Pipe costs assume "out of street" conditions with an additional 30% for engineering and contingency.

Corollaries of **Table 2** include:

1. It is more cost effective to install more detention than it is to upsize the pipe to handle the less-detained flows. Installing offline detention to reduce the design pipe flows only requires up to 28% more volume ($78.7/61.6 - 1$) whereas pipe costs are 53% greater ($15.9M/10.3M - 1$). This pattern follows for each return frequency.
2. Detention to a less frequent storm will increase the volume required and the protection offered.

PREFERRED ALTERNATIVE AND RESULTS SUMMARY

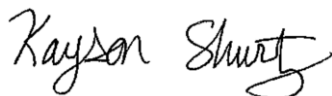
After discussion with the City, they selected the release rate of 0.05 cfs/ac up to the 25-year storm event. This is a stricter design level than for most of the City which is only required to detain to 0.15 cfs/ac for the 10-year and more frequent events. The reason the City has selected a more stringent criteria for this area is because the area is considerably flatter than most other parts of the City and this study identified concerns that if larger storms occurred, the roads in the area would be unable to convey the excess flows if the system were only designed to detain to the 10-year storm. Additionally, the soils are tight and have less ability to infiltrate excess water.

It is recommended that the City require 1 foot of freeboard for all detention and that, rather than the City build regional detention facilities prior to development, detention be required of development as it comes. The detention should be sized to include the runoff flows and volume from the roads. This will require additional inlets and piping that will be connected directly to each detention basin such that excess roadway drainage can be detained. These inlets and pipelines must be completely separate from the mainlines in the road to avoid storing the same water multiple times as it moves through the system. The 100-year design flow which the City requested for conveyance through 4000 South is 175 cfs through the northwestern corner and 127 cfs near the northeastern corner. These 100-year flows are large because they include all overflows not contained by the area's 25-year design and the high percent impervious that is anticipated in the area. The planimetric view of the planned conveyance lines with their sizes and capacities is shown in **Figure 2**.

The study area contains 1,449 acres of developable land, excluding master planned roads. Based on master planned roads, the lineal footage along future main roads in the area is 128,636 feet. While roads only account for 8.7 percent of the study area, they account for 14.6 percent of the total required detention volume ($15.5 \text{ AF} = 106.5 * 0.146$). Therefore, the City has elected to require an additional storage requirement per frontage length. The amount of detention per acre of development is 2,740 cubic feet ($91 \text{ AF} / 1449 \text{ acres}$) and the amount of detention per foot of frontage road is 5 cubic feet ($15.5 \text{ AF} / 128,636 \text{ ft}$).

Please feel free to contact me should you have any questions.

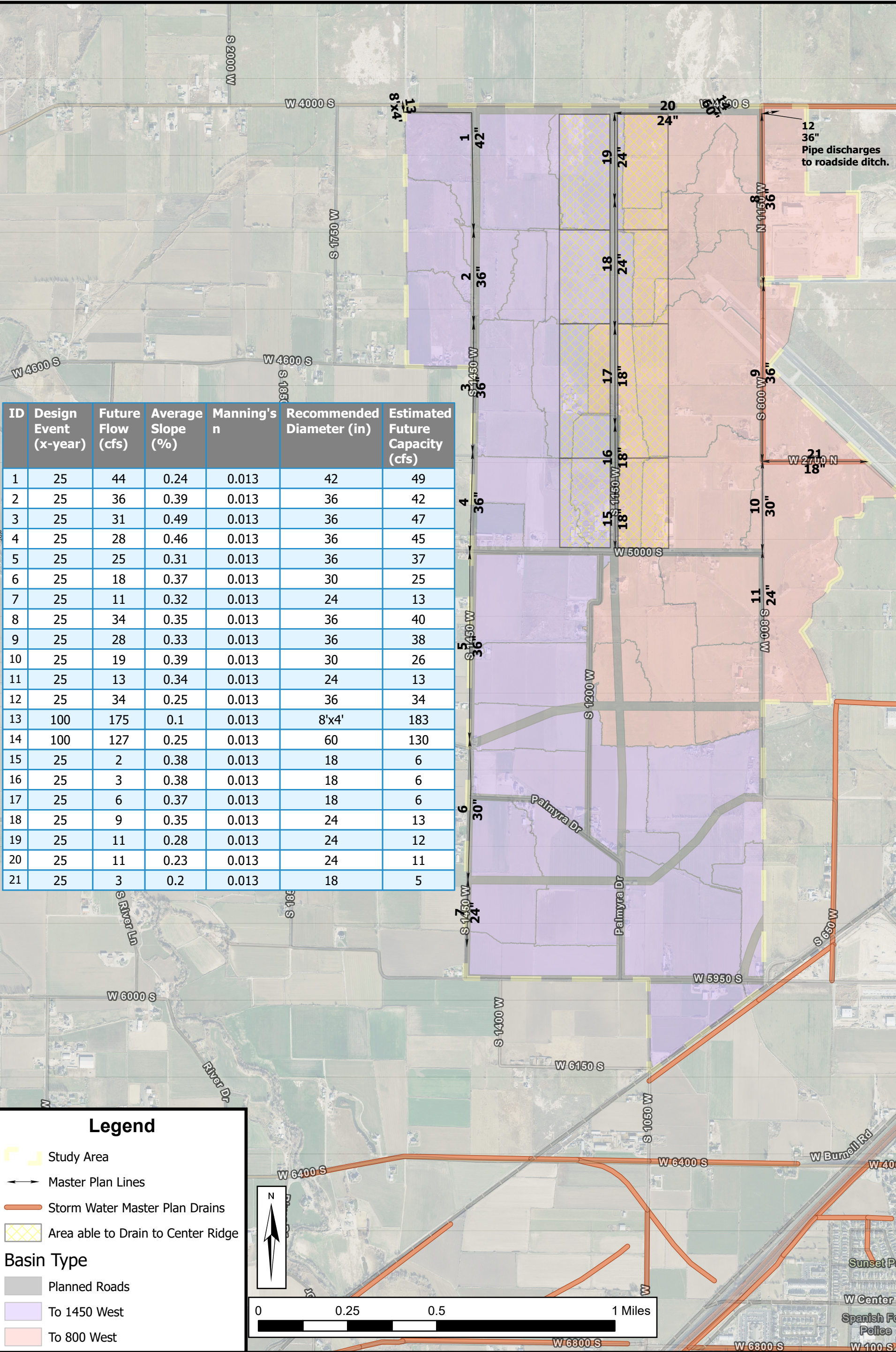
Sincerely,



Kayson Shurtz

Enclosures
HMS 4.9 Model

Date: 6/7/2022
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SPANISH FORK CITY

STORM WATER MASTER PLAN
FOR WEST OF AIRPORT

FIGURE
2

MEMORANDUM

DATE: April 19, 2024
TO: Bruce Ward, P.E.
Salem City Engineer
Chris Thompson, P.E.
Spanish Fork City Engineer

FROM: Kayson Shurtz, P.E.
Hansen, Allen & Luce, Inc. (HAL)
859 West So. Jordan Pkwy – Suite 200
South Jordan, Utah 84095

SUBJECT: Allowable Release Rates from Spanish Fork to Beer Creek (east of I-15)

PROJECT NO.: 406.06.300

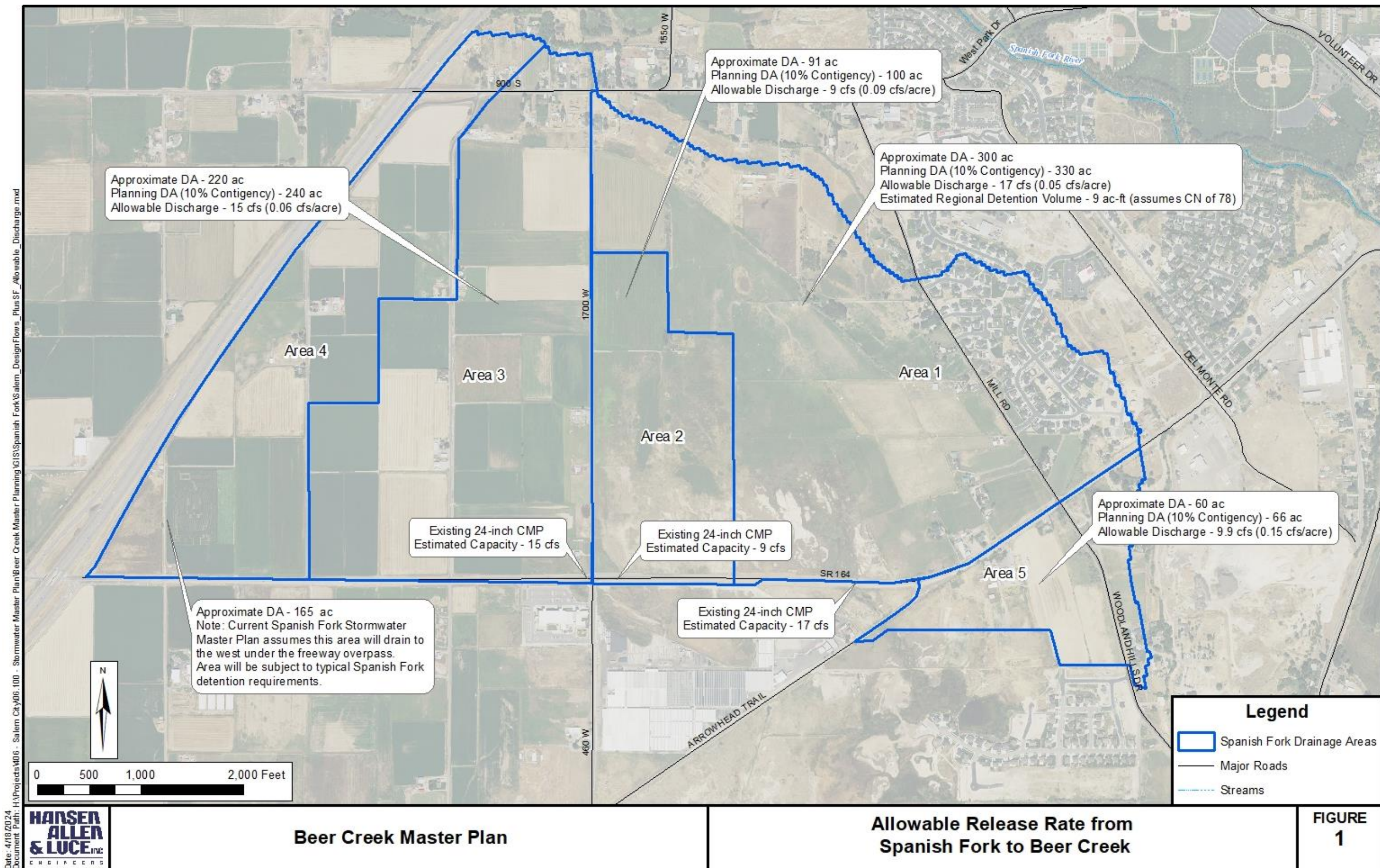
INTRODUCTION

Development is occurring rapidly in Utah County. Some of these development pressures have occurred near the border of Spanish Fork and Salem. City boundaries do not coincide with natural hydrologic drainage basins. The southwest corner of Spanish Fork City drains to Beer Creek which is the major drainage channel for the City of Salem. The Cities determined it would be appropriate to perform some master planning for the shared Beer Creek drainage channel to evaluate existing capacity, identify downstream bottlenecks, and develop a master plan to evaluate what detention or infrastructure would be required to detain or convey projected future flows. The overall study will involve Payson and Utah County since these entities have a significant amount of land that utilizes the Beer Creek drainage channel. This memo covers a portion of the overall Beer Creek study focusing on establishing allowable release rates from Spanish Fork to Salem within the Beer Creek drainage area east of I-15.

EXISTING CROSSING CAPACITY

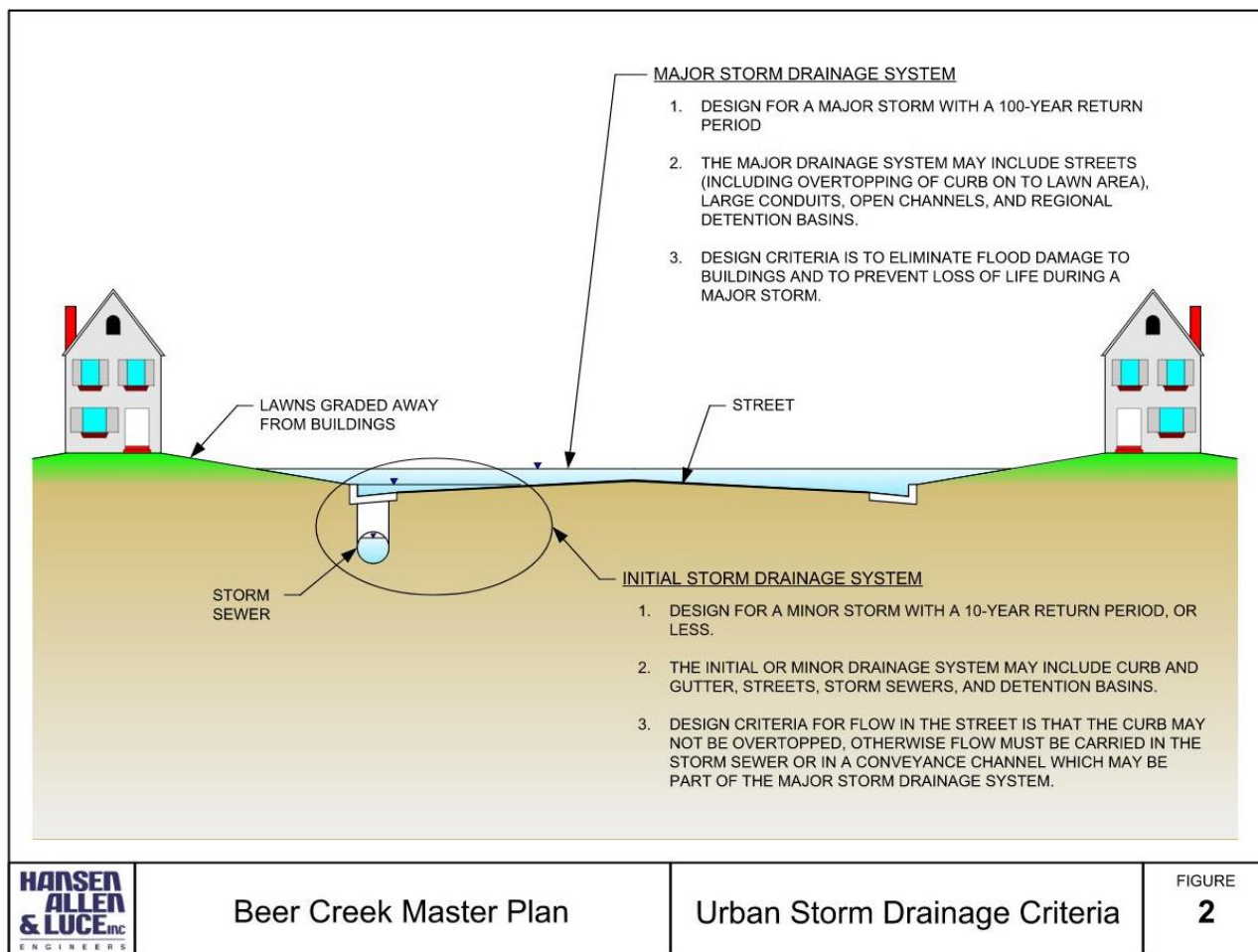
Hansen, Allen, and Luce visited each of the potential crossings that come from Spanish Fork and eventually drain into Salem. The size of each crossing and material type was noted during the field visit. This information was then coupled with elevations extracted from State Lidar data to estimate the existing capacity of each crossing from State Road 164 (SR-164) to just upstream of the I-15 crossing along the Beer Creek drainage.

Based on the field visit it was apparent that much of the Spanish Fork drainage north of State Road 164 has historically acted as de facto detention with the crossings under the road limiting the runoff from Spanish Fork as it enters Salem. The estimated capacity of these crossings were used to determine appropriate release rates from the Spanish Fork drainage areas that are tributary to Beer Creek. Alternatively, the Cities could upsize these conveyances from their existing size to allow more flow to pass under SR 164; however, additional water contributed to Beer Creek will also require increased conveyance sizing in the downstream direction that would propagate a significant distance and is therefore not recommended. The recommended allowable release rates for each of the specified drainage areas from Spanish Fork are presented in Figure 1.



CONVEYANCE CONSIDERATIONS

In areas where the ground is significantly lower than SR-164 careful consideration should be made for how the 100-year surface flows will be conveyed. Typical in storm drainage design underground pipes are designed for a more frequent (i.e. 10-year event) design storm event and roads are utilized to convey runoff in excess of the more frequent design storm up to the 100-year event or greater. We recommend that homes be elevated sufficiently to not flood in a 100-year event as shown in Figure 2.



Low lying ground as shown on Figure 3 will require a significant amount of fill and careful roadway grading to protect future development and structures that will eventually be built in this area. It is recommended that all homes be built at least 1 foot higher than the expected 100-year water surface elevations.

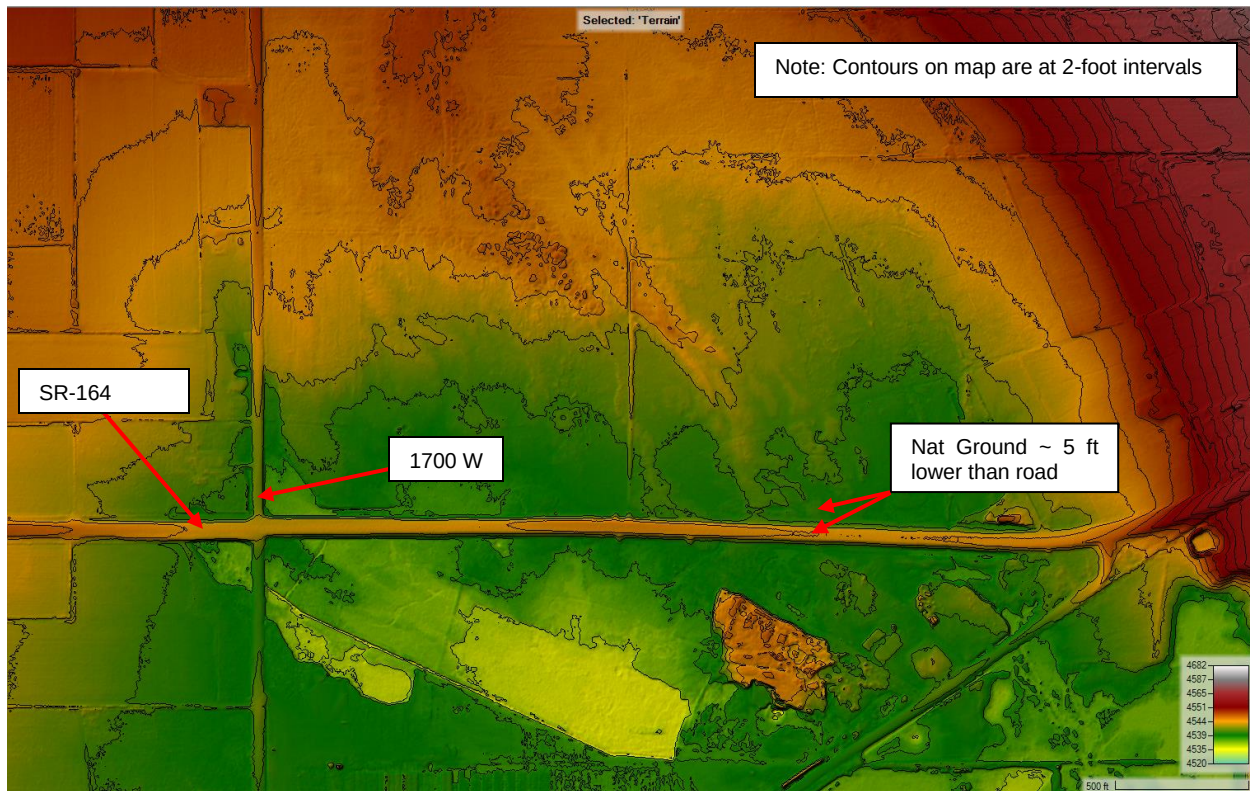


FIGURE 3. EXAMPLE OF EXISTING LOW GROUND ALONG SR-164

SUMMARY

The runoff from Spanish Fork east of I-15 that is tributary to Beer Creek must be detained as development occurs to avoid overwhelming the limited downstream capacity in Beer Creek. This memo specifies allowable release rates from areas within Spanish Fork that will enter the Beer Creek drainage system. The allowable release rates range from 0.05 - 0.15 cfs/acre (based on a 3-hour 100-year design storm). The release rates represent estimated historical release rates that flow from Spanish Fork into the Beer Creek system (east of I-15).

MEMORANDUM

DATE: April 3, 2024

TO: Chris Thompson
Public Works Director/City Engineer
40 South Main St, Suite 240
Spanish Fork, UT 84660

FROM: Kayson Shurtz, P.E.
Ry Weber, M.S.
Hansen, Allen & Luce, Inc. (HAL)
859 West So. Jordan Pkwy – Suite 200
South Jordan, Utah 84095

SUBJECT: 2022 Storm Drain Master Planning Assistance

PROJECT NO.: 348.20.200



BACKGROUND

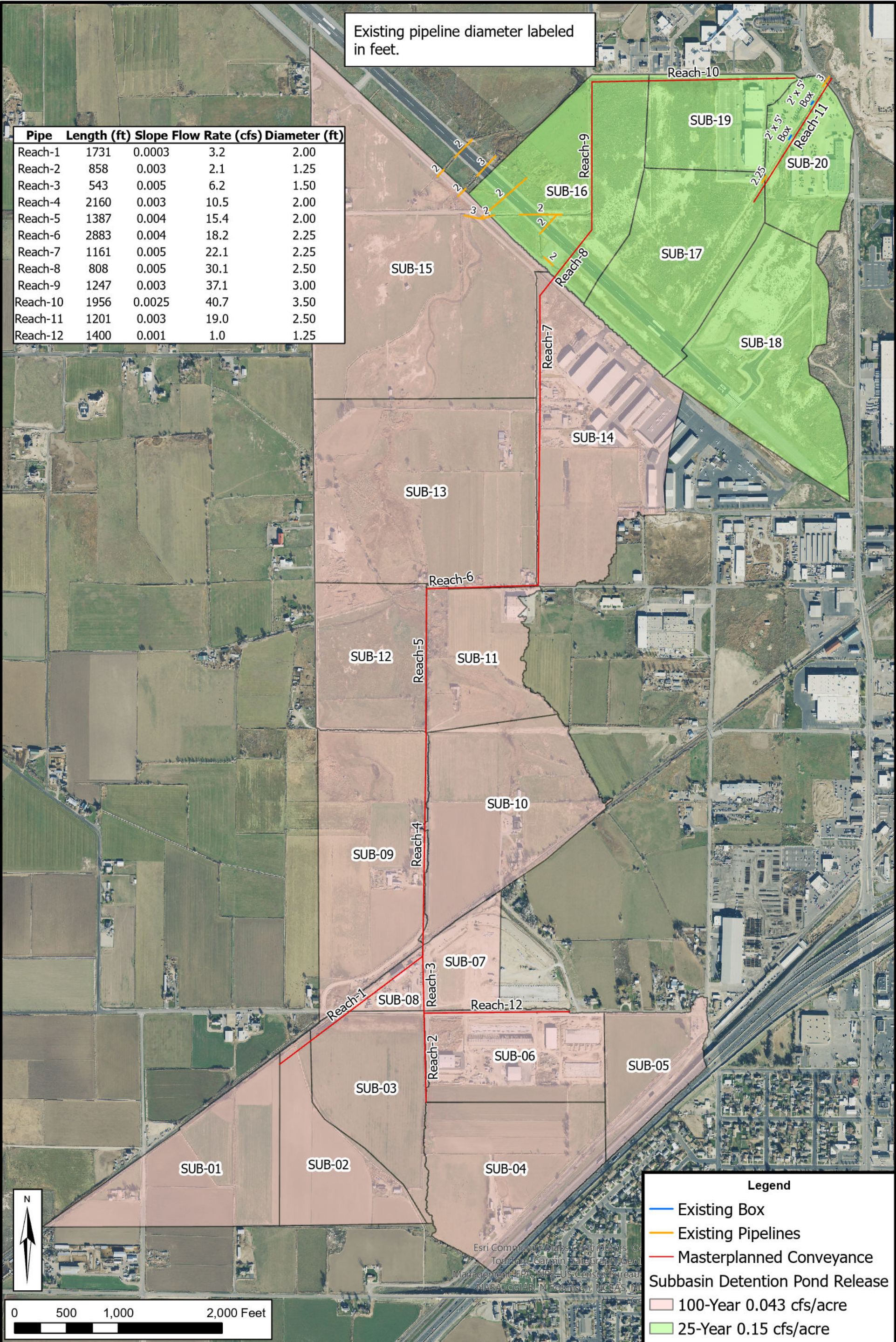
The City of Spanish Fork (City) requested assistance from Hansen, Allen, & Luce, Inc. (HAL) related to the area southeast central of the Spanish Fork Airport. This study this will be referred to as the South Airport development and can be seen in **Figure 1**. Previous master plans had assumed this area would remain agricultural; however, in recent years that has changed; updated master planning solutions are needed in this area. This project's scope of work includes a development of a hydrologic model, evaluation of design flows and project alternatives to handle them.

The study area is approximately 1.4 square miles and is generally bounded on the north by 4800 South, on the east by Main St. and I-15, on the West by 800 West, and on the south by 6400 South. It is expected that this area will completely develop into an industrial land use, rather than an agricultural one. The City has asked that HAL provide them with the master plan sizing recommendations for conveyance and detention, with conveyance across the airport being designed to the 100-year flows.

HYDROLOGY

To estimate the magnitude of peak future flows and total runoff volume, the area which contributes runoff must be delineated. The US Army Corps of Engineers hydrology software HEC-HMS version 4.10 along with amendments by hand were used to do this. 0.5-meter LiDAR elevation flown in 2013-2014 was downloaded from UGRC Raster Data Discovery and was loaded into HMS. Sinks were filled, flow accumulation was run, and outlets were established. The DCIA percent for the future case was assumed to be 72%

Date: 4/3/2024
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(a typical DCIA percentage for industrial developments). Weighted curve numbers were recalculated for the area and the percent DCIA increased from 0% to 72%. Curve numbers for this area range from the high 70s to 80, with the typical CN being 80. This is based on about 90% of the soils being classified as soil type D (the remaining portion classified as soil type C) and an assumed pervious cover as defined by TR-55 of open space, good cover.

Roads were modeled separately as subbasins. Of the 20 subbasins, 17 had roads within. Creating road subbasins was done by calculating the total area of the roads within each subbasin and subtracting that area from the subbasin area those roads fall within. The hydrologic model assumes that the roads within the subbasin drains to the roads parent subbasin detention pond. The model schematic for the north end of the study area can be seen in **Figure 2**. The longest flow path was used to estimate the time of concentration for each basin. Under fully developed conditions it was estimated that the typical velocities would be around 5 fps.

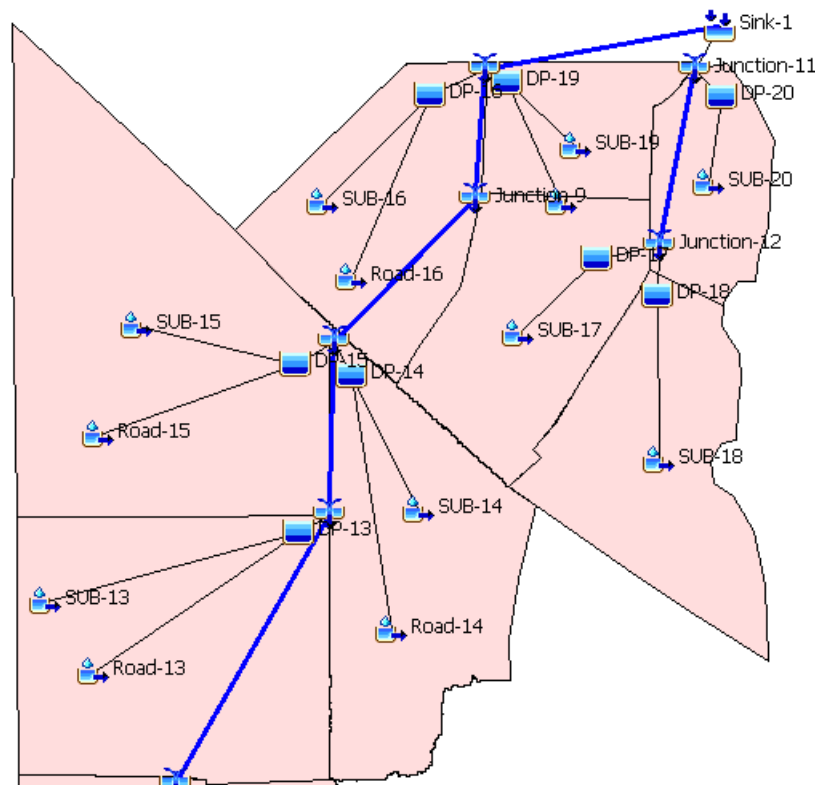


FIGURE 2. HEC-HMS SCHEMATIC FOR NORTH END OF DRAINAGE AREA

The storm distribution selected and used in this study is the modified 3-hour Farmer Fletcher, commonly known as the 3-hr Salt Lake County (SLCo) distribution. The design events evaluated as part of this study included the 25-year event and 100-year event. The SLCo distribution for the 10, 25, and 100-year event can be seen below in **Figure 3**.

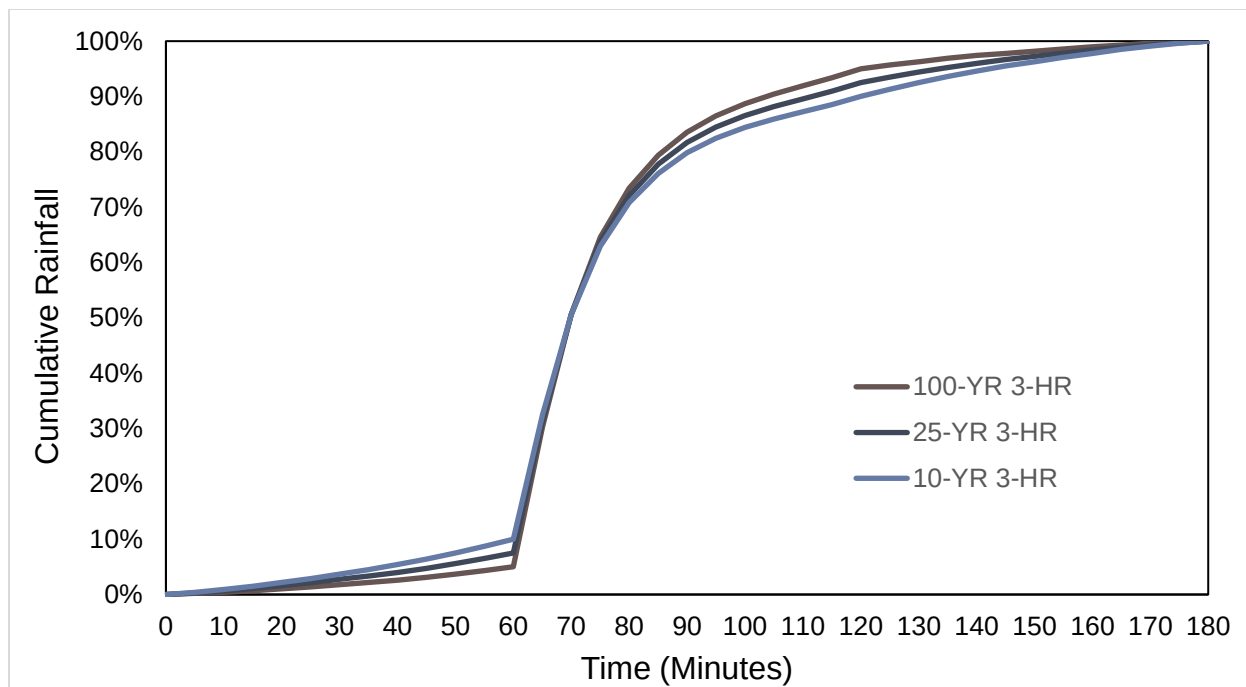


FIGURE 3. SALT LAKE COUNTY 3-HOUR UNITIZED DISTRIBUTION

The total rainfall depth as obtained from NOAA Atlas 14: Precipitation-Frequency Atlas of the United States (Bonnin et al. 2004; NOAA 2013) was 1.29 inches for the 25-year 3-hour duration event and 1.8 inches for the 100-year 3-hour duration event. Due to the size of the South Airport study area, a DARF value was not applied within the study.

COST AND PROTECTION EVALUATION

As release rates increase, the capacity for the pipe must increase, which in turn increases the size of the pipe. However, the more flow allowed to pass in the pipes reduces the required volume of the detention ponds. An analysis was conducted to evaluate the relative trade-offs between conveyance and detention requirements. It is the City's desire to have a 100-year event solution upstream of the Airport to protect facilities at the Airport.

The model was established so that runoff was stored in offline detention basins with an orifice sized appropriately to contribute the determined design flowrate for that subbasin. Design flowrate was determined by calculating the orifice area required to release the subbasins cfs/acre amount with a stage of three feet from the invert of the detention basin. The detention pond volume was then enlarged or reduced using the 25-year (downstream of the Airport) and 100-year (upstream of the Airport) events to ensure the design stage within the detention basin is 3 feet. This in turn ensures an optimal flow rate through the downstream pipe based off the designed release rates. Once the design stage and orifice outflows were reached, the detention basin volume and reach capacities were recorded. The results of the analysis can be seen in **Table 1**.

The initial potential detention release rates for the South Airport area which HAL evaluated were 0.05 cfs/acre, 0.10 cfs/acre, and 0.15 cfs/acre. Upon discussion with the City having knowledge of a new development being built, the subbasin areas for subbasin

13 and 15 increased. The area for subbasins 13 and 15 increased by 0.052 and 0.14 square miles respectively. Increasing the area of the subbasins increased the total area of the South Airport study area from approx. 1.2 square miles to approx. 1.4 square miles. To account for the increase in upstream area above the Airport while still accounting for limiting downstream pipe capacities a new detention pond release rate was determined.

From the 0.05 cfs/acre release rate and the 25- and 100-year event analysis, it was determined that to keep the size of the pipe under the airport runway to 30-inches additional detention would be required. To determine the new detention pond release rate for subbasins above the airport runway a flow of approximately 30 cfs (the approximate capacity of 30-inch pipe) was used divided by the tributary area above the airport of about 687 acres. The release rate required to produce about 30 cfs at the location of the bore under the airport runway is about 0.043 cfs/acre. The areas downstream of the bore a higher release rate of 0.15 cfs/acre for a 25-year event was selected to reduce the size and residence time of the detained water. This was done with the intent that these changes would reduce the chances of attracting birds around the airport. The results for the selected alternative of 0.043 cfs/acre above the airport and 0.15 cfs/acre below the airport can be seen in **Table 1**.

TABLE 1. TOTAL STORAGE REQUIRED TO DETAIN TO DIFFERENT DESIGN RATES

Design Release Rate	Cumulative Design Flow at Outfall	Approximate Pipe Costs	Total Storage (acre-ft)
			25-year Event
Combination 0.043 cfs/ac & 0.15 cfs/acre	41 cfs	\$ 7,043,494	76*,89**
0.05 cfs/ac	38 cfs	\$ 7,158,793	56, 62*,77**
0.10 cfs/ac	77 cfs	\$ 10,236,100	51
0.15 cfs/ac	115 cfs	\$ 11,860,252	47

Note: Pipe costs assume "out of street" conditions with an additional 30% for engineering and contingency. As well as costs for an assumed total of 500 ft of boring under the airport for a 30-inch pipe.

*Detention volume required for the 100-year event upstream of the airport.

**Total detention volume required for the 100-year event above the airport and the 25-year event below the airport.

As detention volume size increase, the cost for the storm water alignment decreases and as storm frequency decreases, detention volume increases.

PREFERRED ALTERNATIVE AND RESULTS SUMMARY

The preferred alternative for this area utilizes a combination of detention and conveyance that will provide the Airport with 100-year protection while minimizing the pipe size under the airport runway. HAL recommends a detention pond sizing and release rate of 0.043 cfs/acre for detention ponds that are tributary to the alignment which flows to the Airport. Detention ponds that were sized for the 100-year event would detain and release above the airport and the 25-year event would be detained with a release rate of 0.15 cfs/acre below the airport. This would require approximately a 43% increase in detention volume as compared to detaining to the 25-year event distributed throughout the subbasins above the airport. The main reason for this recommendation is the 100-year overflows are

approximately 270 cfs which would have to pass underneath the Airport runway, given the cost of boring and utilities restrictions that is not considered feasible. By distributing a 43% increase in detention volume across the 1.07 square miles above the Airport, the 100-year event is attenuated sufficiently to have smaller pipe sizes throughout the basin which will reduce boring costs and reduce the likelihood of downstream conflicts with existing utilities. **Table 2** shows the design configurations for each pipe section seen in **Figure 1**. for the 100-year event at a 0.43 cfs/acre release rate.

Conveyance facilities between the airport runway and the Army National Guard property were surveyed to verify size, material, and slope to estimate existing capacity. The locations of these facilities along with estimated capacities and photos from the site visit performed as part of this study can be found in the attached appendix. The limiting reach along the existing drainage ditch was found to have a capacity of approximately 21 cfs with 6 inches of freeboard. The pipe between the airport and the Army National Guard property is a 27-inch HDPE pipe (ribbed) that has an estimated capacity of about 12.9 cfs. This pipe would require an upsize based on the future design flows of 19 cfs. The rest of the downstream facilities have adequate capacity for the projected future flows.

Army Personnel at the Army National Guard indicated they would be in favor of piping the entire section along their property. Estimated size for piping the entire section can be found in **Table 2**. If this area becomes piped another possibility would be to consolidate the flows from Reach 8 into Reach 11 to convey the total flow from Reach 8 and Reach 11(49.1 cfs) thereby reducing the flows in Reaches 9 and 10.

It is recommended that the City require 6-inches of freeboard for all detention in this study area and that, rather than the City build regional detention facilities prior to development, detention be required of development as it comes. The detention should be sized to include the runoff flows and volume from the roads. This will require additional inlets and piping that will be connected directly to each detention basin such that excess roadway drainage can be detained. These inlets and pipelines must be separated from the mainlines in the road to avoid storing the same water multiple times as it moves through the system. The planimetric view of the planned conveyance lines with their sizes is shown in **Figure 1**. It should be noted that there are currently several pipes which direct water from South to North underneath the airport. There is a potential for the city to use present conveyance pipes as supplementary to the master planned conveyance described in this report.

TABLE 2. DESIGN CONFIGURATIONS FOR PIPE ALIGNMENTS

	Length (Ft)	Slope(ft/ft)	Flow Rate (cfs)	Diameter (Ft)
Reach-1	1,731	0.0003	3.1	2.00
Reach-2	858	0.003	2.1	1.25
Reach-3	543	0.005	6.2	1.50
Reach-4	2,160	0.003	10.5	2.00
Reach-5	1,387	0.004	15.4	2.00
Reach-6	2,883	0.004	18.2	2.25
Reach-7	1,161	0.005	22.1	2.25
Reach-8*	808	0.005	30.1	2.50
Reach-9	1,247	0.003	37.1	3.00
Reach-10	1,956	0.0025	40.7	3.50
Reach-11	1,201	0.003	19.0	2.50
Reach-12	1,400	0.001	1.0	1.25

*Bored reach below airport

The study area contains 865 acres of developable land, excluding master planned roads. Based on master planned roads, the lineal footage along future main roads in the area is 36,440 feet with roads accounting for 3.6 percent of the study area. The detention storage per unit area not including roads is 4,828 ft³/acre with 73 acre-feet of storage above the airport and 2,756 ft³/acre with 13 acre-feet of storage below the airport. The detention required per foot of frontage is 4.1 ft³/foot with 36,440 total feet of frontage and 3.4 acre-feet of storage required just for the roads.

AIRPORT BORING CONSIDERATIONS

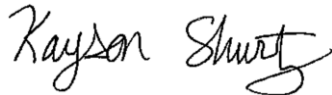
This recommendations from this master planning effort includes boring underneath the existing Airport runway. There are several rules and regulations for any work completed near Airport facilities. Below is a list of several items that need to be considered (list is not meant to be exhaustive but should be representative of the major items that will need to be addressed in design and construction).

- No construction equipment is allowed within 250 feet on either side of the centerline of the airport runway. If it is necessary to have construction equipment in this zone the airport runway would have to be shut down while the required work is being completed.
- No construction equipment is allowed within 110 feet on either side of the centerline of the airport taxiway. If it is necessary to have construction equipment in this zone the airport taxiway would have to be shut down while the required work is being completed.
- Environmental work may be required due to federal regulations.
- City would have to obtain an easement from the Airport for the drainage facilities (even though they are the owner of the airport special easements are still required).
- Advanced notice of at least 45 days is often required for work performed on Airport property.

Careful planning and advanced coordination with the Airport to address these issues as well as others will be critical in the design phase to allow for all parties to be on the same page and come up with a final solution that meets the needs identified in the master plan and reduces disruptions to the operations of the Airport.

Please feel free to contact me should you have any questions.

Sincerely,

A handwritten signature in black ink that reads "Kayson Shurtz". The signature is written in a cursive, flowing style.

Kayson Shurtz

Enclosures
HMS 4.10 Model

Attachments

Date: 4/3/2024
Document Path: H:\Projects\348 - Spanish Fork City\20,200 - 2022 Storm Drain Master Planning Assistance Near Airport\GIS\NearAirport\NearAirport.aprx



SPANISH FORK CITY

EXISTING DRAINAGE FACILITIES
AND SURVEY

FIGURE
A-1

Hydraulic Analysis Report

Project Data

Project Title: Existing Facilities Downstream of Spanish Fork Airport near Army National Guard

Designer:

Project Date: Wednesday, April 3, 2024

Project Units: U.S. Customary Units

Notes: Elevations are in NAVD 88 datum.

Channel Analysis: Airport Channel_XSec3

Notes:

Input Parameters

Channel Type: Custom Cross Section

Cross Section Data

Station (ft)	Elevation (ft)	Manning's n
0.00	4517.45	0.0300
9.20	4516.15	0.0300
17.00	4515.61	0.0300
19.85	4515.53	0.0300
21.90	4515.69	0.0300
33.20	4517.10	-----

Longitudinal Slope: 0.0054 ft/ft

Depth 1.0000 ft

Result Parameters

Flow 33.4264 cfs

Area of Flow 13.0891 ft²

Wetted Perimeter 22.2732 ft

Hydraulic Radius 0.5877 ft

Average Velocity 2.5538 ft/s

Top Width 22.1682 ft

Froude Number: 0.5857

Critical Depth 0.7861 ft

Critical Velocity 3.8448 ft/s

Critical Slope: 0.0171 ft/ft

Critical Top Width 18.94 ft

Calculated Max Shear Stress 0.3370 lb/ft²

Calculated Avg Shear Stress 0.1980 lb/ft²

Composite Manning's n Equation: Lotter method

Manning's n: 0.0300

Channel Analysis: Airport Channel_XSec1

Notes:

Input Parameters

Channel Type: Custom Cross Section

Cross Section Data

Station (ft)	Elevation (ft)	Manning's n
0.00	4517.94	0.0300
6.50	4513.86	0.0300
11.40	4513.40	0.0300
14.66	4513.65	0.0300
21.15	4513.92	0.0300
21.94	4514.30	0.0300
25.60	4515.05	-----

Longitudinal Slope: 0.0025 ft/ft

Depth 1.0000 ft

Result Parameters

Flow 20.5575 cfs

Area of Flow 11.0712 ft²

Wetted Perimeter 17.0541 ft

Hydraulic Radius 0.6492 ft

Average Velocity 1.8569 ft/s

Top Width 16.7648 ft

Froude Number: 0.4027

Critical Depth 0.6727 ft

Critical Velocity 3.5108 ft/s

Critical Slope: 0.0183 ft/ft

Critical Top Width 15.30 ft

Calculated Max Shear Stress 0.1560 lb/ft²

Calculated Avg Shear Stress 0.1013 lb/ft²

Composite Manning's n Equation: Lotter method

Manning's n: 0.0300

Channel Analysis: Airport Channel_XSec2

Notes:

Input Parameters

Channel Type: Custom Cross Section

Cross Section Data

Station (ft)	Elevation (ft)	Manning's n
0.00	4516.28	0.0300
8.80	4512.88	0.0300
12.50	4512.64	0.0300
16.71	4512.73	0.0300
24.40	4516.13	-----

Longitudinal Slope: 0.0025 ft/ft

Depth 3.0000 ft

Result Parameters

Flow 159.5876 cfs

Area of Flow 42.5096 ft²

Wetted Perimeter 22.7783 ft

Hydraulic Radius 1.8662 ft

Average Velocity 3.7542 ft/s

Top Width 21.6430 ft

Froude Number: 0.4721

Critical Depth 2.0263 ft

Critical Velocity 6.7227 ft/s

Critical Slope: 0.0124 ft/ft

Critical Top Width 16.91 ft

Calculated Max Shear Stress 0.4680 lb/ft²

Calculated Avg Shear Stress 0.2911 lb/ft²

Composite Manning's n Equation: Lotter method

Manning's n: 0.0300

Channel Analysis: Pipe 1

Notes:

Input Parameters

Channel Type: Circular

Pipe Diameter 2.25 ft

Longitudinal Slope: 0.0041 ft/ft

Manning's n: 0.0200

Depth 2.2500 ft

Result Parameters

Flow 12.8899 cfs

Area of Flow 3.9761 ft²

Wetted Perimeter 7.0686 ft

Hydraulic Radius 0.5625 ft

Average Velocity 3.2419 ft/s

Top Width 0.0000 ft

Froude Number: 0.0000

Critical Depth 1.2458 ft

Critical Velocity 5.7050 ft/s

Critical Slope: 0.0117 ft/ft

Critical Top Width 2.24 ft

Calculated Max Shear Stress 0.5756 lb/ft²

Calculated Avg Shear Stress 0.1439 lb/ft²

Channel Analysis: Pipe 2

Notes:

Input Parameters

Channel Type: Rectangular

Channel Width 5.00 ft

Longitudinal Slope: 0.0110 ft/ft

Manning's n: 0.0130

Depth 1.5000 ft

Result Parameters

Flow 86.1286 cfs

Area of Flow 7.5000 ft²

Wetted Perimeter 8.0000 ft

Hydraulic Radius 0.9375 ft

Average Velocity 11.4838 ft/s

Top Width 5.0000 ft

Froude Number: 1.6524

Critical Depth 2.0965 ft

Critical Velocity 8.2163 ft/s

Critical Slope: 0.0043 ft/ft

Critical Top Width 5.00 ft

Calculated Max Shear Stress 1.0296 lb/ft²

Calculated Avg Shear Stress 0.6435 lb/ft²

Channel Analysis: Pipe 3

Notes:

Input Parameters

Channel Type: Rectangular

Channel Width 5.00 ft

Longitudinal Slope: 0.0130 ft/ft

Manning's n: 0.0130

Depth 1.5000 ft

Result Parameters

Flow 93.6316 cfs

Area of Flow 7.5000 ft²

Wetted Perimeter 8.0000 ft

Hydraulic Radius 0.9375 ft

Average Velocity 12.4842 ft/s

Top Width 5.0000 ft

Froude Number: 1.7963

Critical Depth 2.2166 ft

Critical Velocity 8.4483 ft/s

Critical Slope: 0.0044 ft/ft

Critical Top Width 5.00 ft

Calculated Max Shear Stress 1.2168 lb/ft²

Calculated Avg Shear Stress 0.7605 lb/ft²

Channel Analysis: Pipe 4

Notes:

Input Parameters

Channel Type: Circular

Pipe Diameter 3.00 ft

Longitudinal Slope: 0.0090 ft/ft

Manning's n: 0.0130

Depth 3.0000 ft

Result Parameters

Flow 63.2756 cfs

Area of Flow 7.0686 ft²

Wetted Perimeter 9.4248 ft

Hydraulic Radius 0.7500 ft

Average Velocity 8.9517 ft/s

Top Width 0.0000 ft

Froude Number: 0.0000

Critical Depth 2.5576 ft

Critical Velocity 9.8561 ft/s

Critical Slope: 0.0084 ft/ft

Critical Top Width 2.13 ft

Calculated Max Shear Stress 1.6848 lb/ft²

Calculated Avg Shear Stress 0.4212 lb/ft²

Site Visit Photos



27-inch corrugated HDPE (Upstream side)



27-inch corrugated HDPE (Downstream side)



Existing Ditch (typ)



2' W x 5' H Concrete Box Culvert (most upstream)



Existing Ditch (typ)



2' W x 5' H Concrete Box Culvert (most downstream)



36-inch RCP (Upstream side)



36-inch RCP (Downstream side)