56th Avenue, Quebec Street to Havana Street Environmental Assessment

DRAINAGE REPORT

Prepared for:

City and County of Denver

in partnership with

US Department of Transportation
Federal Highway Administration

Colorado Department of Transportation

Prepared by:

URS Corporation
Denver, Colorado

May 2008

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1.0 GENERAL LOCATION AND DESCRIPTION

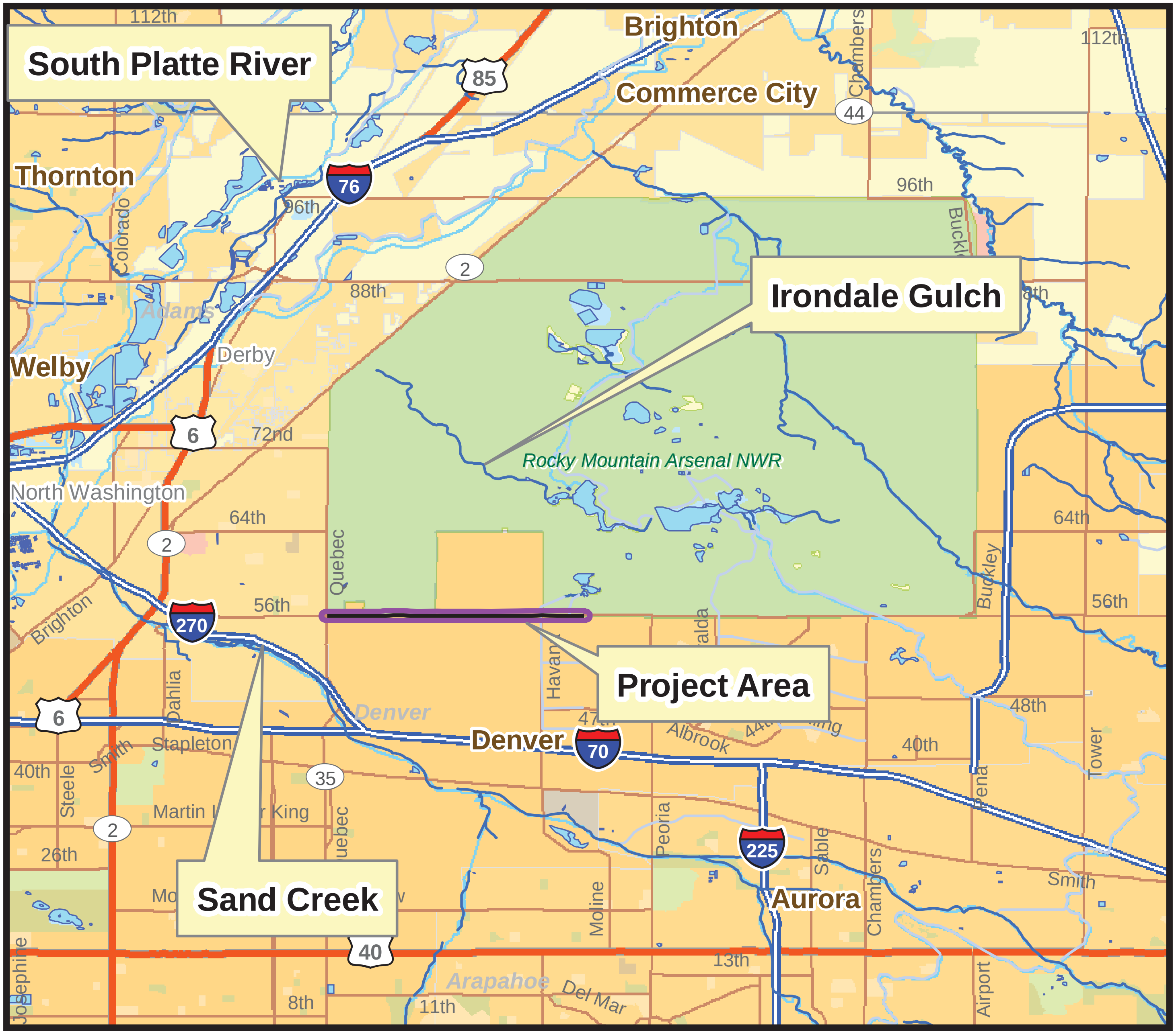
1.1 Description of Project

The 56th Avenue Roadway Improvement Project involves widening the road to accommodate six lanes of traffic, and adding new curb and gutter, a landscaped median, and detached multi-use paths with tree lawns. The planned expansion of the 56th Avenue corridor will extend from Quebec Street on the west to beyond Peña Boulevard on the east (see Figure 1-1). This corridor lies on the boundary between the City and County of Denver and Adams County.

Due to funding limitations, the 56th Avenue corridor is being divided into two segments. The first segment of the project, analyzed herein, begins east of Quebec Street and extends to the east side of the Havana Street intersection. The expansion and widening of this segment of the corridor includes approximately two miles of roadway and drainage improvements along 56th Avenue. The Prairie Gateway development, a U.S. Postal Service Bulk Mail facility, and a Denver Water facility lie on the north side along with the planned North Stapleton Development. The south side of the road is also part of the planned North Stapleton Development. Figure 1-2 shows the proposed roadway typical sections for this project segment.

This report presents drainage improvements proposed for the first segment of the corridor project and documents the analysis that forms the basis of the design. The primary project goal is to furnish storm sewer systems, stormwater retention, and permanent water quality Best Management Practices (BMPs) as required. Analysis of the on-site and off-site drainage basins affecting the project will be conducted to determine peak runoff discharges for use in design of structures to convey stormwater off the roadway, and to size retention and water quality facilities to be built as part of the project.

The 56th Avenue improvements are located in the South Platte River Basin, more or less on the ridgeline between the Sand Creek and Irondale Gulch watersheds. The terrain throughout the project area is flat to gently rolling with a predominant trend to slope to the north and west. Merrick & Company provided survey, obtained May 2007, for existing topography and utilities.

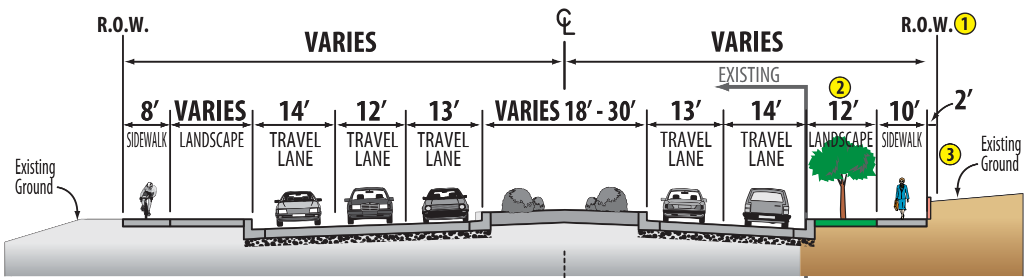


56th Avenue Environmental Assessment
Quebec Street to Havana Street

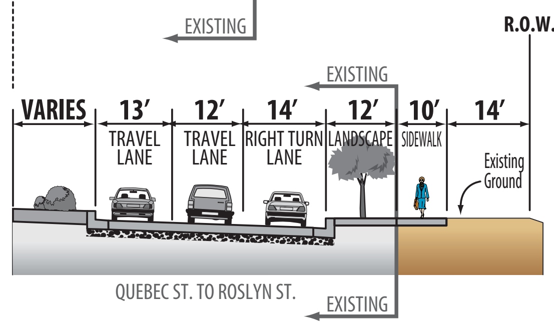
FIGURE 1 -1
Project Location

URS
DENVER THE MILE HIGH CITY

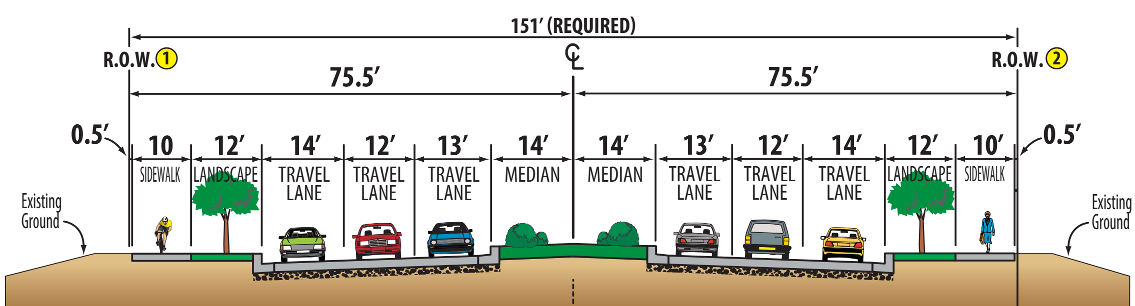
DESIGN CONCEPT: CONSTRUCT SIDEWALK
QUEBEC ST. TO SPRUCE ST., SOUTH SIDE
(APPLIES TO ALTERNATIVES 3 THROUGH 6)
TYPICAL SECTION, LOOKING EAST



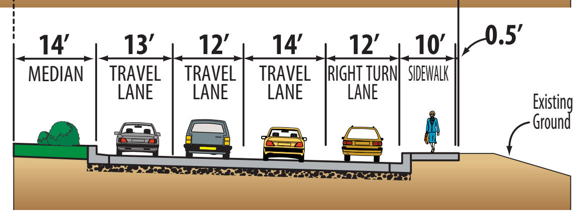
1. 0′ to 12.5′ (varies) of additional ROW required on south side of 56th Avenue for landscape buffer and sidewalk, Spruce Wy. to Valentia St.
2. 7′ landscape buffer in front of Fire Training Center Facility on south side of 56th Avenue.
3. Retaining wall behind sidewalk, as needed, Roslyn St. to Spruce Wy.



DESIGN CONCEPT: WIDEN TO 6 LANES
SPRUCE ST. TO HAVANA ST.
(APPLIES TO ALTERNATIVES 5 AND 7)
TYPICAL SECTION, LOOKING EAST



1. 50.5′ of additional ROW required on north side of 56th Avenue, east of Valentia St.to proposed Central Park Blvd. 10.5′ of additional ROW required on north side, Central Park Blvd. to Havana St.
2. 0.5′ of additional ROW required on south side of 56th Avenue, Valentia St. to Havana St.



Stormwater runoff along the corridor from Quebec Street to Havana Street flows generally to the west and north. Storm drain inlets and pipes, located in 56th Avenue between Quebec Street and Spruce Street, collect flows and convey them to the existing retention pond in the southeast corner of 56th Avenue and Quebec Street. From Spruce Street to future Verbena Street, existing inlets and pipes collect runoff and carry it to a 66" x 48" concrete box culvert located east of Spruce Street that conveys flows north through the Prairie Gateway Development. From future Verbena Street to the Union Pacific Railroad (UPRR) spur located west of Havana Street, there are a series of cross culverts ranging in size from 24" reinforced concrete pipe (RCP) to 30" RCP to twin 24" RCPs that drain to the north. These will be removed as part of the project. At Havana Street, drainage from the east side of the intersection is conveyed west to the Havana Interceptor ditch.

1.2 Project Features

Table 1-1
Project Features

Roadway & Length:	56th Avenue, from Quebec Street to Havana Street, 2 miles
Major Roadway Structures:	Six 24-inch RCP culverts, one 30-inch RCP culvert, Wildlife Crossing
Major Intersections:	Quebec Street, Roslyn Street, Spruce Street, Valentia Street, Central Park Boulevard (future street), Havana Street
Rivers:	Sand Creek to the southwest, Irondale Gulch to the north of project area.
Canals:	Havana Interceptor
County:	Denver and Adams County
Legal Description:	The project is in Sections 15 and 16, Township 03 South, Range 67 West of the 6th Principal Meridian.

1.3 Flood History

Documented floods or drainage problems within the Irondale Gulch watershed area downstream of the project are discussed in the Outfall Systems Plan, Reference 6. There are no reported flooding problems within the 56th Avenue project area.

2.0 MAJOR DRAINAGE BASINS AND SUB-BASINS

2.1 Major Basin Description

Four major drainage basins exist in or partially within the project area as defined by the *Denver Storm Drainage Master Plan*, Reference 9. These basins are Basin 0058-01 (Prairie Gateway), Basin 3900-01 (Irondale Gulch-Stapleton East Section 10), Basin 4000-01 (Stapleton West Section 10), Basin 4400-01 (North Stapleton). These areas are shown in Figure 2-1 and described in more detail below.

Basin 4400-01 (North Stapleton)

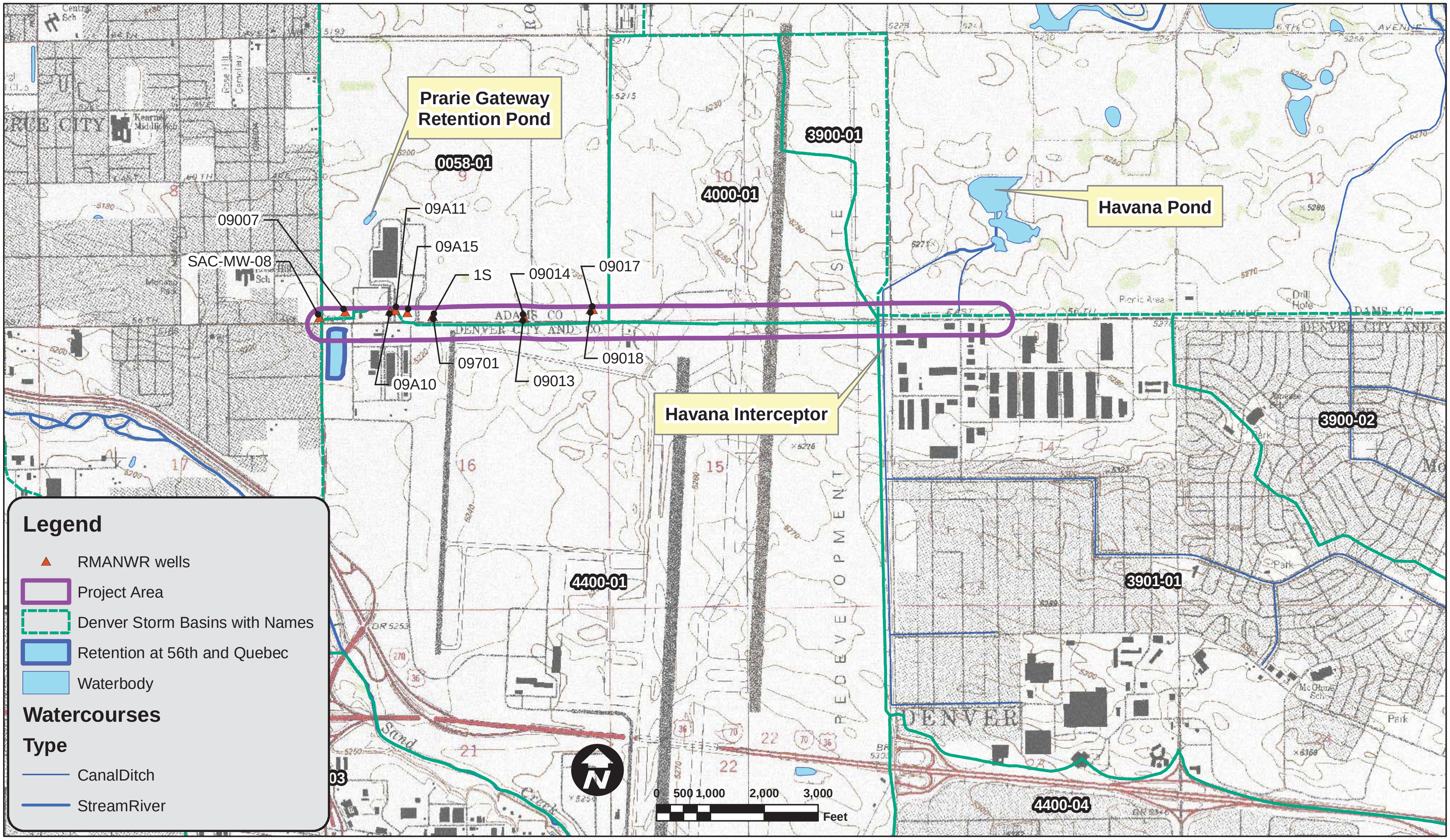
The basin consists of approximately 3,183 acres and is the former Stapleton airport, which will be completely redeveloped. The current plan for redevelopment is outlined in the *North Stapleton Infrastructure Master Plan* (NSIMP), Reference 8, and 56th Avenue is the northern boundary of the North Stapleton area between Spruce Street and Havana Street. A copy of the drainage plan is included in Appendix B.

Historically, the majority of the North Stapleton area drains to the north to Rocky Mountain Arsenal (RMA). The NSIMP changes historical drainage patterns in accordance with previous studies and agreements. The drainage from this basin will outfall to Sand Creek, a tributary to the South Platte River. Careful coordination and planning is required to construct the plan laid out in NSIMP; for example, this reorientation of drainage patterns requires a new 36-inch outfall under I-270. Additionally, multiple detention ponds are required to mitigate the flows from the 100-year storm event. Only the portion of 56th Avenue between Valentia Street and (future) Central Park Boulevard will discharge to the North Stapleton site.

Storm water management features shown in the NSIMP with respect to 56th Avenue are a detention pond that is currently planned for the southeast corner of Spruce Street and 56th Avenue. An open channel will be located immediately south of 56th Avenue from the detention pond to (future) Chester Street. Also, a detention pond is planned to support the commercial area that stretches from (future) Dallas Street to Havana Street. No storm water crossings are planned under 56th Avenue between Spruce Street and Havana Street.

Basin 4000-01 (Stapleton West Section 10)

The basin consists of approximately 498 acres and is also part of the former Stapleton airport. This basin is in the western portion of Section 10, and is largely undeveloped with the exception of the former runways and storage of crushed concrete. *The Stapleton Area Outfall System Plan,* Reference 7, provides details of the proposed redevelopment of Section 10.



Source: URS Corporation

56th Avenue Environmental Assessment
Quebec Street to Havana Street

FIGURE 2 - 1
Major Drainage Basins

This basin is located on the north side of 56th Avenue and west of Havana Street. In general, drainage flows northwest and outfalls toward Basin 0058-01 (Prairie Gateway) at 64th Avenue. Discharges from this basin into the RMANWR must be limited to historic conditions, and retention ponds must be provided with any future development. Current plans for new retention ponds for the development show them placed well north of 56th Avenue in the central and northern part of the basin. The portion of 56th Avenue between Havana Street and (future) Central Park Boulevard will discharge to this basin, and runoff will ultimately be conveyed to the retention ponds provided for the development.

Basin 3900-01 (Irondale Gulch-Stapleton East Section 10)

This small basin is located on the northwest corner of 56th Avenue and Havana Street. The basin drains north to the RMANWR via Irondale Gulch.

The basin consists of approximately 140 acres and is part of the former Stapleton airport, and is located in the eastern portion of Section 10. It is largely undeveloped with the exception of the former runways and storage of crushed concrete. *The Stapleton Area Outfall System Plan,* Reference 7, shows details of the proposed development in this basin. Development in the basin must limit discharges into the RMANWR to historical conditions.

Very little of the basin is along 56th Avenue, which is the southern limit of the basin, and there are no proposed discharges into this area from the project.

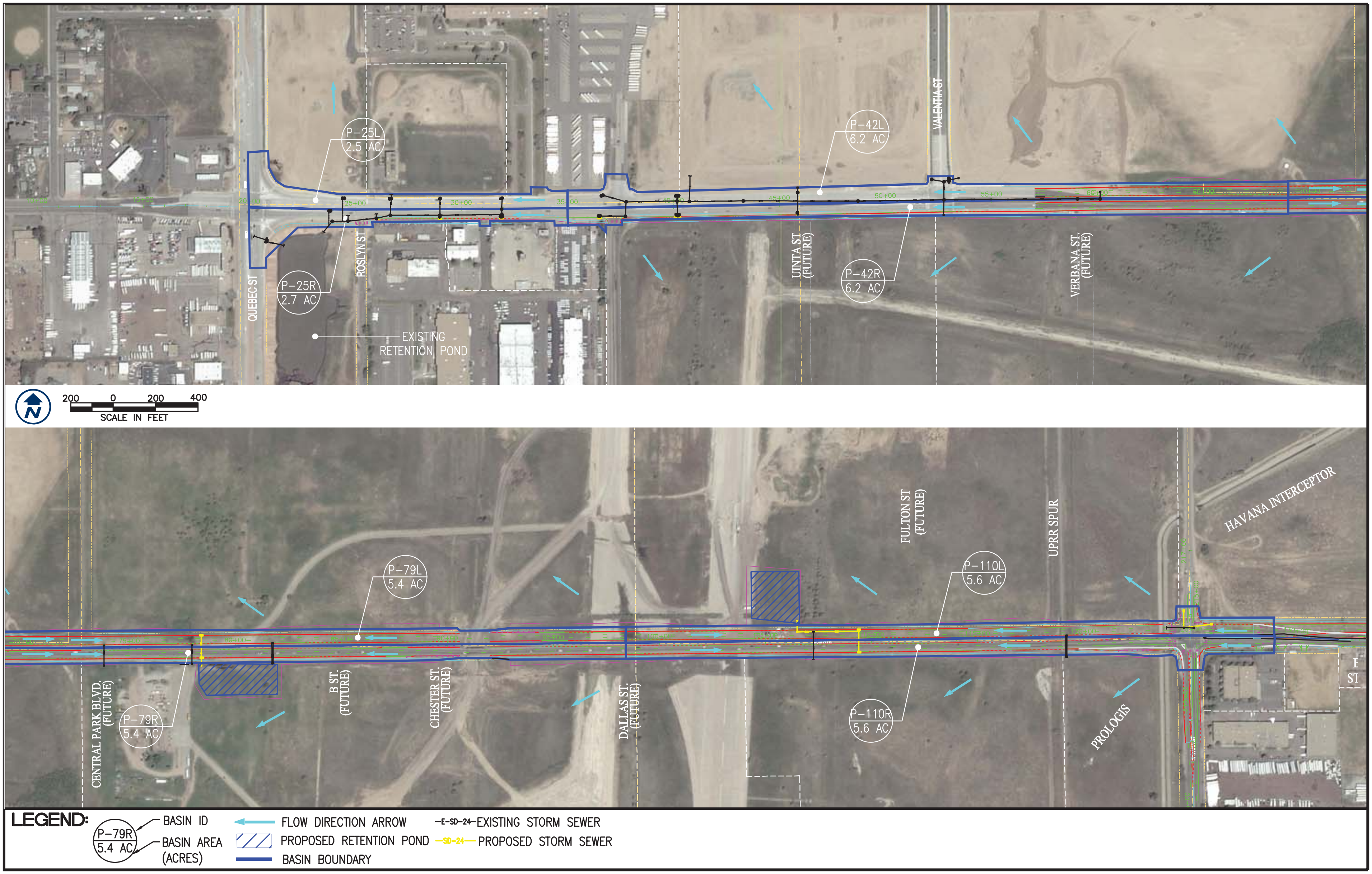
Basin 0058-01 (Prairie Gateway)

The basin is located north of 56th Avenue from Quebec Street to Valentia Street. It consists of approximately 1,017 acres and is being completely redeveloped as Prairie Gateway within Commerce City. A Denver Water Pump Station facility and the U.S. Postal Service's Bulk Mail Center are also located in the northeast corner of 56th Avenue and Quebec Street.

The report *Prairie Gateway, Outfall Systems Planning, Preliminary Design Report*, Reference 6, presents a detailed study of this drainage basin. The 100-year storm event is retained in natural depressions and existing retention ponds. There are no storm drains within the basin, and storm drains west of Quebec Street that convey storm water through Commerce City are undersized to convey even the 2-year storm event. Thus, the redevelopment of Prairie Gateway must preserve the natural retention storage volume. The portion of 56th Avenue between Spruce Street and Valentia Street discharges into this basin.

2.2 Sub-Basin Description

Grades within the right-of-way of the proposed 56th Avenue roadway generally slope downward to the west toward Quebec Street. Eight sub-basins have been defined by the proposed roadway profile or drainage structure locations, see Figure 2-2. Design points are generally located at low points along the profile and drainage is directed toward existing drainage features or to proposed water quality treatment areas and then to existing drainage facilities downstream.



Source: URS Corporation

56th Avenue Environmental Assessment
Quebec Street to Havana Street

FIGURE 2 - 2
Roadway Drainage Basins

3.0 DRAINAGE FACILITY DESIGN

3.1 Hydrology

Drainage design for the proposed roadway is based on the conceptual project configuration, on-site peak flows, historic and existing drainage patterns, and City and County of Denver, and other technical criteria requirements, as follows:

- City & County of Denver; *Storm Drainage Design and Technical Criteria*; Revised January 2006

- Colorado Department of Transportation; *Drainage Design Manual*; CDOT; 2004

- Colorado Department of Transportation; *Erosion Control and Stormwater Quality Guide*; CDOT; 2002

- Urban Drainage and Flood Control District; *Urban Storm Drainage Criteria Manual, Vol I, II and III;* (USDCM) June 2001

Hydrology Procedure. On-site sub-basins are analyzed with the Rational Method as described in the Urban Drainage Criteria Manual, June 2001. See Appendix A for the proposed roadway drainage basin calculations.

Design Storms. The major and minor storm recurrence intervals for the project are 100-year and 5-year, respectively, based on the size and 45-mph design speed of the proposed road. (CDOT, 2004). The 100-year event is used to size cross drainage structures, e.g. the Havana Interceptor, and retention ponds. The roadway storm drain system will be designed using the 5-year event. Flow spread criteria will be based on an arterial road with speeds greater than 45 miles per hour.

Land Use. The project area is bounded by commercial and residential zoned lands to the south and undeveloped areas to the north. Proposed conditions runoff calculations for on-site basins are based on widened roadway conditions with an on-site basin average of 100% imperviousness for paved surfaces and 0% imperviousness for lawns with sandy soils, per Table RO-3 (USDCM June 2001). Roadway medians will be landscaped with grasses or natural vegetation in an effort to minimize runoff. Roadway imperviousness has been calculated for each conceptual sub-basin in order to complete water quality capture volume calculations. Proposed condition calculations are based on conceptual project pavement limits.

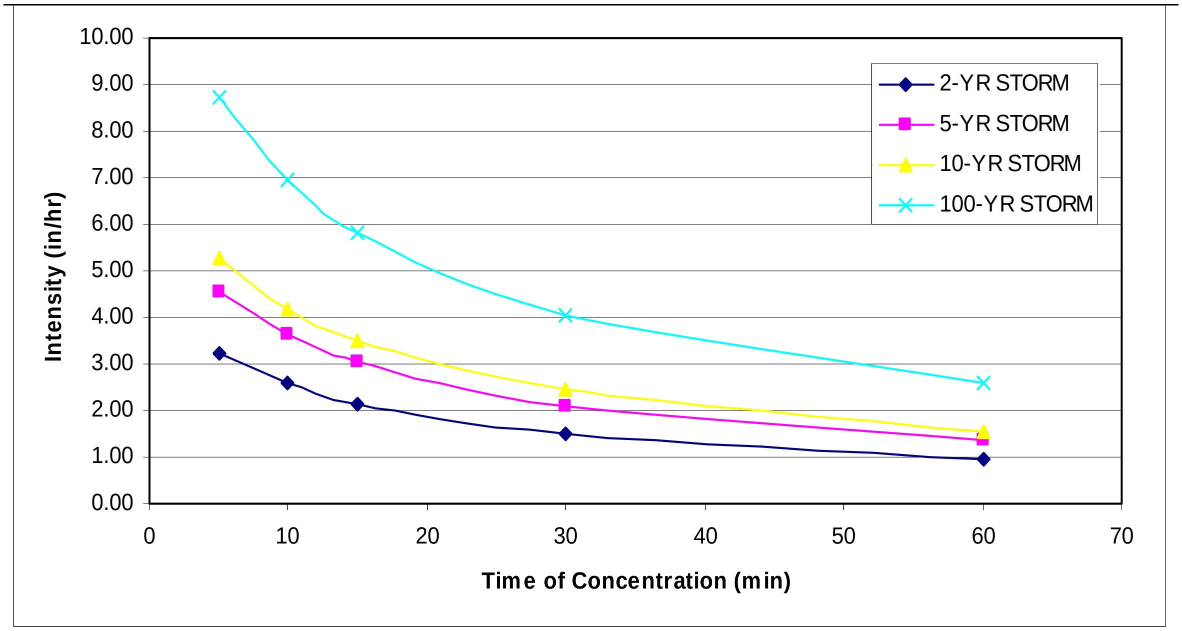
Soils. The United States Department of Agriculture Natural Resources Conservation Service classification Type A and B soils were used to calculate times of concentration for use in the Rational Method. The NRCS Hydrologic Soil Classification Map is in Appendix B. Within the project area, the vegetation is mostly native, except for developed areas that have been landscaped.

Runoff Coefficients. Runoff coefficients for pavement and developed areas are taken from the revised USDCM June 2001, Rational Method, Table RO-5.

Time of Concentration. The times of concentration for use in the Rational Method are calculated using the procedure described in the USDCSM June 2001. A minimum time of concentration of 5-minutes is used for sub-basins that only encompass the roadway. A minimum time of 10-minutes is used for basins that have both roadway and offsite drainage components.

Intensity. The intensity is calculated according to the June 2001 USDCM method, using point rainfall values provided in the *Storm Drainage Design & Technical Criteria* (CCD, 2006). Figure 3-1 shows the intensity versus the time of concentration for the City of Denver.

Figure 3-1
Intensity Duration Curves



3.2 Hydraulics

Storm Sewers: There are several storm sewers within the EA project area. The minimum pipe diameter is 18-inches, and all drainage pipes shall be Reinforced Concrete Pipe (RCP). New storm sewer pipes will be designed to contain the 5-year hydraulic grade line and such that the 100-year hydraulic grade line does not extend above one foot below the proposed finished grade. All improvements to the existing storm drain system will be designed to meet City and County of Denver criteria.

Roadway Inlets: Denver Standard Type 14 and 16 inlets are used to drain the roadway. All pipe outlets will include headwalls or flared end sections. CDOT Type C and D area drains will be used, if necessary, in graded areas outside the paved roadway. Ditch capacities and flow spreads will be calculated during final design.

3.3 Proposed Storm Drainage Improvements

Conceptual drainage improvements for the 56th Avenue Roadway Improvements Project from Quebec to Havana Streets include adjustments to the existing storm drain system to accommodate the roadway improvements, installation of new storm drain, and two water quality/retention basins.

New inlets are proposed where multi-use paths are added to the existing roadway, where the roadways will be widened, and at low points in the roadway profile where curb and gutter is added, and anywhere that gutter capacity or allowable flow spread is exceeded.

Proposed Storm Sewer Improvements

Proposed roadway improvements include widening the road by adding one 12-foot lane, new curb and gutter, a 12-foot landscape buffer and a 12-foot multi-use path on the south side from Roslyn Street to Valentia Street. This is the ultimate roadway section to be built when the Regional Fire Training Facility is relocated. Along this portion of the project, existing inlets will be replaced and new curb inlets will be installed to accommodate the improvements. New inlets will be reconnected to the existing storm sewer, and the runoff will be directed north to an existing drainage facility on the Prairie Gateway property, just east of Spruce Street. Runoff resulting from proposed improvements will be treated in this existing facility.

Interim drainage improvements from Quebec Street to Spruce Street, to be constructed if the Regional Fire Training Facility does not move, will include adjustment of the existing storm drain to accommodate the addition of a 12-foot multi-use path on the south side of the road, beginning at Roslyn Street and extending to Spruce Street. Existing inlets will be replaced, and offsite flows from the south will be redirected to the existing storm sewer in 56th Avenue, which eventually discharges to the retention pond located on the southeast corner of the intersection of Quebec and 56th Avenue. This existing drainage facility acts to provide water quality treatment for runoff generated by all proposed improvements in this area.

The roadway will be widened to the full six lane section to the south and north from Valentia Street to Havana Street. Existing culverts providing cross-drainage for offsite areas will be removed to accommodate the proposed roadway improvements. New inlets will be constructed in the low points of the profile and as required to meet flow spread criteria. New storm sewer storm sewer will be constructed from the inlets to the proposed temporary retention basins.

At Havana Street, the widened section ends and the roadway will be tapered back to the existing four lane section in approximately 1,000 linear feet. The intersection of 56th Avenue and Havana Street will be rebuilt, and the existing inlets relocated. The existing storm sewer in the intersection discharges to the Havana Interceptor, and this discharge point will be maintained. Runoff from the portion of roadway just east of Havana drains toward the intersection to curb inlets that will outlet to the existing Havana Interceptor.

Proposed Temporary Retention Basins

As shown on Figure 3-2, the site has two proposed water quality retention basins, where roadway flows from the eastern portion of the project will be retained and treated. These retention basins are intended as a temporary solution to drainage and water quality needs associated with the proposed improvements, and will remain in place until adequate formal downstream drainageways associated with the Stapleton redevelopment have been constructed. These retention facilities are sized to capture, at a minimum, the runoff equal to 1.5 times the 24-hour, 100-year storm plus one foot of freeboard and are as shallow as feasible to encourage infiltration and other losses of the captured roadway runoff.

Drainage from the (future) Central Park Boulevard to the (future) Dallas Street will be conveyed west via curb and gutter to new curb inlets located east of (future) Central Park Boulevard and outfall to a proposed retention basin on the south side of the road. Stormwater retention and water quality will be provided in this facility. Roadway flows that cannot be contained within the proposed curb and gutter will be captured in curb inlets and directed to the proposed retention facility in storm sewer or roadside ditches.

Drainage improvements from (future) Dallas Street to Havana Street will convey roadway flows via curb and gutter to new curb inlets and outfall north to a proposed retention basin at roadway Sta. 105+00. Stormwater retention and water quality will be provided in this facility. Roadway flows that cannot be contained within the proposed curb and gutter will be captured in curb inlets and directed to the proposed retention facility in storm sewer or roadside ditches.

No offsite flows will be conveyed within the new 56th Avenue roadway to the retention ponds. Between Havana St. and Quebec St., the existing road is on a ridgeline between Irondale Gulch to the north and Sand Creek to the South, and existing contours slope away from the roadway.

During the next phase of design, the need for emergency spillways on the proposed retention ponds, their size and location will be addressed. The proposed ponds are located in land that is currently undeveloped, and there are not structures immediately to the north or south. Final design of the proposed retention ponds will also need to consider the groundwater table and seasonal fluctuations which could affect the depth at which the ponds could be set. Borings at the proposed sites will be performed to identify groundwater constraints and infiltration rates for use in design.

Maintenance of the proposed retention ponds will be similar to maintenance of detention ponds in terms of trash removal, sediment removal and mowing, as the ponds are intended to have forebays and be grass-lined. Standing water will be minimized as much as possible by designing the ponds to infiltrate water. Depending on the soils at each site, infiltration can be enhanced by designing infiltration trenches or wells as part of the facility. If standing water becomes a problem, which is not likely as the only source of water will be runoff from the roadway, algae growth and insects can be controlled with the use of herbicide or pesticides. These temporary retention facilities should require no more maintenance than the existing facility at 56th and Quebec.

Source: URS Corporation

56th Avenue Environmental Assessment
Quebec Street to Havana Street

FIGURE 3-2
Proposed Stormwater Retention Basins

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4.0 WATER QUALITY

4.1 Water Quality

The following is a summary of the existing and proposed stormwater quality measures for the 56th Avenue Improvements project, specifically:

- Design criteria established for BMP design and construction;
- Impact analysis utilized to assess pre- and post- stormwater quality conditions;
- Location of water quality features (BMPs), drainage structures, and outfalls proposed within the project area and,
- A mitigation plan as it relates to the construction stormwater permit (i.e., Storm Water Pollution Prevention Plan)

Best Management Practices (BMPs) for temporary water quality on this project will be selected in accordance with the CDOT 'New Development and Redevelopment Program'. BMPs are described in CDOT's standard plans and construction specifications, and documents such as the *Erosion Control and Stormwater Quality Guide* (CDOT, 2002) and *Drainage Design Manual* (CDOT, 2004). Both documents have been developed to provide design guidance and criteria for engineers performing hydrologic and hydraulic analysis and design on roadway projects.

The *Erosion and Stormwater Quality Guide* promotes the use and proper installation of temporary BMPs to minimize the impact of erosion associated with roadway construction and operations. The *Drainage Design Manual* was developed to provide guidance and to establish criteria for the design of highway drainage features requiring a hydrologic analysis to determine the magnitude and frequency of flows, and a hydraulic analysis to locate and size drainage facilities.

For planning purposes, all water quality treatment BMPs along the project that will be utilized for treatment of developed roadway basins are assumed to be retention ponds, as listed in Table 4-1, and analyzed as outlined in the *Urban Storm Drainage Criteria Manual.* All proposed water quality treatment BMPs along the project will also be retention ponds with preliminary design based on the *Storm Drainage Design & Technical Criteria.*

Table 4-1
Proposed Water Quality Treatment Locations

Water Quality Location	Water Quality Capture Volume (watershed inches)	Design Volume (acre-ft)	Depth (ft)	Area (acre)
Pond at Quebec and 56th Avenue (Existing)	0.29	0.1513	3	0.05
Prairie Gateway Pond (Existing)	0.30	0.3717	3	0.12
Water Quality Location	**Effective Rainfall (inches)**	**Design Volume (acre-ft)**	**Depth (ft)**	**Area (acre)**
Retention Pond (at 79+00)	3.68	4.95	5	1.19
Retention Pond (at 110+00)	3.75	5.22	5	1.25

The goal for BMPs selected for this project is to remove 80% of Total Suspended Solids (TSS) and detain 100% of the Water Quality Capture Volume (WQCV). This criterion is consistent with the CDOT Tier 2, Intermediate Design Criteria. This criterion will protect the receiving waters downstream of the project.

4.2 Stormwater Pollution Prevention Plan

Construction Stormwater Permit

As required under the Clean Water Act amendments of 1987, the Environmental Protection Agency (EPA) has established a framework for regulating municipal and industrial stormwater discharges. This framework is under the National Pollutant Discharge Elimination System (NPDES) program (**Note**: The Colorado program is referred to as the Colorado Discharge Permit System, or CDPS, instead of NPDES). The Water Quality Control Division ("the Division") has stormwater regulations (5CCR 1002-61) in place.

Construction activities that are part of a larger common plan of development which disturb one acre or more over a period of time are also included (CDPHE, 2007). A construction stormwater permit [CDPS (COR-030000) General Permit Stormwater Discharges Associated with Construction Activity] is required for construction activities associated with this proposed project.

The application is due at least ten days prior to the commencement of earth grading activities. A mitigation plan or Stormwater Management Plan (SWMP) needs to be developed before submitting the construction stormwater permit application.

Mitigation Plan

A SWMP is required as part of the General Permit for Stormwater Discharges Associated with Construction Activity. This plan identifies measures, non-structural (i.e., administrative measures, phasing, signs, etc.) and structural, that will be used throughout each phase of the construction project to minimize erosion and protect water quality. The General Notes for the Stormwater Management Plan, provided in construction plans, and the Erosion Control Plans will be included with the Final Construction documents prepared for each phase of the project.

Erosion Control Plan

Section I.B.3.a of the Construction Stormwater Permit requires that erosion and sediment controls be included as part of the SWMP. The primary source of wind and water erosion will be from denuded and disturbed areas during construction of the project. BMPs consisting of gravel filter inlet protection, silt fence on earth embankments and silt sock on paved embankments, and permanent seeding will be utilized to minimize the impact of grading. Once permanent seeding and paving is complete, the potential for wind and water erosion will be minimized.

Erosion and Sediment Control plans prepared for this project will show the location and type of temporary erosion control measures to be installed during construction. These BMPs will be installed according to Colorado Department of Transportation's Erosion Control Manual and specifications in Section 208, or USDCM June 2001 Volume III, as appropriate.

Active areas of earthwork operations will be watered and compacted according to the earthwork specifications contained in the contract. Disturbed areas where construction activities will not occur for long periods will be stabilized. Throughout construction, as unpaved areas are completed, topsoil placement and permanent seeding or landscaping operations will follow.

Mud and dirt carryout onto existing paved streets will be prevented by construction of gravel entryways. Cleanup of paved surfaces will occur as necessary by sweeping.

Wind erosion from all active unpaved roads for this project will be controlled through sprinkling.

General Stormwater Permits

There are several layers of stormwater regulations governing the protection of water quality uses. The Federal agency, EPA, the state of Colorado agency, CDPHE-WQCD, and local jurisdictions, including the City and County of Denver, have administrative responsibilities, enforcement duties, and processes in place as a result of the National Pollutant Discharge and Elimination System (NPDES) Stormwater Phase I Municipal Separate Storm Sewer System (MS4) program.

The Phase I Stormwater Regulations require owners of MS4s to acquire a General NPDES Permit for stormwater discharges from their MS4. The General Stormwater Permit for the project area is:

City and County of Denver Municipal Stormwater Discharge Permit CDPS Permit Number COS-000001

The City and County of Denver has a Construction and Post-Construction Program in place to protect stormwater quality impacts from construction activities within their urbanized area.

5.0 CONCLUSIONS

5.1 Design Criteria

56th Avenue from Quebec Street to Havana Street is planned to become a six lane arterial road with multi-use path access along the entire corridor. For planning the stormwater conveyance system, existing drainage reports and master plans (see References; Section 5) have been reviewed as part of the storm drainage analysis for the EA project area.

Adherence to the appropriate design criteria, as stated in the previous section, will be evaluated during final design.

5.2 Drainage Concept

Between Quebec Street and Valentia Street, the stormwater runoff from the project area will be directed to existing storm sewer outfalls. Between Valentia Street and Havana Street, it will be necessary to create two temporary retention facilities to detain the runoff until proposed drainage infrastructure is constructed in adjacent areas. The method used to evaluate the need for retention and retention facility sizing is based on the drainage requirements of the City and County of Denver.

6.0 REFERENCES

1. Colorado Department of Transportation; *Drainage Design Manual*; CDOT; 2004.

2. Wright Water Engineers, Inc.; *Urban Storm Drainage Criteria Manual*; Urban Drainage & Flood Control District; June 2001.

3. Haestad Methods, *StormCAD*, Version 5.5, Storm Drain Hydraulics Program.

4. Colorado Department of Transportation; *Erosion Control and Stormwater Quality Guide*; CDOT; 2002.

5. Colorado Department of Public Health and Environment (CDPHE). March 2007. *Rationale for Stormwater Discharges Associated with Construction Activity.* http://www.cdphe.state.co.us/wq/PermitsUnit/stormwater/SWConstructionRationale.pdf

6. Urban Drainage and Flood Control District; Pr*airie Gateway Outfall Systems Planning Preliminary Design Report*, Love & Associates, Inc. April 2003

7. Urban Drainage and Flood Control District; *Stormwater Outfall Systems Plan Stapleton Area*, McLaughlin Water Engineers, July 1995

8. Matrix Design Group, Inc; *North Stapleton Infrastructure Master Plan Amendment No. 1*, December 2006

9. City & County of Denver; *Denver Storm Drainage Master Plan*, April 2005

10. Urban Drainage and Flood Control District; *Irondale Gulch and DFA 0055 Stormwater Outfall Systems Plan and Preliminary Design Report*, Wright Water Engineers, Inc. May 1990

11. City & County of Denver; *East 56th Avenue Corridor Concept Plan*, April 2004

12. City & County of Denver; *Storm Drainage Design and Technical Criteria*; Revised January 2006

13. Urban Drainage and Flood Control District; *Urban Storm Drainage Criteria Manual*, Vol I, II and III; (USDCM) June 2001.

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APPENDIX A

Hydrology

NRCS Hydrologic Soil Classification Map

Developed Basin Calculations – Rational Method

Water Quality Design

Retention Basin Calculations

Design References

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Design Basis/References/Assumptions:

Hydrologic criteria were established with reference to the City and County of Denver Storm Drainage Design and Technical Criteria (CCD, January 2006). The following sections describe the methods used to calculate peak flows for basins, sub-basins, and ultimately, design points along the project.

Rainfall

Rainfall intensity calculated using Equation 5.1 from the CCD Criteria.

$$I = \frac{28.5P_1}{(10+T_c)^{0.786}}$$

where: I = rainfall intensity (inches per hour)
P_1 = one-hour rainfall depth (inches)
T_c = time of concentration (minutes)

Point Rainfall Values taken from Table 5.1 in the CCD Criteria are as follows:

$P_2 = 0.95$, $P_5 = 1.34$, $P_{10} = 1.55$, $P_{50} = 2.25$, $P_{100} = 2.57$

Resulting Rainfall Intensity Equations for Denver County, Colorado:

2-YR STORM $$I = \frac{28.5*0.95}{(10+T_c)^{0.786}}$$

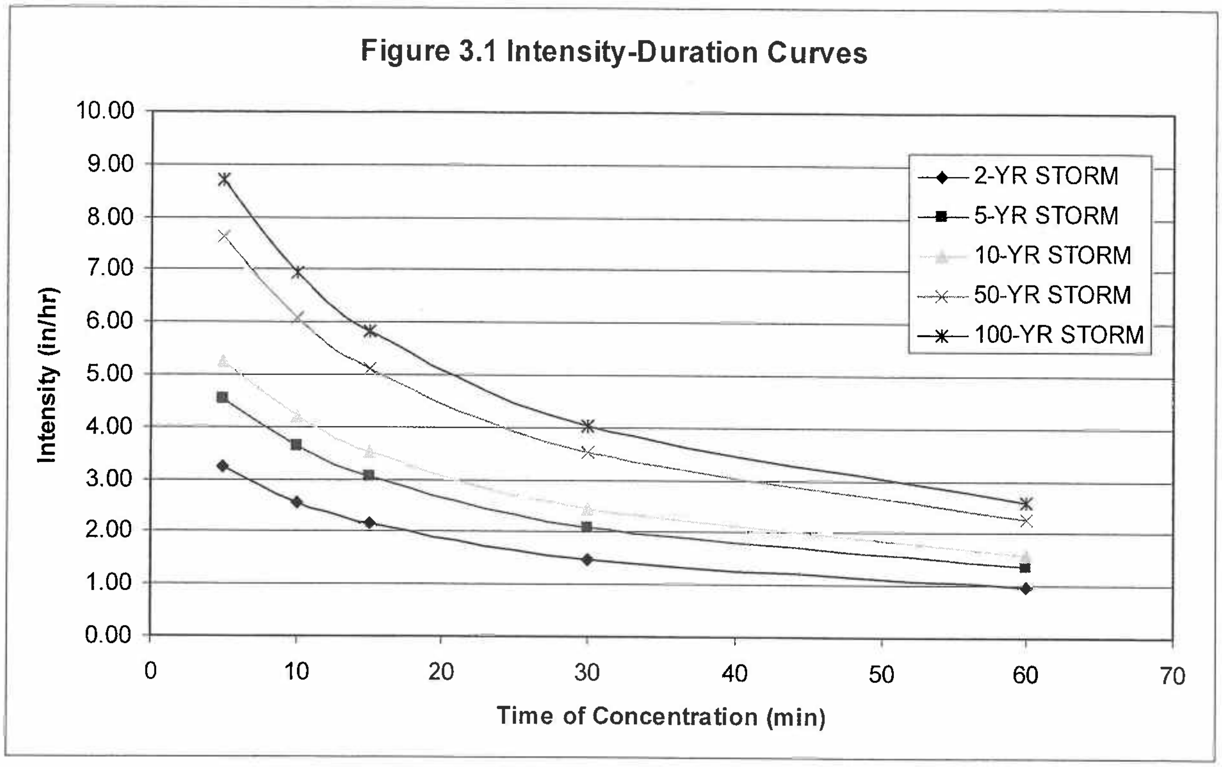
5-YR STORM $$I = \frac{28.5*1.34}{(10+T_c)^{0.786}}$$

10-YR STORM $$I = \frac{28.5*1.55}{(10+T_c)^{0.786}}$$

50-YR STORM $$I = \frac{28.5*2.25}{(10+T_c)^{0.786}}$$

100-YR STORM $$I = \frac{28.5*2.57}{(10+T_c)^{0.786}}$$

Resulting Intensity-Duration Curves for Denver County, Colorado:



Factors for Preparation of Intensity-Duration Curves (Table RA-4 UDFCD June 2001)

Duration (min)	5	10	15	30	60
Intensity (in/hr)	$3.48P_1$	$2.70P_1$	$2.28P_1$	$1.58P_1$	$1.0P_1$

2-YR STORM P_1 = 0.95 inches

Duration (min)	5	10	15	30	60
Intensity (in/hr)	3.306	2.565	2.166	1.501	0.95

5-YR STORM P_1 = 1.34 inches

Duration (min)	5	10	15	30	60
Intensity (in/hr)	4.6632	3.618	3.0552	2.1172	1.34

10-YR STORM P_1 = 1.55 inches

Duration (min)	5	10	15	30	60
Intensity (in/hr)	5.394	4.185	3.534	2.449	1.55

10-YR STORM P_1 = 2.25 inches

Duration (min)	5	10	15	30	60
Intensity (in/hr)	7.83	6.075	5.13	3.555	2.25

100-YR STORM P_1 = 2.57 inches

Duration (min)	5	10	15	30	60
Intensity (in/hr)	8.9436	6.939	5.8596	4.0606	2.57

Values Using Intensity Equations (Equation 5.1, CCD Storm Drainage Design and Technical Criteria, January 2006)

2-YR STORM I=28.5*0.95/((10+t)^0.786)

Duration (min)	5	10	15	30	60
Intensity (in/hr)	3.22	2.57	2.16	1.49	0.96

5-YR STORM I=28.5*1.34/((10+t)^0.786)

Duration (min)	5	10	15	30	60
Intensity (in/hr)	4.55	3.63	3.04	2.10	1.35

10-YR STORM I=28.5*1.55/((10+t)^0.786)

Duration (min)	5	10	15	30	60
Intensity (in/hr)	5.26	4.19	3.52	2.43	1.57

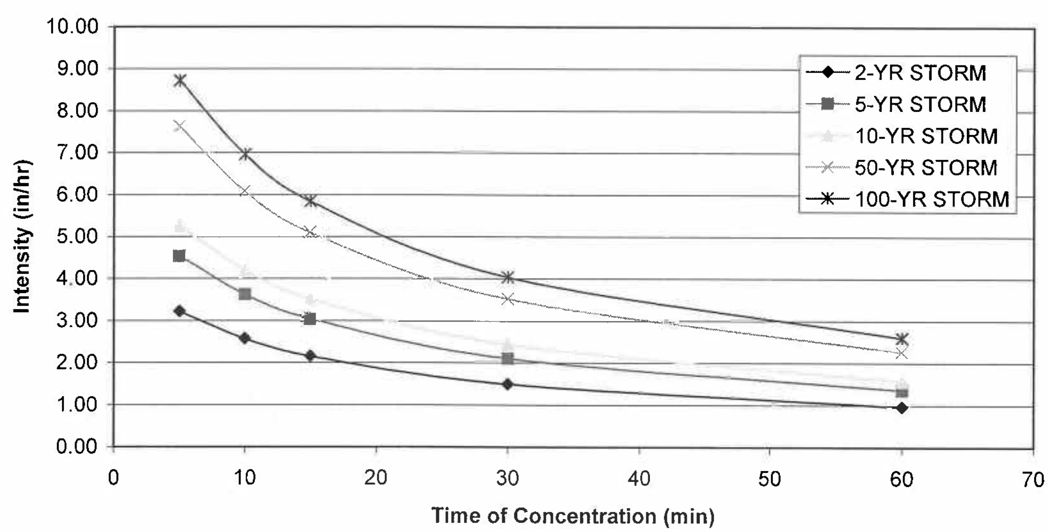
10-YR STORM I=28.5*2.25/((10+t)^0.786)

Duration (min)	5	10	15	30	60
Intensity (in/hr)	7.63	6.09	5.11	3.53	2.27

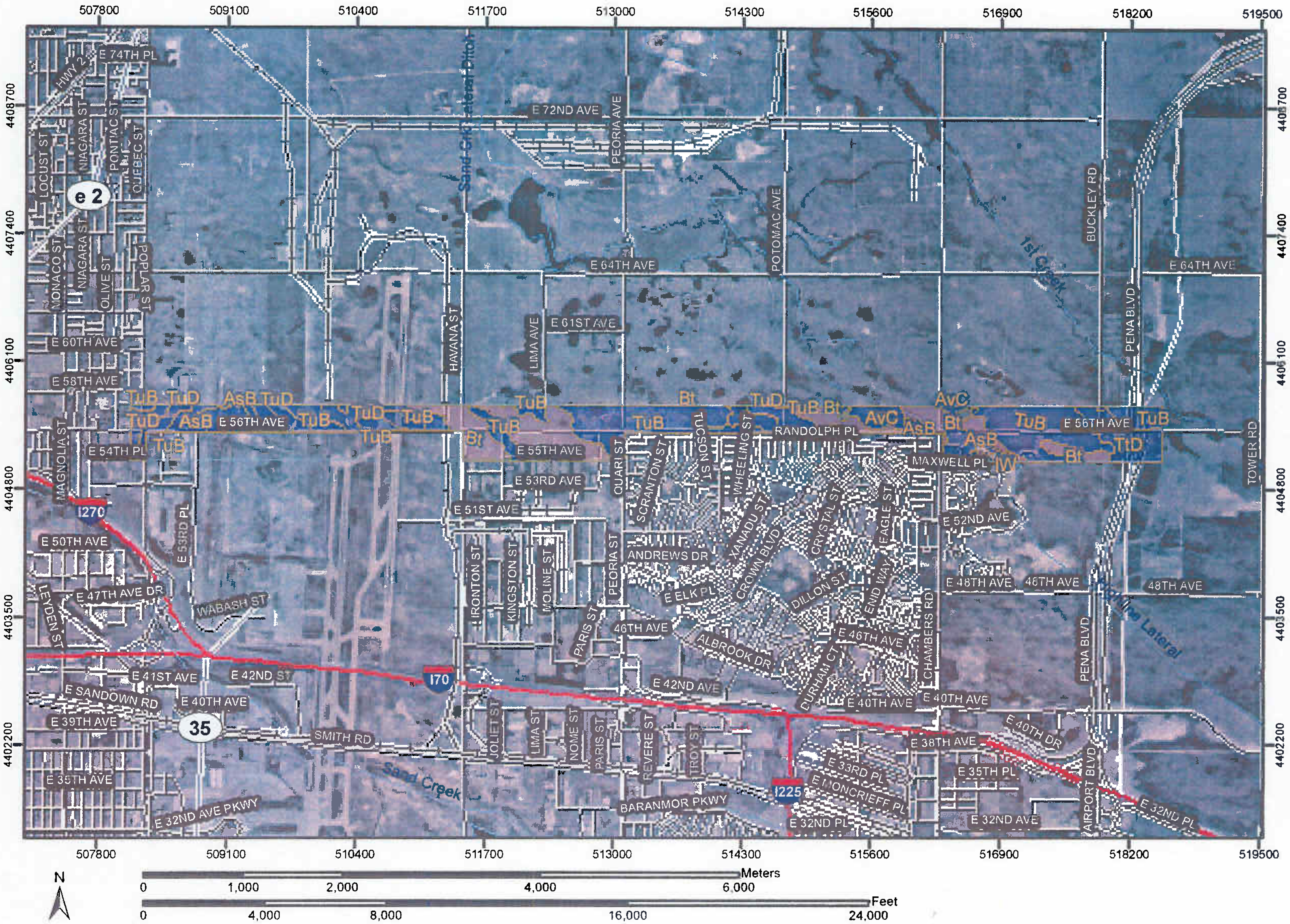
100-YR STORM I=28.5*2.57/((10+t)^0.786)

Duration (min)	5	10	15	30	60
Intensity (in/hr)	8.72	6.95	5.83	4.03	2.60

Figure 3.1 Intensity-Duration Curves CCD Method



Hydrologic Soil Group–Adams County Area, Parts of Adams and Denver Counties, Colorado
(NRCS Hydrologic Soil Group for 56th Avenue)



Natural Resources
Conservation Service

Web Soil Survey 2.0
National Cooperative Soil Survey

MAP LEGEND

Area of Interest (AOI)
- Area of Interest (AOI)

Soils
- Soil Map Units

Soil Ratings
- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Political Features

Municipalities
- Cities
- Urban Areas

Water Features
- Oceans
- Streams and Canals

Transportation
- Rails

Roads
- Interstate Highways
- US Routes
- State Highways
- Local Roads
- Other Roads

MAP INFORMATION

Original soil survey map sheets were prepared at publication scale. Viewing scale and printing scale, however, may vary from the original. Please rely on the bar scale on each map sheet for proper map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL: http://websoilsurvey.nrcs.usda.gov
Coordinate System: UTM Zone 13N

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Adams County Area, Parts of Adams and Denver Counties, Colorado
Survey Area Data: Version 7, Jan 8, 2007

Date(s) aerial images were photographed: 1993

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Hydrologic Soil Group

Hydrologic Soil Group— Summary by Map Unit — Adams County Area, Parts of Adams and Denver Counties, Colorado				
Map unit symbol	**Map unit name**	**Rating**	**Acres in AOI**	**Percent of AOI**
AsB	Ascalon sandy loam, 1 to 3 percent slopes	B	92.8	6.4%
AvC	Ascalon-Vona sandy loams, 1 to 5 percent slopes	B	35.9	2.5%
Bt	Blakeland-Truckton association	A	253.3	17.3%
IW	Intermittent water		4.0	0.3%
Sm	Sandy alluvial land	A	0.1	0.0%
TtD	Truckton loamy sand, 3 to 9 percent slopes	B	62.8	4.3%
TuB	Truckton sandy loam, 1 to 3 percent slopes	B	425.4	29.1%
TuD	Truckton sandy loam, 3 to 9 percent slopes	B	117.8	8.1%
W	Water		3.3	0.2%
Totals for Area of Interest (AOI)			1,461.4	100.0%

Description

Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups according to the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms.

The soils in the United States are assigned to four groups (A, B, C, and D) and three dual classes (A/D, B/D, and C/D). The groups are defined as follows:

Group A. Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C. Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.

Group D. Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. Only the soils that in their natural condition are in group D are assigned to dual classes.

Rating Options

Aggregation Method: Dominant Condition

Component Percent Cutoff: *None Specified*

Tie-break Rule: Lower

56th AVE. (QUEBEC TO PENA) - CONCEPTUAL
DEVELOPED BASIN CALCULATIONS - RATIONAL METHOD
COVER PAGE

BY: CMH
DATE: 12/3/2007
CHECKED BY: KK
DATE: 2/21/2008

Purpose: Generate basin and sub-basin areas, land use, flow paths, times of concentration and runoff coefficients for proposed conditions.

References: Urban Storm Drainange Criteria Manual (UDFCD, June 2001)

Assumptions: For % Impervious calculations, consider roadway pavement areas as USDCM "Paved Streets" and all other areas as "Lawns-Sandy Soils"
Use Rational Method taken from UDFCD to determine basin characteristics and peak flows.
Basin upstream and downstream limits determined by profile high points. Design points determined by profile low points.
Medians will be possibly xeriscaped or landscaped with vegetation other than grass.
Assume turning lane approaches to be 300 ft long
Assume worse-case scenario in areas where more than one surface althernative exists (choose alternative with more impervious area)
Flow path and time of concentration calculations made from proposed and existing topography. For roadway basins, there will be a sheet flow component (from the crown of the road to the ditch or gutter) and then concentrated flow down the ditch or gutter to the design points.
NRCS Type A and B Hydrologic Soils - See NRCS Soil Classification
Assume turning lanes to be 12' unless otherwise called out on typical section, double turn lanes to be 24'
Only consider turning lanes at major intersections
Subtract 0.5' curb top for landscaped corridors

Assume Type B Soils: Quebec to Havana
Oswego to Blackhawk
Chambers to Pena

Assume Type A Soils: Havana to Oswego
Blackhawk to Chambers

* See NRCS Soil Classification Map

Notes: Basins named by design point station - either the downstream end of the structure or basin.

56th AVE. (QUEBEC TO PENA) - CONCEPTUAL
DEVELOPED BASIN CALCULATIONS - RATIONAL METHOD
PROPOSED DRAINAGE BASIN PARAMETERS

BY: CMH
DATE: 11/1/2007
REVISED BY: KK
DATE: 2/21/2008

Conceptual Road Design - Typical Sections

STREET BEGIN	STATION BEGIN	STREET END	STATION END	LEFT ROW (FT)	LEFT PAVED (FT)	LEFT UNPAVED (FT)	RIGHT ROW (FT)	RIGHT PAVED (FT)	RIGHT UNPAVED (FT)	COMMENTS
BEGIN	10+00	QUEBEC ST.	20+00	74	59	15	78	51	27	
QUEBEC ST.	20+00	ROSLYN ST.	25+00	74	59	15	78	51	27	
ROSLYN ST.	25+00	SPRUCE WAY	37+00	74	59	15	73	58	15	Corridor in front of Fire Training Facility (South side) - retaining wall behind sidewalk
SPRUCE WAY	37+00	VALENTIA ST.	52+55	74	59	15	78	51	27	
VALENTIA ST.	52+55	HAVANA APPROACH -WEST	122+00	75.5	53.5	22	79.5	65.5	14	Assume no landscaping on south side of road.
HAVANA APPROACH - WEST	122+00	HAVANA ST.	125+00	79.5	65.5	14	79.5	66	13.5	Single right (Eastbound to Southbound)
HAVANA ST.	125+00	HAVANA APPROACH - EAST	128+00	79.5	68	11.5	79.5	64.5	15	Double left (Westbound to Southbound) and Single Right (Westbound to Northbound)
HAVANA APPROACH - EAST	128+00	JOLIET ST.	138+00	79.5	54.5	25	68	66	2	Excel Substation - no landscape buffer on south side
JOLIET ST.	138+00	PEORIA APPROACH - WEST	175+00	79.5	54.5	25	79.5	54.5	25	
PEORIA APPROACH - WEST	175+00	PEORIA ST.	178+00	79.5	54.5	25	77.5	54.5	23	Single Left (Eastbound to Northbound) and Single Right (Eastbound to Southbound)
PEORIA ST.	178+00	PEORIA APPROACH - EAST	181+00	79.5	77.5	2	112	66	46	Double left (Westbound to Southbound) and Single Right (Westbound to Northbound)
PEORIA APPROACH - EAST	181+00	REVERE ST.	192+00	68	54.5	13.5	112	54.5	57.5	No landscape buffer in fron tof Martinez Army Reserve Center - North side of road
REVERE ST.	192+00	WORCHESTER ST.	224+00	79.5	54.5	25	112	54.5	57.5	
WORCHESTER ST.	224+00	CHAMBERS APPROACH - WEST	280+50	61.75	54.5	7.25	97	54.5	42.5	No landscape buffer (westbound), 15.5' median
CHAMBERS APPROACH - WEST	280+50	CHAMBERS	283+50	61.75	54.5	7.25	97	73.25	23.75	Single right (Eastbound to Southbound)
CHAMBERS	283+50	CHAMBERS APPROACH - EAST	286+50	71	69.5	1.5	97	54	43	Double left (Westbound to Southbound)
CHAMBERS APPROACH - EAST	286+50	BUCKLEY RD.	336+00	75	67.5	7.5	60	44	16	8' Landscape buffer (westbound), 19' median. Use existing recreational corridor for south ROW boundary
BUCKLEY RD.	336+00	PENA APPROACH - WEST	354+56	82.5	54.5	28	82.5	54.5	28	
PENA APPROACH - WEST	354+56	PENA BLVD.	357+56	82.5	54.5	28	82.5	66	16.5	Single right (Eastbound to Southbound)
PENA BLVD.	357+56	PENA APPROACH - EAST	360+56	82.5	71	11.5	82.5	61.5	21	Double left (Westbound to Southbound)

56th AVE. (QUEBEC TO PENA) - CONCEPTUAL
DEVELOPED BASIN CALCULATIONS - RATIONAL METHOD
PROPOSED DRAINAGE BASIN PARAMETERS

BY: CMH
DATE: 11/1/2007
REVISED BY: KK
DATE: 2/21/2008

BASIN ID	STREET BEGIN	STREET END	BASIN BEGIN (STA.) ENGLISH	BASIN END (STA.) ENGLISH	SIDE (LT. or RT.)	AREA LENGTH (ft)2	AREA LENGTH (mi)2	AVG. BASIN WIDTH (ft)	BASIN AREA (SQ. FT.)3	BASIN AREA (AC.)	BASIN AREA (SQ. MI.)	AVG. PAVEMENT WIDTH (FT)	PAVEMENT AREA (SQ. FT)	PAVEMENT AREA (AC.)	SOIL GROUP (%)1 A	B	C/D	BASIN ID
P-25L	QUEBEC ST.	SPRUCE WAY	20+00	35+00	LT.	1500	0.28	74.0	111000	2.5	**0.004**	59.5	89250	2.0		100		**P-25L**
P-25R	QUEBEC ST.	SPRUCE WAY	20+00	35+00	RT.	1500	0.28	78.0	117000	2.7	**0.004**	52.0	78000	1.8		100		**P-25R**
P-42L	SPRUCE WAY	N/A (JUST BEFORE CENTRAL PARK)	35+00	69+00	LT.	3400	0.64	79.5	270300	6.2	**0.010**	53.5	181900	4.2		100		**P-42L**
P-42R	SPRUCE WAY	N/A (JUST BEFORE CENTRAL PARK)	35+00	69+00	RT.	3400	0.64	79.5	270300	6.2	**0.010**	65.5	222700	5.1		100		**P-42R**
P-79L	N/A (BETWEEN VERBANA AND CENTRAL PARK)	DALLAS ST. (FUTURE)	69+00	98+50	LT.	2950	0.56	79.5	234525	5.4	**0.008**	53.5	157825	3.6		100		**P-79L**
P-79R	N/A (BETWEEN VERBANA AND CENTRAL PARK)	DALLAS ST. (FUTURE)	69+00	98+50	RT.	2950	0.56	79.5	234525	5.4	**0.008**	65.5	193225	4.4		100		**P-79R**
P-110L	DALLAS ST. (FUTURE)	N/A (JUST PAST HAVANA)	98+50	129+00	LT.	3050	0.58	79.5	242475	5.6	**0.009**	56.1	171125	3.9		100		**P-110L**
P-110R	DALLAS ST. (FUTURE)	N/A (JUST PAST HAVANA)	98+50	129+00	RT.	3050	0.58	79.5	242475	5.6	**0.009**	66.7	203575	4.7		100		**P-110R**

56th AVE. (QUEBEC TO PENA) - CONCEPTUAL
DEVELOPED BASIN CALCULATIONS - RATIONAL METHOD
PROPOSED % IMPERVIOUS VALUES

BY: CMH
DATE: 11/01/07
REVISED BY: KK
DATE: 2/21/2008

BASIN ID	AREA (ft^2)	LAWNS, SANDY SOIL (% impervious)[1]	% **	RURAL RESIDENTIAL (%)*	% **	NEIGHBORHOOD RESIDENTIAL (%)*	% **	GRAVEL ROADS (%)*	% **	PAVED ROADS (%)*	PAVED AREA (ft^2)	% **	TOTAL PERCENT IMPERVIOUS	TOTAL PERCENT PERVIOUS	BASIN ID
P-25L	111000	0	19.6	17	0	46	0	46	0	100	89250	80.4	80	20	P-25L
P-25R	117000	0	33.3	17	0	46	0	40	0	100	78000	66.7	67	33	P-25R
P-42L	270300	0	32.7	17	0	46	0	40	0	100	181900	67.3	67	33	P-42L
P-42R	270300	0	17.6	17	0	46	0	40	0	100	222700	82.4	82	18	P-42R
P-79L	234525	0	32.7	17	0	46	0	40	0	100	157825	67.3	67	33	P-79L
P-79R	234525	0	17.6	17	0	46	0	40	0	100	193225	82.4	82	18	P-79R
P-110L	242475	0	29.4	17	0	46	0	40	0	100	171125	70.6	71	29	P-110L
P-110R	242475	0	16.0	17	0	46	0	40	0	100	203575	84.0	84	16	P-110R

* % Impervious values obtained from Urban Drainage and Flood Control District Drainage Manual
** Land Use obtained from arial photo of existing area and survey information.

56th AVE. (QUEBEC TO PENA) - CONCEPTUAL
DEVELOPED BASIN CALCULATIONS - RATIONAL METHOD
PROPOSED RUNOFF COEFFICIENTS FOR 2-, 5-, 10-, AND 100-YEAR STORM EVENTS

BY: CMH
DATE: 11/1/2007
REVISED BY: KK
DATE: 2/21/2008

BASIN ID	SOIL GROUP A (LAWNS, SANDY SOIL) (TABLE RO-5, URBAN DRAINAGE)				%	SOIL GROUP B (LAWNS, SANDY/CLAYEY SOIL) (TABLE RO-5, URBAN DRAINAGE)				%	SOIL GROUP C/D (LAWNS, CLAYEY SOIL) (TABLE RO-5, URBAN DRAINAGE)				%	C_2	C_5	C_{10}	C_{100}	BASIN ID
	C_2	C_5	C_{10}	C_{100}		C_2	C_5	C_{10}	C_{100}		C_2	C_5	C_{10}	C_{100}						
P-25L	0.543	0.568	0.600	0.662	0	0.571	0.599	0.633	0.704	100	0.600	0.630	0.665	0.747	0.0	0.571	0.599	0.633	0.704	P-25L
P-25R	0.385	0.421	0.461	0.538	0	0.423	0.463	0.506	0.600	100	0.461	0.504	0.551	0.661	0.0	0.423	0.463	0.506	0.600	P-25R
P-42L	0.391	0.427	0.467	0.543	0	0.429	0.468	0.511	0.603	100	0.466	0.509	0.555	0.664	0.0	0.429	0.468	0.511	0.603	P-42L
P-42R	0.570	0.594	0.625	0.684	0	0.597	0.623	0.655	0.723	100	0.624	0.652	0.686	0.763	0.0	0.597	0.623	0.655	0.723	P-42R
P-79L	0.391	0.427	0.467	0.543	0	0.429	0.468	0.511	0.603	100	0.466	0.509	0.555	0.664	0.0	0.429	0.468	0.511	0.603	P-79L
P-79R	0.570	0.594	0.625	0.684	0	0.597	0.623	0.655	0.723	100	0.624	0.652	0.686	0.763	0.0	0.597	0.623	0.655	0.723	P-79R
P-110L	0.424	0.458	0.495	0.568	0	0.460	0.497	0.537	0.624	100	0.496	0.536	0.579	0.681	0.0	0.460	0.497	0.537	0.624	P-110L
P-110R	0.593	0.616	0.646	0.703	0	0.618	0.643	0.674	0.740	100	0.644	0.670	0.702	0.776	0.0	0.618	0.643	0.674	0.740	P-110R

Arapahoe Parker
Existing Basin Calculations - Rational Method
Time of Concentration

BY: MMM
DATE: 11/14/2006

CHECKED BY: CR
DATE: 12/6/2006

TIME OF CONCENTRATION

SUB-BASIN DATA			INITIAL/OVERLAND TIME (Ti)				TRAVEL TIME (Tt)					TOTAL	Tc CHECK (Urbanized Basins)			FINAL Tc
BASIN ID	COMP. C_5	AREA (acre)	INIT. C_5[3]	LENGTH (ft)	SLOPE (Si)%	Ti (Min)[1]	LENGTH (ft)	LONG. SLOPE (ft/ft)	Paved(P) or Grass Waterway(GW) or Short Pasture(SP)	VEL. (fps)	Tt (Min)[2]	Ti+Tt (Min.)	Urban-Yes (Y) or No (N)	LENGTH. (ft)	Maximum Tc = (L/180) + 10	5 minute min (minutes)
P-25L	0.60	2.5	0.90	45	2.0	2.0	900	0.0070	P	1.7	9.0	10.9	Y	945	15.3	10.9
P-25R	0.46	2.7	0.90	45	2.0	2.0	900	0.0070	P	1.7	9.0	10.9	Y	945	15.3	10.9
P-42L	0.47	6.2	0.90	45	2.0	2.0	2820	0.0070	P	1.7	28.1	30.0	Y	2865	25.9	25.9
P-42R	0.62	6.2	0.90	45	2.0	2.0	2820	0.0070	P	1.7	28.1	30.0	Y	2865	25.9	25.9
P-79L	0.47	5.4	0.90	45	2.0	2.0	1950	0.0135	P	2.3	14.0	15.9	Y	1995	21.1	15.9
P-79R	0.62	5.4	0.90	45	2.0	2.0	1950	0.0135	P	2.3	14.0	15.9	Y	1995	21.1	15.9
P-110L	0.50	5.6	0.90	45	2.0	2.0	1450	0.0135	P	2.3	10.4	12.4	Y	1495	18.3	12.4
P-110R	0.64	5.6	0.90	45	2.0	2.0	1450	0.0135	P	2.3	10.4	12.4	Y	1495	18.3	12.4

N/A Not Applicable
[1] Ti = 0.395(1.1-C5)L0.5 / S1/3 (UDFCD, June 2001)
[2] Tt = Length/60Vel, WHERE Vel = CV*Slope^0.5 and CV = 20 (Paved), 15 (Grass Waterway), 7 (Short Pasture/Lawns)
[2] Initial C5 = Runoff coefficient for overland flow component of time of concentration, for a roadway basin, that would be 0.90
When overland flow path component of time of concentration encompasses both pavement and grassed waterway, used C5 of 0.45
For a basin that is mostly paved, overland tc is time from the crown of the road to the gutter flow line.
Min Overland Slope of 2%

56th AVE. (QUEBEC TO PENA) - CONCEPTUAL
DEVELOPED BASIN CALCULATIONS - RATIONAL METHOD
PROPOSED RUNOFF

BY: CMH
DATE: 2/22/2008
REVISED BY:
DATE:

			RATIONAL FLOWS												
BASIN ID	Tc	AREA (ACRES)	C_2	C_5	C_{10}	C_{100}	I_2 (IN./HR)[1]	I_5 (IN./HR)[1]	I_{10} (IN./HR)[1]	I_{100} (IN./HR)[1]	Q_2 (CFS)	Q_5 (CFS)	Q_{10} (CFS)	Q_{100} (CFS)	COMMENTS
P-25L	**10.92**	2.5	0.571	0.599	0.633	0.704	2.48	3.53	4.05	6.71	3.6	5.4	6.5	12.0	
P-25R	**10.92**	2.7	0.423	0.463	0.506	0.600	2.48	3.53	4.05	6.71	2.8	4.4	5.5	10.8	
P-42L	**25.92**	**6.2**	0.429	0.468	0.511	0.603	1.62	2.31	2.65	4.39	4.3	6.7	8.4	16.4	
P-42R	**25.92**	**6.2**	0.597	0.623	0.655	0.723	1.62	2.31	2.65	4.39	6.0	8.9	10.8	19.7	
P-79L	**15.94**	**5.4**	0.429	0.468	0.511	0.603	2.10	2.98	3.42	5.67	4.8	7.5	9.4	18.4	
P-79R	**15.94**	**5.4**	0.597	0.623	0.655	0.723	2.10	2.98	3.42	5.67	6.7	10.0	12.1	22.1	
P-110L	**12.35**	**5.6**	0.460	0.497	0.537	0.624	2.36	3.35	3.84	6.37	6.0	9.3	11.5	22.1	
P-110R	**12.35**	**5.6**	0.618	0.643	0.674	0.740	2.36	3.35	3.84	6.37	8.1	12.0	14.4	26.2	

[1] I = (28.5 x P1) / (10 +Tc)^0.786 , Eq. (RA-3) Urban Drainage, Where P1(2-yr)=0.95, P1(5-yr)=1.34, P1(10-yr)=1.55, P1(100-yr)=2.57
N/A - Not Applicable

BY: **CMH**
DATE: **12/3/2007**
CHECKED BY: **MMM**
DATE: **12/4/2007**

Purpose:	Determine water quality needs and potential treatment areas based on water quality impacts by proposed conceptual roadway design.
References:	Urban Storm Drainange Criteria Manual (UDFCD, June 2001) CDOT Erosion Control and Stormwater Quality Guide (CDOT, 2002) City and County of Denver Storm Drainage Design and Technical Criteria (January 2006) North Stapleton Infrastructure Master Plan Amendment No. 1 (December 2006) City and County of Denver Storm Drainage Master Plan (April 2005) Prairie Gateway Outfall Systems Planning Preliminary Design Report (City of Commerce City, april 2003)
Assumptions:	For % Impervious calculations, consider roadway pavement areas as USDCM "Paved Streets" and all other areas as "Lawns, Sandy Soil" Assume proposed water quality facilities (conceptual-level) at 79+00 and 110+00 will be retention ponds. Use CCD Retention Pond design criteria to calculate volume. Assume a design depth of 5 ft (including 1 ft freeboard) Assume all other water quality facilities (conceptual-level) will be Water Quality Extended Detention Basins with an average depth of 3 ft.
Notes:	% Impervious and area values for basins taken from the Developed Basin Calculations - Rational Method Water quality treatment locations determined by conceptual design analysis along the entire corridor from Quebec to Pena Blvd.

56th Avenue (Quebec to Pena) - Conceptual
Water Quality Design
WQ Summary for Project WQ Locations

BY: CMH
DATE: 12/3/2007
CHECKED BY: MMM
DATE: 12/4/2007

WQ Location	WQCV (watershed inches)	Design Volume (acre-ft)	Depth (ft)	Area (acre)
QUEBEC & 56TH POND	0.29	0.1513	3	0.05
PRAIRIE GATEWAY SWALE (AT STA. 42+00 LT)	0.30	0.3717	3	0.12
PROPOSED WQ EDB (AT STA. 140+00 LT)	0.28	0.5617	3	0.19
PROPOSED RANDOLPH TRIBUTARY DIVERSION WQ EDB (AT STA. 212+00 LT)	0.27	1.5291	3	0.51
PROPOSED PENA WQ EDB (AT STA. 352+00 LT, NE PENA INTERSECTION))	0.27	0.2352	3	0.08

WQ Location	Effective Rainfall (inches)	Design Volume (acre-ft)	Depth (ft)	Area*1.2 (acre)
RETENTION POND (AT 79+00)	3.68	4.95	5	1.19
RETENTION POND (AT 110+00)	3.75	5.22	5	1.25

56th Avenue (Quebec to Pena) - Conceptual
Water Quality Design
Composite % Impervious for WQ Locations

BY: CMH
DATE: 12/3/2007
CHECKED BY: MMM
DATE: 12/4/2007

BASIN ID WITH RESPECT TO TREATMENT AREA	BEGIN BASIN STATION (ft)	END BASIN STATION (ft)	SIDE	AREA[1] (sq ft)	AREA (acre)	% IMP.[2]	(AREA) x (% IMP.)	WEIGHTED % IMP.
QUEBEC & 56TH POND								
P-25L	20+00	35+00	LT	111000	2.55	80	204	
P-25R	20+00	35+00	RT	117000	2.69	67	180	73
TOTAL					5.23		384	
PRAIRIE GATEWAY SWALE (AT STA. 42+00 LT)								
P-42L	35+00	69+00	LT	270300	6.21	67	418	
P-42R	35+00	69+00	RT	270300	6.21	82	509	75
TOTAL					12.41		926	
PROPOSED ~~WQ EDB (AT STA. 78+00 RT)~~ RET POND								
P-79L	69+00	98+50	LT	234525	5.38	67	361	
P-79R	69+00	98+50	RT	234525	5.38	82	441	75
TOTAL					10.77		802	
PROPOSED WQ EDB ~~(AT STA. 107+00 RT)~~ RET POND								
P-110L	98+50	129+00	LT	242475	5.57	71	395	
P-110R	98+50	129+00	RT	242475	5.57	84	468	78
TOTAL					11.13		863	
PROPOSED WQ EDB (AT STA. 140+00 LT)								
P-139L	129+00	155+00	LT	206700	4.75	69	327	
P-139R	129+00	155+00	RT	196350	4.51	77	347	
P-168L	155+00	184+00	LT	227100	5.21	73	381	71
P-168R	155+00	184+00	RT	249450	5.73	65	372	
TOTAL					20.19		1427	
PROPOSED RANDOLPH TRIBUTARY DIVERSION WQ EDB (AT STA. 212+00 LT)								
P-192L	184+00	204+10	LT	150595	3.46	73	252	
P-192R	184+00	204+10	RT	225120	5.17	49	253	
P-210L	204+10	242+70	LT	404270	9.28	52	483	
P-210R	204+10	242+70	RT	273678	6.28	77	484	
P-255L	242+70	294+00	LT	329490	7.56	89	673	69
P-255R	242+70	294+00	RT	469860	10.79	59	636	
P-306L	294+00	337+60	LT	328200	7.53	89	671	
P-306R	294+00	337+60	RT	265200	6.09	73	444	
TOTAL					56.16		3897	
PROPOSED PENA WQ EDB (AT STA. 352+00 LT, NE PENA INTERSECTION))								
P-357L	337+60	360+55	LT	189338	4.35	69	300	
P-357R	337+60	360+55	RT	189338	4.35	69	300	69
TOTAL (NORTHEAST POND)					8.69		600	

Notes:
[1] Data from Conceptual Basin Calculations - Based on Conceptual Typical Sections
[2] Data from Conceptual Basin Calculations - Based on Conceptual Typical Sections

56th Avenue (Quebec to Pena) - Conceptual
Water Quality Design
Cumulative WQCV's for Project WQ Locations using UDFCD Criteria

BY: CMH
DATE: 12/3/2007
CHECKED BY: MMM
DATE: 12/4/2007

Water Quality Location =	**QUEBEC & 56TH POND (EXISTING)**		
1. Basin Storage Volume - Proposed Roadway Basins			
A) Tributary Area's Imperviousness Ratio ($i = I_a / 100$)	I_a =	73.00	%
	i =	0.73	
B) Contributing Watershed Area (Area)	Area =	5.230	acres
C) Water Quality Capture Volume (WQCV) (WQCV =1.0 * (0.91 * I^3 - 1.19 * I^2 + 0.78 * I))	WQCV =	0.29	watershed inches
D) Design Volume: Vol = (WQCV / 12) * Area * 1.2	Vol =	0.1513	acre-feet
	Vol =	6590.63	ft^3

Water Quality Location =	**PRAIRIE GATEWAY SWALE (EXISTING AT STA. 42+00 LT)**		
1. Basin Storage Volume - Proposed Roadway Basins			
A) Tributary Area's Imperviousness Ratio ($i = I_a / 100$)	I_a =	75.00	%
	i =	0.75	
B) Contributing Watershed Area (Area)	Area =	12.410	acres
C) Water Quality Capture Volume (WQCV) (WQCV =1.0 * (0.91 * I^3 - 1.19 * I^2 + 0.78 * I))	WQCV =	0.30	watershed inches
D) Design Volume: Vol = (WQCV / 12) * Area * 1.2	Vol =	0.3717	acre-feet
	Vol =	16191.25	ft^3

Water Quality Location =	**PROPOSED WQ EDB (AT STA. 140+00 LT)**		
1. Basin Storage Volume			
A) Tributary Area's Imperviousness Ratio ($i = I_a / 100$)	I_a =	70.68	%
	i =	0.71	
B) Contributing Watershed Area (Area)	Area =	20.193	acres
C) Water Quality Capture Volume (WQCV) (WQCV =1.0 * (0.91 * I^3 - 1.19 * I^2 + 0.78 * I))	WQCV =	0.28	watershed inches
D) Design Volume: Vol = (WQCV / 12) * Area * 1.2	Vol =	0.5617	acre-feet
	Vol =	24467.65	ft^3

Water Quality Location = PROPOSED RANDOLPH TRIBUTARY DIVERSION WQ EDB (AT STA. 212+00 LT)	
1. Basin Storage Volume	
A) Tributary Area's Imperviousness Ratio ($i = I_a / 100$)	I_a = 69.38 % i = 0.69
B) Contributing Watershed Area (Area)	Area = 56.162 acres
C) Water Quality Capture Volume (WQCV) (WQCV =1.0 * (0.91 * I^3 - 1.19 * I^2 + 0.78 * I))	WQCV = 0.27 watershed inches
D) Design Volume: Vol = (WQCV / 12) * Area * 1.2	Vol = 1.5291 acre-feet Vol = 66607.60 ft^3

Water Quality Location = PROPOSED PENA WQ EDB (AT STA. 352+00 LT, NE PENA INTERSECTION))	
1. Basin Storage Volume	
A) Tributary Area's Imperviousness Ratio ($i = I_a / 100$)	I_a = 69.00 % i = 0.69
B) Contributing Watershed Area (Area)	Area = 8.693 acres
C) Water Quality Capture Volume (WQCV) (WQCV =1.0 * (0.91 * I^3 - 1.19 * I^2 + 0.78 * I))	WQCV = 0.27 watershed inches
D) Design Volume: Vol = (WQCV / 12) * Area * 1.2	Vol = 0.2352 acre-feet Vol = 10245.31 ft^3

56th Avenue (Quebec to Pena) - Conceptual
Water Quality Design
Retention Pond Design using CCD Criteria

BY: CMH
DATE: 2/8/2008
CHECKED BY: KK
DATE: 2/21/08

Table 13.4 Required Retention Rainfall (CCD Criteria)

% Impervious	Effective Rainfall (r_{eff})	% Impervious	Effective Rainfall (r_{eff})
35	2.56	70	3.54
40	2.7	75	3.68
45	2.84	80	3.82
50	2.98	85	3.96
55	3.12	90	4.1
60	3.26	95	4.24
65	3.4	99	4.35

	% Impervious	**Effective Rainfall (r_{eff})**	**A (acres)**	**V [1] (acre-ft)**
Retention Pond 79	75	3.68	10.77	4.95
Retention Pond 110	78	3.75	11.13	5.22

Notes:
1 - V = 1.5 x ((r_{eff} / 12) x A)

5.0 RAINFALL

5.1 Introduction

The design rainfall data to be used to complete hydrologic analyses described in the RUNOFF chapter of these DENVER CRITERIA are presented in this section. More specifically, this chapter provides: 1) point precipitation values for Denver, 2) information on the Colorado Urban Hydrograph Procedure (CUHP), and 3) an intensity-duration-frequency table for use with the Rational Method. All hydrological analyses within Denver shall use the rainfall data presented herein for calculating storm runoff. There may be cases where the designer needs to consider events more extreme than the 100-year storm (e.g., for public safety).

The design storms and intensity-frequency-duration tables for Denver were developed using the rainfall data and procedures presented in the DISTRICT MANUAL and are presented herein for convenience.

5.2 Rainfall Depth-Duration-Frequency Values

A review of the isopluvial maps presented in the *Precipitation-Frequency Atlas of the Western United States, Volume III-Colorado* (National Oceanic and Atmospheric Administration [NOAA] Atlas) shows that all of Denver can be included in one rainfall zone. The precipitation values for various return periods and duration storms were found to have minimal variation.

The 1-hour point rainfall is necessary for use with both the Rational Method and CUHP and is also the basis for deriving durations less than one hour. For watersheds greater than 10 square miles, the 3-hour rainfall depth is required, and for watersheds 20 square miles and larger, the 6-hour rainfall depth is required for use with CUHP. One-hour point rainfall values are summarized in Table 5.1. To obtain durations less than 1 hour, the factors in Table 5.2 are applied to the 1-hour point rainfall.

Table 5.1. One-hour Point Rainfall Depths

Return Period	One-hour Point Rainfall (inches)
2-Year	0.95
5-Year	1.34
10-Year	1.55
50-Year	2.25
100-Year	2.57
Date: July, 1992 Revised:	Reference: Wastewater Management Division, 1987, as determined based on NOAA Atlas 2, Volume IIII.

Table 5.2. Calculation of Rainfall Durations Less than One Hour

Duration (minutes)	**5**	**10**	**15**	**30**
Relationship to 1-hour Point Precipitation (P_1)	$0.29P_1$	$0.45P_1$	$0.57P_1$	$0.79P_1$
Reference: UDFCD 2001, Volume 1.				

These point rainfall depths must be distributed temporally (e.g., 5-minute increments) for use with the CUHP model. Area adjustment of these point rainfall values is required based on watershed size when using CUHP. CUHP automatically calculates temporal adjustments to rainfall distribution for various storm events and watershed sizes in accordance with the RAINFALL chapter of the DISTRICT MANUAL.

Table 5.3 provides the rainfall intensity-duration values calculated for use with the Rational Method in small watersheds that are 160 acres or less in size, based on the following equation:

$$I = \frac{28.5\, P_1}{(10 + T_c)^{0.786}} \qquad \text{(Equation 5.1)}$$

in which:

I = rainfall intensity (inches per hour)

P_1 = 1-hour point rainfall depth (inches)

T_c = time of concentration (minutes)

Rainfall

CUHP and UDSWMM are based primarily upon rainfall and impervious data specific to the location and layout of the site. For the purposes of modeling, the point rainfall data in Table 5.1: Point Rainfall, have been adopted from the DSDDTC and DCM. (The 24-hour data are from NOAA Atlas 2 for Colorado, the source reference for the DCM).

Table 5.1: Point Rainfall				
Return Period	**One-Hour**	**Two-Hour**	**Six-Hour**	**24-Hour**
Two-Year	0.95	1.11	1.43	2.05
Five-Year	1.34	1.55	1.96	2.65
10-Year	1.55	1.80	2.29	3.1
50-Year	2.25	2.54	3.10	4.5
100-Year	2.57	2.88	3.48	4.8

Rainfall losses were estimated per the UDFCD DCM. Depression losses for impervious soils were set at 0.10 inch, and at 0.35 inch for pervious soils (Table 2-1, DCM). Horton's infiltration parameters were set according to Table 2-2 of the DCM as shown in Table 5.2, Infiltration Rates.

Table 5.2: Infiltration Rates		
	SCS Soil Type A	**SCS Soil Type C**
Initial Infiltration	5.0 in/hr	3.0 in/hr
Final Infiltration	1.0 in/hr	0.5 in/hr
Decay Coefficient	0.0007/second	0.0018/second

Hydrologic Models

Two computer models are used for the Stapleton site: CUHP and UDSWMM. CUHP is the commonly used rainfall-runoff model for generating synthetic flood hydrographs for Denver-area watersheds. It employs the calculation of excess rainfall based on infiltration rates for specific soil types. The excess rainfall is applied to a unit hydrograph, which is determined by the size, length, and slope of the basin. CUHP output is available for independent use, or for transfer to the UDSWMM model.

The UDSWMM model is the UDFCD version of the nationally known Storm Water Management Model (SWMM). This program uses CUHP hydrographs to determine flows through channels and detention ponds. Detention pond area-height relationships define pond volumes. These data are then combined with the hydraulic characteristics of the pond outlet works to define outflows as reduced by attenuation in the pond storage volume. The UDSWMM output provides for sizing of the detention ponds, their outlet works, and the conveyance channels. The time-relative aspects of the stormwater system are used by UDSWMM. That is, flood peaks are not arbitrarily made coincident in time, but rather are time-correct for the input rainfall event, detention pond storage time, and time of travel in conveyance channels.

lowest point in the detention vault. The outlet pipe(s) shall discharge into a standard manhole or standard inlet or into an open drainageway with erosion protection. If an orifice plate is required to control the release rates, the plate(s) shall have a hinge on one side to open into the detention pipes to facilitate back flushing of the outlet pipe(s) and be firmly bolted or secured to the wall to prevent leakage around the edges.

13.6.4 Maintenance Access

Access easements to the detention facility shall be provided in accordance with Figure 13.6. Maintenance access designs shall take into consideration Occupational Safety and Health Administration (OSHA) requirements for confined space entry.

13.7 Design Standards for 100-year Runoff Retention Ponds

13.7.1 Allowable Use

A retention facility (a pond with a zero release rate or a very slow release rate when a trickle outflow can be tolerated) is used when there is no available formal downstream drainageway, or one that is grossly inadequate. When designing a retention facility, the hydrologic basis of design is difficult to describe because of the stochastic nature of rainfall events. Thus, sizing for a given set of assumptions does not ensure that another scenario produced by nature (e.g., a series of small storms that add up to large volumes over a week or two) will not overwhelm the intended design. For this reason, retention ponds are strongly discouraged as a permanent solution for drainage problems. They have been used in some instances as temporary measures until a formal system is developed downstream.

When a retention pond is proposed as a temporary solution to an evolving drainage problem, the pond shall be sized to capture, as a minimum, the runoff equal to 1.5 times the 24-hour, 100-year storm plus 1-foot freeboard. The facility also shall be situated and designed so that when it overtops, no human-occupied or critical structures (e.g., electrical vaults) will be flooded, and no catastrophic failure at the facility (e.g., loss of dam embankment) will occur. Retention facilities shall be as shallow as feasible to encourage infiltration and other losses of the captured urban runoff. A minimum infiltration drawdown of the volume in 72 hours will be required for all retention ponds. If this volume cannot be infiltrated within this time frame, a secondary outlet must be designed to provide additional releases from the pond.

13.7.2 Calculation of Retention Volume

The standard methodology described below in Equation 13.1 and Table 13.4 shall be used for calculating the required volume for retention. The intent of this methodology is to provide a simple, reasonable calculation without compromising Denver's policies for public safety and welfare.

$$V_r = 1.5 \times [(r_{eff} / 12) \times A]$$ (Equation 13.1)

where:

V_r = Volume of retention pond in acre-feet

r_{eff} = Effective rainfall (from Table 13.4) in inches

A = Area of development in acres

Table 13.4 Required Retention Rainfall

% Impervious	Effective Rainfall (r_{eff})	% Impervious	Effective Rainfall (r_{eff})
35	2.56	70	3.54
40	2.70	75	3.68
45	2.84	80	3.82
50	2.98	85	3.96
55	3.12	90	4.10
60	3.26	95	4.24
65	3.40	99	4.35

The proposed site development plan shall be used to determine the percent imperviousness value for use in Table 13.4.

The effective rainfall for retention is based on the 100-year, 24-hour rainfall obtained from the NOAA Atlas. The average value for Denver is considered to be 4.8 inches. The effective rainfall was extrapolated using CUHP to obtain an effective value based on site development characteristics. No reduction in volume will be allowed for pond infiltration during the storm event.

13.7.3 Design Standards for Retention Ponds

Design standards for retention ponds must comply with specific site development, flood proofing, site investigation and physical design considerations, as described below.

1. Site Development: The total development site area must be accounted for when planning for the retention of stormwater runoff. Provide grading for the entire site development to drain to the retention pond. Any off-site basins that historically flow through the site must be provided flow routes around the site and returned to the natural drainageway. Colorado state law maintains that "a property within a natural drainageway is subservient to the historic drainage from upper lands." Off-site drainage cannot be excluded if there is no other discharge location to be used; therefore, in volume calculations, include all off-site drainage basin areas that cannot otherwise be rerouted around the development and returned to the natural drainage path.

2. Floodproofing: The construction of a retention pond is essentially creating an isolated floodplain on the property. Delineate the limits of the 100-year flood area on the design drawing. Provide 1 foot of freeboard from the 100-year maximum water surface elevation of retention pond volume. Provide a 100-year emergency release overflow route from the site, which returns the flow back to its natural drainage path. Ensure finished floor elevations are 1.5 feet above the water surface elevation when the emergency spillway is conveying the maximum design flow or emergency flow.

3. Site Investigation: Site selection for infiltration retention ponds is critical. Factors for evaluating site suitability include:

   - Location of groundwater table
   - Location of bedrock
   - Seasonal fluctuation of water table
   - Soil permeability and porosity
   - Soil profile
   - Environmental conditions (e.g., contaminated soils)
   - Proximity to structures (e.g., basements)

   The following factors would preclude the site's use as a retention infiltration pond:

   - A seasonal high groundwater of less than 4 feet below the pond bottom
   - Bedrock within 4 feet of the pond bottom
   - Pond location over fill
   - Surface and underlying soils classified as NRCS Hydrologic Group D
   - Saturated infiltration rate less than 0.3 inch per hour

   A thorough geotechnical and geohydrological investigation shall be performed to determine site suitability. The following shall be included in the investigation:

   - Soil borings to a depth of 10 feet or to bedrock
   - Percolation tests
   - Soil classification

4. Physical Design Characteristics: The pond construction shall conform to the criteria as explained in Section 13.4 for above-ground detention basins. Section 13.4.2 shall be adhered to for grading

requirements. Section 13.4.8 shall be consulted for embankment protection as required. Section 13.4.9 shall be referred to for landscaping requirements.

13.8 Checklist and Design Aids

All of the design criteria in this chapter must be followed. Several key considerations that the designer must take care to address include:

1) Grade earth slopes 4:1 or flatter.
2) Provide minimum freeboard of 1 foot.
3) Provide trickle channels in above-ground detention areas.
4) Protect embankment from overtopping conditions.
5) Provide proper trash racks at all outlet structures.
6) Provide signs as required.
7) Provide maintenance access.
8) Provide emergency spillway and check emergency overflow path.
9) Check finished floor elevation of any structure near the detention basin.
10) Ensure that failure of underground detention is clearly evident from above ground.
11) Design the invert of the inflow pipe to the detention basin to be higher than the water quality level.

URBAN STORM DRAINAGE

CRITERIA MANUAL

volume 3 - best management practices

Urban Drainage and Flood Control District
Denver, Colorado September, 1999
Latest Revision: October 2007

High groundwater should not preclude the use of an EDB. Groundwater, however, should to be considered during design and construction, and the outlet design must account for any upstream base flows that enter the basin or that may result from groundwater surfacing within the basin itself.

Stable, all weather access to critical elements of the pond, such as the inlet, outlet, spillway, and sediment collection areas must be provided for maintenance purposes.

6.5 Design Procedure and Criteria

The following steps outline the design procedure and criteria for an EDB.

1. Basin Storage Volume

Provide a storage volume equal to 120 percent of the *WQCV* based on a 40-hour drain time, above the lowest outlet (i.e., perforation) in the basin. The additional 20 percent of storage volume provides for sediment accumulation and the resultant loss in storage volume.

A. Determine the *WQCV* tributary catchment's percent imperviousness. Account for the effects of DCIA, if any, on Effective Imperviousness. Using runoff volume reduction practices in the tributary catchment and Figure ND-1, determine the reduction in impervious area to use with *WQCV* calculations.

B. Find the required storage volume (watershed inches of runoff):

Determine the Required *WQCV* (watershed inches of runoff) using Figure EDB-2, based on the EDB's 40-hour drain time.

Calculate the Design Volume in acre-feet as follows:

$$Design\ Volume = \left(\frac{WQCV}{12}\right) * Area * 1.2$$

In which:

Area	=	The watershed area tributary to the extended detention pond
1.2 factor	=	Multiplier of 1.2 to account for the additional 20% of required storage for sediment accumulation

2. Outlet Works

The Outlet Works are to be designed to release the *WQCV* (i.e., not the "Design Volume") over a 40-hour period. Refer to the TYPICAL STRUCTURAL BMP DETAINS AND SPECIFICATIONS chapter for schematics pertaining to structure geometry; grates, trash racks, and screens; outlet type: orifice plate or perforated riser pipe; cutoff collar size and location; and all other necessary components.

For a perforated outlet, use Figure EDB-3 to calculate the required area per row based on *WQCV* and the depth of perforations at the outlet. See the TYPICAL STRUCTURAL BMP DETAINS AND SPECIFICATIONS chapter to determine the appropriate perforation geometry and number of rows. The lowest perforations should be set at the water surface elevation of the outlet micro-pool. The total outlet area is calculated by multiplying the area per row by the number of rows.

Minimized the number of columns and maximize the perforation hole diameter when designing outlets to reduce chances of clogging by

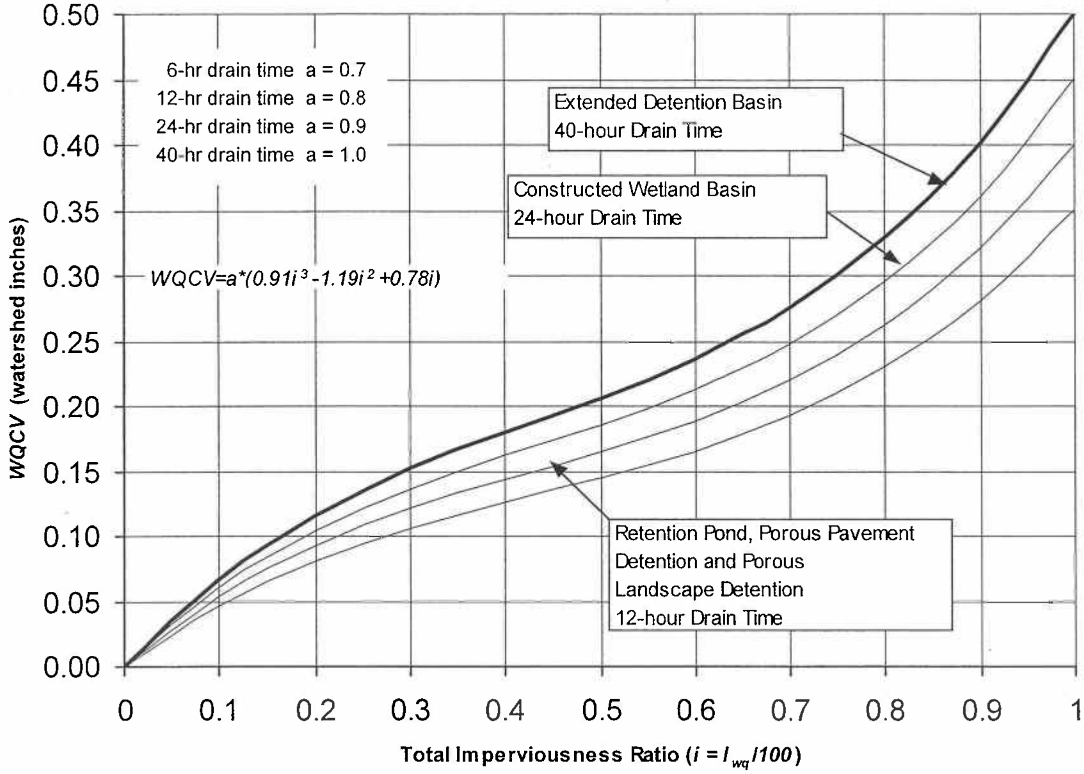


Figure EDB-2—Water Quality Capture Volume (*WQCV*), 80th Percentile Runoff Event

APPENDIX B

North Stapleton Infrastructure Master Plan

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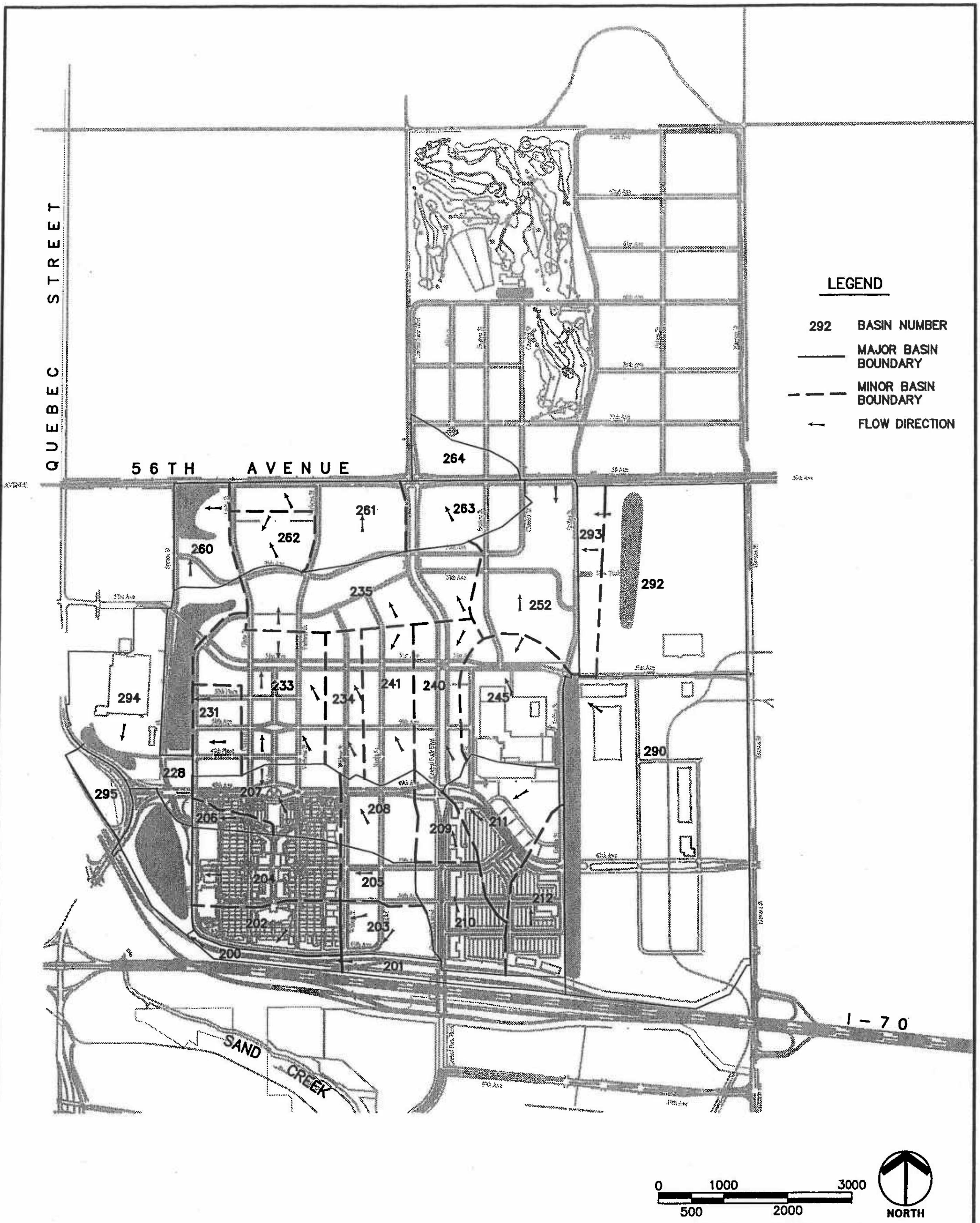

FORESTCITY DEVELOPMENT
7351 E. 29th Avenue, Suite 301 Denver CO 80238

Matrix Design Group, Inc.
Integrated Design Solutions
1601 Blake Street, Suite 200
Denver, CO 80202
Phone 303-572-0200
Fax 303-572-0202

CUHP Basin Delineation Plan

NORTH STAPLETON INFRASTRUCTURE MASTER PLAN - AMENDMENT NO. 1

DECEMBER 2006

Figure 5.2

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