MADISON / WAHSATCH
DRAINAGE IMPROVEMENTS
FINAL DRAINAGE STUDY

Oct. 1972

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DRAINAGE STUDY
MADISON/WAHSATCH DRAINAGE IMPROVEMENTS

OCTOBER 1985
REVISED APRIL 1986

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DEC JOB NO. 307.001
ENGINEER'S STATEMENT

The attached drainage plan and report were prepared under my direction and supervision and are correct to the best of my knowledge and belief. Said drainage report has been prepared according to the criteria established by the City for drainage reports and said report is in conformity with the master plan of the drainage basin. I accept responsibility for any liability caused by the negligent acts, errors or omissions on my part in preparing this report.

Prepared by:

Michael T. Stift, P.E. 10/18/85
Project Engineer

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Water Resources Manager
Denver Engineering Corporation

CITY OF COLORADO SPRINGS

Filed in accordance with Section 15-3-906 of the Code of the City of Colorado Springs, 1980, as amended.

Gary Highes, P.E. Date: August 8, 1986
City Engineer

Subject to review of design and construction location.
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TECHNICAL ADDENDUM

APPENDIX

A. Hydrology
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NOTE: A portion of Appendix A, Figures 1 through 8, have also been included in this book. See the "Technical Addendum" for the complete Appendixes.
INTRODUCTION

The Madison/Wahsatch drainage area, as defined by this Drainage Study is approximately bounded by Cache La Poudre on the south, Nevada Avenue on the west, Chicago/Rock Island Railroad tracks on the north and Shook's Run on the east in north central Colorado Springs, Colorado (Appendix A, Figure 1). This drainage study area was roughly defined by the "Shook's Run Master Drainage Basin Study", dated December 1972. The December 1972 report generalized the overall plan for drainage of 50-year and 100-year storm events within the Shook's Run watershed by means of various drainage improvements and roughly delineated subbasin drainage boundaries for each major drainage improvement. The Madison/Wahsatch drainage area is one of these subbasins and is fully developed with residential housing, schools and parks. It has severe flooding problems due to antiquated and underdesigned drainage facilities. The worst flooding is reported to be occurring at the street intersections of Madison/Wahsatch and Wahsatch/Caramillo. However, flooding is not limited to these specific areas and an overall drainage study to determine specific drainage improvement recommendations has been conducted by Denver Engineering Corporation to relieve or eliminate flooding. We wish to formally extend our appreciation to the many people and the various City of Colorado Springs Divisions in providing the required information to complete this engineering report.

BOUNDARY DELINEATION

The Drainage Study boundaries were chosen based on the following:

a. Existing inlets and storm sewer system on Nevada Avenue.


c. Review of topographic maps.

d. Field reconnaissance of existing curb and gutter flow and overland drainage patterns.

West Boundary: Beginning at the Chicago/Rock Island Railroad tracks, the west boundary travels southerly along the east Weber Street right-of-way line where an existing berm separates Weber Street and the railroad right-of-way to Jackson Street. At Jackson Street, the boundary follows along the centerline of Weber Street to Fontanero Street. All land west of this boundary drains to Nevada Avenue. In accordance with the "Shook's Run Master Drainage Basin Study", if drainage facilities are installed within the Madison/Wahsatch Drainage Area, then existing drainage facilities along Nevada Avenue are adequate to drain the 50-year storm. South of Fontanero Street the west boundary is Nevada Avenue. Again it was assumed that the Nevada Avenue inlets and storm sewer system can safely and adequately convey major storm events.

South Boundary: The south boundary is Cache La Poudre as defined by the Scope of Work. This boundary was chosen due to the intersection of Shook's Run, an abandoned railroad right-of-way and Cache La Poudre.
The abandoned railroad right-of-way may be an excellent location to install a storm sewer with an outfall at Shook's Run.

North Boundary: The north boundary is the Chicago/Rock Island Railroad tracks which is a manmade boundary and cuts off any flow from the north flowing south.

East Boundary: Beginning at the Chicago/Rock Island Railroad tracks, the east boundary follows the centerline of Magellan Street to Jackson Street, then crosses Bonnie Park to just west of Royer Street. The boundary then travels south and southeast following the limits of "Shook's Run Master Drainage Basin Study" Subbasin 3 and connects to Shook's Run at its intersection with Columbia Street. The boundary then follows Shook's Run to its intersection with Cache La Poudre.

A portion of Subbasin 3 shown as Offsite Basin AA in Appendix A, Figure 3, has also been included in this analysis. The area bounded on the north by the Chicago/Rock Island Railroad tracks, on the east by Templeton Gap and El Paso Street, on the south by Madison Street and Jefferson Street, and on the west by Subbasins A and B of this study also drains into the Madison/Wahsatch drainage area and, therefore, was analyzed as part of this drainage study.

EXISTING FACILITIES AND HISTORICAL USAGE

The following description of existing drainage facilities is based on City maps, interviews with City staff, and field reconnaissance. A plan view of the existing drainage facilities is shown in Appendix A, Figure 2. The drainage system drains from north to south/southeast to Shook's Run and Cache La Poudre. Most storm runoff is generally handled by curb and gutter flow to existing storm sewers located as follows:

<table>
<thead>
<tr>
<th>Storm Sewer</th>
<th>From</th>
<th>To</th>
<th>Size (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Uintah Street</td>
<td>Weber Street</td>
<td>Shook's Run</td>
<td>33 to 45</td>
</tr>
<tr>
<td>2. Wahsatch Avenue</td>
<td>Caramillo Street</td>
<td>Platte Avenue</td>
<td>18**</td>
</tr>
<tr>
<td>3. Weber Street</td>
<td>Yampa Street</td>
<td>Dale Street*</td>
<td>18</td>
</tr>
<tr>
<td>4. Royer Street</td>
<td>North Jr. High</td>
<td>Dale Street*</td>
<td>18</td>
</tr>
<tr>
<td></td>
<td>School Roof Drain</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Dale Street is the next street south of the study area and is a 45 inch storm sewer.
** Assumed per interviews with City Maintenance staff.

All these storm sewers are located in the southern third of the study area. Therefore, no drainage facilities are located in the northern two-thirds of the drainage area. Storm flows in the north area of the drainage basin from the Chicago/Rock Island Railroad tracks to Caramillo Street generally drain along curb and gutter either ponding
in the abandoned railroad right-of-way that splits the study area or ponding in the street gutters. The southerly drainage area from Carmillo Street to Cache La Poudre drains along existing curb and gutter to the existing storm sewer system and into Shook’s Run. This area has not had the flooding problems that the northerly area has had.

The abandoned railroad right-of-way historically had a system of ditches, culverts and bubblers or yard inlets with a siphon pipe system to drain runoff along either side of the railroad tracks down to the intersection of the railroad right-of-way and Shook’s Run. This drainage network has been eliminated or plugged with the abandonment of the railroad and the installation of a bike trail along the old railroad tracks. As a result and as verified in interviews with City Park Maintenance crews, water ponds in this right-of-way and is eliminated only through infiltration and evaporation.

Flooding has occurred in some streets even during moderate storm events. Some street grades are less than 0.5 percent. General engineering design standards require a minimum of 0.5 percent grade to drain concrete sections as ponding and deposition of dirt and debris impedes the flow of water along the gutter. In addition, successive street overlays have reduced the curb height of some vertical curbs to less than eight inches which is the City standard curb height. Some vertical curbs are only 4 or 5 inches in height. Ramp curbs are located in the area north of Jackson Street between Wachsatch and Royer Streets. Successive street overlays have covered the road and gutter sections eliminating some ramp curbs or reducing the ramp curb height to between 1 and 4 inches.

There has also been flooding problems at intersections that have caused traffic accidents. Most intersections have concrete crosstabs that flow perpendicular to the flow of traffic. However, there are no inlets or storm sewer system to eliminate water from the street and as a result, the intersection floods. Fontanero Street at Wachsatch Avenue has a 12 foot wide concrete crosstabs and when flowing full a has a depth of flow that would flood into the bottom of a compact car door. In addition, some intersections have bubblers which have drop inlets along the curb radii and water passes through the upstream grated inlet, through a pipe siphon and out the other grated inlet to the other side of the street. The street intersections are crowned between curb radii and if inlets or the pipe are plugged then water ponds and floods the upstream curb radius. Lastly, some intersections have neither crosstabs nor bubblers and sheet flow occurs across intersections which has impeded traffic in the past.

HYDROLOGY

Runoff volumes and peak rates of discharge were determined utilizing the methodology presented in the Soil Conservation Service’s "Urban Hydrology for Small Watersheds, Technical Release No. 55" (TR-55). All hydraulic and hydrologic parameters used in this study and all storm water routings were determined using existing conditions as of September, 1985. Subsequent street overlays, paving of concrete gutters with asphalt and other factors will alter the existing street capacities and routing calculations.
The drainage study area was broken into ten Subbasins, A through J, to provide better accuracy in routing of storm flows. Subbasins A through E generally drain to the abandoned railroad right-of-way where water is ponded to be removed through infiltration and evaporation. The old railroad track bed within the right-of-way is a high point that ponds water behind it. Some of the Subbasins have bubblers or small culverts to pass flows under the high point, but these facilities are inadequate to handle even minor storm flows without ponding due to the plugging with debris or inadequate capacity. Subbasin F drains to the Uintah Street storm sewer which drains to Shook's Run. There have not been any reports of flooding in this area except for where bubblers have been installed to pass water under the crown of a street. Subbasins H and I drain into the remainder of the existing storm sewer system. Again, in the investigation no evidence of flooding problems was found except at bubblor locations. Therefore, it appears that the existing storm sewer system is adequately draining these areas. Subbasin G drains into Shook's Run and Subbasin J is the abandoned railroad right-of-way and immediately adjacent tributary areas. These major drainage Subbasins are delineated in Appendix A, Figure 3.

Each city block within a Subbasin was assigned a block number and the 5-year and 100-year design flows were determined. In cases where a single block was found to drain to two separate Subbasins, that block was broken into two blocks, i.e. 17 and 17A, and the flows for each was determined and applied to the appropriate Subbasin runoff calculations. A preliminary hydrologic analysis was performed for each block to estimate the probable design flows and assist in calculation of the final hydrologic design flows and hydraulic routing analysis for each Subbasin.

Hydrologic volume parameters were determined for each block. These parameters are soil type and type of cover or land use. Soil information was taken from Soil Survey of El Paso County Area, Colorado, prepared by the Soil Conservation Service (SCS) and dated June, 1981. The soil group boundaries are delineated in Appendix A, Figure 4. The soils throughout the study area are all well draining soils. Soils found in the study are as follows:

<table>
<thead>
<tr>
<th>Soil Name</th>
<th>Map* Symbol</th>
<th>Hydrologic Soil Group</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ascalon</td>
<td>2</td>
<td>B</td>
<td>Sandy loam, 1 to 3 percent slopes</td>
</tr>
<tr>
<td>Blakeland</td>
<td>8</td>
<td>A</td>
<td>Loamy sand, 1 to 9 percent slopes</td>
</tr>
<tr>
<td>Blendon</td>
<td>10</td>
<td>B</td>
<td>Sandy loam, 0 to 3 percent slopes</td>
</tr>
<tr>
<td>Chasville</td>
<td>16</td>
<td>A</td>
<td>Gravelly, sandy loam, 1 to 8 percent slopes</td>
</tr>
</tbody>
</table>

*Map symbol corresponds with referenced SCS map symbols

Land use was taken from City base maps, compiled on January 5, 1973, and field verified as a part of this drainage study. For the study area from Cache La Poudre to Caramillo Street. The area from Del Norte Street to the Chicago/Rock Island Railroad tracks was taken from
base maps prepared by Denver Engineering Corporation and IntraSearch Inc. during August 1985.

Both the hydrologic soil group designation and land use were used to determine the runoff curve number utilizing Table 1 in Appendix A, "Runoff Curve Numbers for Selected Agricultural, Suburban, and Urban Land Use". Curve numbers for each block were determined and a weighted curve number for each block was then calculated to determine storm runoff volumes.

The hydraulic time parameters of elevation difference, flow length and slope for all blocks were determined from the base maps. The velocity of overland flow was determined utilizing the TR-55 graph, "Average Velocities for Estimating Travel Time for Overland Flow" as presented in Appendix A, Figure 5. From this information, the time of concentration for each block was calculated and results are presented in Appendix A, Table 2.

The time of concentration was then used to determine the peak runoff in cubic feet per second per square mile per inch of runoff resulting from the incremental summation of the six hour Type IIA storm per the City of Colorado Springs Drainage Criteria and as presented in Appendix A, Figure 6. Runoff in inches was determined for the 5-year and 100-year storm event as a function of the runoff curve number and precipitation as shown in Appendix A, Figure 7. The following storm events were used in accordance with the City of Colorado Springs Drainage Criteria:

<table>
<thead>
<tr>
<th>Design Flow</th>
<th>Precipitation (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q5</td>
<td>2.1</td>
</tr>
<tr>
<td>Q100</td>
<td>3.5</td>
</tr>
</tbody>
</table>

From this data, the 5-year and 100-year peak flows for each block were calculated. The results of this preliminary hydrologic analysis are presented in Appendix A, Table 3.

**HYDRAULICS**

Streets serve an important and necessary service when calculating and sizing drainage facilities. Gutter flow in streets transports runoff to inlets and storm sewers. The capacity of street flow may reduce sizing or eliminate the need for storm sewers in particular areas within a drainage basin. Curb and gutter flow capacities were calculated assuming zero depth of flow at the street centerline providing emergency vehicles with a minimum of one lane of traffic open for access during the design storm event. Street capacities and a discussion on determining street capacities for the various street sections are presented in Appendix B.

Surface conditions were determined from field observation. The following Mannings roughness coefficients (n) were used:
There are no records of City maps showing slopes for any of the existing storm sewer facilities in the study area. Each storm sewer has cleanouts along the storm sewer for maintenance. However, through the years, successive asphalt overlays have covered the 24 inch diameter cleanout covers limiting access to the storm sewer system.

All street flows were considered to be in the gutter, along concrete crossovers and drained into storm inlets. In discussions with the City Maintenance staff, bubbler siphon pipes are always full of water creating noxious odors and mosquito problems. Grass cuttings and other debris easily collect in the bubbler such that rarely do they operate at capacity. Therefore, it is recommended and the routing analysis assumes that all bubbler used to pass drainage under the crown of a street be abandoned by removal or conversion to a grated drop inlet.

Hydraulic calculations indicate that new drainage inlets are required to alleviate flooding problems. This report utilizes the City standard D10 wake opening for all vertical curb locations where a new inlet is required. Existing curb opening inlet dimensions were measured in the field and used to determine calculated inlet capacities. Bubbler locations were also identified through field reconnaissance. The existing bubbler grated drop in -t was used in place of a D10R inlet to save on construction costs when sufficient inflow capacity could be provided. Final design will include analysis of these bubbler to see if they can actually be converted to a grated drop inlet. If they cannot, a D10R inlet should be installed. The City standard grated inlet was reviewed only for areas where rollover curb and gutters are present and no other drainage facilities are present. Curb capacity is limited in these areas due to successive street overlays and curb heights are generally 4 inches or less, making installation of D10R inlets impractical. In all cases, a 60 percent pickup of street flow was used for inlets on straight grades in accordance with the City's "Subdivision Policy Manual". New and existing curb opening and drop inlet capacities, dimensions, calculations, and criteria used in this report are presented in Appendix C.

DRAINAGE IMPROVEMENTS AND RECOMMENDATIONS

Currently, flows overtop street corners and flow from one Subbasin to another. With the criteria as set forth in previous sections of this report, storm sewer inlets, locations, and sizes were chosen to prevent this type of flooding. Therefore, each Subbasin was analyzed individually. The existing drainage patterns indicate that the major storm sewer should follow the abandoned railroad right-of-way. Ponding occurs to the north of the right-of-way and most curbs overtop in areas immediately adjacent to the right-of-way. The Subbasin hydraulic routing indicates that a storm sewer is required from Jackson Street
to Corona Street to alleviate flooding.

Very few improvements are required south of the railroad right-of-way. These improvements are either street improvements or are inlets that connect to existing storm sewers.

Currently, no existing underground utilities were found in the railroad right-of-way, except at road crossings. An existing overhead electric line is located along the entire west boundary line of the abandoned railroad right-of-way and along the east boundary line from Española Street to the Chicago/Rock Island Railroad tracks. An asphalt bike path has been installed and the greenbelt is maintained by the City Parks Division from Cache La Poudre to the Chicago/Rock Island Railroad tracks. Only the area immediately north and south of Fontanero Street is sodded and has underground sprinklers. The remaining greenbelt is native grasses. The minimum width of the abandoned right-of-way is 80 feet and the bike path generally bisects the right-of-way.

A major storm sewer could be installed on either side of the bike path and would be the most cost efficient alternative. However, the City Electrical Division has indicated that they have purchased this right-of-way for a major electrical trunk line to be installed in the mid-1990’s. No construction plans have been engineered to date, but it appears there is room for both the storm sewer and electrical trunk line. Therefore, if a storm sewer is to be installed in the right-of-way the design engineer, City Engineering Division, City Parks Department, and City Electric Division must coordinate alignments.

If the abandoned railroad right-of-way can not be used for installation of a major storm sewer, then the pipeline should be installed in Wahsatch Avenue from Jackson Street to Columbia Street and then down Columbia Street to Shook's Run. This alternative would be much more expensive due to numerous utility conflicts, pavement replacement costs, and traffic and pedestrian safety hazards. Estimated construction costs for each alternative are presented later in this text.

As a result, the hydrological and hydraulic routing analysis considers a future storm sewer within the abandoned railroad right-of-way. The routing calculations are presented in Appendix D. In addition, a summary of new and existing inlet capacities used in the routing analysis and comparison of remaining street flow to curb capacity for the 100-year design flow are presented at the end of Appendix D. Where curb capacity upstream of an inlet is insufficient to handle the 100-year design flow, it was felt that the small amount of flow that will overtop the curb is not enough to require another inlet upstream. In all cases, curb capacities are sufficient to handle the 5-year design flow. A summary of the recommended improvements in each Subbasin are presented in Appendix A, Figure 8 and are as follows.

Subbasin A

All drainage flows to the southwest corner of the Subbasin to the abandoned railroad right-of-way and northeast corner of Weber and Fontanero Streets. Bubblers located at the right-of-way and at the northeast corner are intended to pass flow into the right-of-way or
south along Weber Street to be eventually drained into the Uintah Street storm sewer. As all flows drain to a street sump condition, storm inlets were sized for the 100-year design flows.

Recommended drainage improvements are as follows:

1. Install a concrete crossspan parallel and east of the Corona Street centerline at Madison Street to eliminate sheet flow across the street.

2. Install DIOR inlets at the following locations:
   a. A 4 foot DIOR at the northeast and southeast corners of Madison Street and Wahsatch Avenue.
   b. A 6 foot DIOR at southwest corner of Wahsatch Avenue and the southerly entrance to the adjacent strip mall.
   c. A 10, 8 and 14 foot DIOR at the northeast, southeast and northwest corners of Washington Street and Wahsatch Avenue, respectively.
   d. A 22 foot DIOR at the northeast and northwest corners of Fontanero Street and Wahsatch Avenue.
   e. A 4 foot DIOR at the north and south intersection of Fontanero Street and the railroad right-of-way.
   f. An 8 foot DIOR at the northeast corner of Weber Street and Fontanero Street.

Note that additional drainage facilities should be installed in Subbasin A as a result of drainage from Subbasin AA. These facilities are discussed in Subbasin AA recommendations.
Subbasin II

All drainage is to the south and west to the abandoned railroad right-of-way and Jackson Street where bubblers drain all water into the railroad right-of-way. Therefore, the storm inlet system for Subbasin B was sized for the 100-year design flow.

The existing curb and gutter is a ramp curb that has been overlayed with asphalt and is therefore only 1 to 4 inches deep from the top of curb to gutter flowline. Options discussed with the City staff include:

1. Installation of a standard 8 inch vertical curb and gutter to match existing pavement grade. However, this will pond water behind the new curb and cause drainage problems for homeowners.
2. Lower street grades by removal and reinstallation of existing asphalt pavement and concrete curb and gutter.
3. Install a non-uniform street cross section.
4. Install city standard grated inlets along existing ramp curb and gutters. Very little head will be on top of inlet to develop a flow into the inlet.

In discussions with the City staff, the first two options were eliminated due to drainage problems and high construction costs, respectively. Option 4 is not recommended as any grated inlet system will be obsolete if another asphalt overlay is installed. Option 3 or installation of a non-uniform street cross section would require installation of the city standard ramp curb and gutter resulting in a grade break from street crown to gutter of 2.0 to 2.8 percent. The 8 inch vertical curb is not considered feasible as the grade break would be from 2.0 to 9.3 percent and would be too severe for vehicular traffic.

Recommended drainage improvements for Subbasin B are:

1. Install concrete cross pans at the following locations:
   a. Parallel and west of the Magellan Street centerline at Lasalle Street.
   b. Parallel and north of Jackson Street centerline at Wahsatch Drive.
   c. Replace the asphalt crosspan with concrete west of the "T" intersection of Corona and LaSalle Streets.
2. Install DIOR inlets at the following locations:
   a. A 10 foot DIOR at the northwest corner of Royer and Jackson Streets.
   b. A 12 and 20 foot DIOR at the northwest and northeast corners of Corona and Jackson Streets, respectfully.
c. A 4 and 20 foot DIOR at the northwest and northeast corners of Balboa and Jackson Streets, respectfully.

d. A 20 foot DIOR at the northwest and northeast corners of Wahsatch Drive and Jackson Street.

e. A 4 and 6 foot DIOR at the south and north intersections of Jackson Street and the abandoned railroad right-of-way.

3. Replace the existing curb and gutter on LaSalle and Corona Streets with a non-uniform street cross section. Asphalt overlays have reduced top of curb to flowline depths to less than 3 inches and existing conditions have very little curb capacity.

4. Replace approximately 145 linear feet of existing 4 inch vertical curb located immediately east of the intersection of Jackson Street and Wahsatch Drive on the north side of Jackson Street with a standard 8 inch vertical curb and gutter.

Subbasin C

All drainage is to the south and west to a sump at the abandoned railroad right-of-way and Espanola Street. Therefore, the storm inlet system for Subbasin C is sized for the 100-year design flow.

Recommended drainage improvements for Subbasin C are:

1. Abandon the existing bubbler parallel and north of the Espanola Street centerline at Wahsatch Avenue and install a 6 foot DIOR inlet at the northeast corner. Calculations for converting the bubbler to a grated drop inlet show that it would have insufficient capacity to handle the 100-year design flow, but could handle the 5-year design flow. As an alternative, a concrete crosspan could be installed, however, Wahsatch Avenue traffic will be impeded due to slowing of traffic due to the dip in the crosspan. There are no existing stop signs or traffic signals along Wahsatch Avenue at this intersection so the DIOR inlet is recommended.

2. Convert the median bubbler into a grated drop inlet.

3. Install a concrete crosspan parallel and north of the Espanola Street centerline at Corona Street to eliminate sheet flows across the street crown.

Subbasin D

All drainage is to the south and west to existing inlets at Wahsatch Avenue and Caramillo Street and to an existing bubbler that passes
water under the railroad right-of-way onto Washsatch Avenue where it drains to the Uintah Street storm sewer. This bubbler is to be abandoned and the storm inlet system for Subbasin D is sized for the 100-year design flow.

Recommended drainage improvements for Subbasin D are:

1. Install concrete crossspans parallel and east of the Corona Street centerline at Del Norte Street and parallel and north of the Caramillo Street centerline at Corona Street to eliminate sheet flow over the street crown. Replace existing asphalt crosspan parallel and west of the Washsatch Avenue centerline at Del Norte Street.

2. Convert the existing bubbler on the northeast corner of Del Norte Street and Washsatch Avenue into a grated drop inlet and connect to the existing storm sewer in Washsatch Avenue. If the grated drop inlet is structurally inadequate, then install a 4 foot D10R.

3. Convert the bubbler on Buena Ventura Street into two curb opening inlets and connect to the proposed storm sewer to be installed in or in the vicinity of the abandoned railroad right-of-way.

4. Buena Ventura Street is a gravel road that drains to the two curb openings. This street should be paved and a concrete crossspan installed to eliminate large amounts of dirt and gravel from entering the storm sewer. A standard 8 inch vertical curb and gutter should also be installed on the north and northeast side of Buena Ventura Street. An 8 inch curb and gutter exists on the opposite side and should be left in place.

5. Remove the existing curb opening inlet at the northeast corner of Caramillo Street and Washsatch Avenue. The existing 2 foot 7 inch long inlet has insufficient capacity to drain the 100-year design flow. Replace with a 6 foot D10R.

Subbasin E

Drainage is to the south and west to the intersection of the abandoned railroad right-of-way and Corona Street where drainage ponds, enters the railroad right-of-way, or passes through an existing 10 inch culvert draining under the railroad right-of-way southerly to the Uintah Street storm sewer system. Therefore, design the storm inlet system to drain the 100-year design flow.
Recommended drainage improvements for Subbasin E are:

1. Abandon the 18 inch culvert located parallel and west of Corona Street centerline at the abandoned railroad right-of-way and Columbia Street.

2. Install DIOR inlets in the following locations:
   a. A 4 and 8 foot DIOR at the northwest and southeast corners of Corona and Columbia Streets, respectively.
   b. A 4 foot DIOR immediately west of the abandoned railroad right-of-way on Columbia Street to drain a sump condition.

3. An alternative to the DIOR to be installed immediately west of the railroad right-of-way would be to install a new standard 8 inch curb and gutter and pave Columbia Street to drain to Wahsatch Avenue.

Subbasin F

All drainage is to the south to the Uintah Street storm sewer and east to existing storm inlets at Shook's Run, except for along Caramillo Street immediately west of the railroad right-of-way and east of Weber Street drain to a depression in Caramillo Street. A second storm sewer in Wahsatch Avenue drains the medians from the abandoned railroad right-of-way to Uintah Street. The existing storm sewers and inlet system is adequate to handle the 100-year design flows.

Recommended drainage improvements for Subbasin F are:

1. Abandon all the existing bubblers and install concrete crossspans at the following locations:
   a. Parallel and on both sides of the Weber Street centerline at Columbia Street.
   b. Parallel and on both sides of the Weber Street centerline at San Miguel Street.
   c. Parallel and on both sides of the Wahsatch Avenue centerline at Columbia Street.
   d. Parallel and east of the Wahsatch Avenue centerline at San Miguel Street.
   e. Parallel and west of the Franklin Street centerline at San Miguel Street.

2. Install a concrete crossspan parallel and north of the San Miguel Street centerline at Franklin Street to eliminate sheet flow across the street crown.
J. Remove existing inlets and install two 6-foot DI60R inlets at the depression on the north and south sides of Caramillo Street west of the railroad right-of-way.

The existing sump curb opening inlets on the north and south sides of San Miguel Street at Shook's Run and Uintah Street at Shook's Run are inadequate to handle the 100-year design flow contributed by the study area. Additional flow to these inlets is contributed by the area to the east of the study area. Calculation of these contributing flows is not a part of the Scope of work for this study. Therefore, it is recommended that these additional contributing flows be determined and new inlets sized for all 100-year design flows. The San Miguel Street 100-year design flow from the study area is 20.0 cubic feet per second. The Uintah Street 100-year design flow from the study area is 31 cubic feet per second. Both inlets are sumps and would require a 10-foot and 4-foot DI60R inlet at San Miguel and Uintah Streets, respectively, if the inlets were to be installed and sized for only the 100-year design flows from the study area and not the flows from east of the study area. By installing these inlets, smaller frequency storm events can adequately drain runoff from both areas until the future study is completed to determine all the required 100-year design flow inlet facilities.

Subbasin G

All drainage is to the south and east to an existing 18 inch culvert which discharges directly to Shook's Run at San Rafael Street. The culvert can safely convey the 5-year design flow, but cannot convey the 100-year flow. Two solutions are possible if the 100-year design flow is to be adequately drained without flooding. A second 18 inch culvert should be constructed next to the existing 18 inch culvert or the existing culvert should be removed and a grouted riprap rundance should be installed. The recommended drainage improvement is the grouted riprap rundance.

Subbasins H and I

All drainage is to the south to existing bubblers which all drain to Cache La Poudre and discharge through two grated drop inlets at Shook's Run.

Recommended drainage improvements for Subbasins H and I are:

1. Abandon the existing bubbler and install concrete crossspans at the following locations:
   a. Parallel and on both sides of the Weber Street centerline at San Rafael Street.
   b. Parallel and on both sides of the Corona Street centerline at San Rafael Street.
   c. Parallel and on both sides of the Corona Street centerline at Yampa Street.
2. The bubblers at the northeast and northwest corner of Wansatch Avenue and San Rafael Street and the bubbler at the northeast corner of Wansatch Avenue and Yampa Street should be converted to grated drop inlets. The curb opening bubbler at the northeast corner of Wansatch Avenue and Yampa Street should be converted to a curb opening inlet. These inlets would connect to the existing storm sewer in Wansatch Avenue. The area that drains to these grated drop inlets currently drain to this same storm sewer through curb opening inlets. If these grated drop inlets are structurally inadequate, install 4 foot DIOR inlets at each location.

3. The bubbler located at the "T" intersection of Royer and Yampa Streets should be abandoned and a concrete crosspan installed. Currently, there is an asphalt crosspan at this location. Installation of a concrete crosspan will provide better street flow hydraulics and asphalt crosspans are undesirable due to deterioration of the asphalt at flowline.

4. Remove the existing bubbler on the northwest and northeast corners of El Paso Street and Cache La Poudre as and install a 6 foot and 4 foot DIOR, respectively.

5. Remove the existing bubbler on the northwest corner of Corona Street and Cache La Poudre as the existing inlet is in a sump condition and has insufficient capacity to drain the 100-year design flow and install an 8 foot DIOR.

6. Convert the existing curb opening bubbler at Royer and Cache La Poudre to curb opening inlets.

The two proposed DIOR inlets, Items 4 and 5 above, are required only to drain the 100-year design flow. The existing curb opening inlet system can handle the 5-year design flow. If the new DIOR inlets are not installed, then it is estimated that half the flow of 2.0 and 2.7 cubic feet per second will overtop the street crown of Cache La Poudre at Corona and El Paso Streets respectively and flow into the drainage area south of the study area. The remaining half of this flow will flow to the sump grated drop inlets at the intersection of Cache La Poudre and Shook's Run.

The calculated 100-year design flow from the study area that flows to the two sump grated drop inlets at Cache La Poudre Shook's Run is 2.7 cubic feet per second. Additional flow to these grated drop inlets is contributed by the area to the east of the study area. Calculation of these contributing flows is not a part of the Scope of Work for this study. However, the two existing drop inlets have a calculated sump capacity of 36.5 and 37.4 cubic feet per second under ideal conditions when the inlets are completely free of debris and therefore, can adequately handle smaller frequency storm events from both areas.
Subbasin J

Subbasin J is the abandoned railroad right-of-way which splits the study area running from the northwest drainage area boundary corner at the Chicago/Rock Island Railroad tracks to the southeast drainage area boundary corner at Shook's Run and Cache La Poudre. It is recommended that the runoff in the right-of-way be cut off from entering the cross streets by means of both an earthen ditch installed on both sides of the bike path and yard inlets installed along each earthen ditch that would operate in a sump condition between each cross street. The right-of-way behind the strip mall at Jackson Street and Walsatch Avenue is a known problem for standing water and muddy conditions that necessitate these improvements. In addition, plug the curb drain that allows flow from abandoned right-of-way to enter Fontanero Street and install a yard inlet to work as a sump to drain the right-of-way.

If the major storm sewer is located within the right-of-way, then yard inlets should be installed within the right-of-way down to Cache La Poudre. If the major storm sewer is located in the street section, then the yard inlets should be installed from above Jackson Street to just above Corona Street. The remaining areas would pond and water would drain from infiltration and evaporation as currently occurs, or curb chases could be installed to allow water to drain to the street and, eventually, to an existing storm sewer system.

An existing 2 foot by 1 foot concrete box storm sewer drains the railroad right-of-way located immediately north of Caramillo Street. This storm sewer connects to the existing 18 inch storm sewer that is located in Walsatch Avenue. It is recommended that the existing box storm sewer be abandoned and replaced with yard inlets, as described above.

Offsite Basin AA

Offsite Basin AA shown in Figure 3 is a portion of Subbasin 3, as delineated in the "Shook's Run Master Drainage Basin Study". This area contributes to the flooding problems on Walsatch Avenue between Fontanero Street and Jackson Street. The area is bounded on the north by the Chicago/Rock Island Railroad tracks, on the east by the Templeton Gap and El Paso Street centerlines, on the south by the Madison Street and Jefferson Street centerlines, and on the west by Subbasin A and B of this study. Calculations of the 100-year design flow indicates that by constructing inlets in this area, the flooding problems in Subbasin A and B and Offsite Basin AA should be alleviated. From conversations with the City Staff, flooding in the past in Subbasin 3 has been reported north of Fontanero Street, but major flooding has not been a problem south of Fontanero Street. Therefore, the proposed storm sewer, as delineated in the "Shook's Run Master Drainage Basin Study" for Subbasin 3, may be reduced in length or eliminated. It is recommended that additional inlets be installed as shown in Figure 8 and connected to the proposed Madison/Walsatch Drainage Facilities to save on future storm sewer construction costs and alleviate flooding along Walsatch Avenue.
Recommended drainage improvements for Offsite Basin AA are:

1. Remove the existing asphalt crossspan parallel and west of Royer street at Madison and install a concrete crossspan.

2. Install D10R inlets at the following locations:
   a. A 12 foot D10R at the southeast corner of Madison Street and Corona Street.
   b. A 12 foot D10R at the northeast corner of Madison Street and Royer Street.
   c. A 12 foot D10R at the northeast corner of Madison Street and El Paso Street.
   d. A 20 foot D10R at the downstream end of the existing concrete crossspan on the south side of Monroe Street, east of Magellan Street.
   e. Two 12 foot D10R inlets at the northeast corner of Magellan Street and Monroe Street; one on either side of the existing concrete crossspan.
   f. Two 14 foot D10R inlets on the northwest and northeast corner of Monroe Street and El Paso Street.
   g. A 4 foot and 12 foot D10R inlet on the northwest and northeast corner of Monroe Street and Franklin Street, respectively.
   h. A 12 foot D10R on the east side of Magellan Street at the intersection of Jackson Street.

Existing Storm Sewers

Access to the existing storm sewers is through cleanouts. However, these access points are overlaid with successive layers of asphalt. It is recommended for maintenance purposes that all the cleanout rims be raised to grade.

STORM SEWER IMPROVEMENTS

The proposed storm sewer should follow the abandoned railroad right-of-way as the new inlets are located in the vicinity of the right-of-way. The Hydrologic Base Maps, Existing and Proposed Drainage Facilities, shows the alternative alignments for the storm sewer trunk line. The storm sewer is required to pick up street flows at inlets at the abandoned railroad right-of-way from Jackson Street to Corona Street. Two alignments are shown. One is along the abandoned railroad right-of-way and the second is located in existing paved streets that follow the railroad right-of-way. From Corona Street to Shook's Run, two alternatives are also available. Existing drainage facilities are generally adequately sized to handle the design flows along this area.
Therefore, the storm sewer trunk line can continue south along the railroad right-of-way for approximately 2800 linear feet and outfall at Shook's Run just upstream of the Cache La Poudre arch culvert crossing. An alternative would be to install the storm sewer easterly along Columbia Street for approximately 2400 linear feet to just downstream of the Columbia Street culvert crossing.

The railroad right-of-way alignment from Jackson Street to Cache La Poudre at Shook's Run is the preferred alignment for the following reasons:

1. Least cost alternative.

2. There are less utilities to cross. Major crossings are generally only at road crossings. Utility plat maps indicate that there are no individual service lines to cross. If constructed in the street section, there are numerous gas, sanitary sewer and water service crossings.

3. There appears to be only electrical poles that run parallel to and inside the proposed railroad right-of-way alignment. However, these poles are just inside the right-of-way and should not be a problem during construction of a major storm sewer. If constructed in the street section, sheeting and shoring requirements to protect parallel utilities will significantly add to installation costs.

4. Open cuts in the railroad right-of-way in ground instead of pavement will lessen the time to construct the project.

5. Asphalt replacement is very costly and is kept at a minimum in the railroad right-of-way alignment.

6. The possibility that pavement removal and heavy construction traffic might damage individual property owners improvements by cracking concrete, damaging grass sod, etc., is greater if the storm sewer is installed in the streets. Generally, each home has a back fence that would help protect a home from noise, dust, and property damage.

7. Traffic disruption will be minimized as the railroad right-of-way alignment will impact traffic only at street crossings. If the storm sewer is installed in the street, some streets will probably be closed to traffic during pipe installation and one side of Washaatch Avenue will probably be closed. Traffic control and safety requirements and costs will be significantly higher.

8. Pedestrian safety and neighborhood impact will be less if the storm sewer is installed in the railroad right-of-way. The right-of-way generally faces the back of residential homes in the area and has a bike path. Portions of the bike path would be closed during construction, but pedestrian traffic would primarily be effected only at street crossings. A storm sewer in the street sections would be in
Front of individual homes, may create frequent citizen complaints, and pedestrian safety precautions would be a prime concern.

The storm sewer hydraulics, pipe sizing and proposed alternate alignments are presented in Appendix E.

Reinforced concrete pipe was used for all storm sewers. The City Corrosion Foreman has indicated in telephone conversations that the railroad right-of-way is a very corrosive soil and that corrosion protection measures should be used for any construction in this area. The initial cost of corrugated steel pipe is competitive. Any steel pipe design would probably require pipe coating and an extensive cathodic protection system. Therefore, the total cost of steel pipe installation would be higher and the design life shorter than for concrete pipe.

Ribbed polyethylene pipe was also reviewed due to the pipe's smooth interior which provides a lower pipe roughness coefficient and a higher flow capacity than other types of pipe. As a result, smaller polyethylene than concrete pipe sizes can generally be installed for equivalent flows. In addition, polyethylene pipe is corrosion resistant, which is desirable along the pipe alignments shown for the proposed storm sewer. The City of Colorado Springs is currently installing some polyethylene on an experimental basis to test the manufacturer's claims about this product. Therefore, until the results of the test are complete and ribbed polyethylene pipe becomes a City-accepted standard pipe material, reinforced concrete pipe is the recommended pipe material.

The following utility agencies were contacted relative to existing and future facilities located along the railroad right-of-way and the possible construction at a storm sewer trunk line in this area:

1. Cablevision
2. City of Colorado Springs, Electrical Division
3. City of Colorado Springs, Engineering Division
4. City of Colorado Springs, Gas Division
5. City of Colorado Springs, Wastewater Division
6. City of Colorado Springs, Water Division
7. Mountain Bell

The only existing utilities found in and running parallel to the railroad right-of-way are overhead electric lines located on the west boundary along the entire right-of-way and on the east boundary from Espanola Street to the Chicago/Rock Island Railroad tracks. There are underground and overhead utilities located at the railroad right-of-way street crossings. The utility plat maps do not provide any indication of buried cover depth for existing facilities. There are
numuneral small pipeline crossings. However, major utility crossings 10 inch and larger are as follows:

<table>
<thead>
<tr>
<th>Street Crossing</th>
<th>Facility</th>
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<tbody>
<tr>
<td>Madison Street</td>
<td>20 inch water</td>
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<tr>
<td>Fontanero Street</td>
<td>Telephone line</td>
</tr>
<tr>
<td>Del Norte Street</td>
<td>10 inch gas</td>
</tr>
<tr>
<td>Caramillo Street</td>
<td>24 inch water</td>
</tr>
<tr>
<td>Wanship Avenue</td>
<td>Telephone line</td>
</tr>
<tr>
<td>Columbia Street</td>
<td>12 inch gas</td>
</tr>
<tr>
<td>Uintah Street</td>
<td>24 inch sanitary sewer</td>
</tr>
<tr>
<td></td>
<td>30 inch water</td>
</tr>
</tbody>
</table>

It is recommended that these major utilities and all other utility crossings be potholed to determine pipe elevations to facilitate design of a gravity storm sewer.

The Shook’s Run Master Drainage Basin Study proposed an open channel from Jackson Street to Fontanero Street. A review of existing street grades at Madison and Washington Streets at Wanship Avenue, where inlets are required to alleviate flooding problems, indicates that there is insufficient grade differences to construct an open channel located in the railroad right-of-way. It is estimated that an open channel would have to have a depth of 7 feet allowing a 0.8 percent slope from proposed inlets to the channel. Therefore, a storm sewer pipeline is recommended for the entire length of the drainage improvements.

CACHE LA Poudre EXISTING ARCH CULVERT

The existing Cache La Poudre street crossing at Shook’s Run is a concrete arch culvert with a dirt bottom. The arch opening is approximately 20 feet wide at the base, 6 feet up from the base it is 18.5 feet wide, and is 12.5 feet in height. An earth berm with brush and trees partially blocks the arch culvert inlet and restricts the flow. Flow in Shook’s Run must go around this berm and then enters the culvert. The culvert has two 45 degree bends in it which offset the culvert inlet and outlet by approximately 10 feet. There are numerous utilities exposed that span inside the culvert. These utilities may be “washed out” and fail during a large flood event.

The Shook’s Run Master Drainage Basin Study 100-year design flow hydrograph indicates that the peak flow is 2196.5 cubic feet per second at a time to peak of 1.1 hours. The calculations for this hydrograph appear to assume continuous surface runoff and do not account for ponding at inlets and sumps, storage in bubblers, or detention storage in the railroad right-of-way left to evaporate or infiltrate.

With the recommended design, the total volume of runoff will not vary from the volume calculated in the “Shooks Run Master Drainage Basin Study,” since the same drainage area is under consideration. What
will vary in the shape of the hydrograph. The area will have storm sewers, so the hydrograph rising limb should be steeper and, with a constant volume, the peak discharge should be smaller.

Therefore, for existing and proposed conditions, peak discharge in Shook's Run at Cache La Poudre will be somewhat smaller than predicted in the "Shook's Run Master Drainage Basin Study." A conservative culvert capacity analysis would use the peak discharge of 2196.5 cfs. Channel shape and slope were taken from the base maps used for the study. It was assumed that the culvert inlet restrictions were removed. It was determined from the culvert capacity calculations presented in Appendix E that inlet control will approximate the capacity of the culvert, since the culvert is close to the inlet control/outlet control transition. The existing crossing should safely pass the peak discharge.

It is recommended that the earthen berm and trees be removed from blocking the concrete arch culvert inlet. In addition, the inlet channel banks should have riprap installed for bank stabilization.

NORTH JUNIOR HIGH SCHOOL FLOODING

North Junior High School, a part of El Paso County School District 11, is located on Yampa Street at the "T" intersection with Royer Street. The school roof drains are connected to an 18 inch storm sewer that is located in Royer Street and connects to an existing 45 inch storm sewer in Dale Street where it outfalls at the channel invert of Shook’s Run.

The school district maintenance staff has indicated in transmittal letters to City Engineering Division that flooding in the school boiler room occurs during periods of heavy rainfall or snowmelt. Sump pumps have been installed by the school district to discharge water out of the boiler room to control the flooding. However, on at least one occasion the water level approached 4 inches below the electrical equipment which would have caused an electrical failure and potentially could have closed the school.

The school district maintenance staff reports that the water entering the boiler room sump is full of leaves and sticks. In addition, it appeared that the water rises in the sump faster than water could be collected and drained from the school roof and therefore it is felt that the existing storm sewer system is backing up from Shook's Run into the boiler room sump. The school staff has also reported that they have rodded the school's drain system including from the last cleanout in the school to beyond the property line on Yampa Street and no obstructions were found.

The city maintenance staff is currently raising all the cleanouts to grade in the pavement sections and completed cleaning the storm sewer from the last cleanout in the school prior to entering Yampa Street to Shook’s Run outfall at Dale Street on Thursday, September 19, 1985. The city maintenance staff reports removing 3 to 5 gallon
buckets of asbestos-type material from the Yampa Street cleanout to
the school cleanout. The city staff believes that this material was
blocking the flow into the storm sewer and that the school roof drain
should now function without flooding.

A review of the grade difference between the Shook's Run outfall and
the school sump was taken from the base maps. The outfall elevation is
approximately 6019 feet. The storm sewer cleanout located in Yampa
Street immediately in front of the school is approximately 8 to 10
feet deep and the street grade is approximately 6042 feet. Therefore,
the storm sewer invert is approximately at elevation 6032 feet. This
provides a drop in elevation of 23 feet. The top of bank elevation of
Shook's Run at the outfall is approximately 6020 feet. This is 12 feet
below the storm sewer invert in front of the school.

As a result of the elevation difference and recent storm sewer
cleaning by the city maintenance staff, we recommend that the school
district observe through the 1985-1986 winter snowmelt and see if
flooding continues in the boiler room sump. If flooding does continue,
we recommend two alternatives. First, the roof drains could be
disconnected from the storm sewer and drained to outside on the school
grounds. All drainage inside the building that is below the ground
outside the school would drain to the existing boiler room sump pump
which would pump to outside the school. The existing 18 inch storm
sewer would be plugged at Yampa Street and disconnected from the
school roof drainage system.

Secondly, the outfall invert and all cleanout invert elevations could
be verified by field survey. The slopes of existing storm sewers could
be calculated as there are no record drawings on this storm sewer
system except for pipe sizes. Then the elevation difference could be
verified. This would produce a record to indicate if there is a
problem within the school roof drain system or a problem in the city
storm sewer system.

COST ESTIMATE

The engineer's construction cost estimate is provided in Appendix F.
Unit prices were determined from review of the following cost
information:

2. Recent bid tabulations published by the Colorado Contractors
   Association.
3. Solicitation of estimates from various contractors for the
   various items of work.
4. Past City of Colorado Springs Engineering Division bids.

The following items were assumed for the cost estimate:
1. No utility relocation costs are included in the cost estimate as pitholing of existing utility locations is to be completed as a part of final design. The impact of utility interference cannot be determined until the pitholing has been completed.

2. An 8 inch thick asphalt pavement over 12 inches of Class 5 road base is assumed for all new pavement or pavement replacement. Unit weights of 133 and 145 pounds per cubic feet for road base and asphalt pavement, respectively, were used.

3. Storm sewer installation costs are based on Class III pipe with an average of 5 feet of cover. Final design will determine actual cover requirements for the pipe.

4. Unit prices are September 1985 prices.

The least construction cost estimate for the Madison/Wahsatch Drainage Improvements is $3,070,920 for a major storm sewer located in the abandoned railroad right-of-way from Jackson Street to Shook's Run at Cache La Poudre and appurtenant drainage facilities. The average estimate cost per linear foot of pipeline is $145 for installing the storm sewer trunk line in the abandoned railroad right-of-way. This compares to an average estimated cost of $240 per linear foot (no sheeting costs have been included) for construction of the storm sewer trunk line in the paved street sections.

The estimate costs of construction are intended to provide an indication of the costs involved and are considered to be an estimate only. We, as engineers, have no control over the cost of materials, equipment and labor, or competitive bidding, and cannot guarantee the accuracy of the construction costs. The unit prices used in the estimate reflect estimated current costs and do not provide for inflation.

CONCLUSIONS

The study area has experienced localized flooding in the past. A system of crossspans, inlets, and storm sewers is planned to alleviate this situation. The area north of Jackson Street, Subbasin B, will require street improvements to improve gutter capacity.

Part of the study area drains south and west towards the abandoned railroad right-of-way. The elevated abandoned railroad track bed creates a sump condition for the drainage area located northeast of the old railroad track bed. The storm sewer has been sized for the 100-year event to provide adequate protection in this area.

Two alternative alignments for the main storm sewer system were investigated. The favored alignment follows the railroad right-of-way, and would cost approximately $3.1 million. Coordination will be required with the City Electrical Division and City Parks Department, since they claim ownership of the right-of-way. The other alignment would be installed in the street, but this would entail numerous utility service crossings, extensive asphalt pavement replacement,
greater traffic and neighborhood disturbances and higher construction costs.
EXISTING DRAINAGE FACILITIES

PONDING IN ABANDONED RAILROAD RIGHT OF WAY

PONDING IN ABANDONED RAILROAD RIGHT OF WAY

FIGURE 2
NOTES
1. MADISON/WAHSATCH DRAINAGE AREA IS SUBBASIN 4 AS DELINEATED IN THE "SHOOLS RUN MASTER DRAINAGE BASIN STUDY" DATED DECEMBER 1972.
2. OFFSITE BASIN AA IS A PORTION OF SUBBASIN 3 AS DELINEATED BY THE SAME STUDY, BUT DRAINS TO WAHSATCH AVENUE NORTH OF FONTANERO STREET AND IS INCLUDED IN THIS REPORT.

FIGURE 3
MADISON/WAHSATCH DRAINAGE IMPROVEMENTS
MAJOR DRAINAGE SUBBASINS
FIGURE 5

AVGAGE VELOCITIES FOR ESTIMATING TRAVEL TIME FOR OVERLAND FLOW

REFERENCE: 1. Colorado Springs Drainage Criteria Figure I for
T_c between 0.1 and 3 hours for 6 hour Type II A storm
2. T_c between 0.01 and 0.1 hours is extrapolated.

TIME OF CONCENTRATION AS A FUNCTION OF PEAK DISCHARGE

FIGURE 6

Delmore Engineering Corporation
1849 One Boulevard
River 305 • Building 2
Denver, Colorado 80226
(303) 273-823
RUNOFF EQUATION

\[ Q = \left( \frac{P-0.2S}{P+0.8S} \right)^2 \]

\[ S = \frac{1000}{C} - 10 \]

POINT OF ZERO RUNOFF

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<td>5 YEAR STORM</td>
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<tr>
<td>100 YEAR STORM</td>
<td>36.4</td>
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FIGURE 7

RUNOFF AS A FUNCTION OF CURVE NUMBER & PRECIPITATION
MADISON / WAHSATCH
DRAINAGE IMPROVEMENTS
FINAL DRAINAGE STUDY
TECHNICAL ADDENDUM

Oct, 1972

DENVER ENGINEERING CORPORATION
1626 COLE BOULEVARD SUITE 300 BUILDING 7
GOLDEN, COLORADO 80401
(303)233-0533
DRAINAGE STUDY
MADISON/WAHSATCH DRAINAGE IMPROVEMENTS

TECHNICAL ADDENDUM

OCTOBER 1985

Prepared For:
CITY OF COLORADO SPRINGS
DEPARTMENT OF PUBLIC WORKS
CITY ENGINEERING DIVISION
30 SOUTH NEVADA, SUITE 403
COLORADO SPRINGS, COLORADO 80901

Prepared By:
DENVER ENGINEERING CORPORATION
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(303) 239-0533

DEC JOB NO. 307.001
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C. Curb Opening and Grated Inlet Capacities & Calculations
D. Subbasin Hydrology & Hydraulic Routing Analysis
E. Proposed Storm Sewer Improvements & Hydraulics Analysis
F. Cost Estimate
G. Bibliography
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<td>Single Block Time of Concentration Calculations</td>
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<td>Single Block 5-Year &amp; 100-Year Design Flows</td>
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</table>
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2. OFFSITE BASIN AA IS A PORTION OF SUB BASIN 3 AS DELINEATED BY THE SAME STUDY, BUT DRAINS TO WAHSATCH AVENUE NORTH OF FORTAMER STREET AND IS INCLUDED IN THIS REPORT.

MADISON/WAHSATCH DRAINAGE IMPROVEMENTS

MAJOR DRAINAGE SUBBASINS
FIGURE 5

AVERAGE VELOCITIES FOR ESTIMATING TRAVEL TIME FOR OVERLAND FLOW

REFERENCE: 1. Colorado Springs Drainage Criteria Figure I for
   $T_C$ between 0.1 and 5 hours for 6 hour Type II A storm.
   2. $T_C$ between 0.01 and 0.1 hours is extrapolated.

FIGURE 6

TIME OF CONCENTRATION AS A
FUNCTION OF PEAK DISCHARGE

Denver Engineering Corporation
1921 One-Boulevard
Burlington, VT 05401
Gibsons, Colorado Hills
(303) 221-8343
RUNOFF EQUATION

\[ Q = \frac{(P - 0.2S)^2}{Pr 0.85} \]

\[ S = \frac{1000}{CN} - 10 \]

POINT OF ZERO RUNOFF

5 YEAR STORM \( 48.8 \)
100 YEAR STORM \( 36.4 \)

RUNOFF AS A FUNCTION OF CURVE NUMBER & PRECIPITATION

FIGURE 7
<table>
<thead>
<tr>
<th>LAND USE DESCRIPTION</th>
<th>HYDROLOGIC SOIL GROUP</th>
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<td>A</td>
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<td>Cultivated land 1:</td>
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<td>without conservation treatment</td>
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<td>good condition</td>
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1 For a more detailed description of agricultural land use curve numbers refer to National Engineering Handbook, Section 4, Hydrology, Chapter 9, Aug. 1972.
2 Good cover is protected from grazing and litter and brush cover soil.
3 Curve numbers are computed assuming the runoff from the house and driveway is directed towards the street with a minimum of roof water directed to lawns where additional infiltration could occur.
4 The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.
TABLE 2
Single Block Time of Concentration Calculations

\[ t_C = \frac{L}{V \times 3600} \]

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### TABLE 2

Single Block Time of Concentration Calculations

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<td>Locations</td>
<td>Load (lbs)</td>
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<td>Board Length (ft)</td>
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<td>Area (ft²)</td>
<td>Acia (lb)</td>
<td>Spn</td>
<td>Top v.</td>
<td>Cp y.</td>
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<tr>
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<td>1000</td>
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</table>
The street capacities for the Madison/Wahsatch Drainage Improvement Study were calculated by using a refinement of the Manning's Equation, developed by Izzard, for computation of flows in street sections:

$$Q = 0.56 \frac{z}{n} \sqrt{y}$$

Where:
- $Q$ = Capacity in triangular (half street) section, cubic feet per second
- $z$ = Reciprocal of cross slope, feet per foot
- $n$ = Mannings roughness coefficient
- $y$ = Depth of flow at face of curb, feet
- $S$ = Street grade, feet per foot

A direct solution for a composite section may be obtained by substituting $Z_3$ into the above equation. $Z_3$ is defined as:

$$Z_3 = Z_1 \left[ 1 + \left( \frac{Z_2}{Z_1 - 1} \right) \left( \frac{T - W}{T + W} \right) \right]^{\frac{1}{2}}$$

Where:
- $Z_3$ = Reciprocal of cross slope of equivalent section, feet per foot
- $Z_1$ = Reciprocal of cross slope of gutter, feet per foot
- $Z_2$ = Reciprocal of cross slope of pavement, feet per foot
- $T$ = Top width of water surface, feet
- $W$ = Width of gutter, feet

The depth of flow in streets was limited to zero depth at centerline to allow for passage of emergency vehicles.

A programmable calculator was used to determine the capacities of the streets with varying widths, slopes and curb sizes. The tables for these calculations follow in this section. The maximum depth of flow value allowed in the curb and gutter ($y_{max}$) was limited by the criteria for emergency vehicles.

Ramp curbs are located in the northern most section of the study area. They vary in height from 1 to 4 inches and the gutter width varies from 5 to 9 inches due to overlays performed in the past. In some cases asphalt has been overlayed up to the flow line. In these cases, an "n" value of 0.13 was used.
A standard ramp curb may be used to replace the existing and deteriorating ramp curbs in this area. A non-uniform cross slope was used to achieve maximum capacity in the curb as this area has a tendency for flooding due to the inadequacy of the existing curbs. For calculating purposes, a composite cross slope in the street was used.

In the areas south of Jackson, existing vertical curb capacities were calculated.
**STREET CAPACITIES**

**Design Calculations**

- **Weber, Uintah, Corona**

**Street Width = 60'**

<table>
<thead>
<tr>
<th>Curve Size</th>
<th>15'</th>
<th>10'</th>
<th>7'</th>
<th>5'</th>
<th>4'</th>
</tr>
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<tbody>
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<td>347</td>
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<td>415</td>
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</table>

**Design Calculations**

- **Maximum Flow Rate Calculation**
  
  \[ \text{Maximum Flow Rate} = \text{Curve Size} \times \frac{\text{Slope}}{2} \]

- **Flow at Centerline**
  
  \[ \text{Flow at Centerline} = \text{Maximum Flow Rate} \times \text{Width} \]

- **Flow at Edge**
  
  \[ \text{Flow at Edge} = \text{Flow at Centerline} \times \text{Edge Factor} \]

- **Maximum Flow Rate**
  
  \[ \text{Maximum Flow Rate} = \text{Curve Size} \times \frac{\text{Slope}}{2} \]

- **Flow at Centerline**
  
  \[ \text{Flow at Centerline} = \text{Maximum Flow Rate} \times \text{Width} \]

- **Flow at Edge**
  
  \[ \text{Flow at Edge} = \text{Flow at Centerline} \times \text{Edge Factor} \]

**Table for Curve Size**

<table>
<thead>
<tr>
<th>Curve Size</th>
<th>15'</th>
<th>10'</th>
<th>7'</th>
<th>5'</th>
<th>4'</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>347</td>
<td>347</td>
<td>347</td>
<td>347</td>
<td>347</td>
</tr>
<tr>
<td>2</td>
<td>401</td>
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<tr>
<td>3</td>
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</tr>
<tr>
<td>4</td>
<td>415</td>
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**Flow at Centerline**

\[ \text{Flow at Centerline} = \text{Maximum Flow Rate} \times \text{Width} \]

**Flow at Edge**

\[ \text{Flow at Edge} = \text{Flow at Centerline} \times \text{Edge Factor} \]

**Maximum Flow Rate**

\[ \text{Maximum Flow Rate} = \text{Curve Size} \times \frac{\text{Slope}}{2} \]

**Flow at Centerline**

\[ \text{Flow at Centerline} = \text{Maximum Flow Rate} \times \text{Width} \]

**Flow at Edge**

\[ \text{Flow at Edge} = \text{Flow at Centerline} \times \text{Edge Factor} \]
Maximum Height of Flood in Curb Required Categories to Allow for Access of Emergency Vehicles.

Note: T and Y values in each category are the same.

For Category D:
- T = 1.5
- Y = 0.5

For Category C:
- T = 1.5
- Y = 0.5

For Category B:
- T = 1.5
- Y = 0.5

For Category A:
- T = 1.5
- Y = 0.5

For Category E:
- T = 1.5
- Y = 0.5

Note: Flood depth must be less than the street elevation.

DESIGN CALCULATIONS
STREET WIDTH = 40'

<table>
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<th>Curve Height (in)</th>
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<tbody>
<tr>
<td>5' 10' 15' 20' 25'</td>
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<tr>
<td>.1 11.2 11.2 11.2 11.2 11.2</td>
</tr>
<tr>
<td>.5 31.7 31.7 28.1 25.1 25.1 25.1 25.1 20.7 13.4 8.0 4.3 2.0 0.8</td>
</tr>
<tr>
<td>.8 31.7 31.7 31.7 31.7 31.7 31.7 26.3 16.9 10.1 5.5 2.6 1.0</td>
</tr>
<tr>
<td>1.0 35.4 35.4 35.4 35.4 35.4 35.4 29.5 16.9 11.3 6.1 2.9 1.1</td>
</tr>
<tr>
<td>1.5 43.4 43.4 43.4 43.4 43.4 43.4 35.9 23.1 13.8 7.5 3.5 1.4</td>
</tr>
<tr>
<td>2.0 50.1 50.1 50.1 50.1 50.1 50.1 41.5 26.7 16.0 8.6 4.1 1.6</td>
</tr>
<tr>
<td>2.5 56.0 56.0 56.0 56.0 56.0 56.0 46.4 29.9 17.9 9.6 4.5 1.8</td>
</tr>
</tbody>
</table>

Max. Curvature = 20'

\[ y_{max} = 2/3 + 18(0.05) = 0.71' = 8.5" \]

**Curb**

- **T (in)**
- **v (ft)**

Note: At a curb depth, the capacity of the flood of 8" and below is the same as 6", 5", and 4" street width. The maximum value for depth of flow in curb is not to exceed 0.6". Note: "T" and "v" values needed to calculate street capacities.
**Street Width = 30''**

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<th>30</th>
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<td>5</td>
<td>1</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
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<tr>
<td>15</td>
<td>15</td>
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<tr>
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<tr>
<td>25</td>
<td>25</td>
<td>1</td>
</tr>
<tr>
<td>30</td>
<td>30</td>
<td>1</td>
</tr>
</tbody>
</table>

**Curb Height (in)**

<table>
<thead>
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<th>15</th>
<th>30</th>
</tr>
</thead>
<tbody>
<tr>
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<td>1</td>
</tr>
<tr>
<td>10</td>
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<tr>
<td>15</td>
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<td>25</td>
<td>25</td>
<td>1</td>
</tr>
<tr>
<td>30</td>
<td>30</td>
<td>1</td>
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</tbody>
</table>

**Design Calculations**

**T_max = 15'**

**Curb = 3'-15''**

**Note:** AT A CURB DEPTH OF 6'' AND 15'-15'', THE CAPACITY OF THE 60, 50, 44 AND 30' STREET WIDTHS IS THE SAME.

**Note:** THE MAXIMUM VALUE FOR A CURB IN 15-15'' CURVES IS NOT TO EXCEED A DEPTH OF 6'' AT CURVE CENTER TO ALLOW FOR PASSAGE OF CIVILIAN VEHICLES.
### DESIGN CALCULATIONS

#### DESIGN CALCULATIONS

<table>
<thead>
<tr>
<th>Type</th>
<th>Conduit Diameter (in)</th>
<th>Curvature (in)</th>
<th>Slope (%)</th>
<th>Rate Per Foot (ft/sec)</th>
<th>Total Capacity (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asphalt</td>
<td>3</td>
<td>15”</td>
<td>2.0%</td>
<td>0.21</td>
<td>3.05</td>
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<tr>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rollover AC Gutter</td>
<td>3</td>
<td>15”</td>
<td>2.0%</td>
<td>0.21</td>
<td>3.05</td>
</tr>
<tr>
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<td></td>
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<td></td>
</tr>
</tbody>
</table>

### Rollover AC Gutter

- **Slope**: 0.4% 0.27
- **Rate Per Foot**: 2.31 = 8.58 CFS
- **Slope**: 0.8% 0.39
- **Rate Per Foot**: 3.87 = 3.66 CFS

### Design Calculations

- **Curvature**: 3.75
- **Slope**: 2.0%
- **Slope**: 1.5% 1.67
- **Rate Per Foot**: 0.70 (9') 1.0 (9') 1.2 (9') 1.3 (9')

### Table

<table>
<thead>
<tr>
<th>Slope</th>
<th>Rate Per Foot (ft/sec)</th>
<th>Total Capacity (cfs)</th>
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</thead>
<tbody>
<tr>
<td>1.1%</td>
<td>0.70 (9') 1.0 (9') 1.2 (9') 1.3 (9')</td>
<td>7.7 CFS</td>
</tr>
<tr>
<td>1.1%</td>
<td>0.59 (5') 0.64 (5') 0.69 (5') 0.76 (5')</td>
<td>8.2 CFS</td>
</tr>
<tr>
<td>1.1%</td>
<td>0.59 (5') 0.64 (5') 0.69 (5') 0.76 (5')</td>
<td>8.2 CFS</td>
</tr>
<tr>
<td>1.1%</td>
<td>0.59 (5') 0.64 (5') 0.69 (5') 0.76 (5')</td>
<td>8.2 CFS</td>
</tr>
<tr>
<td>1.1%</td>
<td>0.59 (5') 0.64 (5') 0.69 (5') 0.76 (5')</td>
<td>8.2 CFS</td>
</tr>
</tbody>
</table>
### Modified Rollover Type Curves

\[ \frac{c}{h} = 0.3 \quad \frac{h_0}{d} = 3.3 = 2 \]

\[ \frac{c}{h} = 0.107 \quad \frac{h_0}{d} = 9.3 = 2 \]

### Capacity of Section 'A' + 'B'

<table>
<thead>
<tr>
<th>Slope</th>
<th>Cap 'A'</th>
<th>Cap 'B'</th>
<th>Total Cap</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.4%</td>
<td>5.3</td>
<td>0.7</td>
<td>6.0</td>
</tr>
<tr>
<td>0.8%</td>
<td>7.5</td>
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<td>1.1%</td>
<td>8.8</td>
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<td>10.0</td>
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<tr>
<td>1.6%</td>
<td>10.6</td>
<td>1.4</td>
<td>12.0</td>
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</tbody>
</table>

- **Section 'A'**
  - \( T = 5.7' \)
  - \( y = 0.25' \)
- **Section 'B'**
  - \( T = 7.0' \)
  - \( y = 31.25 - 25 = 6.25' \)
- **Section 'C'**
  - \( T = 0.9' \)
  - \( y = 3.2' \)
MADISON/WAHSATCH DRAINAGE IMPROVEMENT STUDY

APPENDIX C

CURB OPENING & GRATED INLET CAPACITIES & CALCULATIONS

C.1 City Standard Curb Opening and Grated Inlets
C.2 Existing (Bubbler) Grated Inlets
C.3 Existing Curb Opening Inlets
APPENDIX C

C.1 City Standard Curb Opening and Grated Inlets

Where new inlets are required, the standard Colorado Springs D10R curb opening inlet or the standard grated inlet will be used.

Table 6 from the "Subdivision Policy Manual" has been used to size the curb opening inlets. The grated inlet capacities have been calculated on Sheet 3 of this section. Curb heights of 1, 2, 3 and 4 inches have been analyzed. The grated inlets were only investigated in locations in the study area where existing ramp curbs are located.
## Curb Opening Inlet Capacities (cfs)

**Table 6**

<table>
<thead>
<tr>
<th>Opening Length (ft.)</th>
<th>4.0</th>
<th>6.0</th>
<th>8.0</th>
<th>10.0</th>
<th>12.0</th>
<th>14.0</th>
<th>16.0</th>
<th>18.0</th>
<th>20.0</th>
<th>22.0</th>
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<tbody>
<tr>
<td>Sump Capacity (cfs)</td>
<td>7.9</td>
<td>12.8</td>
<td>18.4</td>
<td>23.0</td>
<td>27.6</td>
<td>34.5</td>
<td>39.4</td>
<td>44.4</td>
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<tr>
<td>Street Slope (%)</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>------</td>
<td>------</td>
<td>------</td>
<td>------</td>
<td>------</td>
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<tr>
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<td>5.6</td>
<td>6.8</td>
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<td>9.8</td>
<td>9.4</td>
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<td>10.4</td>
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<td>6.5</td>
<td>12.1</td>
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<td>25.9</td>
<td>26.3</td>
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<td>27.6</td>
<td>28.0</td>
<td>28.3</td>
<td>28.8</td>
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</table>

Revised: C.Aomofd/S-16-74
**CITY OF COMMERCE SPRINGS**  
**STANDARD GRADED INLET**

<table>
<thead>
<tr>
<th>Area</th>
<th>Length</th>
<th>Formula</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>5'-0&quot; Long</td>
<td>(5'x2') - [(2.5'x1')/2] + [(9.5'x1')/2]</td>
<td>15 Bar(s) @ 1&quot;</td>
</tr>
<tr>
<td>B</td>
<td>7'-6&quot; Long</td>
<td>7'6&quot;</td>
<td>15 Bar(s) @ 1&quot;</td>
</tr>
<tr>
<td>C</td>
<td>10'-0&quot; Long</td>
<td>10'</td>
<td>15 Bar(s) @ 1&quot;</td>
</tr>
</tbody>
</table>

**A SF Opening = 5.375 SF**  
**B SF Opening = 8.50 SF**  
**C SF Opening = 11.625 SF**

**\( Q = CA \)**

<table>
<thead>
<tr>
<th>GRADE</th>
<th>'A' 5'-0&quot; Long</th>
<th>GRADE</th>
<th>'B' 7'-6&quot; Long</th>
<th>GRADE</th>
<th>'C' 10'-0&quot; Long</th>
</tr>
</thead>
<tbody>
<tr>
<td>4&quot;</td>
<td>14.9</td>
<td>2'</td>
<td>30.5</td>
<td>3&quot;</td>
<td>32.3</td>
</tr>
<tr>
<td>3&quot;</td>
<td>12.9</td>
<td>2'</td>
<td>20.5</td>
<td>3&quot;</td>
<td>28.0</td>
</tr>
<tr>
<td>2&quot;</td>
<td>10.6</td>
<td>2'</td>
<td>16.7</td>
<td>2&quot;</td>
<td>22.9</td>
</tr>
<tr>
<td>1&quot;</td>
<td>11.8</td>
<td>2'</td>
<td>11.8</td>
<td>1&quot;</td>
<td>16.2</td>
</tr>
</tbody>
</table>

**Q = \text{Area} \times \text{Depth of water} (ft of water)**
C.2 Existing (Bubbler) Grated Inlets

The maximum capacity of the grated inlets was calculated using the orifice equation for a sump condition.

\[ Q = CA \sqrt{gh} \]

- \( Q \) = Capacity in cubic feet per second
- \( C \) = 0.60
- \( A \) = Area in square feet
- \( g \) = The constant value 32.2
- \( h \) = Depth in feet of water over the grate

Presently, the grated inlets are being used as bubbler systems. From discussions with the City Staff, the bubbler systems are not a sufficient drainage facility within this area. Solutions to this problem have been analyzed and the bubbler systems will be abandoned and replaced with either concrete cross pans to carry flow across streets, or with bubbler systems converted to grated drop inlets.

The maximum capacity of each grated drop inlet has been calculated for varying depth of flow corresponding to curb heights on the following pages. The location of each inlet has been called out for easy reference. The capacities calculated do not include a reduction factor. The inlets should be kept free of debris and clogging by periodic maintenance in order to maximize the efficiency of the grated drop inlets.
TYPE 1

N. YAMADA - E. YAMADA
N. WILSON - E. SAN RAPHAEL (ALL)
N. WILSON - SAN RAFAEL (ALL)

\[
\text{SF of Opening} = (2' \times 1.5') - \left[ \frac{2(2' \times 1.5') - 2(1.5' \times 1.5')}{1.5'} \right]
\]

\[
Q = \frac{1875}{CA
\text{ PE}
\frac{C}{C}
\text{ PE}
\frac{C}{C}
\text{ PE}
\]

Core Height

15' = 20.64 cf
14' = 19.16 cf
13' = 17.40 cf
12' = 15.63 cf
11' = 13.84 cf
10' = 12.04 cf
9' = 10.24 cf
8' = 8.44 cf
7' = 6.64 cf
6' = 4.84 cf
5' = 3.04 cf
4' = 1.24 cf
3' = 0.44 cf
2' = 0.04 cf
TYPE 2
CROSSWALL 6.16' X 6.16'

SF Coverage:
\[ Q = \frac{2(1.5)}{2(1.5) + (4.17)(1.5) - (4.17)(1.5)(1.5)} \]

\[ Q = 1.83 \text{ SF} \]

15' = 8.44 cfs
11' = 7.06 cfs
7' = 5.63 cfs
14' = 7.96 cfs
10' = 6.73 cfs
6' = 5.02 cfs
13' = 7.67 cfs
9' = 6.36 cfs
5' = 4.76 cfs
12' = 7.31 cfs
8' = 6.02 cfs
4' = 4.25 cfs
**DESIGN CALCULATIONS**

**Project:**

**Detail:**

**Designer:**

**Date:**

**Sheet No.:**

**Job No.:**

---

**Type B**

**Potte Eman**

**11 West E. San Miguel**

---

\[ q = \frac{a}{C} \cdot \frac{1}{1.32} \]

\[ = 1.68 \text{ SF} \]

\[ Q = 2.0 \]

\[ C = 0.6 \]

\[ 15'' = 9.04 \text{ cu ft} \]

\[ 14'' = 8.74 \text{ cu ft} \]

\[ 13'' = 8.44 \text{ cu ft} \]

\[ 12'' = 8.09 \text{ cu ft} \]

\[ 11'' = 7.74 \text{ cu ft} \]

\[ 10'' = 7.38 \text{ cu ft} \]

\[ 9'' = 7.01 \text{ cu ft} \]

\[ 8'' = 6.64 \text{ cu ft} \]

\[ 7'' = 6.18 \text{ cu ft} \]

\[ 6'' = 5.72 \text{ cu ft} \]

\[ 5'' = 5.22 \text{ cu ft} \]

\[ 4'' = 4.67 \text{ cu ft} \]
DESIGN CALCULATIONS

Type 4
San Miguel
11 Franklin

\[
\text{sf} = \frac{3.21 \times 10^{-3} \times 10^{-3} \times 10^{-3}}{14(0.64 \times 0.125) + \left(4 \times 3 \times 0.125 - 4 \times 0.64 \times 0.125\right)}
\]

\[= 3.48 \text{ sf}\]

\[a = \text{condition}\]

\[C = 0.6\]

\[g = 322\]

<table>
<thead>
<tr>
<th>(1)</th>
<th>(12)</th>
<th>(10)</th>
<th>(9)</th>
<th>(8)</th>
<th>(7)</th>
<th>(6)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5'</td>
<td>18.46 cf</td>
<td>11'</td>
<td>15.81 cf</td>
<td>7'</td>
<td>12.61 cf</td>
<td>6'</td>
</tr>
<tr>
<td>1.4'</td>
<td>17.84 cf</td>
<td>10'</td>
<td>15.08 cf</td>
<td>6'</td>
<td>11.69 cf</td>
<td>5'</td>
</tr>
<tr>
<td>1.3'</td>
<td>11.40 cf</td>
<td>9'</td>
<td>14.30 cf</td>
<td>5'</td>
<td>9.54 cf</td>
<td>4'</td>
</tr>
<tr>
<td>1.2'</td>
<td>16.54 cf</td>
<td>8'</td>
<td>13.48 cf</td>
<td>3'</td>
<td>6.64 cf</td>
<td>2'</td>
</tr>
</tbody>
</table>
TYRES
E. SAN NICOL
N. WOEZER

15'
5 Base @ 1'
7 Base @ 1'

\[
\left(2\times 15\right) - \left(7\left(\frac{1}{8}\times 12\right)\right) + \left(5\left(\frac{1}{8}\times 12\right) - 7\left(\frac{1}{8}\times 12\right)\right) = 1.45 \text{ SF}
\]

\[
Q = \frac{9.32}{0.06}
\]

Flow Table:

- 15' = 7.81 CFS
- 14' = 7.54 CFS
- 13' = 7.27 CFS
- 12' = 6.98 CFS
- 11' = 6.68 CFS

Q = \frac{9.32}{0.06}
TYPE 6

2' 11 BARS @ 3/4"

\[(\text{SF} \times 1.5) - \left( 11 \times 2 \times 12^{9/3} + 2 \times (1.5) \times 12^{9/3} \right) - \frac{1}{2} \times (11 \times 12^{9/3}) \times 12^{9/3} \right] = 1.50 \text{ SF}

\[Q = C \times \frac{\text{SP}}{2} \]

COURT HEIGHT

12" = 6.88 CFS
11" = 7.01 CFS
10" = 7.91 CFS
9" = 6.34 CFS
8" = 5.34 CFS
7" = 4.72 CFS
6" = 3.58 CFS
5" = 2.79 CFS
4" = 2.23 CFS
TYPE 7

E. San Miguel
N. Wheaton

1.5'

\[ 2(1.5) - \left[ \frac{8(0.5)^{1/2}}{1.5} + \left( \frac{8(1.5)(0.5)}{1.5} \right) - \frac{8(0.5)(1.5)}{1.5} \right] \] = 1.6 SF

\[ Q = 1.6 \times 5.5 

1' = 8.67 CFS
1.5' = 6.91 CFS
2' = 5.50 CFS
2.5' = 4.66 CFS
3' = 4.17 CFS

10' = 5.90 CFS
8' = 5.40 CFS
6' = 4.90 CFS
4' = 4.40 CFS

<table>
<thead>
<tr>
<th>Q (CFS)</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
</tr>
</thead>
<tbody>
<tr>
<td>H (ft)</td>
<td>2F</td>
<td>AE</td>
<td>10F</td>
<td>11F</td>
<td>13F</td>
<td>12F</td>
<td>15F</td>
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</tbody>
</table>
**Type B**

**North Side**

Cache la Plance 11°

\[ \text{Q} = \text{CR} \sqrt{\frac{V}{a}} \]

\[ = 9.30 \text{ CF} \]

\[ c = 0.60 \text{ CFS} \]

\[ n = 8'' \]

\[ (25.5 \times \frac{1}{12}) - \left[ 25.5 \left( \frac{1}{12} \right) + \left( \frac{1}{12} \times \frac{1}{12} \right) - 2 \left( \frac{1}{12} \times \frac{1}{12} \times \frac{1}{12} \right) \right] \]

**South Side**

Cache la Plance 11°

\[ \text{Q} = \text{CA} \sqrt{\frac{V}{a}} \]

\[ = 9.51 \text{ CFS} \]

\[ c = 0.60 \text{ CFS} \]

\[ n = 8'' \text{ CFS} \]

\[ (28 \times \frac{1}{12}) - \left[ 28 \left( \frac{1}{12} \right) + \left( \frac{1}{12} \times \frac{1}{12} \right) - 2 \left( \frac{1}{12} \times \frac{1}{12} \times \frac{1}{12} \right) \right] \]
C.3 Existing Curb Opening Inlets

There are two types of curb opening inlets from which the capacities were calculated.

**Type 1:** Sump (sag) condition - Exists where flow from both directions enters the inlet. Under a true sump condition, the backing up of water at the inlet could result in significant flooding.

Curb opening inlets act as weirs in sag vertical curve locations up to a depth equal to the opening height. The curb opening height and length, and water depth at the curb affect inlet capacity. The curb opening inlets were field measured to determine the curb opening height, length, and curb height as the existing inlets in the study area vary in size.

The efficiency of an inlet is the percent of total flow that the inlet will intercept under a given set of conditions. The efficiency of an inlet changes with changes in cross slope, longitudinal slope, total gutter flow and to a lesser extent, pavement roughness.

The equation used for calculating the capacity of curb opening inlets in sag conditions in an orifice condition where the depth of water is greater than the curb opening is:

\[ Q = 0.67hL \sqrt{2g\phi} \]

- \( C \) = Orifice coefficient = 0.67
- \( h \) = Height of curb opening inlet, (ft)
- \( \phi \) = Effective head on the center of the orifice throat, (ft)
- \( L \) = Length of inlet

An example using this equation can be found on Sheet 5 of this section. Also on Sheet 5, a comparison of capacities of existing curb opening inlets and DIOR inlets is shown. The capacities of the inlets have been multiplied by 0.80 (80% efficiency) to allow for debris blocking the inlet opening.

**Type 2:** Continuous Grade - Exists where inlet has flow entering from the uphill side only. The slope on the uphill side governs as the momentum from the uphill side controls the flow entering the inlet. The efficiency of an inlet changes with changes in cross slope, longitudinal slope, total gutter flow, and to a lesser extent, pavement roughness.
The procedure used for the calculation of the inlet capacity is described below. The charts used are located at the end of this section.

Step 1: Each inlet was field measured and the following information reported:

a. Curb size on each side of inlet (to calculate inlet depression)
b. Total length of opening
c. Total curb height at opening
d. Inlet opening height

Step 2: The Q₅ and Q₁₀₀ flows were calculated using the hydrologic procedures of the SCS Method TR-55. The calculated flows are required to calculate the interception rate of the inlet.

Step 3: Calculate total width of spread of flow in gutter and street (see Chart 5).

Procedure:

a. On using Q₅ or Q₁₀₀, multiply by the average Manning's roughness "n" of .016
b. S = Longitudinal slope of street upstream of inlet
c. W = Width of pan, in all cases assume W=2'
d. Sₓ = Cross slope of street, in all cases assume Sₓ=.02

e. SW/Sₓ, SW = Slope of gutter measured from centerline of street, in all cases assume a 2" fall in the 2' gutter

SW = 2"/24" = .083
SW/Sₓ = .083/.02 = 4.18 in all cases

f. From this chart the value of T/W can be calculated by knowing that W is always equal to 2'. The value of T or total width of spread can now be calculated.

Step 4: Chart 5: By using the values of W/T and SW/Sₓ, the value of Eₓ (total percentage of flow in gutter) can be calculated.

By knowing this value, a composite value for the cross slope for the inlet can be calculated.

Sₓ = Sₓ+Sₓ(S'w + Eₒ)

Where:

Sₓ = Composite cross slope
Sₓ = Pavement cross slope = 3.0 percent
S'w = Depressin in inlet gutter-Field measured for each inlet S'w = a/w

Where:

a = Depression depth
w = Width of gutter equal to 2.0 feet
Eₒ = Percent of flow in gutter
Step 5: Chart 9: Using the values of longitudinal slope, $S$, composite cross slope, $S_d$, and value of $Q_5$ or $Q_{100}$, $L_f$ can be calculated. $L_f$ represents the total length needed to intercept 100% of $Q_5$ or $Q_{100}$.

Step 6: Chart 10, by using the value of actual length divided by the length needed for total interception ($L/L_f$) the efficiency of the inlet can be calculated using Chart 10.

Step 7: By knowing the efficiency and multiplying it by the actual $Q_5$ or $Q_{100}$, the flow intercepted by the inlet can be calculated.

If the flow intercepted is not satisfactory for drainage purposes in this study the intent is to replace the inlet with a Colorado Springs standard D10K inlet or construct an additional inlet upstream or downstream of the existing inlet if curb capacities are sufficient.

On Sheet 4 of this section, the procedure for inlet interception was compared to the D10K inlet capacity for the same size inlet. The calculations proved very similar results for inlet capacities on grade.
## EXISTING CURB OPENING INLETS

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<th>LOCATION</th>
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EXAMPLE: CURB OPENING INLET

COMPARISON OF CAPACITY OF D10 R (COLORADO SPRINGS)

1% SLOPE
6% LONG
8% CURB
8% CREST SLOPE

ASSUME STREET FLOWING FULL

4" DEPRESSION

\[ S_x = 0.88, \quad c_{ov} = 4.8 \]

\[ S_w = 0.83 + \frac{Q^2}{2} = 1.13 \]

\[ E_{ov} = 3.8 \]

\[ w/t = 2/16.67 = 0.11 \]

\[ T_c = 18.67 \]

\[ a^* = \frac{18.67}{16.67} = 1.14 \]

\[ E_0 = 0.36 \]

20% of flow is in gutter

TOTAL DISCHARGE \[ Q = \frac{Q_{ov}}{1 - E_0} \]

CHART 3 \[ Q_{ov} \]

\[ T_c = 16.67 \]

\[ S_x = 0.83 \]

\[ Q_{ov} = 20 \text{ CFS} \]

\[ Q = \frac{20 \text{ CFS}}{1 - 0.2} = 29.41 \text{ CFS} \]

\[ Q_w = 29.41 - 20 \text{ CFS} = 9.41 \text{ CFS} \]

CHART 9

FOR 100% INTERSECTION OF FLOW LENGTH OF OPENING

COMPOSITE \[ S_e = S_x + S_w (E_0) \]

\[ = 0.83 + (1.14)(0.08) = 0.98 \]

\[ Q = \]

\[ L_x = 35' \]

ACTUAL LENGTH: \[ 6/85 = 17.1\% \text{ EFFICIENT} \]

\[ 171 (29.41) = 5.04 \text{ CFS CAP} \]

FROM (CLOD. CAP) \[ 5.28 \text{ CFS CAP} \]
EXAMPLE: COMPARISON OF FEDERAL HIGHWAY ADMIN.
TO COLORADO SPRINGS CAPACITY OF CURB OPENING INLET IN SUMP CONDITION

LENGTH = 6'
CURB HT = 12''
SPREAD = 8''
Q = 0.67 * \sqrt{L} * \sqrt{A}

Q = 0.67 * (9/12) * \sqrt{82.5} * (9/12 * 8/12)
   = 17.56

17.56 (80) = 14.0

COMPAED TO 12.8 CFS COLORADO SPRINGS DWIR IN SUMP

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**DESIGN CALCULATIONS**

**NE Corner, Unihan Avenue - Weber**

\[ Q_{s} = 1.9 \text{ cfs} \]

- B' Long.
- E' Curbs
- 11' Opening
- Sufo = 1.2 %
- 4' Depressional Space

**Q Intermittent?**

\[ Q_{m} = (1.9 + 0.016) = 0.03 \text{ cfs} \]

\[ S_{w}/S_{y} = 9.8 \]

**Chart 5**

\[ i = 2.8 \quad T = 5.4 \]

**Chart 4**

\[ \frac{W}{T} = 2/5.4 = 0.357 \quad \text{cm} \]

\[ E_{o} \text{ (Chart 4)} = 0.8 \quad \text{8120 Flow in Gutter} \]

\[ S_{y} = S_{w} + S_{w}E_{o} = 0.3 + 0.8(0.81) = 0.10 \]

**Chart 9**

\[ L_{s} = 7.1 \]

\[ L/L_{s} = 13.7/7.7 = 1.69 \]

**Chart 10**

\[ E = 100 \quad 100\% \text{ Efficient} \]

\[ Q_{c} = 100 \times 1.9 = 19 \text{ cfs} \]

All Q5 will be picked up.
NE CORNER UNTAH & WEBER,

Q_{100} = 13.8 ft/s

B' LENGTH \( r = 0.22 \)
15'' CORR. \( S_{w} = 0.12 \)
11'' OPENING \( S_{w} = 0.04 \)
Slopes 1.2% \( a = 4' \)
W = 2'

\( Q_{\text{interaction}} \)

\( Q_{n} = 13.8 (0.48) = 6.62 \) ft/s

CHART 5 \( T/\|T\| = 0.7 \) \( T = 13.4' \)

\( W/T = 9/13.4 = 0.18 \)

CHART 4 \( E_{n} = 0.48 \) 48% of Flow in Gutter

\( S_{x} = 0.3 + (0.33)(0.48) = 0.11 \)

\( Q_{f} = E_{n} \)

CHART 9 \( L_{f} = 0.2 \)

\( L/L_{f} = 13/0.2 = 65.0 \)

CHART 10 \( \theta = 0.82 \) EFFICIENT

\( Q_{\theta} = 0.82 (13.8) = 11.32 \) ft/s
NE Corner of Weber and Yampa

0.5 = 0.8 CFS

0.5' 6" WIDTH
14" CURB
6" OPENING
6" DEPRESSION
slope = 0.4% 
S = \frac{0.005}{0.0004} = 1.25

Q interception = ?

Q_n = 0.8(0.05) = 0.04
S_{w} = \frac{0.23}{0.005} = 46

CHART 5
T = 23
T = 4.6

W/T = 2/4.6 = 0.44

CHART 4
E = 0.81
81% of flow in gutter

S_x = S_{w} + S_{w}E_x = \frac{0.23}{0.005} \cdot 0.81 = 4.6

CHART 9
L_1 = 2.5
L_1^2 = 2.5^2 = 6.25

CHART 10
L/L_1 = 1
E = 10 = 100%

Q_c = 1.0 \times 0.8 = 0.8 CFS

All flow will be intercepted
NE CORNER OF YAMADA + WEBER

\[ Q_{100} = 5.5 \text{ cfs} \]

\[ L = 14' \]

\[ B = 8' \]

\[ D = 6' \]

\[ S = 0.4 \]

\[ S_v = 0.3 \]

\[ S_w = 0.03 \]

\[ S_{av} = 3.8 \]

\[ Q_{inlet} = (5.5 \text{ cfs})(0.06) - 0.088 \]

\[ S_{av} = 3.8 \]

\[ T/10 = 5.8 \]

\[ T = 58.8 \]

\[ W/T = 9/58.8 = 0.15 \]

\[ E_0 = 0.48 \]

\[ E = 0.37 \]

\[ E_{inlet} = 0.37(5.5) = 2.04 \]

\[ Q_{inlet} = 2.04 \text{ cfs} \]

\[ Q_{100} = 4.1 \text{ cfs} \]

\[ Q_{critical} = 5.5 \]

\[ Q_{critical} = 5.5 \]

\[ Q_{critical} = 5.5 \]

\[ 4.1 \text{ cfs will be intercepted} \]

\[ 1.4 \text{ will flow into inlet} \]
$Q = 2.9 \text{ CFS}$

$Q_{i} = 0.9 \text{ CFS}$

$Q_{n} = 2.9 \times 0.10 = 0.29$

$Q = 2.9 \\ T = 8.8 \\ T/4 = 2/8.8 = 0.23$

$S_{0} = .03 + 0.16 \times 0.23 = 0.14$

$Q = 2.9$

$L_{T} = 7.5$

$L/L_{T} = 0.5/7.5 = 0.06$

$E = 0.95$

$Q_{i} = (2.9)(.50) = 1.45 \text{ CFS}$

$Q = 2.9 \text{ CFS}$

All of $Q_{i}$ will be interrupted.
NE CORNER WAKASATNA AND UNTAH

100 cfs = 4.9 cfs

12' WIDTH
15" CORB
11" OPENING
8" DEPRESSION

Sw = 0.03
Sw = 0.04 + 0.03 = 0.13
Sw/Sx = 3.8

Q interception
Qn = 4.9 (0.16) = 0.78
Sw/Sx = 22.3

CHART 5

T/W = 5.2
T/W = 8/10.4 = 0.77

CHART 4

E = 0.53
58% in gutter

Sx = Sw - 0.73 (0.53) = 0.20
Using Q = 4.9 cfs

L/T = 8'
L/LT = 12/8 = 1.5
E = 1.0
100% EFFICIENT

Q_i = 4.9 (1.0) = 4.9 cfs

ALL OF Q_i will be intercepted
**NE corner of Yampa + Wahsatch**

- \( Q_{100} = 2.6 \text{ cfs} \)
- Width = 2' (Double)
- 11" Curb
- 7" Opening
- 5' depression = a
- Slope = 0.6%

\[ Q_{interception} \]
\[ Q_{in} = 2.6 \times 0.016 = 0.04 \]
\[ Sw/s = 3.8 \]

**Chart 5**
\[ t/w = 4.0 \]
\[ T = 2(4.0) = 8.0 \]
\[ w/T = 2/8.0 = 0.25 \]

**Chart 4**
\[ E_0 = 0.65 \]
\[ 65\% \text{ of flow in curb} \]
\[ S_p = S_v + S_w(0.5) = 0.03 + (2.1)(0.65) = 0.17 \]

**Chart 9**
\[ L_T = 6.3 \]
\[ L/L_T = 2/6.3 = 0.32 \]

**Chart 10**
\[ E = 0.50 \]
\[ 50\% \text{ efficient} \]
\[ Q_{in} = 2.6(0.50) = 1.3 \text{ cfs} \]

Double
\[ 1.8(0) = 0.6 \text{ cfs} = Q_{100} \]

All of \( Q_{100} \) will be intercepted.
Qₜ = 1.6

Corner of Royer and Cache la Poudre

Slope = 1 1/2%
3'-0" Width
7" Curb
6" Opening
W = 2'

Q = 1.67

Qₚ = (1.67)0.116 = 0.03

CHART 5
T/Ø = 2.6
T = 2.6(2) = 5.2'
W/Ø = 2/5.2 = 0.38

CHART 4
E₀ = 0.83
33% of flow in gutter
Sₑ = Sₓ + Sw,E₀ = 0.34 + 0.83(0.83) = 0.10
Q = 1.6

CHART 9
L₁ = 10
L/L₁ = 3/10 = 0.30

CHART 10
E = 0.47
47% Efficient

Qᵢ = 0.47(1.6) = 0.75
Double Inlet Qᵢ = 0.75(2) = 1.5 CFS
01 CFS of Qₜ will pass Inlet
NE CORNER OF ROYER AND CACHE LA Poudre

Q_n = 4.8

Slab = 1.6%

Width = 5' 0"

Curb = 7'

Opening = 5' 0"

DOUBLE

\[ \begin{align*}
Sy &= 0.03 \\
Sw &= 2/2'-0.083+0.05 \times 0.03 \\
Sw/Sw &= 3.8
\end{align*} \]

\[ Q_n = 8.4 \times 0.16 = 0.13 \]

CHART 5

\[ \frac{T}{W} = 5.6 \]

\[ T = 5.6(2) = 11.2 \]

\[ \frac{W}{T} = 0.18 \]

CHART 4

\[ E_o = 0.49 \]

\[ E_o = 0.03 + 0.083 \times 0.49 = 0.07 \]

\[ Q = 8.4 \]

CHART 9

\[ L_i = 2.6' \]

\[ \frac{L}{L_i} = \frac{3}{2.6'} = 0.12 \]

\[ Q_i = 0.12(8.4) = 1.0 \text{ CFS} \]

DOUBLE INLET \[ Q_i = 1.0(2) = 2.0 \text{ CFS} \]

8.4 - 2.0 = 6.4 CFS WILL PASS BY INLET
DESIGN CALCULATIONS

Q\textsubscript{inlet} = 3.7

Q\textsubscript{outlet} = 2.7

3.0° width

7" covers

6" opening

Q\textsubscript{in} = 3.2 \times 0.06 = 0.19

CHART 5

T/\theta = 3.2

T = 3.0 \times 3.2 = 6.0

W/T = 0.2 = 0.3

CHART 4

E\textsubscript{0} = 0.80

Drainage Effort

E\textsubscript{0} = E\textsubscript{1} + E\textsubscript{2}

E\textsubscript{0} = 0.80 + 0.20 = 1.0

Q\textsubscript{in} = \frac{E\textsubscript{0}}{S\textsubscript{r}} = 3.2

CHART 9

L\textsubscript{1} = 15' FOR TOTAL INLET

CHART 10

L/L\textsubscript{1} = 3/15 = 0.20 EFFICIENT

Q\textsubscript{i} = 0.20 \times 3.2 = 0.64 CFS

3.2 - 0.6 = 2.6 CFS WILL PASS INLET
NW Corner, Uintah + Franklin

Q\text{in} = 1.2 \text{ ft}^3 \text{ per sec}

Q\text{in}= 1.8 \text{ ft}^3 \text{ per sec}

Q\text{in} = 0.016 \approx 0.03

\text{CHART 5} \quad \sqrt{w} = 2.8

\text{CHART 4} \quad T = 28 \text{ sec}

\text{CHART 9} \quad \frac{L}{L_t} = \frac{11}{2.8} = 3.9

\text{CHART 10} \quad E = 100 \quad 100\% \text{ efficient}

\text{All of Qs will be intercepted}
**DESIGN CALCULATIONS**

**HW GORNER 1/8 IN. 4FT FRANKLIN**

\[ Q_{o,0} = 0.7145 \]

- **11' Width**
- **15' O.C.C.**
- **10' Openings**
- **7' Depression**
- **Slope = 1.1%**

\[ Q_n = 0.016 (12.7) = 0.20 \]

**CHART 5**

\[ \frac{A_1}{A_2} = 5.5 \]

\[ T = 5.5(2) = 11.0 \]

\[ \frac{w}{T} = \frac{8}{11} = 0.72 \]

**CHART 4**

\[ E = 0.67 \]

\[ 67\% \text{ of flow in gutter} \]

\[ S_n = 0.3 + 29.2(0.67) = 0.23 \]

\[ V_{n,0} = 2.7 \]

**CHART 9**

\[ L_T = L' \text{ INLET FOR TOTAL INTERCEPTED} \]

\[ \frac{1}{L_T} = \frac{11}{11} = 1.0 \]

**CHART 10**

\[ E = 1.0 \quad 100\% \text{ EFFICIENT} \]

\[ Q_{inlet} \text{ will all be intercepted by inlet} \]
DESIGN CALCULATIONS

CURB OPENING INLETS (Locations, Sizes, and CAPACITIES)

NE CORNER - EL PASO & CACHE LA Poudre

3'-0" Width
7" CURB CAP = 2.1 CFS
6" OPENING
No DECREASE
80% Pickup

CAP = 6.16 CFS (80) = 4.9 CFS

NW CORNER - ROYER ST & CACHE LA Poudre

3'-0" Width
10" CURB
6" OPENING
2" DECREASE
4'-0" x 10" x 6" CURB = 8" CURB HEIGHT.

NE CORNER - ROYER ST & CACHE LA Poudre

3'-0" Width
7" CURB
6" OPENING
No DECREASE

CAP = 9.3 CFS (80) (2 INLETS)
= 14.9 CFS

NW CORNER - N CORNER & CACHE LA Poudre

5'-0" Width
10" CURB
6" OPENING
2" DECREASE

CAP = 10.0 CFS (80)
= 8.0 CFS

SW CORNER

3'-0" Width
10" CURB
6" OPENING

CAP = 6.2 CFS (80)
= 5.0 CFS
### Design Calculations

**NW Corner**

- 8.3" Width
- 11" Curb
- 7" Opening
- Sump
  - 2" Depression

**CAP = 8.1 CPS (0.80)**

**CAP = 6.2 CPS (0.80)**

**NE Corner**

- 6.6" Width
- 11" Curb
- 7" Opening
- Sump
  - 6" Depression

**CAP = 11.2 CPS (0.80)**
**Design Calculations**

**NW Corner**  
N Weber + E Yampa

\[
\text{Double } 2' \times 6' \text{ Width } 14'' \text{ Curb} \quad \text{Cap } 8.2 \text{ CFS (80)}
\]

8'' Opening  
6'' Depression  
= 6.6 CFS

**SW Corner**

\[
\text{0.36} \% 6' \text{ Width } 14'' \text{ Curb} \quad 8'' \text{ Opening} \quad 7'' \text{ Depression} \quad \text{Double } 13.2 \text{ CFS}
\]

**NE Corner**

\[
\text{0.5} \% 6' \text{ Width } 14'' \text{ Curb} \quad 8'' \text{ Opening} \quad 7'' \text{ Depression} \quad \text{Double } 13.2 \text{ CFS}
\]

**Center of North Island**

\[
\text{Cap } 12.3 \text{ CFS (80)} \quad \text{= 9.8 CFS}
\]

**SE Corner**

\[
\text{O.5} \% 3' \times 7' \text{ Width } 12'' \text{ Curb} \quad 8'' \text{ Opening} \quad 7'' \text{ Depression} \quad \text{Cap } 6.4 \text{ CFS (80)} \quad \text{= 5.3 CFS}
\]

**Center of North Island**  
N Wascana + S Wascana
**NE Corner**
- Sump
- Width: 12' 0"
- Curb: 12" Opening: 9" Depression:

**Corona + Uintah**
- CAF = 40.7 CFS (80)
- = 32.6 CFS

**NW Corner**
- Sump
- Width: 14' 0"
- Curb: 10" Opening: 4"

**SE Corner**
- Continuous
- Width: 11' 0"
- Curb: 8" Opening: 5"

**North Wasatch + E Uintah**

**NE Corner**
- Sump
- Width: 15' 0"
- Curb: 11" Opening: 8" Depression:

**Center + North Island**
- Width: 12' 0"
- Curb: 5" Opening: 9"

**CAF = 4.5 CFS (80)**
- = 3.6 CFS

**NW Corner**
- Sump
- Width: 18' 0"
- Curb: 11" Opening: ("

**CAF = 42.6 CFS (80)**
- = 42.1 CFS
**North Weber + E Uintah**

NE Corner:
- 12' Width
- 6" Curb
- 11" Opening
- 4" Depression

NW Corner:
- 12'6" Width
- 15" Curb
- 11" Opening
- 5" Depression

Cap = 12.64 CFS

N Wasatch + Sado Miguel:

Center of North Island:
- 8'6" Width
- 12" Curb
- 10" Opening
- 6" Depression

Cap = 8.6 CFS

= 6.9 CFS

North Wasatch + E Columbia:

Center of North Island:
- 8'6" Width
- 12" Curb
- 11" Opening
- 5" Depression

Cap = 6.6 CFS

= 5.3 CFS

Unitah + N. EC Paso:

NW Corner:
- 21' Width
- 14" Curb
- 9" Opening
- 4" Depression

Cap = 43.0 CFS

NE Corner:
- 12' Width
- 6" Curb
- 6" Opening
- 6" Depression

Cap = 51.0 CFS

= 40.8 CFS
N. Feakun & Uintah

NW Corner
Continuous
11.2' Width
15" Curb
10" Opening
7" Depression

NE Corner
6.0' Width
13" Curb
8" Opening
6" Depression

N. Prospect & Uintah

NW Corner
10.0' Width
12" Curb
8" Opening
4" Depression

NE Corner
TRIPLE 8' Width
12" Curb
8" Opening
4" Depression

Eas. of N. Prospect & Uintah

North Side
1.8' Width
1.8" Curb
2" Opening
2" Depression

South Side
6.8' Width
9" Curb
5" Opening
3" Depression

11.8' Width
11.2" Curb
7½" Opening
4" Depression

11.9 CFS
**N. ARCADIA + SANO MIGUEL**

**NORTH EAST OF INTERSECTION**

3' Width
15" Curb
10" Opening
4" Depression

**SOUTH EAST OF INTERSECTION**

12' Width
12" Curb
12" Opening
No Driveway

CAP = 9.8 CFS

**EAST COLUMBIA + NORTH WAYNE ST.**

Corner, North Side

8'6" Width
12" Curb
12" Opening
Sump

CAP = 6.6 CFS (80)

> 5.3 CFS

**W. WAYNE ST + CARRAMALLO**

**EAST SIDE ISLAND @ SOUTH INTERSECTION**

0' Tall Log
12" Curb
8" Opening
Sump

CAP = 8.0 CFS (80)

= 6.4 CFS

**NE Corner**

0' Tall Log
11/2" Curb
8" Opening
Corr

CAP = 7.1 CFS (80)

> 5.7 CFS
**DESIGN CALCULATIONS**

**CENTRE OF NORTH ISLAND**

8'-7" width  CFP = 7.2 CFS (80)  = 5.8 CFS

8'-7" width  12" curb
7" opening  sump

**NW CORNER**

8'-7" width  11" curb
7" opening  sump  CFP = 6.4 CFS (80)  = 5.1 CFS

**WANGARTH + DEL NORTE**

6'-6" width  12" curb
7" opening  sump

**NORTH ISLAND**

6'-6" width  12" curb
7" opening  sump  CFP = 6.6 CFS (80)  = 5.3 CFS

**NORTH MEXICO + N. WANGARTH**

8'-6" width  12" curb
10" opening  sump

**EAST BUENA VENTURA**

6'-6" width  12" curb
7" opening  sump  CFP = 8.6 CFS (80)  = 6.8 CFS

**DOVER C**

8'-6" width  sump
7" opening  12" curb  CFP = 8.8 (80)  > 7.0

2 inches (7.6)

= 14.0 CFS
CHART 4. Ratio of frontal flow to total gutter flow.
CHART 5. Flow in composite gutter sections.
\[ L_T = 0.6Q^{0.42}S^{0.3}(1/nS_x)^{0.6} \]

For composite cross slopes, use \( S_x \) for \( S_x \).

\[ S_x = S + S_wE_w \quad ; \quad S_w = 0/w \]

---

**CHART 9. Curb-opening and slotted drain inlet length for total interception.**

**EXAMPLE:**

**GIVEN:**
- \( n = 0.016 \)
- \( S = 0.01 \)
- \( S_x = 0.02 \)
- \( Q = 4 \text{ ft}^3/\text{s} \)

**FIND:**
- \( L_T = 3.4 \text{ ft} \)
CHART 10. Curb-opening and slotted drain inlet interception efficiency.

\[ E = 1 - (1 - \frac{L}{L_T})^{1.8} \]
Figure 21. Curb-opening inlets.
MADISON/WAHSATCH DRAINAGE IMPROVEMENT STUDY

APPENDIX D

SUBBASIN HYDROLOGY AND HYDRAULIC ROUTING ANALYSIS
SUB BASIN A

1) PUT IN CROSSFEED & GROUNDWATER DRAINAGE SUCH THAT ALL FLOWS ENTER OR EXIT THE FLOWS CONTINUOUSLY TO FOUNTAINE STREET.

2) ALL FLOWS EXIT ROUTE TO BE INTEGRATED INTO FUTURE STORM SEWER SYSTEM PER CITY OF PETROLIA DRAINAGE STUDY REPORT.

3) MAJOR INTERSECTIONS TO INSTALL BUFFER:
   1. MADISON/WASHINGTON
   2. WASHINGTON/MAIN STREET
   3. FOUNTAIN/WASHINGTON
   4. FOUNTAIN/FR. ROW
   5. FOUNTAIN/WACONIA
Q_5 \text{ CALC/Struct A}

1. INTRODUCTION OF MAISEY & WEISBACH

- FIND Q_5 FOR ENTIRE AREA DRAINING TO INTRODUCTION BLOCK 324, 326, 327, 425, 466.
- CHECK WABASH 24 & CURTIS CAPACITIES. 
  - BLOCK 45 FROM SINGLE BLOCK CANT
    \[ Q_5 = 0.6 \text{ cfs} \]

   BUT ONLY CAPACITY
   a) WABASH: 6% 0.97 \( \rightarrow \) 14.5 cfs
   b) CURTIS: 6% 0.3 \( \rightarrow \) 5.1 cfs

2. MAISEY & WEISBACH

   E. SIDE WABASH
   a) BLOCK 45 FOR FROM BLOCK 45
      \[
      t_4 = \frac{Q_4}{1.125 \cdot 1.05} = 0.010
      \]
   b) COUNTER BLOCK FOR 45 TO 46
      \[
      Q_5 = 0.87 \%
      \]

3. BLOCK 46 + 45

   a) FROM SINGLE BLOCK CANT
      \[
      t_{45} = \text{BLOCK} 45 = 0.610
      \]
   b) COUNTER BLOCK FOR 45 TO 46
      \[
      Q_5 = 0.87 \%
      \]
      \[
      t = \frac{104}{1.125 \cdot 1.05} = 0.610
      \]
      \[
      t_4 = t_4 + t_{45} = 0.610 + 0.63 = 0.473
      \]
   c) CANT 46
      \[
      (t_{46} + 0.6) = 0.473
      \]
**Design Calculations**

- **AREA** = 0.0046 + 0.0156 = 0.0222 sq m

- **WEIGHTS**
  - CN = 0.35 (75) + .60 (61) = 46

- **Q_5 Runoff** = 0.15 in (46%)

- **Q_5** = 0.15 in (0.0225 sq m)(0.625 in/100) = 2.16 in/2.92 ft

- **WATER SUPPLY**
  - **WASHING**

  1. **Bath 3BA, 1 SHOWER, 1 TUB**

     \[ Q_5 = 12.6 \text{ cfs} + 0.1 \text{ cfs} \text{ per person} + 0.1 \text{ cfs} \]

  - **CHECK CUBE CAPACITY**

    **WASHING** 6" 0.8% = 14.5 (10 CUBE CUBIC)

    1. All Q in STOUT.

    2. All water will drain to low spot o' floor.

    4. For row 1, eventually will remove all Q.

    3. Site tests for Q. Prior street turn check Q.100

- **INPUT BASED ON Q_5**

  **DIAGRAM**

  - **Q_5 = 3.0 \text{ cfs}**
  - **Q_5 = 1.8 \text{ cfs}**
  - **L2 4'**
  - **L3 6.5'**
  - **Q_5 = 0.1 \text{ cfs}**
2. INTERSECTION OR WASHINGTON & WASHINGTON

- FIND Q3 FOR AREA DRAWING TO INTERSECTION & ADD SUMMATION OF FLOW FROM INTERSECTION & MANHOLE & WASHBATCH.

a) CHECK UPSTREAM BLOCKED FOR CURB CAPACITY
   - EAST SIDE WASHBATCH

   i. Block 48 from single block curb

   \[ Q_3 = 3.2 \text{ cfs} \]

   ADD: 3.1 + 1.8 = 11.1 \text{ cfs} \text{ REST VALUE}

   CURB CAPACITY

   a) Washington: 8\' 6" O.D. = 19.3 \text{ cfs}
   b) Jefferson: 6\' 6" O.D. = 9.1 \text{ cfs}

b) WASHINGTON / WASHBATCH

   - EAST SIDE WASHBATCH

   i. Block 48 from single block curb

   \[ Q_3 = 3.5 \text{ cfs} \]

   ii. Block 48 + 42

   t = Block 48 = 0.150 \text{ hrs}

   CUTTER FROM 42 TO 48

   \[ S = 0.8\% \]

   \[ t = 1.8 \text{ ft} \]

   \[ t_x = \frac{665 \text{ ft}}{1.8} = 0.047 \]

   \[ t = t_x + t_{48} = 0.047 + 0.150 = 0.221 \text{ hrs} \]

   \[ (\text{SM/ft}) = 1050 \text{ ft/l} \]

   - AREA: 0.152 + 0.0071 = 0.159 \text{ ft}^2
DESIGN CALCULATIONS

Project: 
Detail: 
Designer: Date: Sheet 5 of 
Checker: Date: Job No: 

\[ \text{Wet size water supply} \]

1. Block 32 single block tanks

\[ Q = 2 \text{ ft} \times 2 \text{ ft} \times 0.67 \text{ ft} = 2.2 	ext{ ft}^3 \]

\[ Q_f = 2.2 \times 0.1 = 0.22 \text{ ft}^3/\text{min} \]

\[ Q_e = 0.22 \times 0.87 = 0.19 \text{ ft}^3/\text{min} \]

\[ Q_1 = Q_e + Q_f = 0.48 \text{ ft}^3/\text{min} \]

\[ Q_2 = Q_e + Q_f = 0.48 \text{ ft}^3/\text{min} \]

\[ \text{Check cubic capacity} \]

\[ 6'' \times 0.67'' = 14.5 \text{ cu ft} \text{ capacity} \]

\[ \text{Wet size water supply} \]

\[ \text{Block 32 single block tanks} \]

\[ Q = 2 \text{ ft} \times 2 \text{ ft} \times 0.67 \text{ ft} = 2.2 \text{ ft}^3 \]

\[ Q_f = 2.2 \times 0.1 = 0.22 \text{ ft}^3/\text{min} \]

\[ Q_e = 0.22 \times 0.87 = 0.19 \text{ ft}^3/\text{min} \]

\[ Q_1 = Q_e + Q_f = 0.48 \text{ ft}^3/\text{min} \]

\[ Q_2 = Q_e + Q_f = 0.48 \text{ ft}^3/\text{min} \]

\[ \text{Check cubic capacity} \]

\[ 6'' \times 0.67'' = 14.5 \text{ cu ft} \text{ capacity} \]
J. Fontaniero  &  Waheem

1) NE Corner Raintech
   NW & ARCA 1600 ft south Rhires 56 thru 59, 49 3 49 59.  PAA 7 79.  Q: PAIN
   INLET 2  WASH/RAIN.

   \( t_e \) BLOCK 56 = 0.172

   \( Q = \text{Cutter Flood} \text{ from 56 to 49} \)
   \( Q_{\text{ave}} = 0.8 \% \)
   \( N = 1.9 \text{ fps} \text{ (C16 4)} \)
   \( t = \frac{13.65 \text{ ft}}{1.9 \text{ (C16 4)}} = 0.725 \text{ in} \)
   \( t_e = t + t_r = 0.187 + 0.172 = 0.459 + 1 \text{ in} \)
   \( c_s/m^2 = 370 \)

   AREA
   \[
   \begin{array}{cc}
   \text{BLOCK} & \text{AREA} \\
   56 & 10639 \\
   55 & 1004.8 \\
   58 & 10043 \\
   59 & 10043 \\
   49 & 10043 \\
   \hline
   & 10254 \text{ sq ft}
   \end{array}
   \]

   WEIGHT CO CN
   \( 67 \% \text{ R14 - 80} \text{ G14 - 5} \)
   \( 33 \% \text{ R11 - 5} \text{ G14 - 1} \)

   \( c_n = 0.62 (25) + 0.35 (C1) = 7.0 \)

   \( Q_y \text{ RUNOFF} = 5.25 \text{ in} \text{ (1.36 m)} \)

   \( Q_x = 0.25 (0.015^2) (3.75) = 4.9 \text{ cfs} \)

   + 0.1 cfs from MEDIAN 20.1
   + Flow past INLET (WASH/RAIN) 3.0
   \( \text{Total} = 8.0 \text{ cfs} \)

   CHECK 23" 100% 3 100% > 8.0 0
b) Single Interaction of Extending & Concrete for Curb Capacity, Block 56 & 59

1. Block 59 from single block cards
   - Q = 0.4 cm
   - Water Capacity
     - 9" = 0.72 = 35.91 in = 0.4 cm

2. Block 56
   - t = 56 = 0.172 in
   - Gutter Flow for 56 = 0.59
   - S = 0.6 %
   - N = 1.5 ft
   - k = \frac{1.5}{(2.56 + 0.10 t)^2} = 0.251
   - f = 0.251 + 0.172 = 0.423
   - 35% = 600

<table>
<thead>
<tr>
<th>Area</th>
<th>Block</th>
<th>L/C cm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>56</td>
<td>0.037</td>
</tr>
<tr>
<td></td>
<td>57</td>
<td>0.040</td>
</tr>
<tr>
<td></td>
<td>58</td>
<td>0.042</td>
</tr>
<tr>
<td></td>
<td>59</td>
<td>0.044</td>
</tr>
</tbody>
</table>

- Weight (CN)
  - \text{Res} = 0.07 \text{ cm}
  - \text{Res} = 0.11 \text{ in}
  - CN = 0.24 (0.6) + 0.66 (0.11) = 0.32
  - Q = 0.32 in

- Capacity
  - 9" \text{ of water} \Rightarrow 37.5 \Rightarrow 4.5 \text{ ft}
b) NW corner - Contactor / Launder

\[ Q_f = Q_i + Q_{inlet} - Q_{outlet} \]

\[ Q_f = 4.2 \text{ cfs} \text{ for single switch only} \]

\[ + \text{current } 0.1 \text{ cfs} \]

\[ + \text{water usage } 6.6 - 6.0 = 0.6 \text{ cfs} \]

\[ 5.8 \text{ cfs} \]

- Enter flow capacity

\[ 0.2 \text{ cfs} \text{ at } 15.9 \text{ cfs} > 6.1 \text{ cfs} \]

- INLET pipe on \( Q_f \)

\[ Q_{in} = 6.9 \text{ cfs} \]

\[ Q_{out} = 6.1 \text{ cfs} \]

- DRAIN TO SOUTH

4. Flow into 1 stk row

- This is normal flow 10-15 cfs. Take out 60% of flow on 1 stk riser.

- Street flow past inlet. NW corner of street/lawn is split contributing flow

\[ Q_{o1} = 10.1 - 6.1 = 4.0 \text{ cfs} \]

\[ \text{ Use } 4'' \text{ lateral } : \text{ Q}_{o2} = 40 \text{ cfs} \]

\[ \text{ Q}_{o2} = 1.6 \text{ cfs} \]
5. INTERSECTION OR FOUNDATION IS, WISER
   a) WE CORNER ONLY CONTRIBUTING BLOCK 10.

   \[ Q_2 = 2.1 \text{ cfs} \]  \hspace{1cm} \text{(see single flow case)}
   
   \[ + \text{Curb flow at intersection} + 1.6 = 3.7 \text{ cfs} \]

   * CHECK CURB CAPACITY

   \[ 8'' \text{ curb at } 1.37\% \geq 33.4 \text{ cfs} > 3.7 \text{ cfs} \]
   
   * 60' WAST

   HOWEVER, THIS IS IN SOME CONDITION. WILL
   
   * BE INLET BASED ON Q100.
DESIGN CALCULATIONS

1. INTERSECTION 13401 S. LAUREL

**1st Qp Cals Q100 Will be Greatest Contributing**

**East Block of Hardscape (Block 4E Only)**

**The West Side of Hardscape (Block 5A)**

a) **East Side MAO/WALL** (SU SACS Sheet 3 & 4 for Revision)

   - **Q100 Runner = 1:9 in (Single Block Cals)**

   - **Qf0 = 1.3 in (0.0228 m) (62% 55%) = 194,000**

   - **Remove West Q**

     - **Qw = 10.0 CFS @ 600 ft = 4’**

     - **Qmax = 6.0 CFS max**

     - **Qet = 10.0 - 6.0 = 4.0 CFS**

     - **Check Curve Capacity**

     - 8” @ 0.8% = 14.5 CFS > 4.0 CFS

   b) **West Side MAO/WALL**

     - **Block 72A Rain Single Block Cals**

     - **Qmax = 20.9 CFS + 0.6 CFS each manhole = 21.5 CFS**

     - **Rinng West Q**

     - **Qet = 21.5 @ 300 ft = 2.0 CFS**

     - **Qmax = 21.5 - 2.0 = 19.5 CFS**

     - **Check Curve Capacity**

     - 8” @ 0.8% = 14.1 CFS > 13.5 CFS
2. INTERSECTION WASHABEN & WASHINGTON

See Prelim. Sale Sheet 5-6, Q=Q BACKUP + Qf, R Bell, Qf will be can. or lit when block #74 is
on right side & block 74 on west is conditioned as single lanes.

a) EAST SIDE WASHABEN

- $Q_{100}$ Rumble 2; 1.3010 (Fig 2; $P=25$)
- $Q_{10}=1.3010 \times (1.050 \text{ cpg}) (0.8416 \text{ ft}) = 20.3 \text{ epg}$
- $Q_{10} = Q_{100} \times \frac{0.6}{1.3010} = 14.6$

- REMOVE INLET Q

- $Q_{1}=24.8 \text{ gsd in.}$ $Q_{\text{volum}} = 7.6 \text{ man}$ $Q_{v}=4.6 \text{ cfs}$
- $Q_{V}=24.8 - 2.4 = 19.4 \text{ cfs}$
- Check curb capacity:

- $8'' \times 0.62 = 14.5 \text{ cfs} < 19.4 \text{ cfs}$ $Q_{\text{A}}$

b) INCREASE SIZE OF INLET & EAST SIDE WASHABEN

1) Add another inlet on SE corner 3; people

- NE corner: $Q_{1}=10.0 \text{ cfs}$ @ 106% $Q_{v}=6.00$
- $Q_{17} = 10.0 - 6.0 = 4.0 \text{ cfs}$
- LI corner: $Q_{17}=4.0 \text{ gsd in.}$ $Q_{\text{volum}} = 2.6 \text{ cfs}$ $Q_{v}=1.6 \text{ cfs}$
- $Q_{17} = 4.0 - 1.6 = 2.4 \text{ cfs}$

2) INCREASE SIZE INLET @ NE corner WASHABEN/LIN

- $Q_{1}=10.2 \times 0.6 \times 1.6 = 20.4 \text{ cfs}$
- Try DIO at 10'; $Q_{10} = 9.0 \text{ cfs}$
- $Q_{17} = 20.4 - 9.0 = 11.4 \text{ cfs}$
1. TAKE REMAINDER ON SE CURVE W/ INLET
   \[ Q_{11} = 13.4 \text{ cfs} \]
   \[ \frac{Q_{11}}{Q_{in}} = 0.87 \]
   \[ Q_{in} = 15.0 \text{ cfs} \]

   \[ Q_{11} = 13.4 - 8.6 = 5.4 \text{ cfs} \]

   **CHECK W/ SUMP TANK**
   \[ \theta = 0.86 = 14.5 \text{ cfs} > 5.4 \text{ cfs} \]

2. **WEST SIDE W/ INLET**
   **INLET NO. 32**
   \[ Q_{in} = 3.2 \text{ cfs} \]
   \[ + 0.2 \text{ cfs} \text{ (irrigation)} \]
   \[ - 0.2 \text{ cfs} \text{ (manc.)} \]
   \[ = 3.2 \text{ cfs} \]

   **AVERAGE INLET 8.0**
   \[ Q_{11} = 18.1 \text{ cfs} \]
   \[ \frac{Q_{11}}{Q_{in}} = 0.89 \]
   \[ Q_{in} = 7.4 \text{ cfs} \]

   \[ Q_{11} = 18.1 - 7.4 = 10.7 \text{ cfs} \]

   **CHECK W/ INLET**
   \[ \theta = 0.88 = 14.5 > 10.7 \text{ cfs} \]

3. **INTERSECTION 40FT W/ WASHBATCH**
   **NE CURVE (SEE MINIMUM SLOPE SHEET = 2%)**
   \[ Q_{10} = 1.0 \text{ in} \]
   \[ (0.5 \text{ cfs/in}) \]
   \[ Q_{10} = 0.5 \text{ cfs} \]

   **WASH/WARE INLET**
   \[ Q_{in} = 1.5 \text{ cfs} \]

**5.6 cfs**
DESIGN CALCULATIONS

Project:
Detail:
Designer:
Date:
Sheet:
Job No:

Remove weir Q1:

\[ Q_{12} = 25.6 \quad 4' \text{DIA} \times 8' = 0.7 \quad Q_{12} = 5.4 \, \text{ft}^3/\text{sec} \]

\[ Q_{11} = 25.6 - 7.4 = 18.2 \, \text{ft}^3/\text{sec} \]

core cap. = 29.3 ft^3

OK

Check flow & formula core cap.:

\[ Q_{100} \quad \text{runoff} = 1.13 \, \text{in} \]

\[ (216 + 8 \times 0.2 = 22) \]

\[ Q_{100} = 1.13 \, \text{in} \times (0.0432 \, \text{cm}) \times (800 \, \text{cm}/\text{sec}) = 16.0 \, \text{ft}^3/\text{sec} \]

Check core capacity:

\[ \text{Core area} = 0.42 \times 0.75 = 35.4 \, \text{ft}^3 > 16.0 \, \text{ft}^3/\text{sec} \]

Washout: 0.07 (29.7 ft^3) > 16.0 ft^3

b) New corner wash/foot:

\[ Q_{100} \quad \text{block} = 8.2 \, \text{ft}^3 + \text{median} = 0.6 \]

\[ 8.2 + 0.6 = 18.7 \]

\[ 22.7 \, \text{ft}^3 \]

1. Decrease Q2 to total:

\[ Q_{100} \text{ of } 22.5 \, \text{ft}^3 \]

Check = max cap of 25.6 ft^3

NE corner: limit & washout:

\[ Q_{11} = 25.6 \, \text{ft}^3 \] 4' DIA \times 8' = 0.7

\[ Q_{11} = 19.4 \, \text{ft}^3 \]

\[ Q_{11} = 25.6 - 19.4 = 10.2 \, \text{ft}^3/\text{sec} \]

NW corner: wash & wash:

\[ Q_{17} = 18.1 \, \text{ft}^3 \] 14' DIA \times 0.17 = 10.5 \, \text{ft}^3/\text{sec} \]

\[ Q_{17} = 18.1 - 10.5 = 7.6 \]
1. RECO UL CORNER LWA/ROOF

\[ Q_{	ext{ul}} = 8.2 \text{ L} \]

2.-Q_{	ext{ul}} = 27.3 \quad 22^\circ\text{dio elite} \quad 1.1% \quad Q_{\text{in}} = 14.9

\[ Q_{\text{uf}} = 27.3 - 14.9 = 12.4 \]

Check SWL for
5\% 1.17 = 22, 8 cfs \geq 12.4 \text{ cfs}

4. 24 ft. 2. AA ROW

Add in partial surge size at flow through inlet = 1.25

\[ Q_{\text{manip}} = 12.5 \text{ cfs} \]

Check flow capacity 5\% 1.17 = 22.1 cfs \geq 12.5 \text{ cfs}

\[ Q_{\text{st}} = 12.4 \text{ cfs} \]

\[ Q_{\text{uf}} = 4.4 \text{ cfs} \]

5. INTERSECTION CONTINUOUS & WILDER

a) NE CORNER BLOCK 10 ONLY

\[ Q_{\text{ul}} = 9.0 \text{ cfs} \]

\[ Q_{\text{uf}} = 5.0 \text{ cfs} \]

\[ 6^\circ 1.17 = 22.4 \text{ cfs} > 16.0 \text{ cfs} \]

\[ Q_{\text{uf}} = 16.0 \text{ cfs} \quad 8^\circ 0 \text{ cfs} \]

\[ Q_{\text{uf}} = 18.6 \text{ cfs} \quad \text{OK} \]
SUBRAIN B

1) DRAINAGE IS TO THE SOUTH AND WEST TO THE AR ROW & JACKSON.

2) EXIT CURB AND GUTTER IS A RAMP CURB THAT HAS BEEN EVALUATED WITH AC. AND THEREFORE ONLY 3" TO 5" DEEP FROM TOP OF GUTTER TO FE. OPTIONS DISCUSSED WITH CITY.

   INSTALL STD. 8" VERTICAL CURB AND GUTTER AND MAINLY EXIST PAYMENT GRADE. HOWEVER THIS WOULD IMPACT WATER BEHIND THE NEW CURB & WILL CAUSE DRAINAGE PROBLEMS TO DOWNSTREAM.

   LIMESTONE WHICH HAS HIGH CONSTRUCTION COST.

   INSTALL NON-UNIFORM STREET CURB:

   INSTALL GRADED INLET WHICH WILL REACH STORM SLUG PIPE COSTS AND DEVELOPMENTS WOULD INLET AT VERY LITTLE PRICE WILL BE ON TOP OF X-ACT TO DEVIL Q.

   IN DISCUSSION WITH THE CITY STAFF REVIEW OPTIONS 3 AND 4 OF NON-UNIFORM STORM INLET SYSTEM.

3) MAJOR INTERSECTIONS:

   - ALL INTERSECTIONS ALONG JACKSON
   - JACKSON & AR ROW
   - LARabee & CORONA
d) ALL STREETS ARE 28 FEET WIDE FROM CURB TO CURB.

WIDTH OF CURB = 7.2'

WIDTH OF GUTTER = 2.5'

CHECK INSTALLATION OR 8" "C.E.G"
SUCH THAT THERE WOULD BE ANWnde) BEHIND C.E.G.

IF INSTALL 8' C.E.G WOULD REQUIRE FOLLOWING CUM SLICE

**GUTTER DROP = 0.125’ FT**

**CROSS SLICE**

\[ \text{CROSS SLICE} \quad \text{RECOMM} = \frac{0.125}{0.5} = 0.25\% \]

**CHECK STD. RAMP C.E.G.**

**TYPE 2**

**SCALE 1”=1'-0”**

**DROP = 0.125’**

**STANDARD RAMP CURB AND GUTTER**

**CROSS SLICE = 0.25 - 0.125 = 0.125’**

**CROSS SLICE RECOMM:**

\[ \text{CROSS SLICE} \quad \text{RECOMM} = \frac{0.125}{0.5} = 2.5\% \quad 0.1’ \]

**USE SECTION AS FOLLOWS**

[Diagram of standard ramp curb and gutter with specific measurements and calculations.]
1. Lassen & Corona

a) North side of Ms. Currin (Block 26)
   \[ Q_p = 3.0 \text{ cm} \]
   (single block calc)

b) Check curb gap:
   \[ 2'' 	ext{ runoff} = 2.6 \text{ cm} \times 6.5 \text{ ft} \times 0.47\% \]
   \[ \text{ Check installing new uniform street section} \]
   \[ 	ext{Curb gap} = 6.1 \text{ cm} \times 6.5 \]
   \[ 6.0\% \]

2. Jackson & Rotor

a) Check rotor street curb equation (Block 21)
   \[ Q_p = 4.1 \text{ cm} \]
   (single block calc)

b) NE corner (Blocks 20A & 21)
   \[ t_{ch} = 0.833 \]
   (single green curb)
   \[ \text{Cutter flow } S_{ave} = 11.0\% \]
   \[ N = 2.0 \]
   (ICC 50)
   \[ 	ext{Cutter length} = 16.5 \text{ ft} + 0.50 + 7.00 = 11.53 \]
   \[ k_e = \frac{1133}{1260} = 0.897, \text{ HR} \]
\[ t = t_e + t_{num} = 1.0 \text{ sec} \]

\[ C_{MN}/N = 400 \quad \text{(Eq. 6)} \]

\[ \text{Area} \]

\[ \frac{\text{Block 1}}{\text{Area}} = \frac{0.0056}{0.0057} \]

\[ \text{Weighted CN} \]

\[ x_1 \text{ Return} + x_2 \text{ Ref} = 82 \text{ cn w/ 8} \]

\[ CN = 0.5(74) + 0.5(74) = 74 \]

\[ Q_e \text{ Return} = 0.04 \text{ in} \quad \text{(Eq. 7)} \]

\[ \frac{Q_e}{0.04} (4.0) (0.0107) = 3.4 \text{ in} \]

\[ Q_e \text{ from Block 21} = 4.1 \text{ in} > 3.4 \]

USE Block 21: Quadrant 2

\[ \text{Check cn w/ cn w/ Return} \]

\[ 6.02 \times 0.07 = 0.419 > 0.4 \text{ in} \]

\[ 4.1 \times 0.08 \]

\[ \text{Note: CN return from Block 20A does not control block in case above} \]

2. \[ t_e = 0.161 \text{ in} \quad \text{(Eq. 6)} \]

\[ t_e \text{ Return} = 0.47 \% \]

\[ N = 1.2 \times 11 \quad \text{(Eq. 6)} \]

\[ \text{Cutting limit} = 28 + 180 + 19 = 227 \]

\[ t_e = \frac{196}{12 \text{ (sec)}} = 0.048 \]

\[ t_e = t_e + t_{num} = 0.048 \text{ in} \]

\[ \text{CN} = 1.10 \quad \text{(Eq. 6)} \]
WRIGHTON CN 1/2 paH-tool = 2/6 leq = 5610.00 = 0.5
CN = 0.15 (74) + 0.25 (75) = 176
Qf = 0.45 (103.8) (110) = 15.2 CFS
CULC VIN 20 CAM
8°e 0.47 = 18.5 CFS > 15.2 OK

1. CULC CORONA STREET (ALO (11 232)
- EAT 16O DE CORONA
- Qf = 4.5 CFS
- 3° ANGLE 1 0.18% = 3.7 CFS < 4.5 CFS
- CULC NON UNIFORM SECTION
CUT (10 0.25 CFS) > 4.5 OK

2. WEST CORONA
- Qf FROM 4600 = 2.0 CFS
- CULC CORONA (2)
- 3° ANGLE 1 0.45 = 3.2 CFS < 3.8 OK
4. JACKWIN 1. BALK 2
   c) We Contra
   \[ t_{21} = 0.167 \]
   
   Current Flow
   \[ Q_c = 6.42 \text{ cfs} \]
   \[ N = 1.2 \text{ fls} \text{ (6% <)} \]
   Current Length = \[ 146 + 7'8" + 6'8" = 509.87 \]
   
   \[ t = \frac{509.87}{12 (76)} = 0.118 \text{ yr} \]
   
   \[ t_{e} = t_{c} + \frac{1}{e} = 0.167 \]
   
   \[ e \cdot t_{v} = 1000 \]
   
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   Weighted CN
   \[ \frac{1}{F} \cdot \frac{1}{F} \cdot \frac{1}{F} \cdot \frac{1}{F} \cdot \frac{1}{F} \cdot \frac{1}{F} \]
   \[ CN = 0.2 \left( \frac{4.0}{1} \right) + 0.2 \left( \frac{4.0}{1} \right) = 7.6 \]
   
   \[ Q_r = 0.67 \text{ cfs} \text{ (6%)} \]
   
   \[ Q_r = 0.67 \left( 0.0626 \right) (960) = 18.2 \text{ cfs} \]

   Check Current Capacity
   \[ 8'\ 6.42 \text{ cfs} \]

   \[ 4'\ 6.42 \text{ cfs} \]

   \[ 1'\ 6.42 \text{ cfs} \]

   \[ 0.64 \text{ cfs} \]
DESIGN CALCULATIONS

h) \( Q_{u} = 5.0 \) cfs
   \( C_{u} = \text{cuf} \)
   \( \text{check} \quad C_{u} \quad \text{cap} \)
   \( Q_{u} \text{ (uniform)} \leq 5.0 < 6.0 \) cfs

Based on \( Q_{u} \) cfs for above criterion, do \( Q_{u} \) cfs.

Substation B / \( Q_{u} \)

1. L Alvarez & Company
   a) earth side on R.C. column
      \( Q_{u} = 9.5 \) cfs (above 25)
      \( C_{u} = 7.1 \text{ (cuf)} \)
      \( C_{u} \text{ max - uniform celf} \)
      \( C_{u} \text{ cap} = 6.0 < 9.5 \) cfs

2. I. Alvarez & Company
   a) circuit: Ruby
      \( Q_{u} = 12.9 \) cfs (less than 21)
      \( C_{u} = 12.5 \text{ (cuf)} \)
      \( C_{u} \text{ cap} = 9.1 \text{ cfs} \leq 12.5 \) cfs
      \( C_{u} \text{ (uniform)} \)
      \( \text{Non uniform section} \)
      \( Q_{u} \text{ max} = 12.0 \text{ cfs} \leq 12.5 \) cfs
b) NE Corner

- $Q_{lw} = 12.4 \text{ ft}^3$ (August 21)
- Cure Car = 12 ft. (Note: 26)

2. Junc. & Corner

- $Q_{lw}$ Runoff = 1.38 in
  - $Q_{lw} = 1.38 (0.0708) (100) = 96.75 \text{ ft}^3$ (Note: 26)

- Pick up $Q_{lw}$ beginning @ Jackson & Royal
  - $Q_{lw} = 12.9 \text{ ft}^3$

- Put in 10' BOR which removes 60% (1)
  - $Q_{lw} = 12.9$
  - $Q_{lw} = 5.8 \text{ ft}^3$

- REDO to NE Jackson & Corner
  - $Q_{lw} = 0.120 \text{ ft}^3$

- Gutter Flow
  - $h_{av} = 0.6\%$
  - $f = 1.6 \div 1$
  - Gutter Loss = 26' + 12' = 38' = 960
  - $h_{av} = \frac{960}{1.6 (960)} = 0.148$

- $e = t_{30} + t_{70} = 0.318$
- $0.318 \text{ in.}$

- AREA
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DESIGN CALCULATIONS

Project: ____________________________ Date: __________ Sheet: __________

Detail: ____________________________ Date: __________ Sheet: __________

Designer: ____________________________ Date: __________ Sheet: __________

Checker: ____________________________ Date: __________ Sheet: __________

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1) Lining CN
   100% MPS - 50% gray B

   \[ CN = \frac{75}{Q_{100}} \]
   \[ Q_{100} = 1.75 \text{ in} \]
   \[ Q_{400} = 1.30 (0.016) (1000) = 21.4 \]
   \[ Q = \frac{Q_{400}}{12} = 1.8 \]

   \[ CN = 26.6 \]

   - C IRC 2 INLET
   - NW corner: install 20' pipe @ 0.67
     \[ Q_{in} = 12.2 \text{ cfs} \]
     \[ Q_{out} = 14.4 \text{ cfs} \]

   - NW corner: install 12' pipe @ 0.47
     \[ Q_{in} = 8.6 \text{ cfs} \]
     \[ Q_{out} = 5.8 \text{ cfs} \]

2) Corner
   \[ Q_{in} = 115 \text{ cfs} \]
   - Non-uniform section
     \[ CN < 8 < 9.5 \] 2 recommended section

4) Jackson & Paloma
   \[ Q_{in} = (8000 - 27) = 16.0 \text{ cfs} \]
   \[ Q_{out} = \text{max} + \text{Jackson Paloma} = 5.8 \text{ cfs} \]

   \[ 21.8 \text{ cfs} \]

   - install 2 INLET
5. Jacking & Warming
   a) North Wing
      \[ Q_{in} \text{ (60'x12')} = 17.4 \text{ cfs} \]
      \[ + \text{flow out of } C' = 3.9 \]
      \[ = 21.3 \text{ cfs} \]
      Install 20' DIAH \( \theta \) 0.47
      \[ Q_{in} = 21.3 \text{ cfs} \]
      \[ Q_{out} = 12.2 \text{ cfs} \]
      \[ Q_{30} = 9.1 \text{ cfs} \]

   b) NW Corner
      \[ Q_{in} \text{ (60'x30')} = 17.6 \text{ cfs} \]
      \[ + \text{flow out of } C' = 7.1 \text{ cfs} \]
      \[ = 24.7 \text{ cfs} \]
      Install 20' DIAH \( \theta \) 0.47
      \[ Q_{in} = 21.7 \text{ cfs} \]
      \[ Q_{out} = 12.2 \text{ cfs} \]
      \[ Q_{30} = 9.5 \text{ cfs} \]

6. Jacking & Pit Foul

   Site INLET SUMP FOR \( \theta \) DRAIN INLET & NW
   CONN. SUMP & \( \theta \) INLET. \( \theta \) = 4.5 \( 1.3 \)
   Install 6' DIAH
SUB BASIN C

a) DRAINAGE IS TO THE SOUTH & WEST TO NORTH SIDE OF ESPANOLA AT NARROWED RR TRUNK WHERE SOME CADDY OCCURS.

b) GUESS LOCATION NORTH OF INTERSECTION OR ESPANOLA & WAHSAHAN SHOULD BE ABROGATED & ADD GUESS IN ADJACENT RR ROW.

c) MAJOR INTERSECTIONS

- ESPANOLA & WAHSAHAN
- ESPANOLA & RR ROW
1. INTERCUT EXPANOLA & WATERTMCH

a) NE CORNER

- Block 50 x 60

- Flow: 60 = 0.89 ft

- Cutter Flow: 60 to 50

\[ S_{mc} = 1.17 \]

\[ N = 2.15 \text{ ft}^2 \quad (\text{Fig. 5}) \]

\[ L_{cut} = 10 + 60 = 70 \text{ ft} \]

\[ t_c = \frac{L_{cut}}{2} \frac{1}{(70)} = 0.062 \]

\[ t_r = 0.062 + 0.097 = 0.159 \text{ in} \]

\[ c_s = \frac{1150}{(36)} \quad (\text{Fig. 6}) \]

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</table>

- Waterfall CN 100% RS 50% CN group A

\[ CN = 0.1 \]

- Runoff: 0.08 in (Fig. 7)

\[ Q_r = 0.08 \times 0.0156 \times (1150) = 1.2 \text{ cfs} \]

- Check: 1.19% = 45.0 cfs > 1.2 cfs

b) MIDIAN (Block 34)

\[ Q_r = 0.1 \text{ cfs} \]

Q_f = 0.8 cfs

c) NW CORNER (Block 34)

\[ Q_r = 0.8 \text{ cfs} \]
check curb capacity
8" x 99% ≥ 15,400 ≥ 0.6
36" street width

2. Recommend based on O.C. (call) to install bollard, cross bar & collect all flow at
   announced row in surf w/ 5' curb, check curb
   prior to finalizing crossbar.
\( Q_{100} / \text{storage} \ C \)

1. Interaction of Siahono & Waulditch
   a) NC Corner
      - \( Q_{100} \) runoff = 0.57 (Elc. + C = 61)
      - \( Q_{100} = 0.57 \times (1150)(0.0126) = 8.2 \text{ CF} \)
      - therefore runoff capacity = 47.0 \text{ CF} > 8.2
   b) Median (Block 81)
      - \( Q_{100} = 0.24 \) (Single ocean cells)
   c) NW corner (Block 74)
      - \( Q_{100} = 2.5 \text{ CF} \) (Single ocean cells)
      - runoff cap. = 15.8 \text{ CF} > 2.5 \text{ CF}
   d) Size sup. infl. = 4' give a minimum row
      - \( Q_{100} = 7.6 \)
      - \( Q_{100} = 2.17 \text{ CF} \)

Check: Use existing outlet at eastern end until
and connect to 4' dike & NC Siahono

\( Q_{S5} = 0.0 \text{ CF} \) [7 of 6 above] \( Q_{S1} \)

\( Q_{S1} = 8.2 \text{ CF} \)

Install 4' dike until & NC converge

\( Q_{100} = 8.2 \text{ CF} \)

\( Q_{sum} = 12.8 \text{ CF} \)
a) Drainage is to south E W to existing inlet & washout & Carmillo and to a low point in gravel road with a 6% curve in Buena Ventura. Sump occurs in Buena Ventura. Exist inlet in Buena Ventura is a bubbler which project water across RR & RR to east quiter on washout. Narrow bubbler & confined exist inlet to storm sewer in RR & RR.

b) Exist storm sewer (18") begins & intersection of Del Norte & Washout existing south along washout. With additional inlet at Carmillo & Washout. There is a bubbler located at NE & SE corner of Del Norte & Washout. The bubbler in the NE corner will tie into existing SS in Washout. Median. The bubbler at SE corner will be modified.

c) Install curb, crossings at following places:
- Replace existing asphalt crossover west and parallel to Q of Washout at Del Norte.
- Parallel E East of Corona & at Del Norte to channelize sheet flow over pavement.
- Parallel N North of Carmillo at Corona to channelize sheet flow over pavement.

1) Major intersections:
- Carmillo & Washout
- Sump in Buena Ventura
Qs CALLS/SUBBASIN D.

1. INTERSECTION OF DEL NORTE AND WAHSAATCH

a) NE CORNER - ONLY CONTRIBUTING BLOCK S1
   - Qs = 0.8 CFS (PER SINGLE BLOCK CALLS)
   - CHECK CURB CAPACITY
     9" CURB 9 2.070 => 22.9 CFS > 0.8 CFS 330' WIDTH

   THE EXISTING GRADED INLET (USED AS A BUBBLER) WILL BE UTILIZED TO INTERCEPT THE 5-YR FLOW AND WILL THE INTO THE EXISTING 18" STORM SEWER LOCATED IN THE MEDIAN OF WAHSAATCH. THE BUBBLE (LOCATED AT THE SOUTHEAST CORNER OF THE INTERSECTION) WILL BE ABANDONED.

   CAPACITY OF GRADED INLET @ 9" VC = 634 CFS > 0.8 CFS

   ANOTHER ALTERNATIVE WOULD BE TO INSTALL A CROSS-DRAIN BEHIND 2 Q. OF WAHSAATCH ON EAST SIDE TO CARRY FLOW TO NEXT INTERSECTION.

b) MEDIAN @ WAHSAATCH AND DEL NORTE
   - Qs = 0 CFS

   USING CRITERIA SET UNTIL IN SINGLE BLOCK CALLS, THE 5-YR FLOW IS 0 CFS. MOST LIKELY THERE WILL BE A SMALL AMOUNT OF STREET INLET AROUND THE MEDIAN, BUT THE 9" CURB OPENING INLET WILL HANDLE ALL OF THE FLOW.

2. INTERSECTION OF CARRAMILLO AND WAHSAATCH

   AREA INCLUDES BLOCKS 52, 62, 61 AND 65
**DESIGN CALCULATIONS**

- \( t_{c1} \) Block 61 = 0.093
- Gutter Flow from 61 to 62 to 52
  
  \[ S_{ave} = 0.85\% \]
  
  \[ V = 1.8 \text{ fps} \]

**FIGURE 5**

- Gutter Length = 55' + 460' + 60' + 410' = 980'
- \[ t_c = \frac{980'}{(1.8 \text{ fps})(0.85\%)} = 0.151 \text{ HR} \]
  
  \[ t_c = t_r + t_{c1} = 0.151 + 0.093 = 0.244 \]

- \( cs_m/in = 645 \) (FIG 6)

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<td>.0081</td>
<td>.0279</td>
<td>SM</td>
</tr>
</tbody>
</table>

- Weighted CN = 100% Res - Soil Group A
- \( CN = 61 \)

- \( Q_s \) Runoff = 0.08 (FIG 7)
- \( Q_s = 0.08 \text{ in} \times (0.0877 \text{ SM}) \times (645 \text{ cs}_m/in) = 1.44 \text{ ft}^3 \)

- Check Curb Capacity

10' UC @ 0.5% = 765.1 (ft) > 14 (ft)

**existing inlet & corner, Z1 - Z10000**

- Curb Spacing = 5' opening
- 15' to thc curb

**THIS CORNER IS SUMP DUE TO FLOW COMING FROM**

**EXIST INLET WITH 15' NO. 6 CARRILO TO THIS POINT**
\[ Q_{\text{sump}} = 5.1 \text{ CFS} \quad \text{(PER INLET CAPACITY CALLS)} \]

\[ 5.1 \text{ CFS} > 1.4 \quad \text{OK} \]

* INLET WILL TAKE ALL Q_S

b) MEDIAN NORTH SIDE WASATCH AND DEL NORTE

AND MEDIAN SOUTH SIDE OF INTERSECTION

\[ Q_S = 0 \text{ CFS} \]

**USING CRITERIA SET FORTH IN SINGLE BLOCK CALL:**

THE 5-YR FLOOD IS 0 CFS. MOST LIKELY, THERE
WILL BE A SMALL AMOUNT OF STREET RUNOFF ACROSS
THE MEDIAN, BUT THE 2.7 INCH OPENING INLET WILL
HANDLE ALL OF THE FLOW

c) NW CORNER OF WASATCH AND CARRILLO

**BLOCK 35 Routed to Intersection**

\[ t_{c,35} = 0.077 \text{ HR} \]

**GUTTER FLOW FROM 35 TO INTERSECTION**

\[ S = 0.10 \%
\]

\[ v = 1.6 \text{ fps} \quad \text{(Figure 6)} \]

**GUTTER LENGTH = 50 + 420 = 470'**

\[ t_c = \frac{470}{1.6} = 0.085 \text{ HR} \]

\[ t_c = t_e + t_{c,35} = 0.085 + 0.077 = 0.162 \text{ HR} \]

\[ CSM/\text{in} = 1180 \quad \text{(Figure 6)} \]

\[ 2 \text{ REQ:} \quad \text{BLOCK 35} \quad \text{POCP:} \]

\[ 0.018 \text{ in} \]

\[ 0.018 \text{ in} \]
1. Weighted CN: 100% Res Soil Group A
   \[ CN = 61 \]

   - Qs Runoff = 0.08
     \[ (\text{from Figure 7}) \]
   - Qs = 0.08 in \( (0.0018 \times 1160) = 0.2 \text{ CFS} \)

   - Check curb capacity

   \[ 8" \text{ VC @ 0.67} \% \rightarrow 12.5 \text{ CFS} > 0.2 \text{ OK} \]

   ** Existing Inlet on Warehouse: 6.7" Long 
   7" opening 
   11" to top curb 

   This inlet in under full condition

   \[ Q_{\text{inlet}} = 5.1 \text{ CFS} \text{ capacity} > 0.2 \text{ OK} \]

   ** E. Bueno Ventura 

   ** Existing Bubler, overs water across RR Road 
   in east corner of warehouse 

   - Qs = 0.5 cfs (per single block wall)

   - Check curb capacity

   \[ 8" \text{ curb @ 1.5} \% = 35.89 \text{ CFS} > 0.5 \text{ CFS} \]

   The existing curb inlet (used as a bubble) 
   will be used to tie the 5-yr flood from 
   Bueno Ventura into the proposed storm 
   sewer located in the railroad right-of-way.

   - Check inlet capacity

   \[ (2) \]

   \[ 8" \text{ Long} \]

   \[ 11" \text{ to top curb} \]

   \[ Q_{\text{inlet}} = 7.0 \text{ (2.3 cfs of water)} \]

   \[ = 14.0 (2.3) > 0 \]
1. INTERSECTION OF DEL NORTE AND WASHKATI
   a) NE CORNER - ONLY CONTRIBUTION BLOCK 51
      \[ Q_{100} = 6.0 \text{ CFS} \]
      \[ \text{CHECK CURB CAPACITY} \]
      \[ 9'' \text{ CURB @ 20\% = 27.9 > 6.0 OK} \]
      \[ 30' \text{ WIDTH} \]
      \[ \text{USE EXISTING BUBBLER AS INLET TO TIE INTO EXISTING STORM SEWER} \]
      \[ \text{CAPACITY @ 9'' VC = 6.34 > 6.0 CFS OK} \]
   b) MEDIAN
      \[ 0 \text{ CFS FLOW (SEE SHEET 2)} \]

2. INTERSECTION OF CARA\text{MILLO} AND WASHKATI
   Blocks: 59, 62, 61, 60, 65
   \[ Q_{100} \text{ RUNOFF} = 0.57 \quad (\text{FIG. 7 G (N=61))} \]
   \[ Q_{100} = 0.57 \times (0.015 \text{ CFS/IN}) (0.2) = 10.26 \text{ CFS} \]
   \[ \text{CHECK CURB CAPACITY} \]
   \[ = 25.1 > 10.3 \text{ CFS OK} \]
   \[ \text{W' VC @ 0.5'} \]
   \[ = 10.3 \text{ CFS OK} \]
   \[ \text{EXIST INLET & CURB IN SUMP} \]
   \[ \text{CAPACITY = 5.1 CFS < 10.3 \text{ NOT SUFFICIENT CAPACITY WITH EXIST INLET}} \]
   \[ \text{THEREFORE, REMOVE EXIST INLET AND INSTALL A} \]
NOW - 60 FOOT D I O R I N L E T - THIS NEW INLET WILL BE IN THE EXISTING STORM SEWER LOCATED IN MEDIUM OF WAHATCH ONLY. IF THE CAPACITY OF THIS STORM SEWER WILL NOT BE EXCEEDED

OTHERWISE THE INLET WILL DRAIN TO THE PROPOSED STORM SEWER IN THE ABANDONED RAILROAD ROW.

6' DIOR CAPACITY (IN SUMP) = 12.8 CFS > 10.3 CFS

ALL Q100 WILL BE HANDLED BY INLET

b) M E D I A N - NORTHERN AND SOUTHERN SIDES WAHATCH

Q100 = 0  SEE SHEET 4

c) N W CORNER OF WAHATCH AND CARAMELO

- Block 35 Routes to Intersection
- Q100 Runoff = 0.57 (Figure 7)
- Q100' = 0.57 (.0018)(1100) = 1.2 CFS
- Check for Catch Basin

8" VC & 0.69, => 12.5 CFS > 1.2 CFS OK
30' STREET WIDTH

- Capacity of Existing Inlet = 5.1 CFS > 1.2 OK

- 8 E BUENA VENTURA

Q100 (50) = 3.4 CFS (PER STREET FORM 11258)
- CHECK CURE CAPACITY

8" curb @ 1.5% = 36.89 cfs > 3.4 cfs

- CHECK INLET CAPACITY

> 140 cfs > 3.4 cfs

OK

THE 2 EXISTING Curb Catching Inlets #1 & #2 BUBBA VENTORA will be utilized to tie the 5-yr and 100-yr flows into the Proposed Storm Sewer located in the Railroad Ruts.
**Design Calculations**

**Basin D**

- **Exit Inlet 0.6**
  - **Q100 = 6.1 cfs**
  - **Q110 = 1.2 cfs**

**Del Norte**

- **Q48 = 11.1 cfs**
  - Connect to **Proposed SD**

**Caramillo**

- **Q250 = 3.4 cfs**
  - Connect to **Proposed SD**

**Note:** Cross pans will be installed at the intersection of Caramillo and Caramillo, parallel and north of Caramello, and at the intersection of Del Norte and Caramillo, parallel and east of Caramillo.
SUBSYSTEM E

1) DRAINAGE IS TO THE SOUTH AND WEST TO THE RR ROW ON CORONA. DRAINAGE ON THE EAST FLOWLINE ON CORONA ROWS INTO THE 6" ROW. DRAIN ON THE WEST SIDE ENTIRE AU 18" DIAMETER STORM SWARM AND POKES UNDER THE OLD RR TRACK BED AND CONTINUES DOWN CORONA.

2) INTERCEPT OR AT THE RR ROW AND CORONA

3) MAJOR INTERCEPTORS
   - CORONA & RR ROW
1. Corona & RR ROW
   a) NW Corner
      (Block 5A)

      \[ t_e = 0.081 \text{ ft} \] (Single block case)

      GUTTER FLOW
      \[ \frac{S_{gw}}{N} = 1.2 \% \]
      \[ N = 2.2 \text{ ft}^2 \]
      \[ \text{Gutter length} = 480 + 50 = 530 \]
      \[ t_e = \frac{530}{2.2 (1600)} = 0.067 \]

      \[ t_e + t_e + t_e = 0.146 \]

      CE [\text{in} / \text{ft}] = 1080 (Sec. C)

      " Area = Block 5A = 0.021 \text{ in} "

      WEIGHTED CN
      10-7 RES - Silt Group A

      CN = 61

      \[ Q_e, \text{ Runner} = 0.05 \text{ in} \] (Sec. 7)

      \[ Q_e = 0.05 (1080) (1021) = 0.12 \text{ cm}^3 \]

      CHECK LOAD CAP.
      \[ B' = 1.2 \] in. STREET LENGTH

      \[ Q_{jump} = 0.2 \text{ cm}^3 \]
b) SC CONVER. (CASES 6, 64, 86)

\[ t_{6.6} = 0.133 \text{ HR} \]

**Gutter Flow**

\[ q_{gw} = 0.6 \text{ CF} \]

**U** = 1.55 CF/HR (CASE 6)

**Gutter Length**:

\[ L = 56 + 24 + 36 + 100 = 530 \]

\[ t_e = \frac{530}{1.55 \text{ CF/HR}} = 0.102 \]

\[ t_e = t_e + t_{6.6} = 0.235 \text{ HR} \]

**CON/N** = 1005 (CASE 6)

**AREA**

<table>
<thead>
<tr>
<th>Blk C</th>
<th>AREA</th>
</tr>
</thead>
<tbody>
<tr>
<td>67</td>
<td>0.033</td>
</tr>
<tr>
<td>64</td>
<td>0.0069</td>
</tr>
<tr>
<td>66</td>
<td>0.0139</td>
</tr>
<tr>
<td></td>
<td>0.0283 bfn</td>
</tr>
</tbody>
</table>

**WEIGHT CN** 100% RLD - Silt Group A

\[ CN = 61 \]

\[ Q_f \text{ Runoff} = 0.08 \text{ CF/HR} \]

\[ Q_f = 0.06 \text{ (1005)} \cdot 0.0283 = 2.3 \text{ CF/HR} \]

**Check Curb Cap.**

\[ 8''e 0.6 = 22.6 \text{ CF} > 2.3 \text{ CF/HR} \]

**Install 4' 0.102**

\[ Q_{300/m} = 7.7 \text{ CF} > 2.3 \text{ CF/HR} \]
\[ Q_{100} / \text{Subarea E} \]

1. Corner A, RR Row
   a) NW corner
      \[ Q_{100} = 1.6 \text{ cfs} \]
      \[ Curb Cap = 22.1 \text{ cfs} > 1.6 \]
      Install 4' DIOR
      \[ Q_{sump} = 7.9 > 1.6 \text{ OK} \]
   b) SE corner
      \[ Q_{100} = 0.92 \text{ in} \]
      \[ Q_{100u} = 0.92 \times (1.04) \times (0.0293) = 16.2 \text{ cfs} \]
      \[ Curb Cap = 22.1 > 16.2 \text{ OK} \]
      Install 8' DIOR
      \[ Q_{sump} = 18.4 \text{ cfs} > 16.2 \]
SUBRAIN E

a) DRAINAGE IS TO THE SOUTH TO UTAH. AND EAST TO SHOON RUN, THERE IS ONE STORM SEWER IN UTAH
b) BUNKIES LOCATED ALONG WEBER
   -fontana & webber should be redesigned
   -columbia & webber install crosspan
   -san miguel & webber install crosspan
c) BUNKIES LOCATED ALONG Wycliffe
   -parallel & run off wycliffe & at columbia, intent is to have these flows collected in storm part of utah. do not break this drainage pattern. a & b install conc. cross pans
   -parallel & run off wycliffe & at san miguel, intent is to have this drainage be collected at point at utah. c install cross pan as above

d) BUNKIES LOCATED ALONG FRANKLIN
   -franklin & san miguel install conc. cross pans
e) MAJOR INTERSECTION

- Uintah & Utah
- Uintah & LaVeta
- Uintah & Corona
- Uintah and El Paso
- Uintah and Franklin
- Uintah and Prospect
- Uintah and Shucks Run
1. INTERSECTION UINTAH & WEBER,

AREA INCLUDES BLOCK 1A & 6A, NOT 11A OR 16. NOTE ALL GEOS.
A SEWN BLOCK 1B WILL BE CONNECTED IN LAYER 2. NC

Figures 6A, FOUNTAIN & WEBER.

a) NW CORNER UINTAH & WEBER,

\[ t_{e1} = 0.109 \]

GUTTER FLOW FROM 1 TO 6

\[ Q_{ave} = 0.847 \](Fig. 5)

\[ N = 1.8 \]

GUTTER LENGTH:

\[ 440 + 60 + 440 + 55 + 470 + 50 + 920 = 275 \]

\[ + 30 + 610 + 50 + 450 \]

\[ t_e = \frac{275}{1.8 \times 360} = 0.159 \text{ in.} \]

\[ t_c = t_e + t_{c1} = 0.109 + 0.159 = 0.268 \text{ in.} \]

\[ c_m = 0.635 \](Fig. 6)
**DESIGN CALCULATIONS**

- **Block** | **Area**
  - 1 | 0.842
  - 2 | 1.008
  - 3 | 1.008
  - 4 | 1.010
  - 5 | 1.010
  - 6 | 1.008
  - 0 | 1.055

- Weighted CN 100.7% AC5 - see group A

  CN = 4.1

  - Q2 = 0.08 in (0.0 in)
  - Q3 = 0.08 in (1.055 in) 
  - (675 cfs) = 2.8 cfs

- Check curve capacity

  10°F 1.2% ➔ 65.9 cfs > 2.8 cfs OK.

  60°F 1.2%

- Existing inlet:

  - E corner 11 feet length

  - Existing inlet 15" to 15"

  - This corner is sump due to high points

  - Controlling 20, 26.08, 9, & 8

  - Q_1 = 40.3 cfs (plus 10 cfs & 20 cfs)

  - INLET WILL TAKES ALL Q5

  - NE corner vented & WEBER

  1. t max = 0.093 hrs

  2. Cut-in Flow from II to I

  - Q AV = 0.2 cfs
  - n = 1.8 (SC & T)

  - Cut-in Location = 35.5 ft

  - t, = 0.549
\[
\begin{align*}
7_c &= 4_c + 4_n = 0.548 + 0.072 = 0.622 \\
\text{corr}_n &= 6.50 \\
\text{E AREA} &\begin{array}{c|c}
\text{Block} & \text{AREA} \\
11 & 0.0041 \\
12 & 0.0044 \\
13 & 0.0046 \\
14 & 0.0037 \\
15 & 0.0075 \\
16 & 0.0056 \\
17 & 0.0028 \text{ cu}
\end{array}
\end{align*}
\]

- WEIGHTED CN 100% RES. - SOIL GROUP A
CN = 61

- \[Q_3 \text{ RUNOFF} = 0.071 \text{ in} \text{ (see fig 7)}\]
- \[Q_5 = 0.08 (0.56) (0.0772) = 1.9 \text{ cfs}\]
- CHECK CUB EAS

\[10' \text{ UC} \times 1.1% \Rightarrow 66.9 \text{ cfs} > 1.9 \text{ cfs}\]

- 60' START WITH
- NOTE: THAT @ INLET TUB, IS NOT A TEST COLUMN, SEE INLET END, TUBS, WILL TAKE ALL Q_5

-
2. INTERGRATION OF URANIA & WAWATAN

2) NW CORNER (BLOWS 14A TO 16A)

- $t_c$ for block 14A = 0.155 hr
- GUTTER FLOW 14A to 16A

$S_{900} = 0.75\%$

$v_0 = 1.75$ fps

GUTTER LENGTH = 202.630 + 50 + 450 = 1160 ft

$t_e = \frac{1160}{1.75(3600)} = 0.184$ hr

$t_c = l_c + t_{14A} = 0.339$ hr

$C_{500/1000} = 880$ (Fig 6)

<table>
<thead>
<tr>
<th>AREA</th>
<th>BLOCK</th>
</tr>
</thead>
<tbody>
<tr>
<td>16A</td>
<td>0.025</td>
</tr>
<tr>
<td>15A</td>
<td>0.005</td>
</tr>
<tr>
<td>14A</td>
<td>0.043</td>
</tr>
</tbody>
</table>

- WEIGHTED CN 100% RES = 50% Glacial A

CN = 1.1

- QF (RUNOFF) = 0.08 in. (810, 7)

- $Q_f = 0.08 (580)(0.016) = 1.2$ cfs

- No QF flow from Glacial A - see blow in calculations

- CHECK COND. EXP.

10' C 0.1% => 12.5 cfs > 1.2 cfs 70' spacing with

- CHECK INLET CAPACITY: L&I' & SUPP 1100 =

$Q_{1100} = 42.1$ cfs > 1.2 cfs

(PER UNIT 32 CFS 1100)
b) Median

\[ Q_a = 0 \text{ cfs} \] at all sections.

Using criteria, cut depth in single block calc. The \[ Q_a \] is 0 cfs, must limit that will be 0 cfs. Nunlee street flow across median butt 2-6" curb opening multiplies at each median, will handle all flow.

c) NE curve (Blanks 88, 89, 90)

- \[ t_c \] blank 88 = 0.153 hrs.

- Cutoff flow (see NW corner calc.)
  \[ t_c = 0.184 \text{ hrs} \]

- \[ t_c = t_c + t_{88} = 0.437 \text{ hrs} \]

- \[ E5\%N = 790 \] (Fig 6)

- E Area

<table>
<thead>
<tr>
<th>Block</th>
<th>Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>88</td>
<td>0.003</td>
</tr>
<tr>
<td>85</td>
<td>0.0056</td>
</tr>
<tr>
<td>90</td>
<td>0.002</td>
</tr>
<tr>
<td></td>
<td>0.016 S</td>
</tr>
</tbody>
</table>

- Weighted CN 100% R2 - Soil Category A
  \[ E = 6.1 \]

- \[ A_r = 0.08 \] (Fig 7)

- \[ Q_a = 0.08 (0.0109) (200) = 0.7 \text{ cfs} \]

- Elevation curb gap. \[ \theta = 0.6\% \] = 12.5 ft, 70' street 6 ft

- Check street

  \[ Q_r = 0.6\% \] all flow handled by inlet, SUV

  - Check street flow continues
3. INTERIOR UNDRAINED COLLOMNA
   3a) NW corner (blocks 89A & 90A)
      \( t_c = \text{block} \ 89A = 0.147 \ \text{ft} \) (small block case)

   - Cutoff Len
     \( L_{cw} = 1.2 \ \text{ft} \)
     \( N = 2.2 \ \text{fps} \) (C1c 5)
     \( L_{cw} = 50 + 450 = 500 \ \text{ft} \)
     \( t_c = \frac{500}{(1.2)(500)} = 0.063 \)

   - \( t_c = t_e + t_{ema} = 0.210 \ \text{ft} \)

   - CEM/\( \% = 1050 \)
   - CEM/\( \% = 1050 \)

   - Area
     - Block
       \( 89A \) 0.0055
       \( 90A \) 0.0051
     - Area

   - Weighted CN 100% Rel - SW 100\% A
     \( CN = 61 \)

   - \( Q = \text{Runoff} = 0.04 \ \text{in} \) (C1c 4)
   - \( Q = 0.08 (0.006) (100) = 0.9 \ \text{cu ft} \)
- Check curb capacity:
  \[ q_c = 1.2 \frac{ft^2}{hr} = 42.0 \frac{cu ft}{cu hr} \]
  (60' street width)

- Check exist inlet/sump capacity: \( L = 12' \)
  \[ Q_{sump} = 33.1 \frac{cu ft}{cu hr} \]
  (For 12' inlet/sump case)

b) NE corner

- Block 95: \( 0.059 \frac{cu hr}{cu ft} \)
  (Single好人 case)

- Gutter flow:
  (some as new corner cases)
  \( t_e = 0.067 \frac{hr}{cu ft} \)

\[ t_e = t_e + t_{qf} = 0.122 \frac{hr}{cu ft} \]

- \( C/I 
  \]

- AREA:
  \[
  \begin{align*}
  & \text{B/S} \quad 95 \quad 96 \quad .0023 \quad \text{area} \\
  \end{align*}
  \]
  \( 0108 \text{ CM} \)

- Weighted CN:
  \( 100.7 \text{ MS} - \text{Snow Growth A} \)

- CN = 61

- \( Q_s \text{ runoff} = 0.08 \frac{hr}{cu ft} \)
  (FIG 7)

- \( Q_{cf} = 0.05 \text{ (ILL)} \times 0.708 = 0.05 \frac{hr}{cu ft} \)

- Check curb capacity:
  \[ q_c = 1.2 \frac{ft^2}{hr} = 47.4 \frac{cu ft}{cu hr} \]
  (60' street width)

- Check exist inlet/sump capacity: \( L = 12' \)
  \[ Q_{sump} = 32.6 \frac{cu ft}{cu hr} \]
  (For 12' inlet/sump case)

- All \( Q_s \) handled on inlet.
4. INTERSECTION OF UNIVIAH AND EL PASO

A) NW CORNER

(BLOCKS 70, 67, AND 69)

A check must be made to check for greatest flow

\[ t_{67} = 0.143 \]

GORE FLOW

\[ \text{SAV } = 1.45\% \]

\[ V = 0.4 \text{ fps} \]

\[ F = 5 \]

\[ \text{GUTTER LENGTH } = 20' + 100' + 50' + 50' = 220' \]

\[ t_c = \frac{0.52}{0.4(220)} = 0.095 \text{ HR} \]

\[ t_c = t_{67} + t_c = 0.143 + 0.095 = 0.238 \]

\[ 0.5m/\text{in} = 1000 \text{ for } k \]

\[ \text{AREA } = \text{ BLOCK AREA (sq m)} \]

<table>
<thead>
<tr>
<th></th>
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<th>70</th>
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<tbody>
<tr>
<td>AREA</td>
<td>0.0062</td>
<td>0.0075</td>
<td>0.0051</td>
</tr>
<tr>
<td>AREA</td>
<td>0.0198 sq m</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\[ \text{WEIGHTED CN } = \frac{1}{3} \text{ COMM SOIL GROUP A} \]

\[ \frac{1}{3}(89) + \frac{2}{3}(67) = 70 \]

\[ Q_5 \text{ RUNOFF } = 0.246 \]
Design Calculations

\[ Q_5 = 0.26 \times 0.0198 \times 1000 = 5.1 \text{ cfs} \]

**Comparison of** \( Q_5 \) **based on travel time of block 67 (above cells) and the travel time of block 69 (below)**

\[ t_c = 0.12 \text{ hr} \]

**Gutter Flow**

\[ S_{NE} = 0.019 \]

\[ V = 2.7 \text{ fps} \]

**Gutter Length**

\[ = 500 + 450 = 500 \]

\[ t_e = 0.051 \text{ hr} \]

\[ t_c + t_e = 0.172 + 0.051 = 0.223 \text{ hr} \]

**CSM/A**

\[ = 1150 \text{ ft}^2 \]

\[ 2 \text{ Area} = 0.098 \text{ cm} \]

**Wetted CN**

\[ = 70 \]

**Q < runoff = 0.26**

\[ Q_5 = 0.26 \times 0.0198 \times 1000 = 5.1 \text{ cfs} \]

**Check Curb Capacity**

8" VC @ 1.97% = 28.2 > 5.9 OK

**Check Existing Inlet in curb condition**

= 34.4 > 5.9 OK

1) NE Corner (Blocks 72A)

\[ Q_5 = 0.0 \text{ cfs} \] (Single block cells)

**Curb Capacity**

0.4% 60' wide 8" VC = 7.1 > 0.3 cfs

**Inlet Capacity**

(INLET Curb 4/4 foot) = 4.0 m/min > 0.3 cfs

OK
5. INTERSECTION UTAH - FRANKLIN
   
   a) NW CORNER  (BLOCK 68A-10, 71, 68, 74)  
      
      \[ t_0 = \text{Block 68} = 0.151 \]  
      
      \[ \text{GUTTER FLOW} \]  
      \[ S_{100} = 1.00 \]  
      \[ v = 2.0 \text{fps} \]  
      \[ \text{GUTTER LENGTH} = 30 + 100 + 310 + 50 + 460 = 1520 \]  
      \[ t_g = \frac{1520}{(3650)} = 0.21 \]  
      
      \[ t_c = t_{100} + t_g = 0.151 + 0.214 = 0.365 \text{ HR} \]  
      
      \[ C_{S100} = 870 \]  
      \[ (F = 6) \]  
      
      \[ \text{WEIGHTED CN} = 10\% \text{ READ, SOIL GROUP A} \]  
      \[ \text{CN} = 61 \]  
      
      \[ Q_e = 0.08 \text{ in} \]  
      \[ Q_s = 0.08 \times 0.0571 	imes 870 = 1.8 \text{ CFS} \]  
      
      \[ \text{CHECK CURB} \]  
      \[ 8'' \text{ UC} \text{ @ } 11\% \]  
      \[ \text{STREET} = 30' \]  
      \[ \text{Cap} = 17.0 > 1.8 \text{ OK} \]  
      
      \[ \text{CHECK CAPACITY INLET} \]  
      \[ \text{ALL FLOW WILL BE PRACT} \]  
      \[ \text{SET INLET CFB: CN65} \]
b) NE Corner Block 75

\[ Q_s = 0.8 \]

From previous calls, a 13' Inlet on Continuous Grade will handle all of \( Q_s \).

6. Intersection of Uintah + N. Prospect

a) NW Corner Blocks 75A

\[ Q_s = 1.0 \]

From Inlet Calls & TRIPLE (9) Wide Curv
Condition has a capacity = 66.3 CFS > 1.0

b) NE Corner Blocks (H) 79

\[ Q_s = \frac{1}{6}(1.0) = 1.0 \]

From previous calls, a 13' Inlet on Continuous Grade will handle all of \( Q_s \).

c) Inlet East of Inlet, Uintah + Prospect

North Side

Block = \( \frac{1}{6}(79) = \frac{1}{6}(1.0) = 1.0 \)

There is contributing flow from the east which is out of the scope of this project. The 96' long Inlet in Surf Condition with a capacity of 63 CFS which is greater than 1.0 CFS entering from this aspect of the project.

7. Intersection of Shoo's Run + San Miguel North Side

Block 77

\[ Q_s = 63 \text{ CFS} \]

Capacity of Inlet II Surf Condition

9.8 ft > \( \frac{1}{6} \) 73 CFS CFS
1. Interception Unit at L.W.A.R.
   a) NW corner, Unit # 1, WRB
      • Q_{100} Runoff = 0.57 in. (Fig 7 & CN = 61)
      • Q_{100} = 0.57 in. (0.0372) = 19.9 cfs
      • Check curb cap.
        10" x 1.27" = 6.5.9 cfs > 19.9 cfs
      • Exist inlet & forming 11.5' width in sump
        Q_{sump} = 6.5.8 cfs (11.5' width) > 19.9 cfs
        = inlet will handle all Q_{100}
   b) NE corner
      • Q_{100} Runoff = 0.57 in. (Fig 7 & CN = 61)
      • Q_{100} = 0.57 in. (0.0372) = 19.9 cfs
      • Check curb capacity.
        10" x 1.27" = 6.5.9 cfs > 19.9 cfs
      • Inlet capacity: curb opening = 13"
        Face inlet cap. & sides
        B_{in} = 13.8" cfs Q_{in} = 11.3 cfs

\[ Q_{100} \text{ to washout} = 2.7 \text{ cfs} \]
2. INTEGRATING W rivulets & unseepage

   a) NEw DIVERTER (sec Q1, CaI)

      \[ Q_{1m} \text{ runoff} = 0.52 \text{ in} \quad \text{(fig 2 on 0.5 in. = 6.1)} \]

      \[ Q_{1m} = 0.52 \text{ (gpm)} \left( 0.0169 \right) = 8.6 \text{ cfs} \]

      \[ + 2.5 \text{ cfs main under inlet} + 2.5 = 11.0 \]

      \[ \text{Capacity} = 12.5 \text{ cfs} > 11.0 \text{ OK} \]

      \[ \text{Inlet supply capacity} = 12 \text{ in.} \]

      \[ Q_{1m} = 42.1 \text{ cfs} > 11.0 \text{ (for inlet area cals)} \]

      \[ \text{Inlet will handle all } Q_{1m} \]

   b) MEDIANS

      \[ \text{Each median has } Q_{1m} = 0 \text{ cfs} \text{ at } \text{all} \]

      \[ \text{sec runs CaI}; \quad Q_{1m} \text{ will all be handled} \]

      \[ \text{by exists, wells}. \]

   c) NEw CURVE

      \[ Q_{10m} \text{ runoff} = 0.57 \text{ in} \quad \text{(fig 2 on 0.5 in. = 6.1)} \]

      \[ Q_{10m} = 0.57 \left( 0.0169 \right) = 9.9 \text{ cfs} \]

      \[ + 2.5 \text{ cfs main under inlet} = 12.5 \text{ cfs} > 4.9 \text{ OK} \]

      \[ \text{Exist inlet cap 12' length @ 0.6 ft} \]

      \[ Q_{1m} = 4.9 \text{ in. } Q_{1m} = 4.9 \text{ (for inlet cap cals)} \]

      \[ \text{Inlet will handle all } Q_{10m} \]
3. Interaction Uinta and Corona

a) NW Corner

- \( Q_{\text{in}} \) run-off = 0.57
  
- \( Q_{\text{in}} = 0.57 \left( 1.05 \right) \left( 0.106 \right) = 0.3 \text{ cm} \)

- Curbside capacity = 4.70 cfs > 0.3 cfs

- Exit NWE surf drain = 3.5 cfs > 0.3 cfs

- All \( Q_{\text{in}} \) to be drained by NWE

b) NE Corner

- \( Q_{\text{in}} \) run-off = 0.57
  
- \( Q_{\text{in}} = 0.57 \left( 1.27 \right) \left( 0.0108 \right) = 0.7 \text{ cfs} \)

- Curbside capacity = 4.70 cfs > 0.7 cfs

- Exit NNE surf drain = 3.6 cfs > 0.7

- All \( Q_{\text{in}} \) to be drained by NNE
Q_{0.00} \text{ Sub Basin F}

A) Intersection of Unthank and SE Pk Rd
   1) HW Corner (70, 67, +69)
      \[ \text{\%C} = 0.163 \text{ HR} \quad (Q_{0.00} = \text{CALC}) \]
      \[ \text{CSP/m} = 1150 \]
      \[ \text{Area} = 0.0198 \text{ SM} \]
      \[ \text{CN} = 70 \]
      \[ Q_{0.00} \text{ RUNOFF} = 1.03 \]
      \[ Q_{0.00} = 1.03 (0.0198)(1150) = 23.5 \text{ CFS} \]

CHECK Existing Inlet In Sump
   \[ \text{CSP} = 34.4 > 23.5 \quad \text{OK} \]

CHECK Curb Capacity
   \[ \text{CSP} = 22.2 < 23.5 \]

Flow will overflow curb but all flow will be intercepted by inlet. A small amount of flushing will occur as water overtops curb. However, this amount is not enough to overfill the inlet and store water. End of inlet.

B) NE Corner, Block Top \( \frac{1}{2} \) (Block 76)
   \[ Q_{0.00} = \frac{1}{2} (4.3) = 2.2 \text{ CFS} \]

CHECK Inlet Capacity
   \[ \text{SUMP} 40.8 > 2.2 \quad \text{OK} \]
5. Intersection of Uintah & Franklin

a) NW Corner, Block 7S1

Q_{100} = 3.6 cfs

From previous cells & inlet on continuous grade will handle all of Q_{100}

b) NE Corner, Block 7S9

Q_{100} = 3.6 cfs

From previous cells & inlet on continuous grade will handle all of Q_{100}

6. Intersection of Uintah & Prospect

a) NW Corner, Block 7S8A

Q_{100} = 5.1 cfs

The capacity of a temple (3) is inlet in gorp conditions is 20.3 cfs > 5.1 OK

b) NE Corner, Block 7S9

Q_{100} = 5.1 cfs

From previous cells & inlet on continuous grate will handle all of Q_{100}
c) Inlet East of Intersection Unstress Project North Side

\[
Q_{100} = \frac{1}{10} (61) = 3.1 \text{ cfs}
\]

Capacity of Inlet in Sump Condition = 0.3 cfs
Inlet must be sized to handle Q_{100}
Use a 4" DIOR in Sump Condition.
Max Capacity 7.9 cfs

There is flow coming from the East which is not in this scope of work. This flow must be analyzed to properly size the DIOR inlet.

7. Intersection of Streets and San Miguel North

Block 77

\[
Q_{200} = 20.0 \text{ cfs (Per Single Flow MALS)}
\]

Capacity of Inlet in Sump Condition = 0.8 cfs ≤ 20.0

4" DIOR Inlet in Sump Condition will handle 0.8 cfs.

There is flow coming from the East which is not in this scope of work but will be entering the inlet at this point. In order to properly size this inlet, a study of flow entering from the East must be calculated.
Design Calculations

Project

Detail

Designer Date Sheet

Checker Date Job No

\[ Q_{\text{sum}} = 24.1 \text{ cm}^3 \]
\[ Q_{\text{in}} = 6.3 \text{ cm}^3 \]

\[ Q_{\text{sum}} = 26.6 \text{ cm}^3 \]
\[ Q_{\text{in}} = 7.6 \text{ cm}^3 \]
NOTE: THERE IS A CONTRIBUTING FLOW FROM THE EAST WHICH IS OUT OF THE SCOPE OF THIS PROJECT. THIS FLOW MUST BE ACCOUNTED FOR IN ORDER TO PROPERLY SIZE THE DIAM IN LEWS.
**DESIGN CALCULATIONS**

Project: 
Detail: 
Designer: Date: Sheet: 1 of 3 
Checker: Date: Job No: 

**Subgrade G**

a) All drainage to south & east to exist 18" culvert which discharges to swale run, thus no drywall content point.

---

**Q5 / Subgrade G**

1) 18" culvert Q: 300 cfs run (Block 73 + 76)

- **Qc (Block 73) = 0.111 (Single block cfs)**

- **Cutter Run**
  - **Saw** = 1.07
  - **N** = 1.8 (Fig. 5) Gravel Runo Atrch
  - **Cutter Length** = 30 + 250 = 280 ft
  - **t = 280 \( \frac{1.8}{600} \) = 0.043

- **t_c = t_k + t_c' = 0.154 hrs**

- **C29** = 1000

- **EADA**
  - **AC Area**
    - **Block**
      - 73
      - 36
      - 1000
      - 1000
      - 0.0140 SM

- **Weighted CN**
  - 0.4 cm
  - 0.25

- **CN = 0.25 (12) + 0.75 (26) = 79**

- **Qc Runoff = 0.56**
  - (Fig. 2)
Design Calculations

Project: 
Detail: 
Designer: 
Date: 
Sheet: L of 7

**Design Calculations**

\[ Q_{100} = 0.56 (0.0194) (1020) = 8.9 \text{ cfs} \]

**Capacity Calculation**

\[ Q_{culvert} = \frac{4.16}{1.33} = 3.1 \text{ cfs} \]

\[ Q = 8.0 \text{ cfs} \]

**Impoundment:**

a) Install 2 - 18" Culverts

b) Install Ground Riprap Runoff
SUBBASIN H

a) DRAINAGE IS TO THE SOUTH TO CATCH LA RIVER ALONG WEBER. EXIST "STORM SEWER" BETWEEN YAMPA & WEBER (18") B. ASSUME 18" SL GULLY, SL GULLY DI STREET FOR CAPACITY OR SI CALCULATIONS. 50% RECLAIM ING AVAILABLE.

b) BUBBLERS ARE LOCATED AT INTERSECTIONS

- SAN RAFAEL & WEBER (INSTALL LONG CROSSWALK FOR BUBBLERS ON WAISATCH SEE CALC.)

C) MAJOR INTERSECTIONS

- YAMPA & WEBER
- CATCH LA RIVER & WEBER
- CATCH LA RIVER & WAISATCH
- YAMPA & WAISATCH
- SAN RAFAEL & WAISATCH
LINE AREA & WEIR

L. NO. CRANK, TAPAS & WHEEL. BLOOM TO BE A, FIT MOUNTING LEGS AT INLET OF CONE OR UNTIL CORNER TO OTHER EXHAUSTIVE DETAIL DEF.

\[ t_e = 0.15 \text{ kpa} \] (single block call)

- Gutter Fun = 7 to 8
- \( S_{wG} = 0.55 \)
- \( A = 1.5 \text{ ft}^2 \) (includes gutter)
- Gutter Length = 50 + 490 = 540
- \[ t_e = \frac{540}{18} = 30 \text{ ft} \]
- \[ t_e = t_e + t_e = 0.012 + 0.150 = 0.242 \text{ kpa} \]
DESIGN CALCULATIONS

1. \( Q \) = Runoff \( = 0.08 \text{ in} \) (FIG 1)

2. \( Q = 0.08 \times 10.4 = 0.83 \text{ cfs} \)

3. Check storm capacity

4. INLET 2 \( D = 0.6 \text{ ft} \) \( B = 2.5 \text{ ft} \) \( W = 2.5 \text{ ft} \)

5. \( Q_{\text{ave}} = 13.2 \text{ cfs} \) (per inlet cap. tables) \( > Q \)

6. INLETS WILL TAKE ALL \( Q \)

b) NE CORNER 1 INLET 1 SOUTHWEST BLOCK 17 + 18

7. \( t_{17} = 0.113 \text{ hr} \)

8. Curve flow in curve

9. \( t_{17} = 0.093 \)

10. \( t = t_{17} + t_{17} = 0.093 + 0.113 = 0.205 \text{ hr} \)

11. \( C = \frac{Q}{t} = 10.75 \text{ cfs} \)

12. Block and Area

<table>
<thead>
<tr>
<th>Block</th>
<th>Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>17</td>
<td>0</td>
</tr>
<tr>
<td>18</td>
<td>0.012</td>
</tr>
</tbody>
</table>
DESIGN CALCULATIONS

4. CACHE LA PRAIRIE & WEBB

a) NW CORNER - BLOCK 9 ONLY, NO OTHER CONTRIBUTING AREAS FOR Q_5 DUE TO SUM WELTS AND SUFFICIENT CAPACITY

- Q_5 = 0.8 CFS (SINGLE BLOCK CALC)

- CHECK SUMP CAPACITY
  - 5' x 0.75
  - 13.4 CFS > 6.8 CFS
  - 60' STREET WIDTH
  - EXIT WELTS & ELAYNE, 2.5 FT LENGTH WELTS & 3 FT SUMP
  - Q_{sump} = 11.2 CFS (PER WELT CAP CALC) > 6.8 CFS
  - WELT WILL HANDLE ALL Q_5

b) NE CORNER

- Q_5 = 0.8 CFS (SINGLE Block CALC)

- CHECK WELT CAPACITY
  - 9' x 0.75
  - 26.25 CFS > 6.8 CFS
  - 60' STREET WIDTH
  - EXIT WELTS (2 WELTS REQUIRED 12.9' x 3.0' 9.7"")
DESIGN CALCULATIONS

2. Bubblers have been incorporated in transmitting flow downstream. Use bubblers by drop inlets and connect to existing 3/8 in. manhole.

a) New Manhole

1. Qf = 0.4 cfd (single bubber case)

b) Median (block by)

Qf = 0 cfd (single bubber case)

Whatever nuisance flow will be handled by existing curb opening inlet

c) NC Manhole

Qf = 0.5 cfd (small bubber case)

Gated inlet bubbler capacity 5" curb

Qf = 5.0 cfd > 0.5 cfd

(see gated inlet case)

b) Yampa 2 Manhole

a) New Corner

Qf = 0.4 cfd (single manhole case)

Gated inlet case 10" curb

Qf = 8.2 cfd > 0.4 cfd

(see gated inlet case)
Design Calculations

1) Median Block #7

* Crushed results as at San Rafael median

2) NW Corner (Block 19A)

- Q = 6.4 cfs (Single above case)
- Exist curb opening inlet capacity L3 = 2.5 ft
- Flow equation
  \[ Q_{\text{sum}} = Q_{\text{inlet}} + Q_{\text{outlet}} \]
  \[ Q_{\text{sum}} = 6.4 \text{ cfs} > 0.4 \text{ cfs} \]
  (See inlet cap case)

b) Median Block #7

Same results as at San Rafael & Tamarac.
Q_{100} = \frac{\text{Subbasin Area}}{100} = 4 \text{ in.} 

b) NE Corner

\[ Q_{\text{vow}} = 0.57 \text{ in.} \]

\[ Q_{100} = 0.57 \times 100 = 57 \text{ in.} \]

Check curb capacity \( 18.5 \text{ cu} > 57 \text{ in.} \)

\[ Q_{\text{curb}} = 12.25 > 57 \text{ in.} \]

\[ Q_{100} = 5.5 \text{ in.} \]

\[ Q_{\text{curb}} = 5.5 \text{ in.} \]

\[ \text{ALL Q}_{100} \text{ TO BE HANDLED BY \ EXIST INLET} \]
2. Cover caulk to prevent water penetration.

a) No caulk

- \( Q_{100} = 5.8 \text{ cm}^3 \) (above flow rate)
- Check curb capacity: \( 13 + C_e > 5.8 \text{ cm}^3 \)
- Exit rules: use jump

\[ Q_{35} = 5.8 \quad Q_{jump} = 11.2 \]

1. All \( Q_{100} \) to be handled by jumps.

b) No cover

- \( Q_{100} = 2.9 \text{ cm}^3 \) (above \( Q_{100} \) single block curve)
- Check curb capacity: \( 71 + C_e > 2.9 \text{ cm}^3 \)
- Exit double curb curves meet L + UT 5\%< 0.5%:

\[ Q_{35} = 2.9 \quad Q_{100} = 2.9 \text{ (see \% UT cm3)} \]

All \( Q_{100} \) to be handled by rules.
5. DRAINAGE TO BE CONVERTED TO GRADED INFLOW INLET &
CONNECTION TO SWIT 46 IN LAWN. S

\[ Q_{\text{in}} \] Bulk 12 = 3.6 cfs (single block cfs)

- GRADED INFLOW CAP. = 5.8 cfs > 3.6 OK

b) NE Corner

- Q_{\text{in}} Bulk 12 = 3.6 cfs (single block cfs)
- GRADED INFLOW CAP. = 5.8 cfs > 3.6 OK

b) Van Vl. Washout

a) NW Corner

- Q_{\text{in}} Bulk 18 = 2.8 cfs (single block cfs)

- GRADED INFLOW CAP = 8.2 cfs > 2.8 OK

l) NE Corner

- Q_{\text{in}} Bulk 93 = 2.6 cfs (single block cfs)

- DOUBLE INFLOW L2 = 2.5 8 = 20.57.

Q_{\text{in}} = 2.6 cfs Q_{\text{in}} = 2.6 cfs

ALL Q_{\text{in}} TO BE HANDLED BY INFLOWS.

3. CAUSE LA Poudre & Washout

c) NW Corner

- Q_{\text{in}} Bulk 19 = 2.8 cfs (single block cfs)

- INFLOW SUM CAP = 4.5 cfs > 2.8

ALL Q_{\text{in}} TO BE HANDLED BY INFLOWS.
SUBTASK I

6) DRAINAGE 13 TO SOUTH TO EAST SIDE OF CEMETARY LA PEROUSE TO DUNGEE THAT ROUTE LATER TO TUBBERMILL AT CANCE LA PEROUSE & SPRAGUE AV.

b) DUNGE, THE LOWER END, FOLLOW & INTERSECT:

- PARALLEL AND EITHER SIDE OF CEMETARY Q AT 320 RAPID STREET, REMOVE DUNGE, INSTALL CONCRETE, CURB, CURBING.

- PARALLEL AND EITHER SIDE OF CEMETARY Q AT YAMBA, REMOVE DUNGE, INSTALL CONCRETE, CURB, CURBING.

- NE CORNER OF ROCHEL & YAMBA "T" INTERSECTION, CONNECT CUSTOMER BUILDING TO 18" SS IN RAPID, OTHER END TO REMOVE BUILDING & INSTALL CONCRETE, CURB, CURBING.

6) MASON INTERSECTIONS

- CEMENT LA PEROUSE & CONCRETE
- CEMENT LA PEROUSE & CURBING
- CEMENT LA PEROUSE & SPRAGUE

6) INSTALL 53 TO CANCE TO SPRAGUE & DUNGE TO PERUSE CURB & SUPERBAND AT CONCRETE CURBING.
1. **INTERSECTION W3 CARR T & CARE LA PUYNE**
   
   **a) NO. 13 METER (BLOCK 92B, 92A & 94)**
   
   - **T_{c} Block 92A = 0.128 in**  
     (Block 94 - Block 92A)
   
   - **CUTTER TOW**
     
   - **K = 0.65**
   
   - **N = 1.8**  
     (Fig. 5)
   
   - **CUTTER LENGTH = 450 x 50 = 22,500 ft**  
     1.8 (in)
   
   - **t_{c} = t_{e} + t_{c} = 0.35**
   
   - **cm/ln = 860**
   
   **EAVME**
   
   - **AREA**
     
   - **QLA**
     
   - **CIA**
     
   - **Q4**
   
   - **0.0418 ft**
   
   - **0.0694 ft**
   
   - **0.0186 ft**
   
   - **0.203 ft**
   
   **WEIGHTED CN**
   
   **100% RDI - SOL GRO 1A**
   
   **CN = 5.1**
   
   - **Q_{c} Runner = 0.08 in**  
     (Fig. 7)
   
   - **Q_{c} = 0.08 (0.0203) (80) = 1.4 cm**
   
   **CHECK Curb CAP**
   
   10% 1.5%  
   60' STREET WIDTH
   
   **CHECK EXIT INLET**  
   (See Master CAS)
   
   **Q_{SUM} = 3.0 CFS > 1.0 CFS**
   
   *All Q_{c} to QC HANDLE BY INLET*
2. INTERSECTION R0.4 & CIRCLE 8-BRICK
   a) NEW CURVE
      Length (30M) 92m

      x = 0.097  y = 0.002
      (from line CARS)

      Center Tilt
      S = -1.15%
      N = 2.0  (Fig. C)
      Cut-Off Length = 1070 + 441 + 18.5 = 1239.5
      1.5 = L / 4.0  3.6

      1.5 = 0.257

      L = 0.357

      csp/n = 860  (Fig. C)

      860

      92  10027
      90  10040
      94  10041
      99  10160  SM

      90.7% RRA. - 50% GROWTH
      R = 61

      Qf  Runoff = 0.08 10
      Qf = 0.08 10 1000 = 1.1 CFS

      Check curve cap
      Qe < 1.5% 30 40 60

      Check exit double inlet sum capacity C + 2
      Qsum = 14.9 CFS > 1.1  (exit inlet cap. OMS = 11)

      All Qf to DE handled by inlet.
b) NE Corner

\[ t_e = \text{block} \cdot g \cdot A = 0.417 \text{ hr} \]

**GUTTER BAY**

\[ S_{ac} = 1.57 \]

\[ N_0 = 2.5 \text{ cm} \cdot (2 \text{ in}) \]

**Center Location**

\[ x = 150 + 0.8 \cdot x + 470 \times 0.6 \text{ in} \]

\[ t_0 = \frac{0.072}{2(360)} = 0.04 \text{ hr} \]

\[ t_e = t - t_0 = 0.417 - 0.04 = 0.377 \text{ hr} \]

\[ E_{sm/n} = 740 \]

\[ z = \text{AREA} \]

**BLOCK**

<table>
<thead>
<tr>
<th>Block</th>
<th>Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.007</td>
</tr>
<tr>
<td>B</td>
<td>0.0036</td>
</tr>
<tr>
<td>C</td>
<td>0.0035</td>
</tr>
</tbody>
</table>

**WEIGHTED CN**

\[ \frac{1}{2} \text{ cm} = 1.014 \text{ cm} \]

\[ C_m = 0.33(0.9) + 0.33(1.1) + 0.33(4) = 6.6 \]

**Q_3 Runner**

\[ Q_3 = 0.15 \text{ in} \]

**Q**

\[ Q = 0.15(740)(0.0141) = 1.6 \text{ CFs} \]

**Check Area Capacity**

\[ C'' \text{ cm} = 1.0 \% \quad \Rightarrow 14.2 \text{ CFs} > 1.6 \quad \text{CFs} \]

**Check Exit Duct:**

\[ Q_{35} = 1.6 \text{ CFs} \quad Q_{35} = 1.6 \text{ CFs} \]

\[ Q_{0.2} \text{ CFs} \text{ down to } 0.2 \text{ CFs} \]

**Check Exit Location:**

\[ z = 3.1 \% \quad 8.1 \% \]

\[ Q_{35} = 1.6 \text{ CFs} \quad Q_{0.2} = 0.2 \text{ CFs} \text{ down to } 0.2 \text{ CFs} \]
3. WATERWORKS WATER MAIN (FIGURE 3.1)

(a) NC JUXTAPOSED HORIZONTAL (SMALL BENDS)
(c) Size: 3000, 4000, 5000

(b) NC JUXTAPOSED HORIZONTAL (SMALL BENDS)
(c) Size: 3000, 4000, 5000

Check: 0.46

Cut: 0.47

Use Table 3.4 (5000, 6000, 7000)

Check: 0.35

Cut: 0.36

Use Table 3.4 (5000, 6000, 7000)

(d) New Camber: 96

Cut: 0.35

Use Table 3.4 (5000, 6000, 7000)

(e) New Camber: 96

Cut: 0.35

Use Table 3.4 (5000, 6000, 7000)
1. INTEGRATE CORONA & CACHÉ LA POUVRE

- \( Q_{1000} \) = 0.1 ft³/s (FIG. 2, Cvir = 6)

- \( Q_{100} = 0.1 \times (0.025)(864) = 10.0 \text{ cfs} \)

- Check curl cap = 6.5 ftcs > 10.0 ft/s

- Check inlet sum cap = 9.6 < 10.0 ft/s

2. Assume 2.0 cfs will flow south from Caché La Poudre.

POUDRE CORONA & NORTH CORONA ST. CROWN, E2UW
TURBULENT (WAR CAP IN SUPERWOM, OR REMOVE EXIT INLET & INSTALL A BID INLET)

3. INTEGRATE POULDER & CACHÉ LA POUVRE

(a) NW CORONA

- \( Q_{1000} \) = 0.1 ft³/s (CIR = 6, Cvir = 6)

- \( Q_{100} = 0.1 \times (0.010)(864) = 5.2 \text{ cfs} \)

- From NW CORONA & CACHÉ LA POUVRE & CORONA OUTLET

- Check curl cap = 7.5 ft/s > 6.2 ft/s
b) NE CORNER

\[ Q_{100} = 0.147 \text{ in} \]

\[ Q_{100} = 0.81 (340)(0.0143) = 8.4 \text{ cfs} \]

Check curb cap \[ Q_{100} = 14.3 \text{ cfs} \]

Check exist inlet cap \[ L = 3' \]

\[ Q_{110} = 2.0 \text{ cfs} \]

[6.4 cfs from NE corner came in dispute? RAO]

\[ Q_{110} = 2.0 \text{ cfs} \]

\[ Q_{110} = 2.0 \text{ cfs} \]

3. INTERSECTION EL PASO & CACHÉ LA PUEBLA

a) NE CORNER

\[ Q = 3.2 \text{ cfs} \]

Check curb cap

10" curb = 2.27 \text{ in} \]

30' Run = 0.075

Check inlet cap

\[ L = 3' \]

\[ Q_{110} = 0.5 \text{ cfs} \]

\[ Q_{110} = 2.7 \text{ cfs} \]

Grant inlet cap to Caché La Puebla & Sycamore Run.

b) NW CORNER

\[ Q_{100} = 2.4 \text{ cfs} \]

Check curb cap

\[ Q_{100} = 2.4 \text{ cfs} \]

\[ Q_{110} = 8.3 \text{ cfs} \]
CHECK Curb CAP = 8.6 CFS < 8.8 CFS

CHECK WET SUMP CAP = 99 CFS < 8.8

If flooding will occur, it may be necessary to provide additional drainage. Based on the data provided:

Q_{in} = 2.7 ft³
Q_{out} = 2.1 ft³
Q_{loss} = 0.6 ft³

Q_{def} = 0.6 ft³ (in dead water cases)

Note that additional drainage cells are needed in the lower elevations of the study area.
NW CORNER  CACHE LA PRAIRIE

S' WIDTH  CAPACITY = 10.0 CPS (80) = 8.0 CPS

Qw = 10.0 CPS

Qw = 10.0 CPS

NO CORNER RESIDENCE  CACHE LA PRAIRIE

EXIST DOUBLE 3' INLET SUMP  CAP = 14.0 LPS

Qw = 6.3 < 14.0 OK

CONVERGENCE INLET TO THE EXISTING INLET

NE CORNER RESIDENCE  CACHE LA PRAIRIE

EXIST DOUBLE CONTINUOUS Qw = 8.0

Qw = 8.4  6.9 will pass

TOTAL IN MH = 8.2
OFFSITE BASIN A-A

OFFSITE BASIN A-A HAS BEEN ANALYZED BECAUSE ITS FLOWS DIRECTLY CONTRIBUTE TO THE FLOODING OF THE MADISON / WAINSCOTT INTERSECTION.

THE LOCATION OF OFFSITE BASIN A-A IS SHOWN BELOW

THE MAJOR INTERSECTIONS INCLUDE:
- FRANKLIN + MONROE
- EL PASO + MONROE
- MACELLAN + MONROE
- JACKSON + MACELLAN
- ROGER + MONROE
- EL PASO + MADISON
- ROGER + MADISON

The 100-year flows calculated from this basin are large and must be intercepted before reaching the Madison / Wainscott intersection.
DESIGN CALCULATIONS

OFFSITE BASIN A-A

\[
\begin{align*}
\text{AREA TOTAL} &= 51.17 \text{ acres} \\
\text{AREA STREAMS} &= 0.0079 \text{ sq. miles} [10\%] \\
\text{AREA RESID.} &= 0.0823 \text{ sq. miles} [18\%] \\
\text{AREA FALLOW ROW} &= 0.0097 \text{ sq. miles} [2\%] \\
\text{SOIL GROUP B}
\end{align*}
\]

Basin Area Soil Land Curve Basin Drainage Area

\[
\begin{align*}
Q_s &= (0.0799 \times 1000 \times 0.4) = 32.1 \text{ CFS} \\
Q_{100} &= (0.0799 \times 160.8 \times 1.35) = 110.8 \text{ CFS}
\end{align*}
\]

DRAIN AGENT B B.B

\[
\begin{align*}
\text{AREA TOTAL} &= 0.0422 \\
\text{100\% RESID} \\
\text{SOIL TYPE B} \\
\text{CURVE NO. 75}
\end{align*}
\]

Basin Area Soil Land Curve Basin Drainage Area

\[
\begin{align*}
Q_s &= (0.0422 \times 1000 \times 0.4) = 16.9 \text{ CFS} \\
Q_{100} &= (0.0422 \times 160.8 \times 1.35) = 67.8 \text{ CFS}
\end{align*}
\]
### DESIGN CALCULATIONS

**Basin** | **Basin #** | **Basin Length** | **Slope** | **Vol.** | **Ft.**
---|---|---|---|---|---
SB-1 | 18 | 1000 | 1.85% | 8.44 | 0.101
SB-2 | 18 | 1100 | 1.64% | 2.34 | 0.122
SB-3 | 19 | 1200 | 1.53% | 2.42 | 0.139
SB-4 | 19 | 1400 | 1.34% | 2.32 | 0.169

**Flow Calculation**

- **SB-1**:  
  \[ \text{5yr Flow} = \frac{15}{1270} \cdot 100 \approx 7.67 \text{ CFS} \]
- **SB-2**:  
  \[ \text{5yr Flow} = \frac{1.34}{1240} \cdot 100 \approx 0.69 \text{ CFS} \]
- **SB-3**:  
  \[ \text{5yr Flow} = \frac{0.15}{1200} \cdot 100 \approx 0.72 \text{ CFS} \]
- **SB-4**:  
  \[ \text{5yr Flow} = \frac{0.18}{1400} \cdot 100 \approx 0.13 \text{ CFS} \]

**Capacity of Mound E & FRANKLIN**

- 40' Wide x 0.75' Bx = 34.5 CFS
- 24.5 > 7.67 CFS 5yr OK
- 24.5 < 26.10 CFS 100yr OK

There will be some curve overtopping but not enough to warrant an inlet at this intersection.

**Mound E & FRANKLIN**

- **SB-2**:  
  \[ t_c = 0.125 \]
  \[ + t_c \text{ in curve from SB-1} \]
  \[ t_c = 0.15 \]  
  \[ V = 2.4 \text{ ft.} \]
  \[ t_c = \frac{1500}{2.4} = 0.179 \text{ hrs} \]

- **SB-1**:  
  \[ t_c = 0.179 + 0.125 = 0.304 \]
  \[ C_{franklin} = 0.4 \]
  \[ C_{franklin} = \text{franklin} \]
Design Calculations

@ Monroe + Franklin

Q8 = \(0.0285 \times 3.2 \times 920 \times 0.4\) = 10.49 CFS

Q100 = \(0.0285 \times 3.2 \times 920 \times 1 \times 1.36\) = 35.66 CFS

All flow will be coming from SB-1 down Monroe

Capacity of Monroe @ Franklin

40' wide @ 0.7% 8' curb = 24.5 CFS

\(Q_8 > 10.49\) \(Q_{100}\) OK

\(Q_8 < 35.66\) \(Q_{100}\) No Good

Add Inlet NE corner Monroe + Franklin

12' DIOR 7.7% continuous grade will intercept 9.4 CFS

\(Q_{100} = 35.66 - 9.4 = 26.26\) Flow by

\(Q_8 = 10.49 \times 0.66\) = 6.30 Flow by

24.5 < 26.26 No Good

Add Inlet N North Corner

4' DIOR Intercepts 7.2 CFS

\(Q_{100} = 26.26 - 7.2 = 19.06\) Flow by

\(Q_8 = 4.19 \times 0.66\) = 2.5 Flow by

24.5 > 19.06 OK

@ Monroe + El Paso

SB-8 4 YR = 7.25 + 1.69 = 8.92 CFS

100 YR = 24.64 + 19.00 = 43.70 CFS

Capacity of Monroe @ El Paso

40' 01/0 = 0.7% 8' curb = 24.5 CFS

24.5 < 24.5 = OK

24.5 < 43.70 = No Good
INSTALL INLET @ NE CORNER EL PASO + MONROE

14' DIOR WILL INTERCEPT 10.34 CFS

Q<sub>60</sub> 43.70 - 10.34 = 33.36 CFS Flows By
Q<sub>6</sub> 8.92 (60) 5.35 INTERCEPTED 3.57 Flow By
24.5 < 33.36  NO GOOD
24.5 > 3.57  OK

INSTALL INLET @ NW CORNER EL PASO + MONROE

14' DIOR WILL INTERCEPT 10.34 CFS

Q<sub>60</sub> 33.36 - 10.34 = 23.02 Flows By
Q<sub>6</sub> 3.57 (60) = 21.43 INTERCEPTED 1.43 Flow By
24.5 > 23.02  OK
24.5 > 1.43  OK
@ Mangelo Macelain

Q5 = 10.19 + 1.48 = 11.62 CFS
Q100 = 34.40 + 23.04 = 57.52 CFS

There is a pan located @ Mangelo + Monroe which will carry flow across Monroe and then down Rover.

Capacity of Monroe @ this ft.

40' wide slope = 0.79% cover 8''
= 24.5 CPS

24.5 > 16.44 CFS Q5 OK
24.5 < 58.49 CFS Q100 No Good

Sub-basin A will be split in half. Inlets will be installed at Jackson + Macellain and Monroe + Macellain.

Area: SB-4 A: .0100 SM A01340
Area: SB-13: .0000 SM

Basin Area Soil Unit Curve Basin Basin Unit Curve 5% 10% 25% 50% 98%
SB-4A: C10 B 8.19 1.00 100 0.4 1.98
SB-4B: C10 B 8.19 1.00 100 0.4 1.98

Q5 = (0.066 x 1.00 x 0.9) + 5.30
Q100 = (0.066 x 1.00 x 0.9) + 18.50
INSTALL INLET ON MAGELLAN AT INTERSECTION OF
JACKSON + MAGELLAN
12" D.I.R. INTERCEPT 11 1/2" CAP 20.1 CFS
Q = 7.18
5 1/2" WIDE

CHECK STREET CAPACITY OF MONROE @ MAGELLAN
CAP = 20.7 CFS

CHECK CAPACITY OF MAGELLAN = 1.65%
40' WIDE = 30.1 CFS
5' VC.

CHECK CAPACITY OF MONROE
0.79%
5' VC
= 24.5 CFS
50' WIDE

CHECK CAPACITY OF MONROE @ INTERSECTION MAGELLAN
40' WIDE
0.59%
5' VC
= 20.7 CFS

INTERSECTION OF MAGELLAN + MONROE

Total Flow Q100
= 7.18 (From Jackson) Qs 9.12
+ 15.34 (From SB-4B) Qs 4.48
+ 23.02 (From El Presidio) Qs 1.43

Q100 = 45.54 CFS
Qs = 8.03 CFS

* INSTALL 12" D.I.R. ON MAGELLAN + MONROE
@ 1.2% INTERCEPT 11 1/2" CFS Qs 9.12

* INSTALL 12" D.I.R. ON MONROE
@ 0.79% INTERCEPT 9.4 CFS Qs 4.48

Total Q100 = 60.92 CFS
Total Qs Interceptors = 60(.802) = 48.12 CFS

* INSTALL 2 INLETS - 1 ON EACH SIDE OF EXISTING
Flow by Down Pan to Roger =

$$45.54 - 20.52 = 25.02 \text{ cfs} Q_{f1}$$

$$8.03 - 4.82 = 3.21 \text{ cfs} Q_{f2}$$

A 30' DIOR INLET WILL BE INSTALLED AT THE LOWER END OF THE LESS PAN LOCATED AT MAGUEAN AND MENAGE.

THE 100 yr FLOW DEPTH IN THIS PAN WILL BE 0.4' DEEP. THE DIOR WILL BE 30' WIDE TO INTERCEPT ALL OF THE FLOW AT THIS POINT.
### Lower 1/2 of Sub-Basin 1

**Flow to Madison + Boxer**

<table>
<thead>
<tr>
<th>Basin</th>
<th>Area (sq ft)</th>
<th>Land</th>
<th>Curve No.</th>
<th>Basin Vel</th>
<th>Lc</th>
<th>Cm</th>
<th>Cm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.18</td>
<td>76</td>
<td>12</td>
<td>1800</td>
<td>2.0</td>
<td>0.81</td>
<td>120</td>
</tr>
</tbody>
</table>

\[ Q_L = 0.18 \times 76 \times 12 \times 1800 \times 2.0 \times 0.81 \times 120 \times 0.4 = 8.18 \text{ cfs} \]

\[ Q_H = 0.18 \times 120 \times 1.37 = 29.02 \text{ cfs} \]

**Total Flow to Madison + Boxer**

\[ Q = 8.18 + 29.02 = 8.69 \text{ cfs} \]

\[ 8.69 \times 7.61 = 35.63 \text{ cfs} \]

**Capacity of Madison**

\[ 0.350 \text{ MGD} = 8.9 \text{ cfs} \leq 35.63 \text{ cfs} \]

8' UC

**Flow Must Be Intercepted Further Up**

**Split Up Lower 1/2 of Sub-Basin 1 into 3 Basins**

<table>
<thead>
<tr>
<th>Area 1</th>
<th>Area 2</th>
<th>Area 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.048</td>
<td>0.078</td>
<td>0.068</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Basin</th>
<th>Area (sq ft)</th>
<th>Land</th>
<th>Curve No.</th>
<th>Basin Vel</th>
<th>Lc</th>
<th>Cm</th>
<th>Cm</th>
</tr>
</thead>
<tbody>
<tr>
<td>L/b-4</td>
<td>0.048</td>
<td>76</td>
<td>12</td>
<td>1800</td>
<td>2.0</td>
<td>0.81</td>
<td>120</td>
</tr>
<tr>
<td>L/b-2</td>
<td>0.078</td>
<td>76</td>
<td>12</td>
<td>1800</td>
<td>2.0</td>
<td>0.81</td>
<td>120</td>
</tr>
<tr>
<td>L/b-1</td>
<td>0.068</td>
<td>76</td>
<td>12</td>
<td>1800</td>
<td>2.0</td>
<td>0.81</td>
<td>120</td>
</tr>
</tbody>
</table>

\[ Q_{L/b-4} = 0.048 \times 76 \times 12 \times 1800 \times 2.0 \times 0.81 \times 120 \times 0.4 = 2.09 \text{ cfs} \]

\[ Q_{L/b-2} = 0.078 \times 120 \times 1.37 = 12.50 \text{ cfs} \]

\[ Q_{L/b-1} = 0.068 \times 120 \times 1.37 = 13.07 \text{ cfs} \]
CAPACITY OF MADISON & TRUMLEN

LSB 1
\[ q_s = 2.0 \text{ CFS} \]
\[ q_{0.07} = 6.85 \text{ CFS} \]

40' wide
0.7% = 24.5 CFS > 6.85 OK
8" VC

LSB 1 and 2

Total Area = 0.13

\[ q_s = 0.10 \text{ CFS} \]
\[ q_{0.07} = 0.139 \text{ CFS} \]

\[ q_L = 0.104 + 0.139 = 0.243 \text{ CFS} \]

New CSM = 1000

5' YR EROSION = 0.4
100' YR EROSION = 1.3

\[ Q_s = (0.113)(1000)(0.4) = 45.2 \text{ CFS} \]

\[ Q_{0.07} = 0.113(1000)(1.37) = 154.8 \text{ CFS} \]

CAPACITY OF STREET = 64.5 715.48 OK

MADISON & ROYER

\[ q_{0.07} = 8.02 + 7.41 = 35.63 \text{ CFS} \]

\[ \text{FLOW MUST BE INTERRUPTED BEFORE IT REACHES MADISON & ROYER} \]

NE SENIOR INSTALL 12" DIAM @ EL PHO 0 + MADISON

Flow Interruption = 9.44 CFS

\[ q_{0.07} = 4.52(40) = 8.71 \text{ CFS} \]

Flow passing by inlet = 15.48 - 9.44 = 6.04 CFS

\[ q_{0.07} = 4.52(2.7) = 12.18 \text{ CFS} \]
Total Q100 @ NE Corner Royer and Madison:

\[ Q_{100} = 12.11 \, \text{cfs} + 6.04 \, \text{cfs} + 1.81 \, \text{cfs} = 19.96 \, \text{cfs} \]

\[ Q_5 = 3.54 \, \text{cfs} \]

\[ Q_{100} \text{ from ROGER} = 6.04 \, \text{cfs} \]

\[ Q_{100} \text{ from FND} = 1.81 \, \text{cfs} \]

\[ \frac{18.15 \, \text{cfs}}{5.35 \, \text{cfs}} \]

Capacity of Madison:

80' wide

0.33% = 8.9 cfs < 18.15 cfs no overflow.

Flow Must Be Intercepted by Inlets Before It Is Carried Down to Madison.

18.15 - 8.9 = 9.25 cfs Must Be Intercepted

Install 12' D10R

Intercept 0.44 cfs

18.15 - 9.44 = 8.71 Flow By

Madison CAP = 8.9 > 8.71 Q100 OK

\[ Q_5 = 5.35 \times 0.60 = 3.21 \, \text{cfs} \] Intercept

2.14 cfs Flow By
MADISON/ CORONA

Block 56

Qₐ = 1.8 cfs
Qᵥ = 5.6 cfs

FROM SINGLE BASIN CALLS

Qₐ = 1.8 + 2.14 = 3.94 cfs

Qᵥ = 5.6 + 8.71 = 14.31 cfs

FROM MADISON + ROYER

INSTALL 12' DIOR @ SE CORNER CORONA + MADISON

Flow Intercepted = 8.16 cfs, Qₒ = 2.36 cfs
Qₒ = 14.31 - 8.16 = 6.15 cfs Flow By Down Corona
Qₒ = 3.94 - 2.36 = 1.58 cfs

Capacity of Corona
55' Wide 0.57' 6' UC = 20.7 cfs
60.7 > 6.15 OK

FROM BASIN A CALLS SHEET

Check Intercept of Down Corona + Corona for Cure
Capacity Kills 5% - 0.45 flow from Madison + Royer

Qₒ = 16.6

.55 = Difference in Flow from Basins
16.55

Check Cure Capacity

Corona 9" @ 0.7% = 36.9 > 16.55 OK

Tapped
60' Wide
6' UC = 69.3 > 16.55 OK
@ 1.0%
**Inlet Capacities**

\[ Q_{st} = \text{100-year design flow in street section upstream of the inlet} \]

\[ Q_{in} = \text{portion of 100-year design flow that flows into the inlet} \]

\[ Q_{st} = \text{remaining portion of } Q_{100} \text{ that flows downstream into street gutter to next inlet.} \]

\[ Q_{st} = Q_{in} + Q_{st} \]

\[ 0.8% \text{ S} = 0.8\% \text{ gutter slope} \]

<table>
<thead>
<tr>
<th>Location</th>
<th>TWC Outlet</th>
<th>( Q_{st} ) (cfs)</th>
<th>( Q_{in} ) (cfs)</th>
<th>( Q_{st} ) (cfs)</th>
<th>Existing Catch (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NE Corner Madison/William St</td>
<td>4.8 in. on 0.8% S</td>
<td>10.0</td>
<td>6.0</td>
<td>4.0</td>
<td>14.5</td>
</tr>
<tr>
<td>SE Corner Madison/William St</td>
<td>4.8 in. on 0.8% S</td>
<td>6.1</td>
<td>3.7</td>
<td>2.4</td>
<td>14.5</td>
</tr>
<tr>
<td>W Side Madison/William St</td>
<td>6.9 in. on 0.6% S</td>
<td>21.5</td>
<td>12.7</td>
<td>8.8</td>
<td>14.5</td>
</tr>
<tr>
<td>NE Corner William/Aurora Ave.</td>
<td>15.8 in. on 0.8% S</td>
<td>21.4</td>
<td>9.0</td>
<td>12.4</td>
<td>14.5</td>
</tr>
<tr>
<td>SE Corner William/Aurora Ave.</td>
<td>6.9 in. on 0.8% S</td>
<td>13.4</td>
<td>6.0</td>
<td>7.4</td>
<td>14.5</td>
</tr>
<tr>
<td>NW Corner William/Aurora Ave.</td>
<td>15.8 in. on 0.8% S</td>
<td>15.1</td>
<td>10.5</td>
<td>4.6</td>
<td>14.5</td>
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<tr>
<td>NC Corner Fontain/Aurora Ave.</td>
<td>21.2 in. on 1.0% S</td>
<td>25.6</td>
<td>14.7</td>
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<td>29.3</td>
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<tr>
<td>NLS Corner Fontain/Aurora Ave.</td>
<td>21.2 in. on 1.1% S</td>
<td>27.2</td>
<td>16.4</td>
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<td>30.8</td>
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<tr>
<td>N. Side Fontain &amp; RR Rth</td>
<td>6' DIOR E 12%</td>
<td>6.4</td>
<td>3.4</td>
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<td>32.1</td>
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<tr>
<td>NE Corner W St/Alvord</td>
<td>8' DIOR SUP</td>
<td>14.0</td>
<td>1.0</td>
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<tr>
<td>S Side Fontain/Belle Rth</td>
<td>5.0 DIOR E 12%</td>
<td>1.0</td>
<td>1.0</td>
<td>0.0</td>
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<tr>
<td>SUBDRAIN</td>
<td>LOCATION</td>
<td>TVE WEST</td>
<td>Q1 (cfs)</td>
<td>Q2 (cfs)</td>
<td>Q3 (cfs)</td>
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<tr>
<td>----------</td>
<td>----------</td>
<td>----------</td>
<td>-----------</td>
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<td>-----------</td>
</tr>
<tr>
<td>B</td>
<td>NW CORNER/ JAKAU</td>
<td>10' DIA C 0.425</td>
<td>12.7</td>
<td>7.7</td>
<td>5.2</td>
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<tr>
<td></td>
<td>NE CORNER/ JAKAU</td>
<td>20' DIA C 0.425</td>
<td>26.6</td>
<td>12.2</td>
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<td>12' DIA C 0.425</td>
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<td>20' DIA C 0.425</td>
<td>21.8</td>
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<td>9.6</td>
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<tr>
<td></td>
<td>NW CORNER/ BALBOA/JAKAU</td>
<td>4' DIA C 0.425</td>
<td>4.6</td>
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<td>3.9</td>
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<td></td>
<td>NE CORNER/ WASHINGTON/ JAKAU</td>
<td>20' DIA C 0.425</td>
<td>21.3</td>
<td>11.2</td>
<td>9.1</td>
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<td></td>
<td>NW CORNER/ WASHINGTON/ JAKAU</td>
<td>20' DIA C 0.425</td>
<td>21.3</td>
<td>11.2</td>
<td>9.1</td>
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<tr>
<td></td>
<td>N SIDE/ SUMP/ RR ROW</td>
<td>6' DIA C 0.425</td>
<td>9.5</td>
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<tr>
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<td>S SIDE/ SUMP/ RR ROW</td>
<td>4' DIA C 0.425</td>
<td>1.0</td>
<td>1.0</td>
<td>0.0</td>
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<tr>
<td>C</td>
<td>NE CORNER/ WASHINGTON/ JAKAU</td>
<td>6' DIA C 0.425</td>
<td>8.2</td>
<td>8.2</td>
<td>0.0</td>
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<tr>
<td></td>
<td>NE CORNER/ WASHINGTON/ JAKAU</td>
<td>6' DIA C 0.425</td>
<td>8.2</td>
<td>8.2</td>
<td>0.0</td>
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<tr>
<td></td>
<td>N SIDE/ SUMP/ RR ROW</td>
<td>4' DIA C 0.425</td>
<td>2.5</td>
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<tr>
<td></td>
<td>S SIDE/ SUMP/ RR ROW</td>
<td>4' DIA C 0.425</td>
<td>1.0</td>
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<td>0.0</td>
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<tr>
<td>Section</td>
<td>Location</td>
<td>VPC</td>
<td>QC</td>
<td>QW</td>
<td>WF</td>
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</tr>
<tr>
<td>A</td>
<td>West 2.5' Sump</td>
<td>11.0</td>
<td>1.0</td>
<td>0.0</td>
<td>32.9</td>
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<tr>
<td></td>
<td>NE 2.5' Sump</td>
<td>6.0</td>
<td>6.0</td>
<td>0.0</td>
<td>22.9</td>
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<td>1.2</td>
<td>1.2</td>
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<tr>
<td></td>
<td>N 2.5' Sump</td>
<td>1.0</td>
<td>1.0</td>
<td>0.0</td>
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<tr>
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<td>NE 2.5' Sump</td>
<td>10.3</td>
<td>10.3</td>
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<td>1.0</td>
<td>0.0</td>
<td>14.5</td>
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<tr>
<td></td>
<td>Sump</td>
<td>2.4</td>
<td>2.4</td>
<td>0.0</td>
<td>35.7</td>
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<tr>
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<td>NE 2.5' Sump</td>
<td>1.6</td>
<td>1.6</td>
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<tr>
<td></td>
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<td>Q_{out} (cu. ft.)</td>
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<td>Q&lt;sub&gt;out&lt;/sub&gt; (cfs)</td>
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MADISON/WAHSATCH DRAINAGE IMPROVEMENT STUDY

APPENDIX E

PROPOSED STORM SEWER IMPROVEMENTS AND HYDRAULICS
Values of $r_{full}$ and $r_{nom}$

$A_{nom}$ = Area of pipe (ft$^2$)
$A_{full}$ = Area occupied by flow (ft$^2$)
$v$ = Actual velocity of flow (fps)
$v_{nom}$ = Velocity flowing full (fps)
$q$ = Actual quantity of flow (cfs)
$q_{nom}$ = Capacity flowing full (cfs)
$r$ = Actual hydraulic radius (ft.)
$R_{nom}$ = Hydraulic radius of full pipe (ft.)

FIGURE 9

HYDRAULIC ELEMENTS OF CIRCULAR CONDUITS
<table>
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<th>Location</th>
<th>Q in.</th>
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<th>C = 0.85</th>
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<th>V = 1.5</th>
<th>V = 2.0</th>
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<td>21° 15° 85.77 .98 6.59 6.46 6.0</td>
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**Note:** Represents total flow in main line of Jackson storm sewer.
<table>
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<th>TO</th>
<th>CDRE</th>
<th>DIAM</th>
<th>QG4</th>
<th>QG7</th>
<th>QG9</th>
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<th>V7</th>
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<td>.90</td>
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*NOTE: REPRESENTS TOTAL FLOW IN MAIN LINE OF JACKSON STORM SEWER*

| INLET & RR Roux + Jackson | 1.0   | 50.1 | 61' | 1585 | .98 | 10.4 | 6.69 | 6.9 | 30'    |
| MH @ RR Roux + Jackson   | 1.0   | 15.6 | 15.6| 1585 | .98 | 10.4 | 6.69 | 6.9 | 30'    |

TOTAL @ RR Roux + Jackson 97.5
**Madison ST Basin A-A**

Including Storm Sewer in Offsite

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<th>Q, in.</th>
<th>Q, cu. ft.</th>
<th>V, cu. ft.</th>
<th>V, ft</th>
<th>Length</th>
<th>Notes</th>
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<td>TO</td>
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<td>1.80</td>
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**Note:** *Represents total flow in main line of Manhole @ Morgan + NE side.

**Design Calculations**

Project: Madison/Weinacht

Detail:

Designer: Date: 03/15/95 Sheet: 8 of 20
Checker: Date: 03/15/95 Sheet: 8 of 20

Job No: 387-001
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<th>DI m</th>
<th>Q cu ft</th>
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<td>1.0</td>
<td>16.2</td>
<td>24.2</td>
<td>22.60</td>
<td>80.0</td>
<td>90.0</td>
<td>7.00</td>
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<tr>
<td>MH @ MAECKENZIE + MONROE</td>
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<td></td>
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<tr>
<td>INLET SOUTH SIDE MONROE &amp; MANCHESTER</td>
<td>1.0</td>
<td>25.02</td>
<td>27.00</td>
<td>0.11</td>
<td>0.90</td>
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<td>8.85</td>
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<td>MH + ROEVER + MAECKENZIE</td>
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<td>MH @ ROEVER + MAECKENZIE</td>
<td>0.1</td>
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<td>1.0</td>
<td>9.44</td>
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<td>MH @ EL PASO + MASON</td>
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<td>21&quot;</td>
<td>13.26</td>
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<tr>
<td>MH @ ROEVER + MAECKENZIE</td>
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<tr>
<td>Location</td>
<td>Sump</td>
<td>Qin</td>
<td>Dia</td>
<td>Qin</td>
<td>V/4</td>
<td>V/4</td>
<td>Vps</td>
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<td>12&quot;</td>
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<tr>
<td>TO MAHOLE @ MADISON + ROYER</td>
<td>0.3</td>
<td>99.2</td>
<td>64</td>
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<td>92</td>
<td>1.04</td>
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<tr>
<td>INLET @ SW CORNER, CORNER + MADISON</td>
<td>1.0</td>
<td>4.16</td>
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<td>10.5</td>
<td>7.8</td>
<td>5.94</td>
<td>5.8</td>
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<tr>
<td>TO MAHOLE @ MADISON + ROYER</td>
<td>0.3</td>
<td>1015</td>
<td>54</td>
<td>1011</td>
<td>1.0</td>
<td>6.71</td>
<td>7.0</td>
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* Note: Flow area varies from 11.0 to 16.0 depending on flow rate at Madison.
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<th>Qin</th>
<th>Dia</th>
<th>Qout</th>
<th>G4</th>
<th>V/</th>
<th>V</th>
<th>Length</th>
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<td>10.3</td>
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<tr>
<td>TO</td>
<td>0.4</td>
<td>11724</td>
<td>54&quot;</td>
<td>103.7</td>
<td>9.4</td>
<td>103</td>
<td>7.82</td>
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<td>80'</td>
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<td>INTLET ON DR</td>
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<td>TO</td>
<td>0.15</td>
<td>16094</td>
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<td>R.I.</td>
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<tr>
<td>TO</td>
<td>0.4</td>
<td>11724</td>
<td>54&quot;</td>
<td>103.7</td>
<td>9.4</td>
<td>103</td>
<td>7.82</td>
<td>8.1</td>
<td>80'</td>
</tr>
<tr>
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## Washington/Warsatch to FR Row

<table>
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<tr>
<th>Location</th>
<th>% Grade</th>
<th>Q in</th>
<th>Q out</th>
<th>Q50</th>
<th>V/4</th>
<th>V</th>
<th>Vp</th>
<th>Length</th>
<th>Notes</th>
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<tbody>
<tr>
<td>Inlet &amp; N.E. corner Washington/Warsatch to</td>
<td>1.0</td>
<td>9.0</td>
<td>8.0</td>
<td>8.0</td>
<td>8.0</td>
<td>35'</td>
<td>10' Dior, Qin=9.0</td>
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<tr>
<td>Inlet &amp; SE corner Washington/Warsatch</td>
<td></td>
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<td>8' Dior, Qin=8.0</td>
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<tr>
<td>Inlet &amp; NW corner Washington/Warsatch move into line from east side</td>
<td>0.3</td>
<td>57.5</td>
<td>82.897</td>
<td>0.95</td>
<td>10.0</td>
<td>5.0</td>
<td>300'</td>
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**Note:** A represents total flow to main line of Washington.
### DESIGN CALCULATIONS

#### Fontanero & Railroad Row

<table>
<thead>
<tr>
<th>Location</th>
<th>Qa (%)</th>
<th>Qa CES</th>
<th>Dia (in)</th>
<th>Qa (ft³/s)</th>
<th>V1</th>
<th>V2</th>
<th>V3</th>
<th>Length ft</th>
<th>Notes</th>
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</thead>
<tbody>
<tr>
<td>Inlet @ Heizer</td>
<td>1.0</td>
<td>14.77</td>
<td>23.15</td>
<td>15.65</td>
<td>0.93</td>
<td>1.02</td>
<td>959</td>
<td>100</td>
<td>6.71</td>
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<tr>
<td>Fontanero &amp; Laramie</td>
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<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>To</td>
<td>1.0</td>
<td>14.4</td>
<td>27</td>
<td>8047</td>
<td>96</td>
<td>1.04</td>
<td>7.79</td>
<td>30</td>
<td>8.1</td>
</tr>
<tr>
<td>Inlet @ Hill</td>
<td>1.0</td>
<td>209.6</td>
<td>27</td>
<td>8047</td>
<td>96</td>
<td>1.04</td>
<td>7.79</td>
<td>30</td>
<td>8.1</td>
</tr>
<tr>
<td>Fontanero &amp; Laramie</td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>To</td>
<td>1.0</td>
<td>209.6</td>
<td>27</td>
<td>8047</td>
<td>96</td>
<td>1.04</td>
<td>7.79</td>
<td>30</td>
<td>8.1</td>
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<tr>
<td>Hill @ Fontanero &amp; RR Row</td>
<td>1.0</td>
<td>8.8</td>
<td>18</td>
<td>10.52</td>
<td>0.84</td>
<td>0.5</td>
<td>0.94</td>
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<tr>
<td>Yard Inlets</td>
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<td>8.8</td>
<td>18</td>
<td>10.52</td>
<td>0.84</td>
<td>0.5</td>
<td>0.94</td>
<td>59</td>
<td>59</td>
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<tr>
<td>To Inlet on North Side</td>
<td>1.0</td>
<td>8.8</td>
<td>18</td>
<td>10.52</td>
<td>0.84</td>
<td>0.5</td>
<td>0.94</td>
<td>59</td>
<td>59</td>
</tr>
<tr>
<td>Fontanero &amp; RR Row</td>
<td>0.8</td>
<td>14.0</td>
<td>24</td>
<td>100</td>
<td>0.88</td>
<td>1.01</td>
<td>959</td>
<td>5.1</td>
<td>260</td>
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<tr>
<td>Inlet @ Heizer</td>
<td>0.8</td>
<td>14.0</td>
<td>24</td>
<td>100</td>
<td>0.88</td>
<td>1.01</td>
<td>959</td>
<td>5.1</td>
<td>260</td>
</tr>
<tr>
<td>Fontanero &amp; RR Row</td>
<td></td>
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<td></td>
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<td></td>
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<td></td>
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</tr>
<tr>
<td>To</td>
<td>1.0</td>
<td>18</td>
<td>1.13</td>
<td>1046</td>
<td>10</td>
<td>1.51</td>
<td>0.99</td>
<td>3.2</td>
<td>30</td>
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<tr>
<td>Fontanero &amp; RR Row</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Notes:**
- 62' Dior, Qa = 14.7
- 93' Dior, Qa = 14.9
- 1.0' Dior, Qa = 14.0
- 1.4' Dior, Qa = 14.0
- 1.0' Dior, Qa = 14.0
- 1.0' Dior, Qa = 14.0

**TOTAL FLOW ENTERING @ Fontanero:** 53.4
### Design Calculations

**Project:** Madison/Esparaza
**Designer:**
**Checker:**

<table>
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<tr>
<th>County</th>
<th>S.R.</th>
<th>Q in.</th>
<th>DIA</th>
<th>Q_G</th>
<th>V</th>
<th>R</th>
<th>Length</th>
<th>Notes</th>
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<td>INLET @ NE Corner ESPANOLA</td>
<td>1.0</td>
<td>8.2</td>
<td>18&quot;</td>
<td>10.5</td>
<td>78</td>
<td>98</td>
<td>5.94</td>
<td>5.8</td>
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<tr>
<td>ESPANOLA - RR ROW</td>
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<td>9.2</td>
<td>18&quot;</td>
<td>10.5</td>
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<td>101</td>
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<tr>
<td><strong>TOTAL</strong></td>
<td>1.2</td>
<td>9.2</td>
<td>18&quot;</td>
<td>11.5</td>
<td>80</td>
<td>99</td>
<td>6.51</td>
<td>6.5</td>
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</tbody>
</table>

- **NOTE:**
  - Storm sewer @ ESPANOLA.
## DESIGN CALCULATIONS

### Del Norte + Railroad ROW

<table>
<thead>
<tr>
<th>Diameter</th>
<th>Inlet to NE Corner of Del Norte to Existing 54&quot; in Wahsatch</th>
<th>2 Yard Inlets in Row to Manhole in Row</th>
<th>Carmanillo + Existing Storm Sewer in Wahsatch</th>
<th>Buena Ventura + Storm Sewer in Row</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1.4</td>
<td>1.0</td>
<td>6.0</td>
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<tr>
<td></td>
<td>0.0 18&quot; 0.4 0.47 0.86 7.03 0.0 50'</td>
<td>1.6 2.2 18&quot; 10.28 21 0.66 5.94 3.9 40'</td>
<td>1.5 10.3 18&quot; 10.5 9.8 1.04 5.94 5.2 80'</td>
<td>1.0 25 18&quot; 10.5 2.4 0.69 5.94 4.1 80'</td>
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<tr>
<td></td>
<td></td>
<td>8.2 (6.05) TOTAL FLOW ENTERING MANHOLE AT DEL NORTE + RAILROAD ROW</td>
<td>4&quot; DIOR Gain 7.9</td>
<td>4&quot; DIOR Gain 7.9</td>
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</table>

### Notes
- Convert to Grade Rule
- Convert to Grade Rule
- Convert to Grade Rule
- Convert to Grade Rule
<table>
<thead>
<tr>
<th>Location</th>
<th>Shape</th>
<th>QA</th>
<th>Dia (in)</th>
<th>Q (cu ft/sec)</th>
<th>V (ft/sec)</th>
<th>V (fps)</th>
<th>Length (ft)</th>
<th>Notes</th>
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<td>INLET @ NW CORONA TO SS IN ROW</td>
<td>1.0</td>
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<td>160</td>
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<td>3.1</td>
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<td>1.0</td>
<td>1.5</td>
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<td>1050</td>
<td>10</td>
<td>5.94</td>
<td>3.1</td>
<td>20</td>
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<td>INLET ON SE SIDE CORONA TO COLUMBIA RUN INTO STORM SEWER</td>
<td>1.0</td>
<td>1.62</td>
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<td>2212</td>
<td>7.2</td>
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<td>YARD INLETS E SHADY GLEN IN ROW</td>
<td>1.0</td>
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<td>18&quot;</td>
<td>1050</td>
<td>1.0</td>
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<td>5.9</td>
<td>50</td>
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<tr>
<td>YARD INLETS E EDGEWORTH IN ROW</td>
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<td>1.7</td>
<td>18&quot;</td>
<td>1050</td>
<td>1.0</td>
<td>5.94</td>
<td>3.6</td>
<td>50</td>
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</tbody>
</table>
**INTERSECTION OF JACKSON AND RAILROAD ROW**

**AREA CONTRIBUTING**

**OFFSITE BASIN**: 55.4 ft², 0.162, 75.5

**Basin #** | **Area (ft²)** | **100 Yr Runoff (in)** | **Weighted CN**
--- | --- | --- | ---
20 | 0.0049 | 75 | 76
20A | 0.0058 | 75
21 | 0.0056 | 75
22 | 0.0063 | 75
23 | 0.0114 | 75
24 | 0.0136 | 75
25 | 0.0087 | 75
26 | 0.0087 | 79

**TOTAL**: 0.0818

**TIME TO TRAVEL IN PIPE (t_c)**

\[ t_c = \frac{0.261 + 0.054}{0.321} = 0.91 \text{ in} \]

@ **t_c = 0.321 in**

\[ CSM/n = 9.10 \text{ FIGURE 6} \]

@ **CN = 76**

**100 YR RUNOFF**: 1.82 in

\[ Q_{100} = (\text{Area})(CSM/n)(100 \text{ yr runoff}) \]

\[ = 0.0818 \times 9.10 \times 1.31 = 97.5 \text{ CFS} \]

**QDP**

\[ \text{DIA} \quad \text{Q} \quad Q/\text{ft} \quad V/\text{ft} \quad V \quad V \text{ LENGTH} \quad SLOPE \]

97.5 | 48" | 126.48 | 6.76 | 0.86 | 0.86 | 10.0 | 940 | 0.80

**New t_c**: 0.321 + \( \frac{0.940}{(0.913 \times 0.60)} \) = 0.347

**Q_{DPP}**: 100 YR FLOW @ THE DESIGN POINT IN RAILROAD ROW
## INTERSECTION OF MADISON & RAILROAD ROW

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<th></th>
<th>0.0018</th>
<th>76</th>
<th>75.5</th>
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<td>OFFSET P-A</td>
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<td>B A S H A</td>
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<td>0.011</td>
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<td>0.1811</td>
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<tr>
<td>TOTAL =</td>
<td>0.1012 SM</td>
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</table>

Tc CALCULATED AT THIS INTERSECTION = 0.347

G Tc = 0.347

<SM/Vn = 880

Figure = 6

**WEIGHTED CURVE NUMBER: J A C K S O N = 76**

2.0418 / 0.021 = 71.595(3rd) 71.595(3rd) + 76.0(880) = 138.5871.11 = 75

G CURVE NUMBER = 75

100 YL EROSION = 1.3  P. W. = 17

Q = 0.1811(1.3)(880) = 307.2 FT.

Q = 0.347 + 880 / 307.2

New Tc = 0.347 + 2.826 = 0.8269
**DESIGN CALCULATIONS**

**Project:** N/W  
**Detail:** N/W  
**Designer:** N/A  
**Date:**  
**Checker:** N/A  
**Date:**  
**Job No:** 307,600

**INTERSECTION OF WASHINGTON + FR FNL**

**AREA CONTRIBUTING**
- OFFSET BASIN A-F:
- JAXSON (PAWN F):
- MADISON (BASIN A):

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<th>Contrib</th>
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</tr>
<tr>
<td>47</td>
<td>0.74</td>
<td>0.0296</td>
</tr>
<tr>
<td>58</td>
<td>0.10</td>
<td></td>
</tr>
<tr>
<td>90</td>
<td>0.11</td>
<td></td>
</tr>
<tr>
<td><strong>Total Area</strong>: 0.2134 SM</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\[
Q_c = 0.369
\]
\[
C_s = 6.369
\]

**WEIGHTED CONTRIBUTIONS**

\[
\begin{align*}
\text{BASIN E:} & = 76 \quad (0.0818 SM) \\
\text{BASIN F:} & = 76 \quad (0.0796 SM) \\
\text{TOTAL:} & = 76 \quad (0.1614 SM)
\end{align*}
\]

100 YR RUNOFF = 1.36

\[
Q_{100} = (0.2134) (1.36) = 0.28488 \text{ CFS}
\]

Qp  Dia  C/S  G/S  Y/F  L/\n
\[
\begin{align*}
2449 & \quad 60^\circ & 60^\circ A & 0.94 & 12.26 & 1.02 & 13.71 & 470 & 1.00
\end{align*}
\]

NEW \[Q_c = 0.369 - \frac{470}{(0.369 SM)} = 0.379 \text{ HR} \]
DESIGN CALCULATIONS

Project: N/11
Detail: F.M.E.

Designer: N/A
Date: N/A
Sheet #: N/A
Checker: N/A
Date: N/A
Job No: N/A

\( Q_{c} = 0.849 \)  
\( Q_{w} = 0.881 \)  
\( Q_{m} = 0.0296 \)  
\( Q_{f} = 0.0389 \)

\( R_{c} = 0.6 \)
\( R_{w} = 0.6 \)
\( R_{m} = 0.6 \)
\( R_{f} = 0.6 \)

\( Q_{t} = 0.2555 \)

\( \frac{Q_{c}}{Q_{w}} = 0.379 \)
\( \frac{Q_{w}}{Q_{m}} = 8.40 \)
\( \frac{Q_{m}}{Q_{f}} = 1.0 \)

\( W = 8.40 \times 79.9 = 74.5 \)

\( Q_{t} = 25.0 \times 4.8 \times 130 = 79.1 \)

\( \frac{Q_{c}}{Q_{w}} = 0.379 + \frac{510}{1470} = 0.389 \)}
### DESIGN CALCULATIONS

**Project:** CAM SEWER SPRING M.E.W.  
**Designer:** N.H.  
**Checker:**  
**Date:**  
**Sheet No.:**  
**Job No.:**

---

**INTERSECTION OF ESPANOLA AND SODIUM SEWER IN ROW**

<table>
<thead>
<tr>
<th>AREA</th>
<th>OFFSET A-P</th>
<th>75.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>JACKSON</td>
<td>0.0818</td>
<td>76</td>
</tr>
<tr>
<td>MADISON</td>
<td>0.0276</td>
<td>77</td>
</tr>
<tr>
<td>WASHINGTON</td>
<td>0.0329</td>
<td>75</td>
</tr>
<tr>
<td>FONTAINE</td>
<td>0.0415</td>
<td>71</td>
</tr>
</tbody>
</table>

**ESPARANOLA Areas Cn**

| 59         | .0029  | 79   |
| 39         | .0020  | 60   |
| 41         | .0011  | 34   |
| 50         | .0081  | 61   |
| 60         | .0045  | 61   |
| 60         | .0160  | 61   |

**WEIGHTED CN = 63**

**Total Area = 0.2741**

\[C = 0.389\]

\[CSM/in = 880\] FIGURE 6

**WEIGHTED CN FOR ENTIRE BASINS CONTRIBUTING**

\[\text{RS=0.2741} \]

\[\text{100 YR ROOF R=1.25 in} \] FIGURE 7

\[Q_{100} = 0.2741 (1.25)(880) = 289.4 \text{ cfs} \]

\[Q_{DR} = 289.4 \text{ cfs} \]

\[Q/Q = 0.74\]

\[V/U = 97\]

\[U_a = 16.12\]

**LENGTH SLOPE TO DEL.**

\[\text{1.2\%} \]

**New t_c = 0.389 + \frac{500}{(15.6)(0.03)} = 0.899 HR**
INTERSECTION OF DEL NORTE + STORM SOUFF IN ROW

YARD INLET CONTRIBUTES ONLY
THE FLOW IN INTERSECTION WILL BE CARRIED IN
DOWN WASHBATCH

OFF WASHBATCH:
       JACOB     0.018    76
       MADISON   0.076    71
       WASHINGTON 0.039   76
       FOOTHILLS 0.045    71
       ESPANOLA  0.016    63

DEL NORTE:  30.0022   68

TOTAL AREA 0.2763SM

@ Q = 0.399    CSM/m: 820   Figure 6

WEIGHTED CN FOR ENTIRE BASIN CONTRIBUTING = 73.6

100 YR RUNOFF = 1.2 in   Figure 7
Q100 = 0.2763(820)1.2 = 271.88 CFS

Q:N 6.6"   Q:LI 818.58 0.88  1.00  15.41 13.4  5SD: 0.90

NEW Tc = 0.399 + 5SD = 0.410 HR
INTERSECTION OF CALAMITO + RAILROAD ROW

Flow at intersection of Calamito + Wahsatch will be intercepted by existing storm sewer.

The only flow intercepted by proposed storm sewer is from yard north basin 51.

<table>
<thead>
<tr>
<th>Area</th>
<th>0.049</th>
<th>0.001</th>
<th>0.018</th>
<th>0.029</th>
<th>0.036</th>
<th>0.040</th>
<th>0.080</th>
<th>0.082</th>
</tr>
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<tbody>
<tr>
<td>Jackson</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Madison</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Washington</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Portland</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Español</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Del Norte</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Calamito 0.0029, 64.5

@ tc = 0.410, CSW/m = 800

Fig. 6

Weighted CN value for contributing basins = 73.6

100 Yr. Runoff = 1.25 in. Fig. 7

Q_{100} = 0.212x(800)(1.25) = 279.2 CF/s

Q_{95.2} DIA 96 in. q = .79 V_{95.2} = 14.88 ft V = 14.5 ft

Length slope

New tc = 0.410 + 480

(145)(500) = 0.219 HRs
INTERSECTION OF BUENA VENTURA RAILROAD ROW

<table>
<thead>
<tr>
<th>Area</th>
<th>ENSA A-A</th>
<th>0.44</th>
<th>76.5</th>
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<tbody>
<tr>
<td></td>
<td>JACKSON</td>
<td>0.81B</td>
<td>75.6</td>
</tr>
<tr>
<td></td>
<td>MADISON</td>
<td>0.296</td>
<td>71.1</td>
</tr>
<tr>
<td></td>
<td>Washington</td>
<td>0.329</td>
<td>76</td>
</tr>
<tr>
<td></td>
<td>FREDERICK</td>
<td>0.045</td>
<td>71</td>
</tr>
<tr>
<td></td>
<td>ESPANOLE</td>
<td>0.016</td>
<td>63</td>
</tr>
<tr>
<td></td>
<td>DEL NARTE</td>
<td>0.002</td>
<td>68</td>
</tr>
<tr>
<td></td>
<td>CRANSTON</td>
<td>0.029</td>
<td>64.5</td>
</tr>
</tbody>
</table>

TOTAL = 0.877

\[ e_t = 0.419 \quad C_S = 7.95 \quad \text{Figure 6} \]

100 yr. Runoff = 1.2 \quad \text{Figure 7}

\[ Q_{100} = (1.2808 \times 1.218) = 267.9 \quad \text{CFS} \]

\[ Q_{DR} = 0.74 \quad Q_{OS} = 0.76 \quad V_{OS} = 0.98 \quad V_{DR} = 14.8 \quad 14.5 \quad 750 \quad 11 \%
\]

New \[ e_t = 0.419 - \frac{750}{(1.5 \times 800)} = 0.498 \quad \text{HRS} \]
**INTERSECTION OF CORONA AND RENEWAL FLOW**

<table>
<thead>
<tr>
<th>AREA</th>
<th>Unit</th>
<th>Flow</th>
<th>los</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jackson</td>
<td>0.0618</td>
<td>76</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maricopa</td>
<td>0.0216</td>
<td>71</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Washington</td>
<td>0.0831</td>
<td>76</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fontenelle</td>
<td>0.0415</td>
<td>71</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Emanuele</td>
<td>0.0189</td>
<td>63</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Devereaux</td>
<td>0.0024</td>
<td>62</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Corona</td>
<td>0.0029</td>
<td>64</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bola Venata</td>
<td>0.0046</td>
<td>61</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Corona 55 0.0021 61
64 0.0021 61
63 0.0077 61
64 0.0049 61
66 0.0137 61

Total Area = 0.3162 SM

@ tc = 0.433 HR  CSM/N = 790  Figure 6

Weighted CN Value for Contributing Basins = 72.1

100 YR Runoff = 1.15 in  Figure 7

Q_{100} = \left( 0.3162 \times 1.15 \times 790 \right) = 287.2 \text{ cfs}

Q_{100} \text{ DIA} \quad Q/Q_4 \quad V/V_4 \quad V_4 \quad V \quad \text{LENGTH} \quad \text{Slope}

287.3 66\" 350.2 .82 1.00 14.82 14.8 2800 1.1

TO SHOcks' RUN

\text{New } \tau_c = 0.433 + \frac{2800}{\left(14.8 \times 3600\right)} = 0.486

\text{Q}_{100} \text{ @ OUTFALL} = 287.3 \text{ cfs}
<table>
<thead>
<tr>
<th>Location</th>
<th>Slope</th>
<th>Q in CF/s</th>
<th>DIA</th>
<th>Q in CF/s</th>
<th>Ys</th>
<th>Yh</th>
<th>Yv</th>
<th>V</th>
<th>L (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0% GROSS</td>
<td>0.90%</td>
<td>10</td>
<td>21</td>
<td>1308</td>
<td>0.07</td>
<td>0.80</td>
<td>0.56</td>
<td>450</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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</tr>
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</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.67%</td>
<td>675</td>
<td>24</td>
<td>2223</td>
<td>0.94</td>
<td>1.03</td>
<td>0.31</td>
<td>9.59</td>
<td>36.0</td>
</tr>
</tbody>
</table>
CHECK for inlet or outlet control

IF \( \text{actual depth} \frac{H}{S} < \text{critical depth} \frac{H_c}{S_c} \),

INLET control assumed

USE \( \alpha \text{mm flowing} \) from City for channel parameters

\[
\begin{align*}
\alpha &= 0.060 \text{ (field erosion)} \\
S &= 0.01 \text{ ft/ft} \\
B &= 0.3 \text{ ft} \\
\frac{H_c}{S_c} &= 2.8 \\
Q &= 2.905 \text{ cfs}
\end{align*}
\]

USE "HIT" Hitting Program

\[
\begin{align*}
Q &= 2.905 \text{ cfs} \\
\alpha &= 1.3 \quad (\text{city pg 48}) \\
\gamma &= 2.8 \\
\gamma_c &= 4.41 \\
\frac{H_c}{S_c} &= 7.84 \text{ ft}
\end{align*}
\]

Since \( \frac{H_c}{S_c} \) for INLET control calculation should give close approximation of OILMAX.
**DESIGN CALCULATIONS**

\[ Q = C_d A_0 \sqrt{2gh} \quad \text{for wide channel} \]

\[ 0.98C_e \geq C_d \geq 0.95C_e \]

\[ C_d = 0.95C_e \]

Assume square edge opening: \( C_e = 0.82 \)

\[ C_d = 0.95(0.82) = 0.78 \]

\[ A \approx 2.11 \text{ SF (planimeter or scale drawing)} \]

\[ h = 12.5 \text{ ft} \left( \frac{4}{12} = 1 \right) \]

\[ Q = 0.78(2.11)\sqrt{2(32.2)(12.5)} = 4.469 \text{ cfs} \]

Cache LA possible crossing capacity of
MADISON/WAHSATCH DRAINAGE IMPROVEMENT STUDY

APPENDIX F

COST ESTIMATE
<table>
<thead>
<tr>
<th>SUBBASIN</th>
<th>COST ($)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>$200,965</td>
</tr>
<tr>
<td>B</td>
<td>363,090*</td>
</tr>
<tr>
<td>C</td>
<td>36,080</td>
</tr>
<tr>
<td>D</td>
<td>88,160*</td>
</tr>
<tr>
<td>E</td>
<td>16,570</td>
</tr>
<tr>
<td>F</td>
<td>48,220</td>
</tr>
<tr>
<td>G</td>
<td>5,000</td>
</tr>
<tr>
<td>H</td>
<td>28,000</td>
</tr>
<tr>
<td>I</td>
<td>180,610</td>
</tr>
<tr>
<td>J (Storm Sewer Trunk Line)</td>
<td>1,382,680</td>
</tr>
<tr>
<td>Offsite Basin AA</td>
<td>442,370*</td>
</tr>
<tr>
<td>Subtotal</td>
<td>2,791,745</td>
</tr>
<tr>
<td>Contingencies (10 Percent)</td>
<td>279,175</td>
</tr>
<tr>
<td>Total Estimated Cost**</td>
<td>$3,070,920</td>
</tr>
</tbody>
</table>

*See Subbasin Cost Estimate

**No utility relocation costs are included.
<table>
<thead>
<tr>
<th>SUBBASIN</th>
<th>ITEM</th>
<th>DESCRIPTION</th>
<th>QUANTITY</th>
<th>UNIT</th>
<th>COST ($)</th>
<th>COST ($)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1.</td>
<td>4 Foot DIOR Inlet</td>
<td>4</td>
<td>EA</td>
<td>2,000.00</td>
<td>8,000.00</td>
</tr>
<tr>
<td></td>
<td>2.</td>
<td>6 Foot DIOR Inlet</td>
<td>1</td>
<td>EA</td>
<td>2,500.00</td>
<td>2,500.00</td>
</tr>
<tr>
<td></td>
<td>3.</td>
<td>8 Foot DIOR Inlet</td>
<td>2</td>
<td>EA</td>
<td>3,000.00</td>
<td>6,000.00</td>
</tr>
<tr>
<td></td>
<td>4.</td>
<td>10 Foot DIOR Inlet</td>
<td>1</td>
<td>EA</td>
<td>3,500.00</td>
<td>3,500.00</td>
</tr>
<tr>
<td></td>
<td>5.</td>
<td>14 Foot DIOR Inlet</td>
<td>1</td>
<td>EA</td>
<td>4,200.00</td>
<td>4,200.00</td>
</tr>
<tr>
<td></td>
<td>6.</td>
<td>22 Foot DIOR Inlet</td>
<td>2</td>
<td>EA</td>
<td>5,200.00</td>
<td>10,400.00</td>
</tr>
<tr>
<td></td>
<td>7.</td>
<td>18 Inch RCP</td>
<td>145</td>
<td>LF</td>
<td>25.00</td>
<td>3,625.00</td>
</tr>
<tr>
<td></td>
<td>8.</td>
<td>21 Inch RCP</td>
<td>100</td>
<td>LF</td>
<td>27.00</td>
<td>2,700.00</td>
</tr>
<tr>
<td></td>
<td>9.</td>
<td>24 Inch RCP</td>
<td>260</td>
<td>LF</td>
<td>30.00</td>
<td>7,800.00</td>
</tr>
<tr>
<td></td>
<td>10.</td>
<td>27 Inch RCP</td>
<td>210</td>
<td>LF</td>
<td>38.00</td>
<td>7,980.00</td>
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<tr>
<td></td>
<td>11.</td>
<td>33 Inch RCP</td>
<td>300</td>
<td>LF</td>
<td>50.00</td>
<td>15,000.00</td>
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<tr>
<td></td>
<td>12.</td>
<td>54 Inch RCP</td>
<td>370</td>
<td>LF</td>
<td>118.00</td>
<td>43,660.00</td>
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<td></td>
<td>13.</td>
<td>48 Inch Manhole</td>
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<td>EA</td>
<td>1,800.00</td>
<td>1,800.00</td>
</tr>
<tr>
<td></td>
<td>14.</td>
<td>Box Base Manhole</td>
<td>1</td>
<td>EA</td>
<td>3,500.00</td>
<td>3,500.00</td>
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<tr>
<td></td>
<td>15.</td>
<td>30 Foot Concrete Crossspan</td>
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<td>1,000.00</td>
<td>1,000.00</td>
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<tr>
<td></td>
<td>16.</td>
<td>Remove Bubbler Inlet</td>
<td>4</td>
<td>EA</td>
<td>1,000.00</td>
<td>4,000.00</td>
</tr>
<tr>
<td></td>
<td>17.</td>
<td>Road Base (Class 5)</td>
<td>1500</td>
<td>Ton</td>
<td>15.00</td>
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<tr>
<td></td>
<td>18.</td>
<td>Asphalt Paving</td>
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<td>Ton</td>
<td>48.00</td>
<td>52,800.00</td>
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Subtotal

$200,965
<table>
<thead>
<tr>
<th>SUBBASIN</th>
<th>ITEM</th>
<th>DESCRIPTION</th>
<th>QUANTITY</th>
<th>UNIT</th>
<th>UNIT COST ($)</th>
<th>COST ($)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td>1</td>
<td>4 Foot DIOR Inlet</td>
<td>2</td>
<td>EA</td>
<td>2,000.00</td>
<td>4,000</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>6 Foot DIOR Inlet</td>
<td>1</td>
<td>EA</td>
<td>2,500.00</td>
<td>2,500</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>10 Foot DIOR Inlet</td>
<td>1</td>
<td>EA</td>
<td>3,000.00</td>
<td>3,000</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>12 Foot DIOR Inlet</td>
<td>1</td>
<td>EA</td>
<td>4,000.00</td>
<td>4,000</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>20 Foot DIOR Inlet</td>
<td>4</td>
<td>EA</td>
<td>5,000.00</td>
<td>20,000</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>18 Inch RCP</td>
<td>20</td>
<td>LF</td>
<td>25.00</td>
<td>500</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>21 Inch RCP</td>
<td>90</td>
<td>LF</td>
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<td>2,430</td>
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<tr>
<td></td>
<td>8</td>
<td>24 Inch RCP</td>
<td>40</td>
<td>LF</td>
<td>30.00</td>
<td>1,200</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>27 Inch RCP</td>
<td>20</td>
<td>LF</td>
<td>38.00</td>
<td>760</td>
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Note: If non-uniform street sections are not installed in Corona and La Salle Streets the estimated subtotal cost is $190,890.
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Subtotal: $36,080
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Subtotal: $88,160

Note: If paving of Buena Ventura Street is not part of the improvements, reduce Subbasin D Subtotal estimated cost to $94,650.

| E        | 1.   | 4 Foot DIOR Inlet | 2        | EA    | 2,000.00      | 4,000   |
|          | 2.   | 8 Foot DIOR Inlet | 1        | EA    | 3,000.00      | 3,000   |
|          | 3.   | 18 Inch RCP      | 40       | LF    | 25.00         | 1,000   |
|          | 4.   | 24 Inch RCP      | 20       | LF    | 27.00         | 540     |
|          | 5.   | Road Base (Class 5) | 150      | Ton   | 15.00         | 2,250   |
|          | 6.   | Asphalt Paving    | 110      | Ton   | 48.00         | 5,280   |

Subtotal: $16,570
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<p>| J        | 1.       | 18 Inch RCP               | 800      | LF   | 25.00    | 20,000   |
|          | 2.       | 48 Inch RCP               | 940      | LF   | 98.00    | 92,120   |
|          | 3.       | 60 Inch RCP               | 1450     | LF   | 130.00   | 188,500  |
|          | 4.       | 66 Inch RCP               | 5590     | LF   | 150.00   | 838,500  |
|          | 5.       | Yard Inlet                | 21       | EA   | 2,000.00 | 42,000   |
|          | 6.       | Box Base Manhole          | 19       | EA   | 3,500.00 | 66,500   |
|          | 7.       | 8&quot; Curb &amp; Gutter         | 360      | LF   | 12.00    | 4,320    |
|          | 8.       | Road Base (Class 5)       | 1200     | TON  | 15.00    | 18,000   |
|          | 9.       | Asphalt Paving            | 900      | TON  | 48.00    | 43,200   |
|          | 10.      | Asphalt Bike Path         | 2250     | LF   | 10.00    | 22,500   |</p>
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<th>UNIT</th>
<th>UNIT COST ($)</th>
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Note: If these improvements are not installed, flooding will continue to occur for higher frequency design storms on Washatch Avenue between Madison Street and Fontanero Street.


